NCHRP 24-17

LOAD AND RESISTANCE FACTOR DESIGN (LRFD) FOR DEEP FOUNDATIONS

APPENDIX D
DESIGN EXAMPLES

Prepared for
National Cooperative Highway Research Program
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APPENDIX D

DESIGN EXAMPLES

1. DRILLED SHAFTS

1.1. Design Methodology

The Federal Highway Administration (FHWA) design of drilled shafts founded in sand, clay, and intermediate geomaterial (i.e. soft rock) follows O'Neill and Reese (1998) and O'Neil et. al (1996).

According to FHWA, the axial capacity of a drilled shaft may be calculated as:

$$Q_t = Q_s + Q_b \tag{1}$$

where: Q_t = shaft capacity
Q_s = skin friction capacity
Q_b = end bearing capacity

In equation 1, shaft capacity or failure is defined as the applied load, which will result in settlement of the top of the drilled shaft equal to five percent the diameter of the shaft. An explanation of the computation of Qs and Qb for each material (i.e. sand, clay, and intermediate geomaterials) is presented below with examples given in Appendix A.

1.2. Skin Friction, Qs, and End Bearing, Q_b, for Clay

1.2.1. Skin Transfer

The load transfer in side resistance for drilled shafts founded in clay is a variant of Tomlinson's Alpha (α) method (1977). The undrained shear strength C_u of clay (found from laboratory tests or insitu correlations) is multiplied by alpha, α, to compute the unit skin friction (stress) at the depth z below the ground surface as follows,

$$f_{su} = \alpha C_u \tag{2}$$

where f_{su} = unit skin friction (stress) at depth z
α = empirical factor that varies with depth, (see Table 1) and
C_u = undrained shear strength at depth z,

Due to disturbances (i.e. drilling tool entering and exiting the hole frequently), the unit skin friction for the top five foot (see Fig. 1 and Table 1)) is neglected (i.e. set to zero). The setting of $\alpha = 0$ for a distance of one diameter above the base is from the work of Ellison et al. (1971). They showed that the downward movement of the base of the shaft can result in the development of a tensile crack in the soil near the base resulting in a lateral stress reduction. Consequently the unit skin friction in this zone (i.e. one diameter above the base) is set to zero.

Table 1 Recommended Values for α for Drilled Shafts in Clay

Location along Drilled Shaft	Value of α	Maximum Value of fsu (tsf)
From ground surface to depth of 5 ft. (1.52 m.)	0.0	0.0
From ground surface to length of casing	0.0	0.0
Bottom 1 diameter of shaft or 1 stem diameter above top of bell	0.0	0.0
All other points along drilled shaft sides	0.55	2.75 tsf (275 kPa)

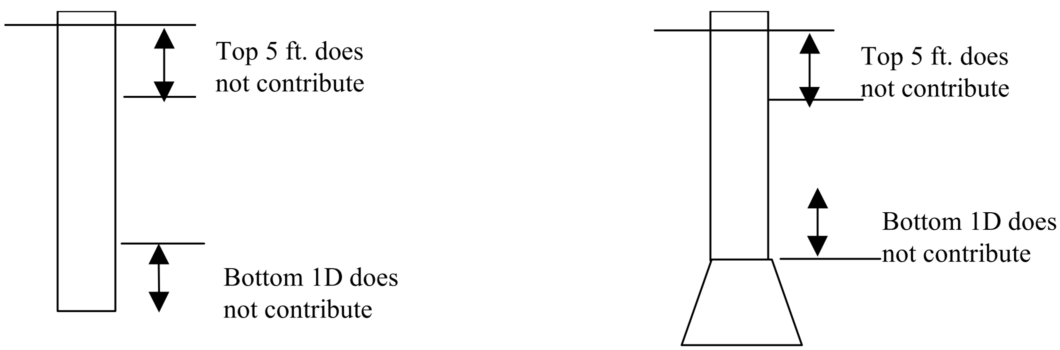


Figure 1 Portions of Drilled Shaft Non-Contributory in Friction

The total side resistance (i.e. force), Q_s for a given layer located at depths L_1 and L_2 below the ground surface is given as:

$$Q_s = \int_{L_1}^{L_2} f_{su} dA \qquad (3)$$

where dA = differential area of the perimeter along the side over a specific depth.

1.2.2. End Bearing

FHWA's unit end bearing (stress) for a drilled shaft founded in clay is based on the work of Skempton (1951):

$$q_b = N_c C_u, \quad q_b < 40 \text{ tsf } (4000 \text{ kPa}) \qquad (4)$$

where:
q_b = unit end bearing for drilled shafts in clay
$N_c = 6.0[1 + 0.2(L/B)]$ (bearing capacity factor) $N_c < 9$
C_u = average undrained shear strength of clay for 1.0 B below the tip
L = total embedment length of shaft in the ground

B = diameter of shaft at the base.

It should be noted that the limiting value of q_b (40 tsf) given in Equation. 4 is the largest measured end bearing recorded for drilled shafts and not a theoretical limit (Engling and Reese, 1974)

In the case of drilled shaft diameters (at the base: B_b) exceeding 75 inches (1.9 m), the FHWA reduces the unit end bearing, q_b, to q_{br} to ensure tolerable settlements under service load conditions. Again, failure capacity of a shaft is the load, which develops settlements equal to five percent the diameter of the shaft. The reduced bearing resistance is defined as:

$$q_{br} = Fr\, q_b \tag{5}$$

where: $F_r = 2.5/[aB_b \text{ (inches)} + 2.5\, b]$ $\quad F < 1.0$

in which $a = 0.0071 + 0.0021\,(L/B_b)$, $\quad a < 0.015$

$b = 0.45\,(C_u)^{0.5}$ $\quad 0.5 < b < 1.5$ and C_u in ksf

The latter expressions were based upon load tests of large under-reamed drilled shafts in very stiff clay (O'Neill and Sheikh, 1985). The reduced bearing resistance, q_{br}, gave similar results to the measured net bearing stress at a base settlement of 2.5 inches (6.35 cm). In addition, when more than half the design load is carried by end bearing, a global factor of safety greater than 2.5 is recommended by FHWA, unless site specific load tests are performed.

The failure end-bearing load, Q_b, is computed as q_b or q_{br} times the cross-sectional area of the drilled shaft's base. An example of capacity prediction for a drilled shaft founded in clay follows.

1.3. Skin Friction, Qs, and End Bearing, Q_b, for Sand

1.3.1. Skin Transfer

The unit side resistance on a drilled shaft founded in sand is based on Coulombic friction, i.e. equal to the normal (horizontal) effective stress times a coefficient of friction ($\tan \phi_c$).

$$f_{sz} = K\,\sigma_z \tan\phi_c \tag{6}$$

The total side resistance (i.e. force), Q_s for a given layer located at depths L below the ground surface is given as

$$Q_s = \int_0^L K\,\sigma_z \tan\phi_c\, dA \tag{7}$$

where f_{sz} = ultimate unit side shear resistance in sand at depth z,

K = a parameter that combines the lateral pressure coefficient

σ_z = vertical effective stress at depth z

ϕ_c = interface friction angle for soil-concrete

L = depth of embedment for drilled shaft in sand

dA = differential area of perimeter along sides of drilled shaft

Generally, the normal stress at the interface of the drilled shaft and the soil is relatively low when the excavation is completed; however the fluid stress from the fresh concrete will impose a normal stress that is dependent on the characteristics of the concrete. Experiments have shown that concrete with moderate slump (up to 6 inches, 15 cm.) act hydrostatically over a depth of 10 to 15 ft. (3 to 4.5 m.) followed by a leveling off of lateral stress at greater depths, probably due to arching (Bernal and Reese, 1983). Concrete with higher slump (about 9 inches,

23 cm.) act hydrostatically to a depth of 32 ft. (10 m.). Thus, construction procedures and the concrete characteristics will probably have a strong influence on the magnitude of the lateral stress at the soil-concrete interface.

As a result of the drilling and concreting influences, the K tan ϕ in Equation 6 is replaced by a simple constant, β, as a function of depth to account for variation in lateral stresses (i.e. K):

$$\beta = 1.5 - 0.135\sqrt{z} \qquad 1.2 > \beta > 0.25 \tag{8}$$

Consequently, the unit skin friction (stress) is given by

$$f_{sz} = \beta \sigma_z \tag{9}$$

The total side resistance (i.e. force), Q_s for a given layer located at depths L below the ground surface is given as

$$Q_s = \int \beta \sigma_z dA \tag{10}$$

It should be noted that the limiting unit skin friction (Equation 9) is again not a theoretical limit, but rather is merely the largest value that has been measured (Owens and Reese, 1982). Higher values can be used if justified via a load test.

1.3.2. End Bearing

Generally, an experimental tip resistance curve for a drilled shaft in sand shows that the end bearing is still increasing at settlements equal to five percent the diameter of the shaft (i.e. FHWA defined shaft capacity). For instance, settlements of more than fifteen percent the diameter have been recorded. However, since such large settlement is not tolerated for most structures; FHWA limits end bearing and settlements to five percent of the shaft's base diameter.

The values of the unit end bearing (stress) q_b are tabulated as a function of N_{SPT} (uncorrected field values) in Table 3 for shaft diameters less than 50 inches. In the case of large diameter shafts [i.e. Shaft diameter, D> 50 in. (1.3m)], equation 11 is used:

$$q_{br} = 50 * (q_b/B_b);\ B_b \text{ in inches}$$

$$\text{or } q_{br} = 1.3 * (q_b/B_b);B_b \text{ in meters} \tag{11}$$

Table 3 Recommended Unit End Bearing Values for Cohesionless Soils

N_{SPT} Values (Uncorrected)	Value of q_b (TSF) [kPa]
0 to 75	($0.60\ N_{SPT}$) [$60\ N_{SPT}$]
above 75	(45) [4500]

Table 3 limits the unit end bearing to 45 tsf (4500 kPa) at a settlement of 5 percent of the base diameter. Higher values, i.e. 58 tsf (5800 kPa) was measured for a settlement of 4 percent of the base diameter in Florida (Owens and Reese, 1982), are viable with load testing. An example of capacity prediction for a drilled shaft founded in sand is given in Appendix A.

1.4. Skin Friction, Qs, and End Bearing, Q_b, for Intermediate Geomaterials (Soft Rock)

FHWA's determination of skin and tip resistance for a drilled shaft founded in soft rock is based on a recent publication of O'Neill and et. al (1996). The equations for unit skin friction and end bearing are presented separately.

1.4.1. Side Resistance

Requires a six step approach as identified:

1. Find the average E_m (mass modulus of rock) and f_{su} (ultimate unit skin friction) along the side of the rock socket:

$$E_m = \Sigma E_{mk} L_k / \Sigma L_k \tag{12}$$

where $E_m = 115\ q_u$ and

$$f_{su} = \Sigma f_{su} L_k / \Sigma L_k \tag{13}$$

where f_{su} = ultimate side friction.
The values selected for f_{su} depend whether the socket is considered "smooth" and failure occurs at the interface (α values) or "rough" where failure occurs through the rock. The rough assumption was used in this study and f_{su} was set equal to $0.5\sqrt{q_u}\sqrt{q_t}$.

2. Calculate Ω given by Equation 14:

$$\Omega = 1.14(\frac{L}{D})^{0.5} - 0.05[(\frac{L}{D})^{0.5}\ \log_{10}(\frac{E_c}{E_m}) - 0.44 \tag{14}$$

where L = socket length and modulus of concrete is given as $E_c(\Psi) = 57{,}000\sqrt{q_{uc}}$

3. Calculate Γ given by Equation 15

$$\Gamma = 0.37\sqrt{(\frac{L}{D})} - 0.15[\sqrt{(\frac{L}{D})} - 1]\log_{10}(\frac{E_c}{E_m}) + 0.13 \tag{15}$$

4. Find n (socket surface roughness)
For "rough" sockets;

$$n = \sigma / q_u \quad \text{where } \sigma = \text{normal stress of concrete} = \gamma_c Z_c M \tag{16}$$

where γ_c . 130 pcf or 20.5 kN/m ^{3} and M is given in Table 4 below based on concrete slump and socket depth

Table 4 Values of M

Socket Depth (m)	Slump (mm)		
	125	175	225
4	0.50	0.95	1.0
8	0.45	0.75	1.0
12	0.35	0.65	0.9

Also, if a water table is present, then $\sigma_n = \gamma_c (Z_c - Z_w) + \gamma_c Z_w$, where Z_c = depth to water table.

In the case of a "smooth" socket, n is estimated from Figure 2.

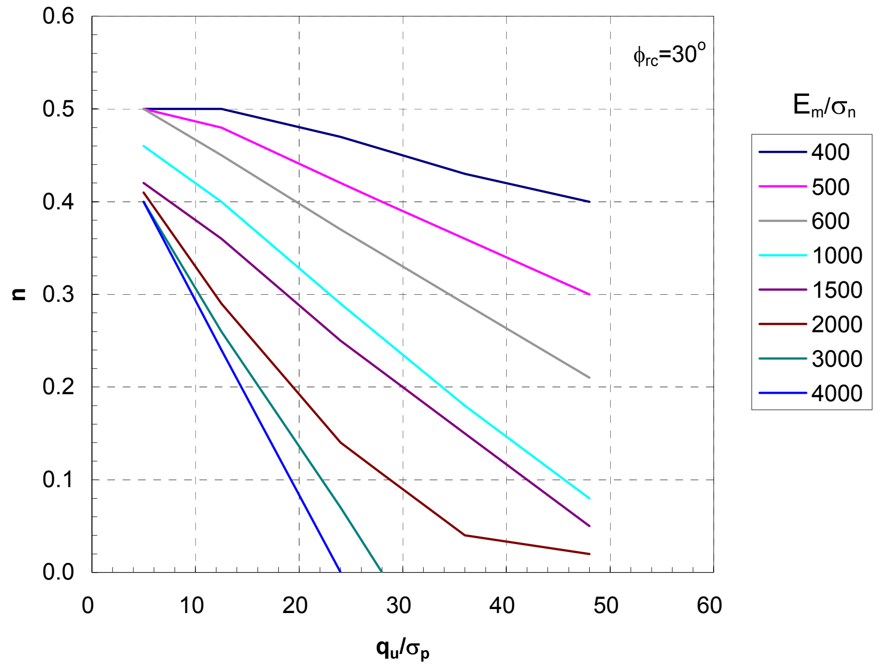


Figure 2 N Factors for Smooth Sockets

5. Next calculate Θ_f and K_f as given below:

$$\Theta_f = \frac{E_m \Omega}{\pi L \Gamma} W_t \tag{17}$$

$$K_f = n + \frac{(\Theta_f - n)(1 - n)}{\Theta_f - 2n + 1} < 1 \tag{18}$$

where W_t = deflection at top of rock socket

6. Finally, calculate the side shear load transfer vs. deformation from:

$$Q_s = \pi D L \Theta_f f_{su} \quad \Theta_f < n \tag{19}$$

$$Q_f = \pi D L K_f f_{su} \quad \Theta > n \tag{20}$$

1.4.2. End Bearing

The total tip resistance as a function of displacement according to O'Neill et al. (1996) is as follows:

$$Q_b = \frac{\pi D^2}{4} q_b \qquad (21)$$

where $\qquad q_b = \Lambda\, W_t^{0.67}$, and

$$\Lambda = 0.0134\, E_m \frac{(L/D)}{(1+L/D)} \left\{ \frac{[200(L/D)^{0.5} - \Omega][1+(L/D)]}{\pi L \Gamma} \right\}^{0.67} \qquad (22)$$

The total shaft resistance, Qt, for a rock socket is the sum of $Q_s + Q_b$. An example of a drilled shaft design in soft rock is given in the following section.

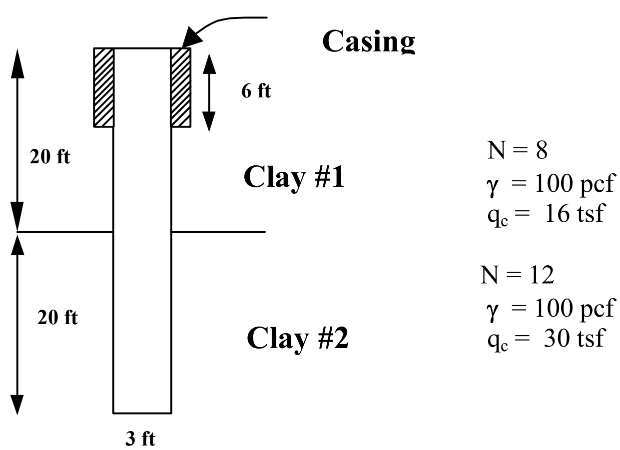
1.5. Examples

<u>Example 1. Drilled Shaft in CLAYS:</u>

Consider Two Cases calculated by FHWA method/α method, i.e., Reese, L.C. and M.W. O'Neill (1988):

1. Multi Layer Clay with Casing, 3 ft diameter straight shaft

2. Multi Layer Clay with Casing B > 75 ", 8 ft diameter straight shaft



$$c = \frac{q_c - \sigma_0}{15}$$

$$\text{Clay Layer \# 1 :c} = \frac{16 * 2000 - 10 * 100}{15} = 2{,}066.67 \; psf \;\; (1.0333 \; tsf)$$

$$\text{Clay Layer \# 2 :c} = \frac{30 * 2000 - 30 * 100}{15} = 3{,}800 \; psf \;\; (1.90 \; tsf)$$

1.1 ***Multi Layer Clay with Casing***: Full Capacity (40 ft Shaft)

a) Skin Friction:

$$Q_s = \pi * 3.0 * \left[(20' - 6')(0.55 * 1.033) + (20' - 3')(0.55 * 1.9)\right]$$
$$= 9.4248 * \left[7.9567 + 17.765\right]$$
$$= 242.42 \ \ Tons$$

b) End Bearing:

$$Q_b = q_b . \frac{\pi \ b^2}{4},$$

$$q_b = N_c C_u ,$$

$$N_c = 6.0 * \left[1 + 0.2 \frac{40}{3}\right] = 22 \ > \ 9 \ \ (use \ \ 9)$$

$$Q_b = (9 * 1.9 tsf) . \frac{\pi \ 3^2}{4} = 120.87 \ Tons$$

c) Total Capacity = Skin Friction + End Bearing
= 242.42 + 120.87
= 363.29 Tons (ultimate)

d) Unit Skin Friction with depth:

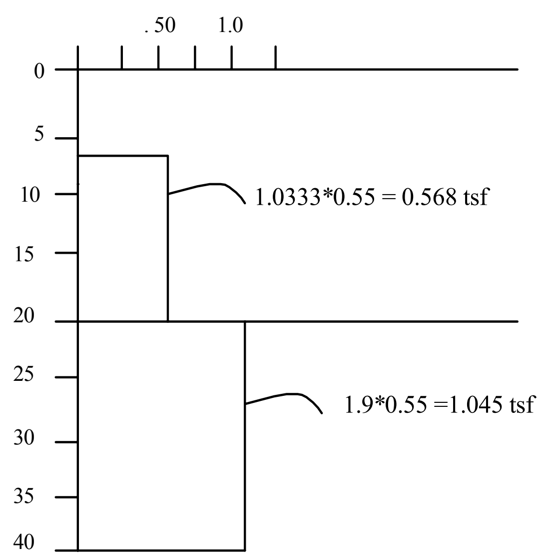
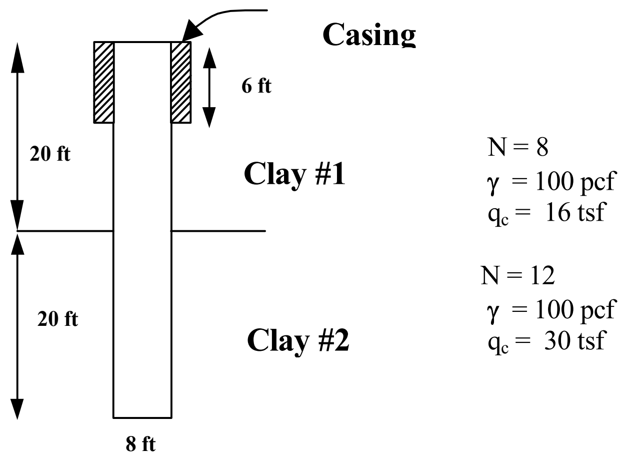


Table 29 recommended resistance factor for total resistance of a drilled shaft in clay using the FHWA method, and for all construction methods, is $\phi = 0.3$ w/Non-Redundant condition

Factored Capacity = 0.3 * 363.29 = **109.0 Tons**

1.2 **Multi Layer Clay with Casing, but B>75" (1.9m):**



a) Skin Friction: $Q_s = \pi * 8.0 * \left[(20' - 6')(0.55 * 1.033) + (20' - 8')(0.55 * 1.9)\right]$

$= 25.1327 * \left[7.9567 + 12.5\right]$

$= 515.14\ Tons$

b) End Bearing: If B > 75 " , then $q_{br} = F_r\ q_b$

$$F_r = \frac{2.5}{[a\ \ B_b(inches) + 2.5\ \ b]}$$

a = 0.0071 + 0.0021(L / B_b)
= 0.0071 + 0.0021(40' /8')
= 0.0176, but a < 0.015

$$b = 0.45\sqrt{C_u} \quad = 0.45\sqrt{1.9 * 2.0} \quad , C_u\ \ in\ \ ksf$$

$$= 0.8772 \quad , \ 0.5 < b < 1.5$$

$$F_r = \frac{2.5}{[0.015\ \ (96") + 2.5\ \ (0.8772)]} \quad = 0.6881$$

$$Q_b = \frac{\pi * 8^2}{4}(0.6881)(9 * 1.9) = 591.48\ \ Tons$$

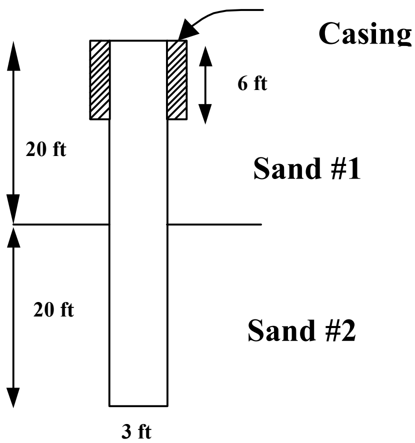
Q_t = 515.14 + 591.48 = 1106.62 Tons

c) Table 29 recommended resistance factor for total resistance of a drilled shaft in clay using the FHWA method, and for all construction methods, is ϕ = 0.3 w/Non-Redundant condition
Factored Capacity = 0.3 * 1106.62 = **332.0 Tons**

Example 2. Drilled Shaft in SANDS:

Calculated by FHWA method/β method, i.e., Reese, L.C. and M.W. O'Neill (1988):

1. Multi Layer Sand with Casing, 3 ft diameter straight shaft



Depth	N	γ,pcf
0	0	100
10	10	100
20	10	100
25	15	100
60	15	100

a) Skin Friction:

$\beta = 1.5 - 0.135\sqrt{z}$ $0.25<\beta(tsf) < 1.2$

or Z<4.94ft, β = 1.2 tsf, and Z>85.73ft, β = 0.25

$$\int_6^{40} \beta\sigma_v dz = \int_6^{40} 150Z - 13.5Z^{3/2} dZ = \frac{150Z^2}{2} - 13.5Z^{5/2} * \frac{2}{5} \Big|_6^{40}$$

$$= 65{,}355.84 - 2{,}223.82 = 18{,}116.37 * \frac{3\pi}{2000} = 297.50^T$$

b) End Bearing: above 8*B and below 3.5*B ,

above: 40.0 - 8*B = 40.0 - 8*(3)= 16' ;

below: 40.0 + 3.5*B = 40.0 + 3.5*(3) = 50.5'

for z = 16' q_b = 0.6*N = 0.6*(10) = 6 tsf
z = 20' q_b = 0.6*N = 0.6*(10) = 6 tsf
z = 25' q_b = 0.6*N = 0.6*(15) = 9 tsf
z = 60' q_b = 0.6*N = 0.6*(15) = 9 tsf

$$\therefore \; q_b = \left[\frac{\left[6*(20-16) + \frac{9+6}{2}*(25-20) + 9*(50.5-25) \right]}{\left[50.5-16 \right]} \right] = 8.4348$$

So, $\quad Q_b = 8.4348 * \left[\frac{\pi .3^2}{4} \right] = 59.622^T$

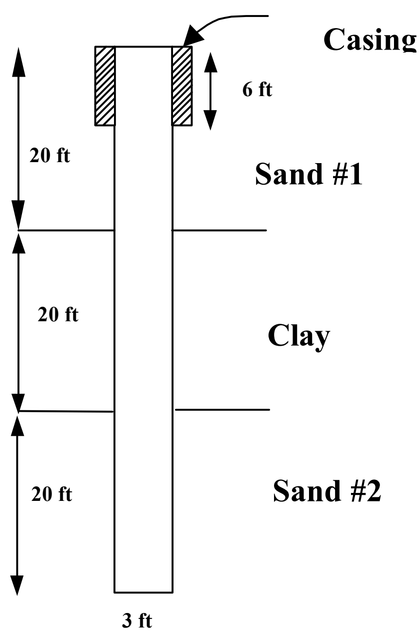
$$Q_T = 297.5 + 59.62 = 357.12$$

c) Table 29 recommended resistance factor for a total resistance of a drilled shaft in sand using the FHWA method for all construction methods is $\phi = 0.40$ w/Non-Redundant conditions.
Factored Capacity = 0.4 * 357.12 = **142.8 Tons**

Example 3. Drilled Shaft in MULTILAYER- SAND-CLAY-SAND:

Calculated by FHWA method/α and β method, i.e., Reese, L.C. and M.W. O'Neill (1988):

1. Multi Layer Sand and clay with Casing, 3 ft diameter straight shaft



Depth	N	γ,pcf	C_u ,tsf
0	0	100	
10	10	100	
20	10	100	
20		100	1.90
40		100	1.90
40	15	100	
70	15	100	

a) Skin Friction (6-20ft) : $Q_s = \dfrac{3.\pi}{2000} \displaystyle\int_6^{20} (1.5 - 0.135\sqrt{z}) \; \gamma \; z \; dz$

$$= 0.0047\left[\frac{150*z^2}{2} - 13.5*z^{5/2}*\frac{5}{2}\right]_6^{20}$$
$$= 0.0047\left[75*(20^2 - 6^2) - 5.4*(20^{5/2} - 6^{5/2})\right]$$
= 0.0047[27,300-9,183.6]
= 85.371^T

b) Skin Friction (20-40ft) : $Q_s = 3.\pi[(40-20)(0.55*1.9)]$
= 196.978^T

c) Skin Friction (40-60ft) : $Q_s = \frac{3.\pi}{2000}\int_{40}^{60}(1.5 - 0.135\sqrt{z})\ \gamma\ z\ dz$

$$= 0.0047\left[\frac{150*z^2}{2} - 13.5*z^{5/2}*\frac{5}{2}\right]_{40}^{60}$$
$$= 0.0047\left(75*(60^2 - 40^2) - 5.4*(60^{5/2} - 40^{5/2})\right)$$
= 0.0047[150,000-95,937.4]
= 254.764^T

ΣQ_s = 85.371+196.978+254.764 = 537.11 tons

d) Tip Resistance: above 8*B and below 3.5*B,

Above: 60.0 - 8*B = 60.0 - 8*(3)= 36 ft ;
Below: 60.0 + 3.5*B = 60.0 + 3.5*(3) = 70.5 ft

For z = 40 ft q_b = 0.6*N = 0.6*(15) = 9 tsf
z = 60 ft q_b = 0.6*N = 0.6*(15) = 9 tsf
z = 75 ft q_b = 0.6*N = 0.6*(15) = 9 tsf

So, $Q_b = \left[\frac{\pi.3^2}{4}\right]*9 = 63.62^T$

Check q_b of overlaying Clay:
$q_b = 9*C_u = 9*1.9 = 17.1$ tsf stronger, ∴ stop @ 40ft.

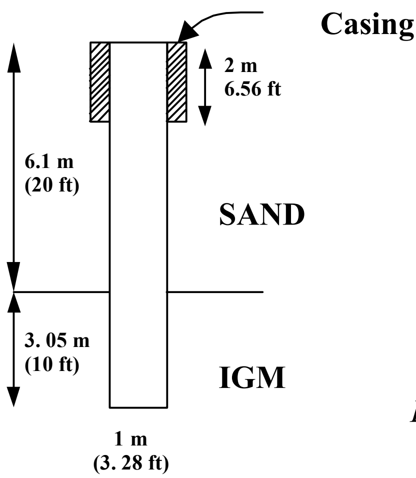
e) Q_T = 537.11 + 63.62 = 600.73 Tons

f) Table 29 recommended resistance factor for a tital resistance of a drilled shaft in sand and clay using the FHWA method using cased construction is $\phi = 0.50$ w/Non-Redundant condition.
Factored Capacity = 0.5 * 600.73 = **<u>300.4 Tons</u>**

Example 4. Drilled Shaft in IGM: (Sand & Limestone)

Calculated using IGM Method (Intermediate Geomaterials), i.e., O'Neill, et al (1996) "Load Transfer for Drilled Shafts in Intermediate Geomaterials" FHWA-RD-95-172, and O'Neill, M. W. and L. C. Reese (1999) 'Drilled Shaft: Construction Procedures and Design Methods", FHWA-IF-99-025.

1. Multi Layer Sand and Limestone with Casing, 1.0 m diameter straight shaft



γ = 100 pcf (15.708 kN/m^3)
N = 10

LimeStone:
q_u = 10 tsf (957.6 kPa, 0.96 Mpa)
q_t = 1 tsf (95.76 kPa, 0.096 Mpa)
γ = 135 pcf (21.2 kN/m^3), γ_c = 20.4 kN/m^3

$$E_c = 57{,}000\sqrt{f_y'} = 57{,}000\sqrt{5000 psi}$$
$$= 4.03E6psi \;\; (27.77E6 \;\; kPa)$$

Because of unit comparison problems, calculate Sand using English and Rock using SI units.

1. Skin Friction (Sand): $Q_s = \dfrac{3.28 * \pi}{2000} \displaystyle\int_{6.56}^{20} (1.5 - 0.135\sqrt{z}) \;\; \gamma \;\; z \;\; dz$

$$= \frac{3.28 * \pi}{2000}\left[\frac{150 * z^2}{2} - 13.5 * z^{5/2} * \frac{5}{2}\right]_{6.56}^{20}$$

$$= \frac{3.28 * \pi}{2000}\left[75 * (20^2 - 6.56^2) - 5.4 * (20^{5/2} - 6.56^{5/2})\right]$$

= 0.00515[26,772.5-9064.6]
= 91.23^T = 91.23*2000/224.809 = **811.66** kN

2. Analysis of Rock resistance is based on O'Neill (FHWA) intermediary geo-materials method, which is deformation based.

3. O'Neill IGM: (Note: Must enter values for E_c, slump, E_m/E_I, E_m, and IGM_Type = 2)

   a. E_m = 115 q_u = 115 (0.96 MPa) = 110.4 MPa.

b. $\Omega = 1.14\left(\frac{L}{D}\right)^{1/2} - 0.05\left(\left\{\frac{L}{D}\right\}^{1/2} - 1\right)\log(\frac{E_c}{E_m}) - 0.44$

$$\Omega = 1.14(3.05)^{1/2} - 0.05(3.05^{1/2} - 1)\log(\frac{27{,}777}{110.4}) - 0.44 = 1.46$$

c. $\Gamma = 0.37\left(\frac{L}{D}\right)^{1/2} - 0.15\left(\left\{\frac{L}{D}\right\}^{1/2} - 1\right)\log(\frac{E_c}{E_m}) + 0.13$

$$\Gamma = 0.37(3.05)^{1/2} - 0.15(3.05^{1/2} - 1)\log(\frac{27{,}777}{110.4}) + 0.13 = 0.507$$

d. $\frac{\theta}{w} = \frac{E_m \Omega}{\pi\ L\ \Gamma f_{su}}; \quad f_{su} = \frac{1}{2}\sqrt{q_u}\sqrt{q_t}$

$$= \frac{110.4 * 1.46}{\pi * 3.05 * 0.507 * (\frac{1}{2}\sqrt{0.96}\sqrt{0.96})} = \frac{161.18}{0.7374} = 218.586 / m$$

e. $\Lambda = 0.0134\ E_m \frac{(\frac{L}{D})}{(\frac{L}{D}+1)}\left\{\frac{200\ \left[\sqrt{\frac{L}{D}} - \Omega\right]\left[1 + \frac{L}{D}\right]}{\pi\ L\ \Gamma}\right\}^{0.67}$

$$\Lambda = 0.0134\ (110.4 MPa)\frac{3.05}{4.05}\left\{\frac{200\ \left[\sqrt{3.05} - 1.46\right]\left[1 + 3.05\right]}{\pi * 3.05 * 5.07}\right\}^{0.67}$$

=1.1141[4.7757]$^{0.67}$
=3.159 MPa m$^{-0.67}$

$$\Lambda = (1114.1\ kPa)\left\{\frac{200\ \left[\sqrt{3.05} - 1.46\right]\left[1 + 3.05\right]}{\pi * 3050 * 0.507}\right\}^{0.67}$$

= 1.1141[0.1316]
= 146.65 kPa mm$^{-0.67}$

f. Determine n for deformation criteria Fig (2) $\frac{q_u}{\sigma_p} = \frac{957.6\ kPa}{100} = 9.576$

$$\frac{E_m}{\sigma_n}; \quad \sigma_n = M\ \gamma_c\ Z_c; \quad Since\ \ Z_c = 6.1 + \frac{3.05}{2} = 7.625m \quad (use\ 8m)$$

$$For\ a\ slump = 175\ mm, \quad M(Fig3.5) = 0.78$$

$$\therefore \sigma_n = 0.78 * 20.4 * 7.625 = 121.33 \quad kPa$$

$$\therefore \frac{E_m}{\sigma_n} = \frac{110{,}400}{121.33} = 909.9 \quad \therefore n \approx 0.42$$

g. Select values of ‘w’ for calculating

$$Q_t = \pi\ D\ L\ \theta\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta < n\ ; \quad q_b = \Lambda\ w^{0.67}$$

$$Q_t = \pi\ D\ L\ k\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta > n$$

1) Let w = 2 mm; θ / w =218.586,

∴θ = 218.586 * 0.002m = 0.437 < n = 0.45

$$Q_t = \pi * 1 * 3.05 * 0.437 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.65 * 2^{0.67}$$

$$= 634 + 182.8$$

$$= 816.7\ kPa$$

2) Let w = 5 mm; θ / w =218.586,

∴θ = 218.586 * 0.005m = 1.093 > n = 0.45

$$k = n + \frac{(\theta - n)(1-n)}{(\theta - 2n + 1)} = 0.45 + \frac{(1.093 - 0.45)(1 - 0.45)}{(1.093 - 2(0.45) + 1)} = 0.7706$$

$$Q_t = \pi * 1 * 3.05 * 0.77 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.65 * 5^{0.67}$$

$$= 1118 + 336.8$$

$$= 1455\ kPa$$

h. Now go back and calculate sand capacity using trend lines when w = 2mm and 5mm.

1. S = (s*100/B);

@ 2mm S=(0.2cm*100/100cm) = 0.2
@ 5mm S=(0.5cm*100/100cm) = 0.5

2. $q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

$= -2.16*(0.2)^4 + 6.34*(0.2)^3 - 7.36*(0.2)^2 + 4.15*(0.2)$
= 0.5829 for w = 2mm
q_s= 0.5829 * (811.66 kN)
= 473.1 kN for 2 mm

$q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

$$= -2.16*(0.5)^4 + 6.34*(0.5)^3 - 7.36*(0.5)^2 + 4.15*(0.5)$$
$$= 0.892 \quad \text{for } w = 5\text{mm}$$

3. q_s = 0.892 * (811.66 kN)
   = 724.4 kN for 5 mm

i. Total Shaft Capacity (Sand + Rock)

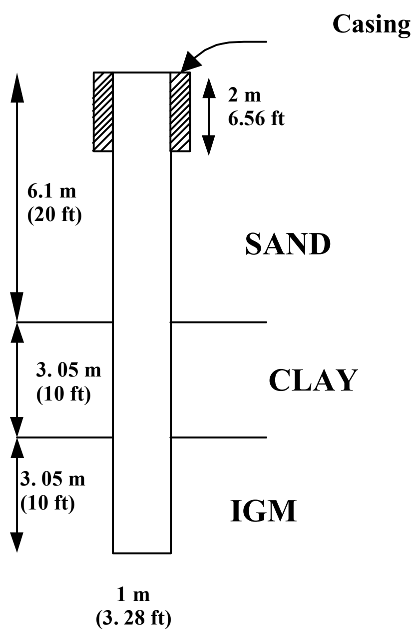
   1) @ 2mm Q_T = 473.1 kN + 634 kN+ 182.8 kN = 1289.9 kN
   2) @ 5mm Q_T = 724.4 kN + 1118 kN+ 336.8 kN = 2179.2 kN

j. Table 29 recommended resistance factor for a total resistance of a drilled shaft in IGM using the Reese and O'Neill FHWA method for all construction methods is $\phi = 0.75$ w/Non-Redundant condition.
   Factored Capacity = 0.75 * 1289.9 = **967.4 kN** @ 2 mm
   = 0.75 * 2179.2 = **1634.4 kN** @ 5 mm

Example 5. Drilled Shaft in IGM: (Sand, Clay & Limestone)

Calculated using IGM Method (Intermediate Geomaterials), i.e., O'Neill, et al (1996) "Load Transfer for Drilled Shafts in Intermediate Geomaterials" FHWA-RD-95-172, and O'Neill, M. W. and L. C. Reese (1999) 'Drilled Shaft: Construction Procedures and Design Methods", FHWA-IF-99-025.

1. Multi Layer Sand, Clay and Limestone with Casing, 1.0 m diameter straight shaft



γ = 100 pcf (15.708 kN/m^3)
N = 10

γ = 100 pcf (15.708 kN/m^3)
c = 1.9 tsf (181.94 kPa)

LimeStone:
q_u = 10 tsf (957.6 kPa, 0.96 Mpa)
q_t = 1 tsf (95.76 kPa, 0.096 Mpa)
γ = 135 pcf (21.2 kN/m^3), γ_c = 20.4 kN/m^3

$$E_c = 57{,}000\sqrt{f_y^{'}} = 57{,}000\sqrt{5000}\,psi$$
$$= 4.03E6\,psi \;\; (27.77E6 \;\; kPa)$$
$$f_{su} = \tfrac{1}{2}\sqrt{q_u}\sqrt{q_t} = 151.41 \;\; kPa$$

Smooth socket IGM_type = 2.0

1. Skin Friction (Sand): $Q_s = \frac{3.28 * \pi}{2000} \int_{6.56}^{20} (1.5 - 0.135\sqrt{z})\ \gamma\ z\ dz$

$$= \frac{3.28 * \pi}{2000} \left[\frac{150 * z^2}{2} - 13.5 * z^{5/2} * \frac{5}{2} \right]_{6.56}^{20}$$

$$= \frac{3.28 * \pi}{2000} \left[75 * (20^2 - 6.56^2) - 5.4 * (20^{5/2} - 6.56^{5/2}) \right]$$

= 0.00515[26,772.5-9064.6]

= 91.23^T = 91.23*2000/224.809 = 811.66 kN

2. Skin Friction (Clay): $Q_s = \pi\ D\ L\ \alpha\ C_u = \pi\ (1)\ (3.05)\ (0.55*181.94)$

= 958.85 kN (107.78^T)

3. FHWA IGM Calculations: (Note: Must enter values for E_c, slump, E_m/E_I, E_m, and IGM_Type = 2)

a. $E_m = 115\ q_u$ = 115 (957.6 kPa) = 110.4 MPa.

b. $$\Omega = 1.14\left(\frac{L}{D}\right)^{1/2} - 0.05\left(\left\{\frac{L}{D}\right\}^{1/2} - 1\right)\log\left(\frac{E_c}{E_m}\right) - 0.44$$

$$\Omega = 1.14(3.05)^{1/2} - 0.05(3.05^{1/2} - 1)\log\left(\frac{27,777}{110.4}\right) - 0.44 = 1.46$$

c. $$\Gamma = 0.37\left(\frac{L}{D}\right)^{1/2} - 0.15\left(\left\{\frac{L}{D}\right\}^{1/2} - 1\right)\log\left(\frac{E_c}{E_m}\right) + 0.13$$

$$\Gamma = 0.37(3.05)^{1/2} - 0.15(3.05^{1/2} - 1)\log\left(\frac{27,777}{110.4}\right) + 0.13 = 0.507$$

d. $$\frac{\theta}{w} = \frac{E_m \Omega}{\pi\ L\ \Gamma f_{su}}; \quad f_{su} = \frac{1}{2}\sqrt{q_u}\sqrt{q_t}$$

$$= \frac{110.4 * 1.46}{\pi * 3.05 * 0.507 * (\frac{1}{2} * 0.151 MPa)} = \frac{161.18}{0.7336} = 219.73/m$$

e. $$\Lambda = 0.0134\ E_m \frac{(\frac{L}{D})}{(\frac{L}{D} + 1)} \left\{ \frac{200\ \left[\sqrt{\frac{L}{D}} - \Omega\right]\ \left[1 + \frac{L}{D}\right]}{\pi\ L\ \Gamma} \right\}^{0.67}$$

$$\Lambda = 0.0134\ (110{,}112.5kPa)\frac{3.05}{4.05}\left\{\frac{200\ \left[\sqrt{3.05}-1.46\right]\left[1+3.05\right]}{\pi * 3050 * 0.507}\right\}^{0.67}$$

$=146.27$ kPa mm$^{-0.67}$

f. Determine n for deformation criteria Fig 36 $\frac{q_u}{\sigma_p} = \frac{957.6\ kPa}{100} = 9.576$

$$\frac{E_m}{\sigma_n};\quad \sigma_n = M\ \gamma_c\ Z_c;\quad Since\quad Z_c = 6.1 + 3.05 + \frac{3.05}{2} = 10.675m$$

$$For\ a\ slump = 175\ mm,\quad M(Fig3.5) = 0.68$$

$$\therefore \sigma_n = 0.68 * 20.4 * 10.675 = 148.1\quad kPa$$

$$\therefore \frac{E_m}{\sigma_n} = \frac{110{,}112.5}{148.1} = 743.6\quad \therefore n \approx 0.4\quad <\quad n = 0.45$$

g. Select values of ‘w’ for calculating

$$Q_t = \pi\ D\ L\,\theta\ f_{su} + \frac{\pi D^2}{4} q_b\quad for\quad \theta < n\ ;\quad q_b = \Lambda\ w^{0.67}$$

$$Q_t = \pi\ D\ L\,k\ f_{su} + \frac{\pi D^2}{4} q_b\quad for\quad \theta > n$$

1) Let w = 2 mm; θ / w =219.73 m^{-1},

$\therefore$θ = 219.73 * 0.002m = 0.439 < n = 0.45

$$Q_t = \pi * 1 * 3.05 * 0.439 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.27 * 2^{0.67}$$

$$= 636.85 + 182.8$$

$$= 819.2\ kPa$$

2) Let w = 5 mm; θ / w =219.73 m^{-1},

$\therefore$θ = 219.73 * 0.005m = 1.099 > n = 0.45

$$k = n + \frac{(\theta - n)(1-n)}{(\theta - 2n + 1)}\quad = 0.45 + \frac{(1.099 - 0.45)(1 - 0.45)}{(1.099 - 2(0.45) + 1)} = 0.75$$

$$Q_t = \pi * 1 * 3.05 * 0.75 * (151.4\ kPa) + \frac{\pi * 1^2}{4} * 146.27 * 5^{0.67}$$

$$= 1084.6 + 335.9$$

$$= 1420.5\ kPa$$

h. Now go back and calculate sand capacity using trend lines when w = 2mm and 5mm.

1. S = (s*100/B);

   @ 2mm S=(0.2cm*100/100cm) = 0.2 , and

   @ 5mm S=(0.5cm*100/100cm) = 0.5

2. $q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

   $= -2.16*(0.2)^4 + 6.34*(0.2)^3 - 7.36*(0.2)^2 + 4.15*(0.2)$

   = 0.5829 for w = 2mm

3. q_s = 0.5829 * (811.66 kN)

   = 473.1 kN for 2 mm

4. $q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$

   $= -2.16*(0.5)^4 + 6.34*(0.5)^3 - 7.36*(0.5)^2 + 4.15*(0.5)$

   = 0.892 for w = 5mm

5. q_s = 0.892 * (811.66 kN)

   = 724.4 kN for 5 mm

6. Clay: S = s*100/B; @ 2 mm S=0.2 & 0.5 @ 5 mm 0.12 < S < 0.74

$$\frac{q_{st}}{Q_s} = \frac{S}{[0.095155 + 0.892937 * S]} = \frac{0.2}{0.2737} = 0.731$$

$$= \frac{0.5}{0.5416} = 0.9232$$

   q_s = 0.7310*958.85 = 700.55 kN @ 2 mm

   q_s = 0.9232*958.85 = 885.16 kN @ 5 mm

i. Total Shaft Capacity (Sand + Rock)

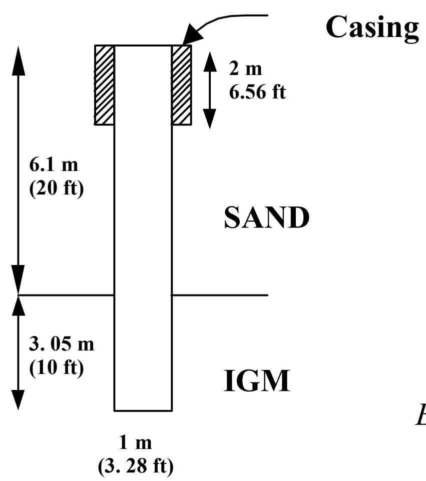
1) @ 2mm Q_T = 473.1 kN + 700.5 kN+ 636.85 kN + 182.4 = 1992.8 kN
2) @ 5mm Q_T = 724.4 kN + 885.16 kN+ 1084.6 kN + 335.9 kN = 3030.1 kN

j. Table 29 recommended resistance factor for a total resistance of a drilled shaft in IGM using the Reese and O'Neill FHWA method for all construction methods is ϕ = 0.75 w/Non-Redundant condition.
Factored Capacity = 0.75 * 1992.8 = **1494.6 kN** @ 2 mm
= 0.75 * 3030.1 = **2272.6 kN** @ 5 mm

Example 6. Drilled Shaft in IGM: (Sand & Limestone) Consider "Rough" Socket:

Calculated using IGM Method (Intermediate Geomaterials), i.e., O'Neill, et al (1996) "Load Transfer for Drilled Shafts in Intermediate Geomaterials" FHWA-RD-95-172, and O'Neill, M. W. and L. C. Reese (1999) 'Drilled Shaft: Construction Procedures and Design Methods", FHWA-IF-99-025.

1. Multi Layer Sand and Limestone with Casing, 1.0 m diameter straight shaft



γ = 100 pcf (15.708 kN/m^3)
N = 10

LimeStone:
q_u = 10 tsf (957.6 kPa, 0.96 Mpa)
q_t = 1 tsf (95.76 kPa, 0.096 Mpa)
γ = 135 pcf (21.2 kN/m^3), γ_c = 20.4 kN/m^3

$$E_c = 57{,}000\sqrt{f_y'} = 57{,}000\sqrt{5000}\,psi$$
$$= 4.03E6\,psi\ \ (27.77E6\ \ kPa)$$
$$f_{su} = \tfrac{1}{2}\sqrt{q_u}\sqrt{q_t} = 151.41\ \ kPa$$

1. From Previous Example,

a) Skin Friction (Sand): $Q_s = \dfrac{3.28*\pi}{2000}\displaystyle\int_{6.56}^{20}(1.5-0.135\sqrt{z})\ \gamma\ z\ dz$

$$= \frac{3.28*\pi}{2000}\left[\frac{150*z^2}{2} - 13.5*z^{5/2}*\frac{5}{2}\right]_{6.56}^{20}$$

$$=\frac{3.28*\pi}{2000}\left[75*(20^2-6.56^2)-5.4*(20^{5/2}-6.56^{5/2})\right]$$

=0.00515[26,772.5-9064.6]

$=91.23^T = 91.23*2000/224.809 = 811.66$ kN

2. O'Neill (FHWA) Rock - Rough Socket: (Note: Must enter values for E_c, slump, E_m/E_I, E_m, and IGM_Type = 1..0)

   a) If "Rough" $n = \sigma_n / q_u$

   $$\sigma_n = M\ \gamma_c\ Z_c; \quad Since \quad Z_c = 6.1 + \frac{3.05}{2} = 7.625m \quad (use\ 8m)$$

   $$For\ a\ slump = 175\ mm, \quad M(Fig3.5) = 0.78$$

   $$\therefore \sigma_n = 0.78*20.4*7.625 = 121.33 \quad kPa$$

   b) $n = \sigma_n / q_u = 121.33 / 95.76 = 0.13$

   c)

   $$Q_t = \pi\ D\ L\ \theta\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta < n\ ; \quad q_b = \Lambda\ w^{0.67}$$

   $$Q_t = \pi\ D\ L\ k\ f_{su} + \frac{\pi D^2}{4} q_b \quad for \quad \theta > n$$

   d) $\theta / w = 218.586\ m^{-1}$

   e) Let w = 2 mm; $\therefore \theta = 218.586 * 0.002m = 0.437 > n = 0.13$

   $$k = n + \frac{(\theta - n)(1-n)}{(\theta - 2n + 1)} = 0.13 + \frac{(0.437-0.13)(1-0.13)}{(0.437-2(0.13)+1)} = 0.356$$

   $$Q_t = \pi*1*3.05*0.356*(151.4\ kPa) + \frac{\pi*1^2}{4}*146.65*2^{0.67}$$

   $$= 516.48 + 182.83$$

   $$= 699.3\ kPa$$

   f) Calculate sand capacity using trend lines when w = 2mm
      1. S = (s*100/B); @ 2mm S=(0.2cm*100/100cm) = 0.2

2. $q_{st} / Q_s = -2.16*S^4 + 6.34*S^3 - 7.36*S^2 + 4.15*S$
$= -2.16*(0.2)^4 + 6.34*(0.2)^3 - 7.36*(0.2)^2 + 4.15*(0.2)$
$= 0.5829$ for w = 2mm

3. $q_s = 0.5829 * (811.66 \text{ kN})$
$= 473.1$ kN for 2 mm

g) Σ Q = 473.1 + 516.48 + 182.83 = 1172.4 kN

h) Table 29 recommended resistance factor for a total resistance of a drilled shaft in IGM using the Reese and O'Neill FHWA method for all construction methods is $\phi = 0.75$ w/Non-Redundant condition.
Factored Capacity = 0.75 * 1172.4 = **879.3 kN** @ 2 mm

2. DRIVEN PILES

2.1. Axial Pile Capacity Prediction by Static Methods

Axial pile capacities can be predicted by various static capacity prediction methods. These methods use certain properties of the soil, such as the effective friction angle, ϕ', or the CPT cone bearing q_c. However, the computed capacities may not reflect the actual capacities exactly, so designs based on static methods must be conservative, which is reflected in the resistance factor, Φ. This section will discuss specific ways to predict the axial pile capacities.

2.1.1. Axial Loading Capacity of a Pile

The ultimate resistance of a pile, R_{ult} (or R_n--Norminal resistance), is given below:

$$R_{ult} = R_P + R_s \qquad (1)$$

where: pile tip resistance $R_p = q_p A_p$,
pile side resistance $R_s = \sum q_{si} \Delta z_i U$,
q_p = unit tip resistance. Predictions of q_p are in Sections 2.1.2.2, 2.1.2.4 and 2.1.3.
q_s = unit side resistance which is regarded as constant along segment Δz_i of the pile. Predictions of q_s can be found in Sections 2.1.2.1, 2.1.2.3 and 2.1.3,
U = perimeter of the pile's shaft, and
A_p = area of the tip of the pile.

In this project, the eight methods listed below are considered in predicting the axial capacities:

1. α-method, which include α-Tomlinson (Tomlinson, 1980/1995) and α-API (Reese et al., 1998) for cohesive soil

2. λ-method (Vijayvergiya and Focht, cited in US Army Corps of Engineers, No.7, 1992) for cohesive soil,

3. β-method (Bushan--cited in Bowles, 1996) for cohesionless soil,

4. β-method (Burland, Esrig and Kirby, 1979) for cohesive soil,

5. Nordlund method (Nordlund, 1963) for cohesionless soil,

   (Methods number 4 to 5 above are cited in Hannigan et al., 1995).

6. Nottingham and Schmertmann CPT method (McVay and Townsend, 1989) for all soil types,

7. Meyerhof SPT (Meyerhof, 1976/1981) for cohesionless soil,

8. Schmertmann SPT method (Lai and Graham, 1995) for all soil types and rock.

The first five methods, as discussed in Section 2.1.2, are semi-empirical methods. These methods are based on the empirical relationship between soil properties and the stress states

(both effective and total stress analyses are used). The last three methods, as discussed in Section 2.1.3, predict the pile capacities based on the data from in-situ tests (SPT and CPT).

2.1.2. Semi-Empirical Methods

Semi-empirical methods are used to relate the adhesion between the pile and the surrounding soil to the internal friction angle, ϕ, or the undrained shear strength, S_u. This section presents five different semi-empirical methods, will be divided into 2 sub-sections: Section 2.1.2.1 for cohesive soil and 2.1.2.2 for cohesionless soil.

2.1.2.1. Side Resistance In Cohesive Soil

In cohesive soil, the side resistance of a pile is usually predicted using the undrained shear strength, S_u, or the over-consolidation ratio, OCR. This section reviews different methods predicting the side resistance in cohesive soil.

2.1.2.1.1. α-Tomlinson method

The α-Tomlinson method (Tomlinson, 1980/1995), based on total stress analysis, is used to relate the adhesion between the pile and a clay to the undrained shear strength of the clay, S_u. The ultimate unit side resistance may be taken as:

$$\mathbf{q_{si} = \alpha S_{ui}} \qquad \mathbf{2}$$

where: α = adhesion factor (Figure 1), which depends on the bearing embedment in stiff clay and the width of the pile,
S_u (or C_u) = average undrained shear strength of the soil in the segment of interest.

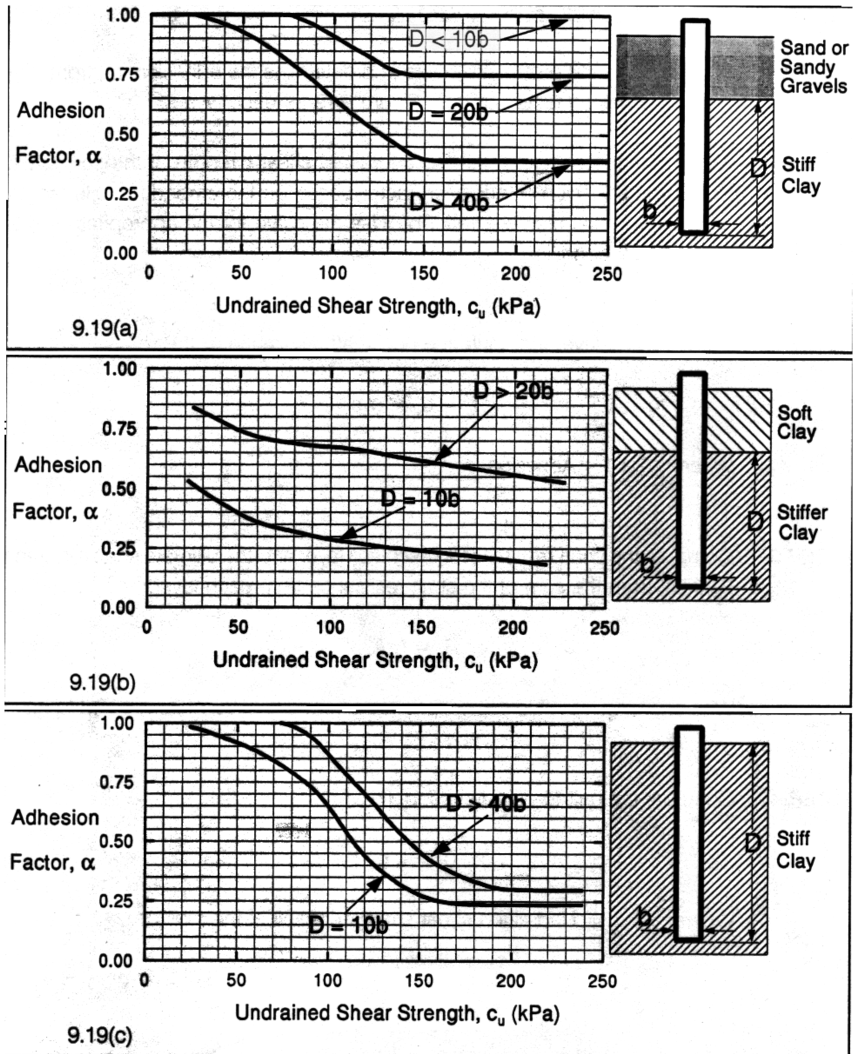


Figure 3: α factors; Tomlinson method (Tomlinson, 1995)

Following is the discussion on the α-Tomlinson method:

The α method is simple to use and it has been used over many years. However, it is a total stress analysis and does not depend on the ground water level; therefore, the resistance based on the α method may not be close to the measured capacity, which depends on the in-situ stress state and the ground water level.

Tomlinson originally developed the α-method for large displacement piles. Therefore, for small-displacement piles (H and pipe piles), the α method may not be suitable. Similarly, the Tomlinson method is only suitable for pile embedded in stiff clay. For cohesive layers that lie above the bearing layer, other methods should be used.

The assumptions behind the α Tomlinson method include the following:

- If there is a soft clay layer above the bearing stiff clay, then the soft clay will be dragged down to the stiff layer and will lower the α factor (see Figure 1),

- Similarly, the cohesionless soil in Figure1 will be dragged down and therefore, the α factor will be increased,

- Other intermediate cases, such as the layer right above the stiff layer is silt, or is only a very thin lens, etc. will decrease the accuracy of the prediction.

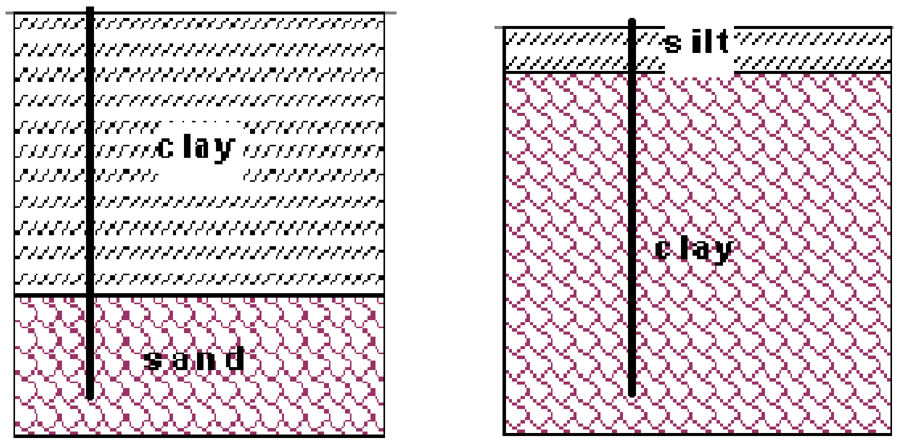


Figure 4: Examples when α-Tomlinson is not applicable.

2.1.2.1.2. α-API revised method (1987)

The α-API method (cited in Reese et al., 1998) is similar to the α-Tomlinson method, and the ultimate unit side resistance, in the same unit as S_u, is taken as:

$$q_{si} = \alpha S_{ui} \qquad 2$$

where: $\alpha = 0.5\ \psi^{-0.5}$ if $\psi \leq 1.0$,

$\alpha = 0.5\ \psi^{-0.25}$ if $\psi > 1.0$, and max $\alpha = 1.0$,

$\psi = S_u / \sigma_v'$, and

σ_v' (or p_v') = the vertical effective overburden pressure at the depth of interest.

From Eq. 2, the adhesion factor, α, with different effective stresses is generated in Figure 3.

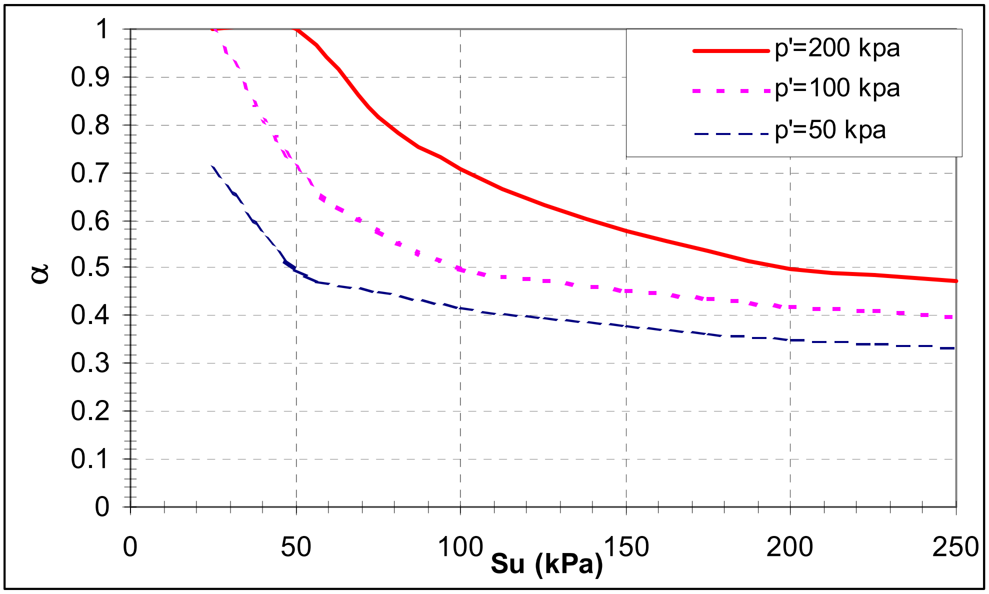


Figure 5: α factors; Revised API 1987 method (generated from Eq. 2)

The α-API method is a mixed method between total stress analysis (S_u) and effective stress analysis (σ_v'). It is much easier to use than the Tomlinson method. For example, the user will have no issue with considering other layers that lie above the bearing layer. Finally, the α-API method has simple equations; thus it is easily automated.

Following is the discussion on the unit side resistance of both α methods:

As shown in Figure 4, the unit side resistance $q_s = \alpha\ S_u$ for the α-API method follows a logical trend: the stiffer the soil (higher S_u), the more the side resistance.

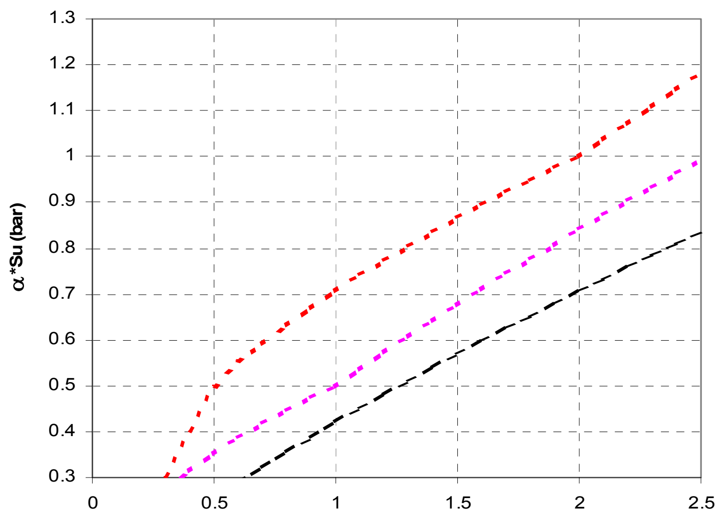


Figure 6: The unit side resistance of α-API method (generated from Eq. 2)

However, as indicated in Figure 5, for the α-Tomlinson 1980 method, when S_u is in the range of about 0.8 to 1.7 bar (80 to 170 kPa), the unit side resistance decreases as the soil

becomes stronger, which does not follow traditional wisdom on pile behavior. (In that range, there is very sharp decrease of the α factor).

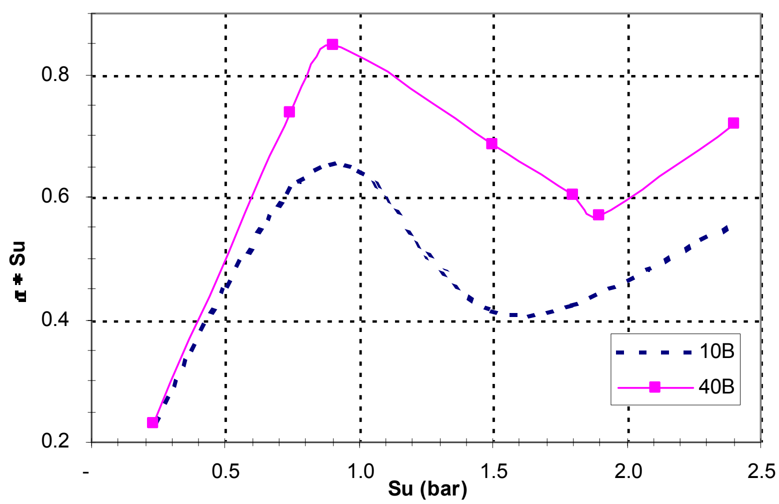


Figure 7: The unit side resistance of α-Tomlinson 1980 case #3 (generated from Figure 1)

2.1.2.1.3. β-Burland method (1973)

From the theory of soil mechanics, based on effective stress analysis, we have $q_s = K (\tan \delta) \sigma_v'$,

where: K = horizontal stress ratio,

δ = adhesion angle between soil and piles,

σ_v' = vertical effective stress.

The equation can be rewritten as:

$$q_s = \beta\sigma_v' \qquad 3$$

where: σ_v' = vertical effective stress,

β = factor depended on the over-consolidation ratio OCR (Figure 6).

Following is the discussion on the β-Burland method:

Esrig and Kirby (1979) (cited in Hannigan et al., 1995) suggested that for heavily over-consolidated clays, the value of β should not exceed two.

Practically, the OCR ratio is not usually measured in the laboratory. Therefore, this method is difficult to implement. In this project, all OCR ratios are obtained from in-situ tests through correlations (Section 2.2).

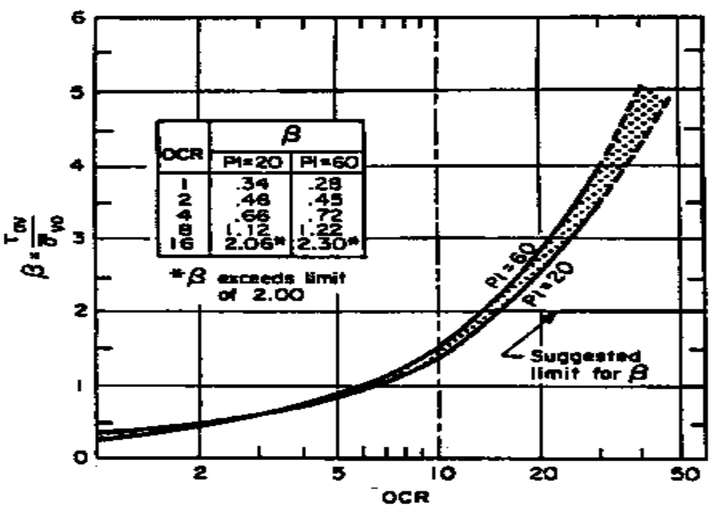


Figure 8: β factors (AASHTO 1996/2000)

2.1.2.1.4. λ-Method

The λ-method (cited in US Army Corps of Engineers, 1992), based on effective and total stress analysis, may be used to relate the unit side resistance to the passive earth pressure as:

$$q_s = \lambda(\sigma' + 2S_u) \qquad 4$$

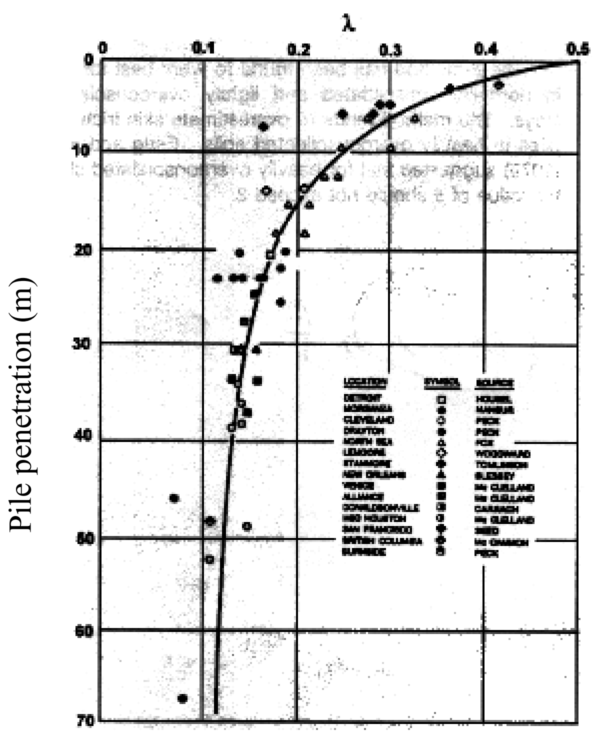


Figure 9: λ factors (US Army Corps of Engineers, 1992)

where: λ = an empirical coefficient, which depends on the pile embedment as shown in Figure 7.

The λ factor was empirically suggested by examining pipe piles in only 15 locations. The main drawback of this method is that it assumes a single value of λ for the whole pile.

2.1.2.2. Tip resistance in cohesive soil

The ultimate unit tip resistance of piles in saturated clay (Reese at al., 1998) may be taken as:

$$q_p = 9\ S_u \qquad 5$$

where: S_u = average undrained shear strength in the range from 2B to 3.5B below the tip, and B is the diameter of the pile.

With unsaturated clay, Eq. 5 is still used to predict tip resistance, which may reduce the accuracy of the answer.

2.1.2.3. Side resistance in cohesionless soil

In cohesionless soil, the side resistance of a pile is usually predicted using the adhesion angle, δ, or the relative density, D_r. The adhesion angle, δ, is related to the internal friction angle of the soil, φ, through the volume displacement, the material, the shape of the pile and the roughness of the pile. This section reviews different methods predicting the side resistance in cohesionless soil.

2.1.2.3.1. β-Bushan method (1982)

Similar to the β method in cohesive soil, the unit side resistance is related to effective stress as (cited in Hannigan et al., 1995):

$$q_s = \beta\sigma_v' \qquad 3$$

where: $\beta = 0.18 + 0.65\ D_r$, and

D_r = Relative density in decimals. Therefore, max $\beta = 0.83$ when $D_r = 1$ (much lower than max β in cohesive soil--Section 2.1.2.1.3.)

2.1.2.3.2. Nordlund method

Nordlund (cited in Hannigan et al., 1995) developed the following equation for the unit resistance:

$$q_s = K_\delta C_F \sigma_v' \frac{\sin(\delta + \varpi)}{\cos\varpi} \qquad 6$$

where: K_δ = Coefficient of lateral earth pressure at the depth of interest. (Figures 8 to 11),

δ = friction angle between pile and soil. For non-taper piles: $\delta \le \phi$. (Figure 12),

C_F = Correction factor for K_δ when $\delta \ne \phi$. $C_F \approx 0.6$ to 1.0. (Figure 13),

σ_v' = effective over-burden pressure at the center of the layer of interest, and

ω = angle of the pile taper from vertical.

For a uniform cross section pile (ω = 0), the Nordlund equation becomes

$$q_s = K_\delta C_F \sigma_v' \sin\delta \qquad 7$$

The equation of this semi-empirical method is somewhat similar to the theory of soil mechanics, in which

$$q_s = K_\delta\, \sigma' \tan\delta \qquad 8$$

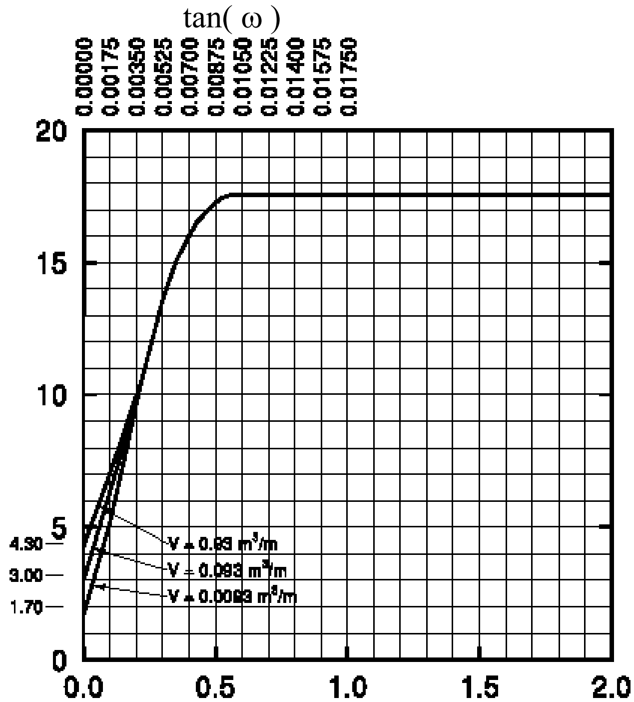


Figure 10: Design curve for evaluating K_δ when ϕ = 40

(Norlund 1963--cited in Hannigan et al., 1995)

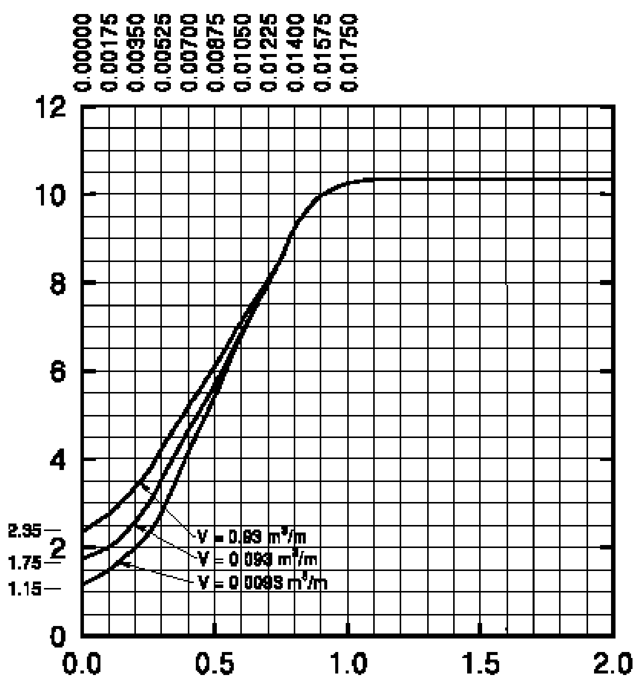
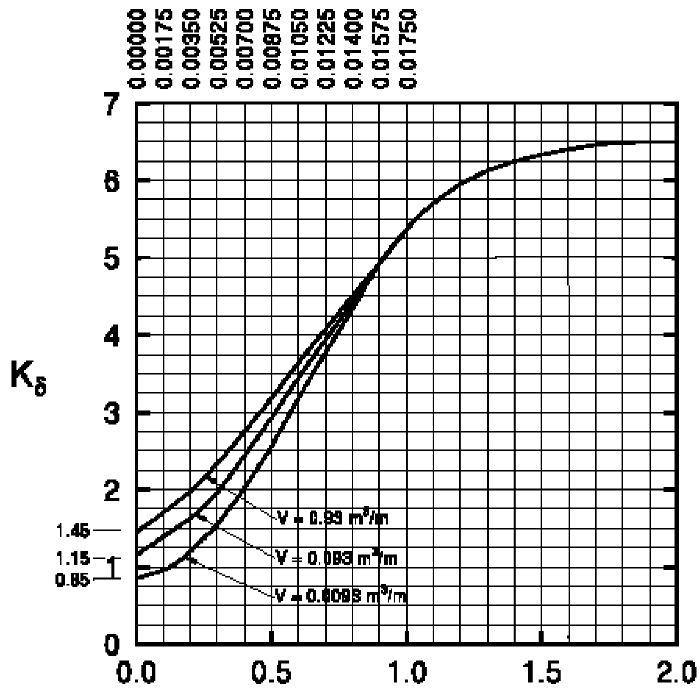


Figure 11: Design curve for evaluating K_δ when ϕ = 35

(Norlund 1963--cited in Hannigan et al., 1995)

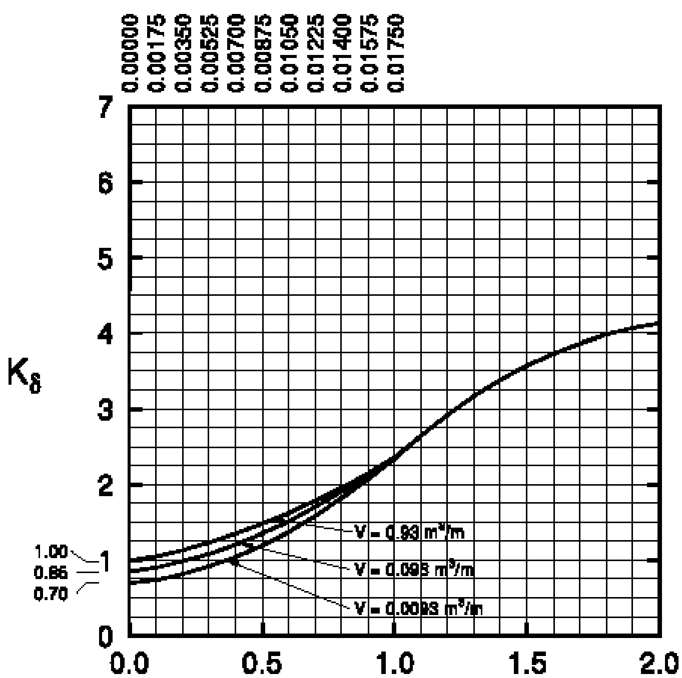
tan(ω)



ω (degrees)

Figure 12: Design curve for evaluating K_δ when ϕ = 30
(Norlund 1963--cited in Hannigan et al., 1995)

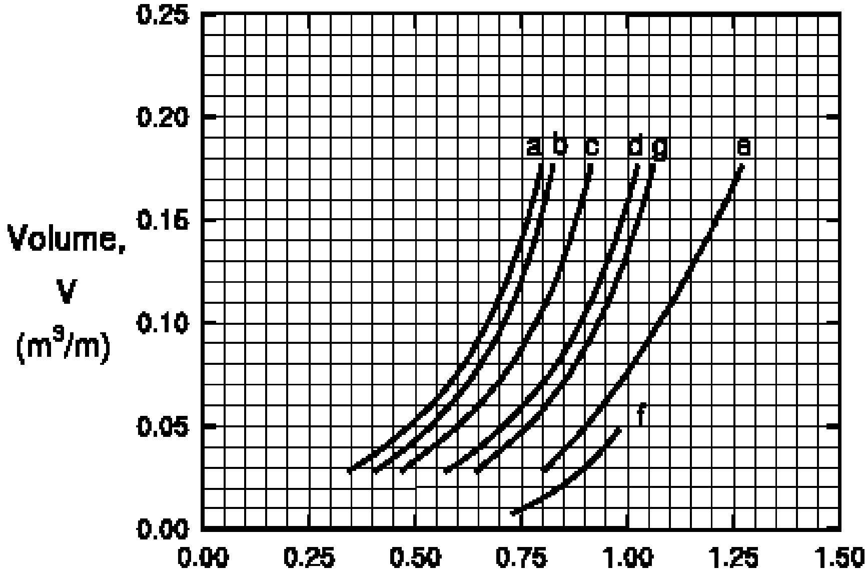
tan(ω)



ω (degrees)

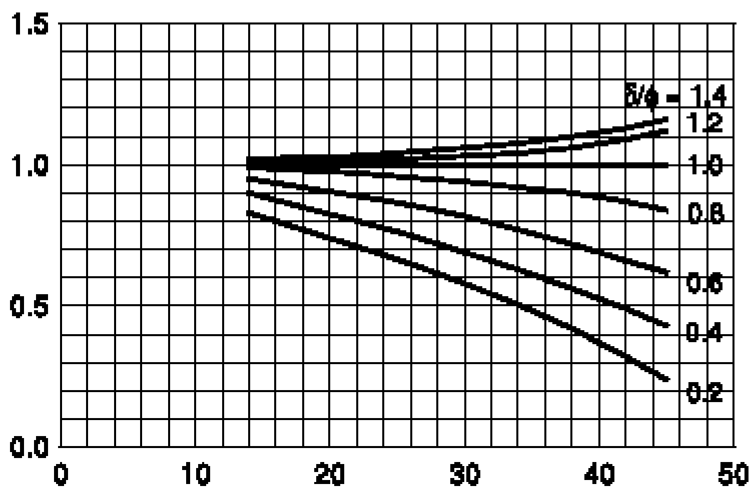
Figure 13: Design curve for evaluating K_δ when ϕ = 25
(Norlund 1963--cited in Hannigan et al., 1995)

δ / φ



a. Pipe piles
b. Timber piles
c. Concrete piles
d. Raymond step taper piles
e. Raymond uniform taper piles
f. H piles
g. Tapered portion of monotube piles

Figure 14: Relation δ/ϕ and pile displacement
(Norlund 1963--cited in Hannigan et al., 1995)



φ (degrees)

Figure 15:Correction factor (C_F) for K_δ

(Norlund 1963--cited in Hannigan et al., 1995)

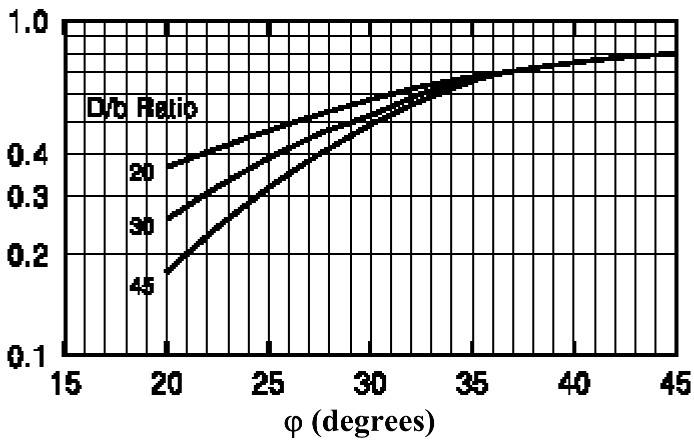
2.1.2.4. Tip resistance in cohesionless soil--Thurman method

From bearing capacity theory, Thurman (cited in Hannigan et al., 1995) related the unit tip resistance in sand with effective stress as:

$$q_p = \alpha_t N'_q \sigma_v' \qquad 9$$

where: α_t = dimensionless factor (Figure 14),
N'_q = Bearing capacity factor (Figure 15),
σ_v' = effective overburden pressure at the pile tip. σ_v' is limited to 150 kPa (tip resistance reaches a limiting value at some distance below the ground),
q_p also has a limit as shown in Figure 16.

N'_q is very high at high internal friction angles (N'_q>250 when ϕ>42^o). Therefore, some software, e.g. DRIVEN (FHWA, 1998) recommends the limit of only 36^o for ϕ.



D: Embedded pile length
b: Pile diameter or width

Figure 16: α_T coefficient (FHWA--DRIVEN, 1998)

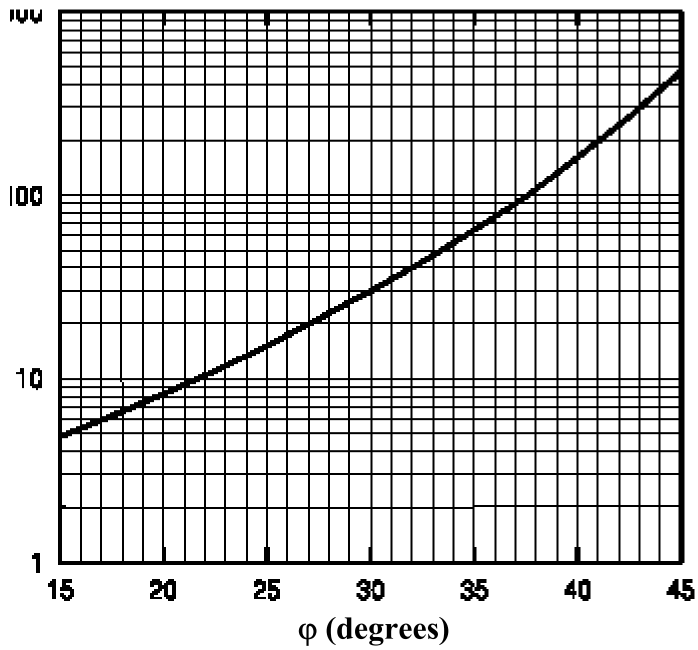


Figure 17: Bearing capacity factor Nq'(FHWA--DRIVEN, 1998)

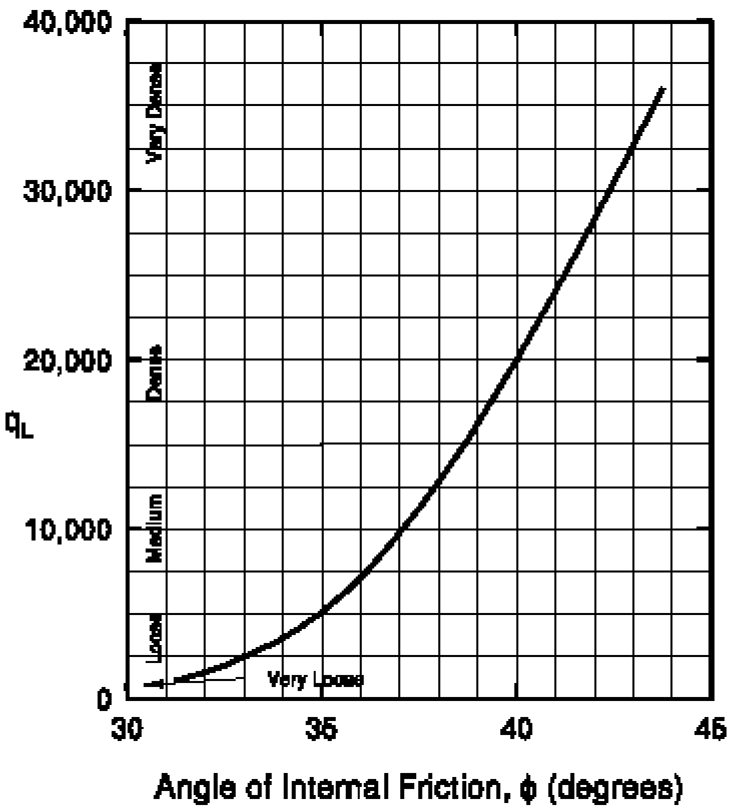


Figure 18: Relationship Between Maximum Unit Pile Toe Resistance q_L (kPa) and Friction Angle for Cohesionless Soils (Meyerhof, 1976/1981).

2.1.3. Empirical Methods

This section presents different empirical methods, which directly relate the resistances with the in-situ test results, i.e. the N-value and the CPT readings.

2.1.3.1. Meyerhof method for piles in cohesionless soil

There are more than four different variations of Meyerhof 1976 method. Different agencies have different guidelines, e.g. EM 1110-1-1905 (US Army Corps of Engineers, 1992), leading to different results, even though the method is still referred to as Meyerhof 1976 method. The primary variations of the side resistance of the method are listed below:

1) AASHTO provision (AASHTO 1996/2000): $q_s = k' N$ (kPa).
2) Meyerhof original paper (Meyerhof, 1976/1981): $q_s = k\ N$ (kPa).
3) Bowles (Bowles, 1996): $q_s = k\ N'$ (kPa).
4) U.S. CORPS of Engineers: No guidelines.

where: N = Uncorrected blow count,
N' = Corrected blow count,
k = 2 for displacement piles, and k = 1 for small displacement piles.

AASHTO (AASHTO 1996/2000) uses strict unit conversion, therefore k' = 1.9 for displacement piles and 0.96 for small displacement piles.

The side resistance should be limited as given below:

$$q_s \leq 100 \text{ kPa}$$

The tip resistance for Meyerhof method is similarly obtained as:

1) AASHTO provision (AASHTO 1996/2000): $q_p = 0.38\ N_t'\ D/B$ (bar).
where: N_t' = Corrected blow count near the pile tip,
D = The embedment of the pile in cohesionless soil, and
B = The diameter or width of the pile cross-section.
2) Original paper by Meyerhof (Meyerhof, 1976/1981): $q_p = 0.4\ N_t\ D/B$ (bar).
where: N_t = Uncorrected blow count near the pile tip.

It should be noted that, in 1976, the concept of correcting the N-value due to the overburden pressure was very new. Therefore, Meyerhof did not correct N. Similarly, in Meyerhof 's paper, he used approximate conversion: 1 tsf ≈ 100 kPa (1 bar).

3) Bowles (Bowles, 1996): $q_p = 0.4\ N_{8+3B}'\ D/B$ (bar).
where: N_{8+3B}' = Corrected blow count in the depth of 8B above tip and 3B below tip.
4) EM 1110-1-1905 (U.S. Army CORPS of Engineers, 1992)
$q_p = 0.38\ N_{8+3B}\ D/B$ (bar) $= 0.8\ N_{8+3B}\ D/B$ (ksf).
where: N_{8+3B} = Uncorrected N in the depth of 8B above tip and 3B below tip.

Similar to the side resistance calculations, the value of the maximum tip resistance is also limited as shown below:

1) AASHTO provision (AASHTO 1996/2000)

$q_L = 4\ N_t'$ in sand,

$= 3\ N_t'$ in silt.

2) Meyerhof original paper (Meyerhof, 1976/1981)

$q_L = 4\ N_t$ in sand,

$= 3\ N_t$ in silt.

3) Bowles (Bowles, 1996)

$q_L \quad = 3.8\ N_{8+3B}'$.

4) EM 1110-1-1905 (U.S. Army CORPS of Engineers, 1992)

$q_L \quad = 3.8\ N_{8+3B}$ (bar) $= 8\ N_{8+3B}$ (ksf).

2.1.3.2. Schmertmann method for SPT

The procedure of the Schmertmann method for SPT (Lai and Graham, 1995) is described below. First of all, the SPT blow count N is adjusted as shown below:

If $N < 5$ then $N = 0$ (ignores side resistance in weak soil), and

If $N \geq 60$ then $N = 60$ (limit on side resistance)

Discussion:

It is conservative to ignore side resistance when N is in the range of 4 to 5. For example, in Luling bridge, ID 1001, the soil is mostly clay with N < 5. The truncation of N < 5 significantly lowers the predicted capacity compared to the measured capacity.

The ultimate side resistance for different types of piles and soil types is presented in Table 1.

Table 1: Side resistance --Schmertmann method for SPT

Ty-Pe	Description	Ultimate unit side resistance q_s(tsf)		
		Concrete	Steel H piles	pipe piles
1	Plastic clay	2.0N(110-N) /4006.6	2N(110-N)/5335.94	0.949+0.238ln N
2	Clay-silt-sand mixtures Very silty sand, silts	2.0N(110-N) /4583.3	$-0.0227+0.033N-4.57610^{-4}*N^2 +2.465E-6*N^3$	0.243+0.147ln N
3	Clean sands	0.019N	0.0116N	0.058+0.152ln N
4	Soft limestone, very shelly sand	0.01N	0.0076N	0.018+0.134ln N

At any point A, the unit tip resistance is

$$q_{p@A} = \frac{\text{weighted average of } q_p \text{ 8B above A} + \text{weighted average of } q_p \text{ 3.5B below A}}{2}$$

The weighted average of q_p is based on values calculated from Table 2.

Table 2: Tip resistance –Schmertmann method for SPT

Type	Description	Ultimate unit end bearing q_p (tsf)	
		Concrete and H piles	Pipe piles
1	Plastic clay	0.7 N	0.48 N
2	Clay-silt-sand mixtures Very silty sand, silts	1.6 N	0.96 N
3	Clean sands	3.2 N	1.312 N
4	Soft limestone, very shelly sand	3.6 N	1.92 N

For concrete and H piles, the mobilized tip resistance is expected to be one third (1/3) of the ultimate tip resistance. For pipe piles, the mobilized tip resistance is expected to be one half (1/2) of the ultimate tip resistance.

The ultimate resistance is only fully mobilized when the bearing embedment is sufficient, i.e. $D_A = D_C$

where: D_A = Actual bearing embedment, and D_C = Critical bearing embedment, which is shown in Table 3.

Table 3: Critical depth ratio—Schmertmann method for SPT

Soil Type	Description	Critical depth ratio (D_C/B)
1	Plastic clay	2
2	Clay-silt-sand mixtures Very silty sand, silts	4
3	Clean sands N = 12 or less N = 30 or less N greater than 30	6 9 12
4	Soft limestone, very shelly sand	6

If $D_A < D_C$ and the bearing layer is stronger than the overlying layer, then:

$$q_p = q_{LC} + \frac{D_A}{D_C}(q_T - q_{LC}) \tag{10}$$

where: q_p = Reduced tip resistance,

q_{LC} = Unit tip resistance at layer change, and

q_T = Uncorrected unit tip resistance at pile tip.

$$CSFBL = \frac{SFBL}{q_T}\left[q_{LC} + \frac{D_A}{2D_C}(q_T - q_{LC})\right] \tag{11}$$

where: CSFBL = Reduced side resistance in the bearing layer, and

SFBL = Uncorrected side resistance in the bearing layer.

If $D_A > D_C$ and the bearing layer is stronger than the overlying layer, then:

$$CSFACD = \frac{USFACD}{q_{CD}}\left[q_{LC} + 0.5(q_{CD} - q_{LC})\right] \quad 12$$

where:

CSFACD = Corrected side resistance between the top of the bearing layer and the critical depth,
USFACD = uncorrected side resistance in the bearing layer from the top of the bearing layer to the critical depth, and
q_{CD} = unit tip resistance at critical depth.

2.1.3.3. Nottingham and Schmertmann method for CPT

The procedure of the Nottingham and Schmertmann method for CPT (McVay and Townsend, 1989) is described below.

The ultimate side resistance of piles may be taken as:

From ground level down to the depth of 8B $\quad q_s = K\ f_s /2$ 13

From 8B down to the tip $\quad q_s = K\ f_s$ 14

where: f_s = sleeve friction from CPT data,

In cohesionless soil, the ratio K (or Ks) is a function of the ratio between penetration depth(Z) and pile width (D)--see Figure 17.

In cohesive soils: K is written as α (or Kc), which is the α factor, similar to the α-method--see Figure 17.

Similarly, the ultimate tip resistance of piles may be taken as:

$$q_p = \frac{q_{c1} + q_{c2}}{2} \quad 15$$

where: q_c = The tip resistance in CPT data (if q_c>100 tsf, limit q_c to 100 tsf),

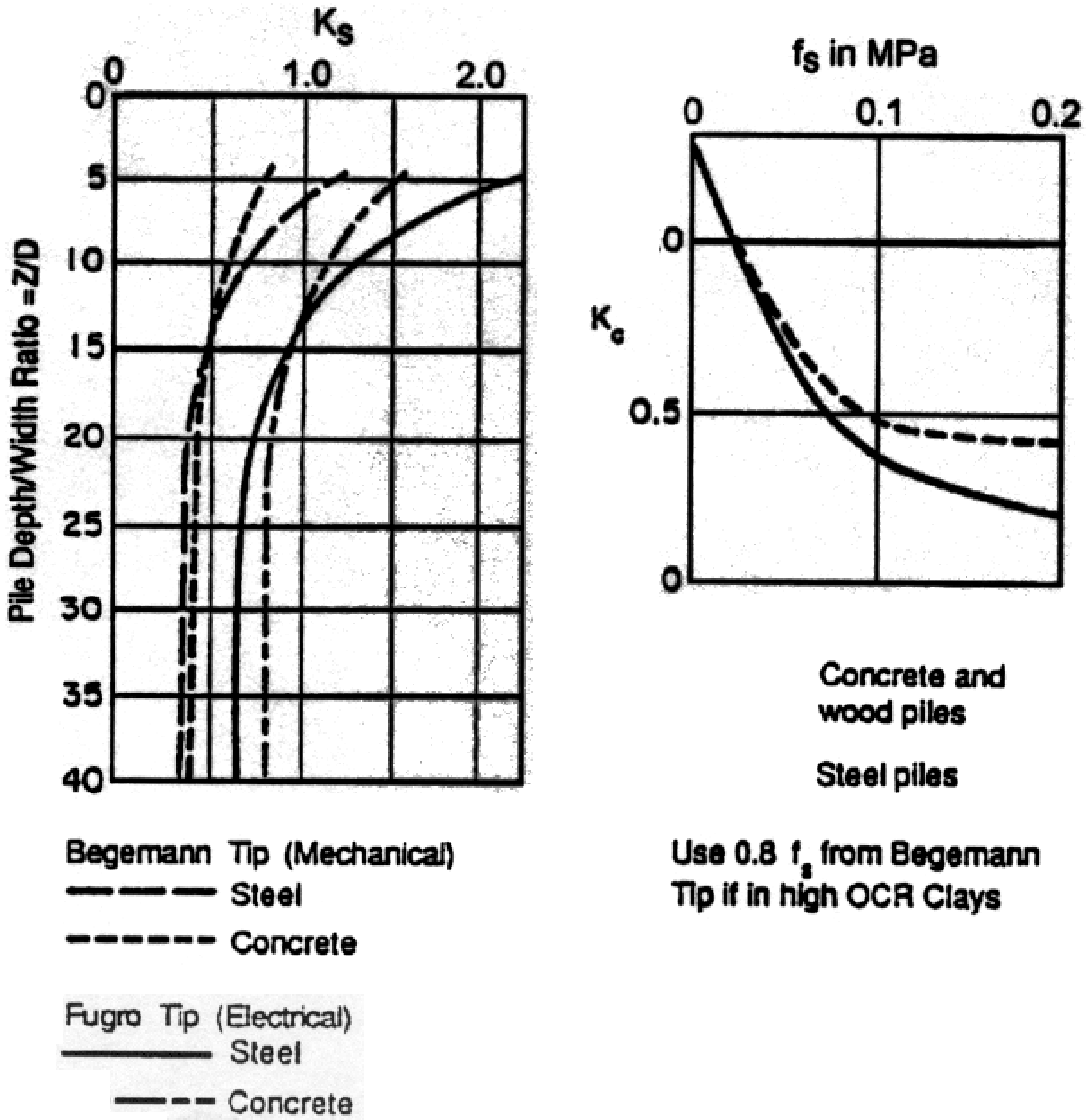
q_{c1} = The average q_c over a distance of xB below the tip, which is shown below:

Sum of the minimum values (upward)/ xB + Sum of the actual values (downward)/ xB

Use the minimum q_{c1} with different xB ranging from 0.7B to 3.75B,

q_{c2} = The average q_c over a distance of 8B above the tip using the minimum q_c values (minimum path). Figure 18 depicts the procedure for tip resistance prediction.

For mechanical cone in cohesive soils, due to the effects of the base of the friction sleeve on q_c, the q_p value should be reduced by about 40%.



Side friction

Figure 19: Ks and Kc ratio in cohesionless and cohesive soil, respectively

(cited in McVay and Townsend, 1989)

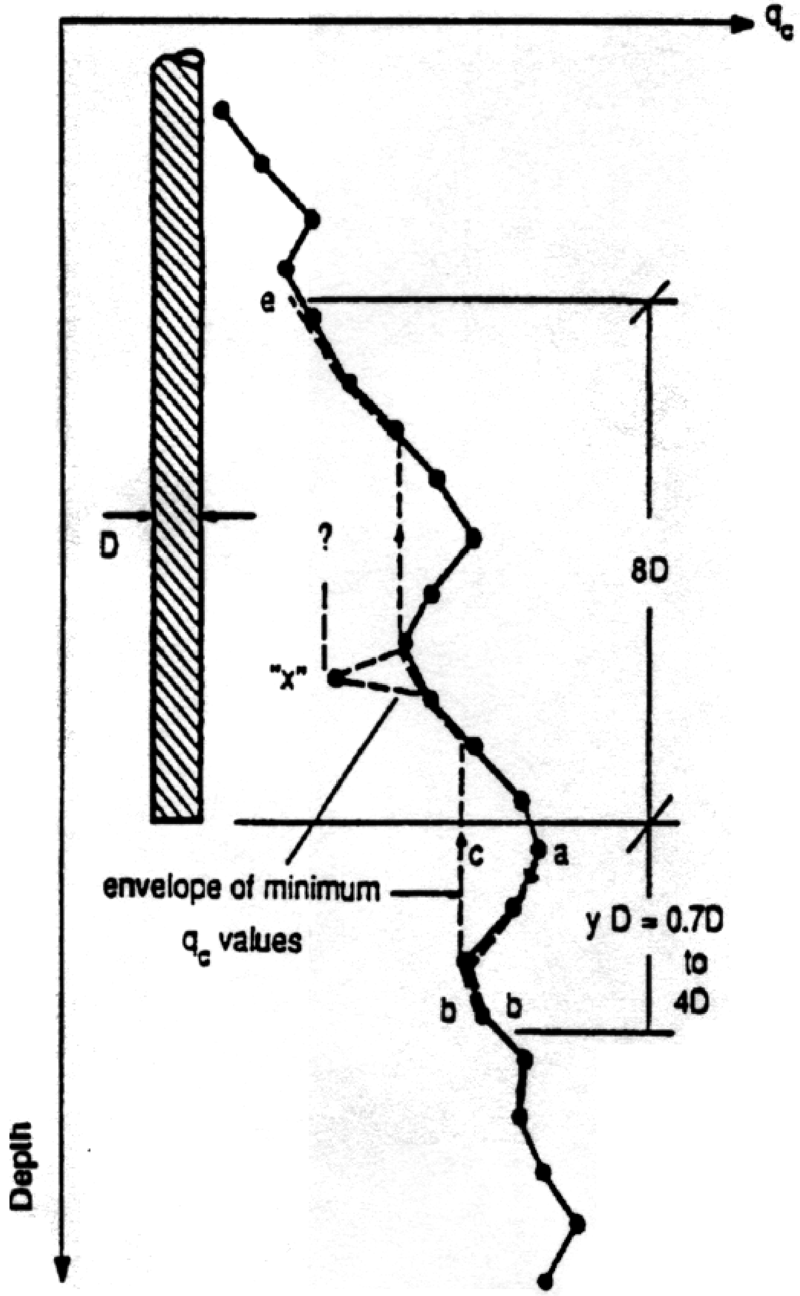


Figure 20: Tip resistance computation procedure--Nottingham 1975.

(cited in McVay and Townsend, 1989)

A summary of all the methods is presented in Table 4 below.

Table 4: Summary of the methods

Methods	Side resistance	Tip resistance	Parameters required	Constraints
α-Tomlinson (Tomlinson, 1980/1995)	$q_s = \alpha S_u$	$q_p = 9\ S_u$	S_u; D_b (bearing embedment)	+Bearing layer must be stiff cohesive + Number of soil layers ≤ 2
α-API (Reese et al., 1998)			S_u	
β in cohesive (AASHTO, 1996/2000)	$q_s = \beta\sigma'$		OCR	
λ (US Army Corps of Engineers, 1992)	$q_s = \lambda(\sigma' + 2S_u)$		S_u	Only for cohesive soils
β in cohesionless (Bowles, 1996)	$\beta\sigma'$		D_r	
Nordlund and Thurman (Hannigan et al., 1995)	$q_s = K_\delta C_F \sigma' \dfrac{\sin(\delta + \varpi)}{\cos\varpi}$	$q_p = \alpha_t N'_q \sigma'$	φ	
Meyerhof SPT (Meyerhof, 1976/1981)	$q_s = k\ N$	$q_p = 0.4D/BN'$	N	+ For cohesionless soils + SPT data
Schmertmann SPT (Lai and Graham, 1995)	q_s = function(N)	q_p = fn(N)	N	SPTdata
Schmertmann CPT (McVay and Townsend, 1989)	q_s = function(f_s)	q_p = fn(q_c)	q_c, f_s	CPT data

2.1.4. Evaluation of Axial Capacity Prediction Software

Four common programs are used in predicting the pile axial capacity. This section briefly evaluates the programs, such as the methods they use, and the results they get.

2.1.4.1. APILE 3.0 (EnSoft)

APILE 3.0 is a commercial Windows-based software developed by EnSoft, Inc (Reese et al., 1998).

For cohesive soil, the following static methods are used: The α-Tomlinson method, the α-API method and the λ method. For cohesionless soil, the following static methods are used: The Nordlund method and the API method. The combination of the α-Tomlinson and Nordlund methods in mixed soils is called the FHWA procedure.

The following problems were detected in using this program:

- For piles with non-circular sections, when the unit is changed from SI to English, the predicted axial capacity as well as the shapes of the capacity graph change,
- Outputs of EnSoft's examples 2 and 4 are different from the User Manual's printout,
- In FHWA procedure, the program does not correctly interpret the data when there are more than two layers: The bearing embedment depth will be calculated by the program as the distance from the tip to the nearest interface, which may be less than the actual bearing embedment,
- There is no box for inputting ground-water level, the user has to use buoyant unit weight whenever applicable.

2.1.4.2. DRIVEN 1.1 (FHWA)

DRIVEN is a freeware Windows-based software provided by the Federal Highway Administration (FHWA, 1998). Driven is the Windows version of the old SPILE MSDOS-based program. The α-Tomlinson method is used in cohesive soils, while the Nordlund and Thurman methods are used in cohesionless soils.

For concrete pile, DRIVEN does not support the circular section.

The α-Tomlinson method is only suitable for pile embedded in stiff clay. However, DRIVEN allows the user to specify the α-Tomlinson method in cohesive layer(s) that lies above the bearing layer.

2.1.4.3. PL-AID (University of Florida)

PL-AID is a MSDOS-based program developed at the University of Florida predicting capacities of piles using CPT data, i.e. Nottingham and Schmertmann method (McVay and Townsend, 1989).

PL-AID works correctly under the following conditions:

- CPT data (raw truck format or interpreted format) are in SI units,
- The increments in data input are equal (e.g. 5 or 10 cm).

PL-AID was not originally developed for H piles and unplugged open-ended pipe piles.

2.1.4.4. SPT-97 (FDOT and University of Florida)

SPT-97 is a Windows-based program predicting capacities using SPT data, i.e. Schmertmann method (Lai and Graham, 1995). SPT97 works normally well, except two following cases:

- When $D_c < D_a$ (the critical depth is smaller than the actual depth), there is a problem in correcting the resistance.

- The capacity is erroneously computed for pipe piles.

A summary of the issues related to the above software can be found in Table 5.

Table 5. Issues with using Driven Pile Axial Capacities Analysis Software

APILE 3.0 Plus	DRIVEN 1.1	SPT 97	PL-AID
All results using SI unit are erroneous (this problem was corrected on September 18, 00)	The computed results for H piles, SI units are erroneous.	When $D_c < D_a$ (critical depth is smaller than actual depth), there is a problem in correcting the resistance.	Increments must be equal. This is always true if the data are originally collected from the tests. However, some of the data are actually digitized from graphs. In this case, interpolation must be made to get equal increments.
Error in critical depth (this problem was corrected on November 27, 00)	Concrete piles: There is no option to input circular pile.		
Wrong handling of volume displacement for H and pipe piles (this problem was corrected on January 04, 01)			
The λ method is only suitable for pile in clay. In sand, APILE still has λ capacities for piles by converting ϕ to S_u (but in the APILE manual, it said that it took capacities from API)	The Nordlund method: Open ended pipe piles: Curve f (for H piles) is used for open ended pipe piles, which leads to higher capacities of plugged open ended pipe than those of close ended pipe with the same dimensions		
The α-Tomlinson (1980) method: The layer system must be converted to 1 or 2 layer(s). This causes an approximation and lowers the reliability, especially when the soil system is complex.	The α-Tomlinson 1980 method: In order to get correct results, the layer system must be converted to 1 or 2 layer(s). This causes an approximation and lowers the reliability, especially when the soil system is complex.	For pipe piles, the capacity from SPT97 is erroneously computed	
Error in user specified q_L (this problem was corrected on January 24, 01)			
Different capacities with different increments. (This problem was unsuccessfully corrected on February 14, 01)			

2.2. Interpretation of the Pile-Load Tests

Static load tests to failure determine the load capacities directly, therefore they are more precise than the prediction methods presented in Section 2.1. However, from load-settlement curves, a determination of the ultimate capacity or of the mobilized capacity is required. Many different methods of interpretations have been proposed and some of most common methods are presented in this section.

2.2.1. Debeer Method

The DeBeer capacity (Bowles, 1996) is determined as follow.
The load test data is plotted in a log-log scale.
The intersection between the two straight portions of the graph will correspond to the DeBeer capacity.

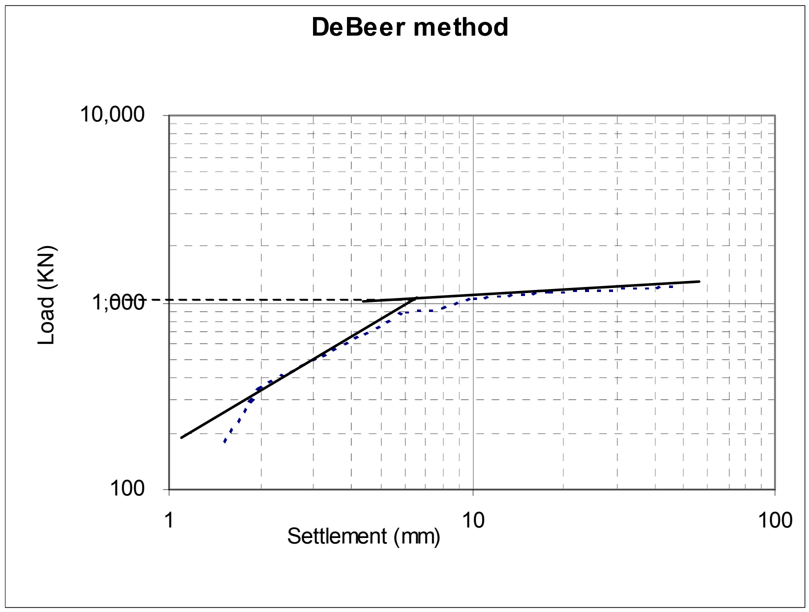


Figure 21: Example of the Debeer method

One of the most common problems with this method is that, in some cases, the two straight portions in the graph are not clearly defined.

2.2.2. Davisson Method

The Davisson method (Coduto, 2001) is one of the most popular methods and it is based on the elastic compression of the pile. The procedure to define the the Davisson capacity is as described below:

1. In the load test graph, plot the base line with the slope of: AE/L
   where: A = Cross sectional of the material,

   E = Modulus of elasticity of the material, and

   L = Embedment length of the pile

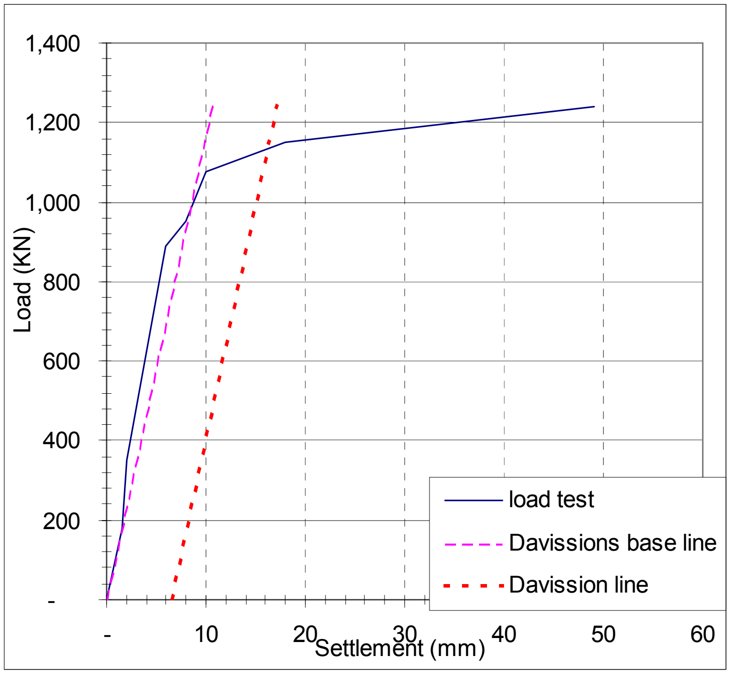


Figure 22: Example of the Davisson method

2. Plot the Davisson line parallel to the base line. The distance between the two lines is $0.15+\frac{B}{120}$ (in)
3. The intersection of that line and the load test graph represents the Davisson capacity.

Davisson method is most suitable for quick load tests on friction piles. For end bearing piles, as the load test curve does not flatten, the Davisson method is usually not applicable.

2.2.3. Other Methods

The pile capacity can be at a point where the settlement of the pile is 5% of its diameter. (or 2%, 3%, or even 10% depending on the pile shape and load test procedure). These methods are very simple. However, they do not account for the length and the material of the pile.

2.3. Soil Properties Correlated from Insitu Tests

About 95% of the soil data from the database are from SPT and CPT tests. Moreover, the α, β, λ and the Nordlund method all use the angle friction ϕ and/or the undrained shear strength S_u or the over-consolidation ratio OCR. Therefore, correlations are required to obtain these properties from the SPT and CPT tests.

There are usually two or more correlations for the same property. Therefore, in the following, the capacities are predicted based on the various different correlations available. The correlations used in this study to get engineering soil properties from SPT are presented in Table 6. Similarly, Table 7 shows the correlations to get engineering soil properties from CPT.

Table 6: Correlations of soil properties from SPT

Properties	From SPT	Figure	Reference	
ϕ	Peck, Hanson and Thornburn: $\approx 54 - 27.6034 \exp(-0.014N')$	2`	Figure 4.12	Kulhawy and Mayne, 1990
	Schmertmannφ' $\approx \tan^{-1} [N / (12.2 + 20.3 \sigma')]^{0.34}$	22	Figure 4.13 and Equation 4.11	
S_u (bar)	Terzaghi and Peck (1967): 0.06 N		Equation 4.59	
	Hara 1974: $0.29 N^{0.72}$		Equation 4.60	
OCR for clay	Mayne and Kemper $\approx 0.5 N / \sigma'_o$ (σ'_o in bar)	23	Figures 3.9 and 3.18	
Dr	Gibbs and Holtz's Figures	24	Figures 2.13 and 2.14	

where: N is the uncorrected blow counts, and
N' is the corrected blow counts

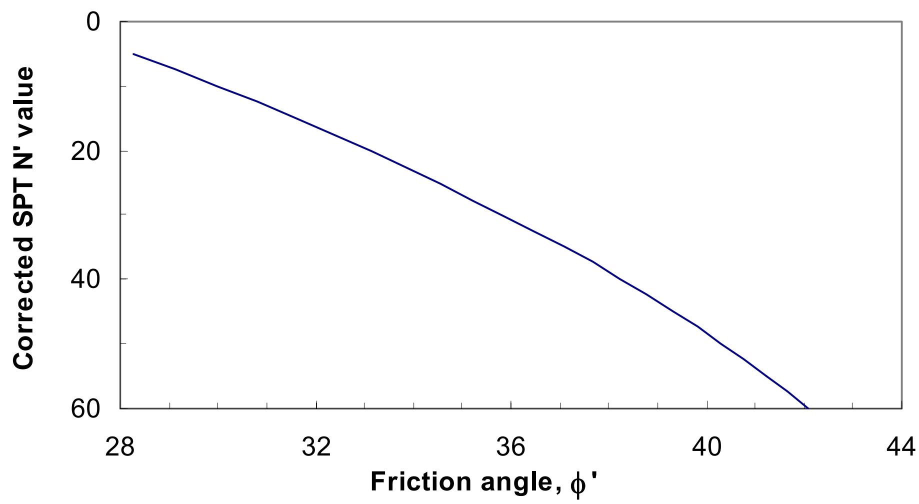


Figure 21: ϕ' by Peck, Hanson and Thornburn (Kulhawy and Mayne, 1990)

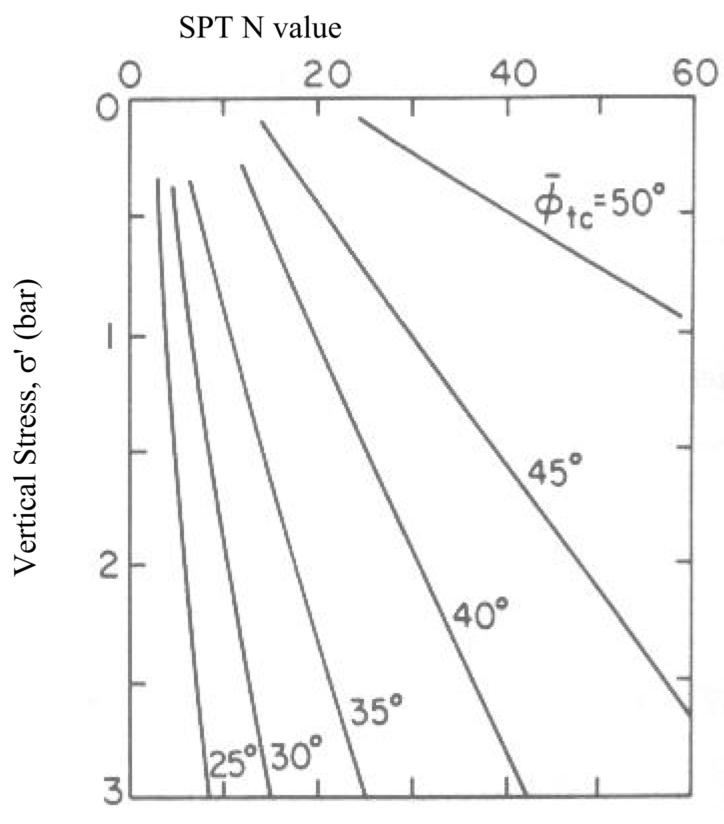


Figure 23: ϕ' by Schmertmann (Kulhawy and Mayne, 1990)

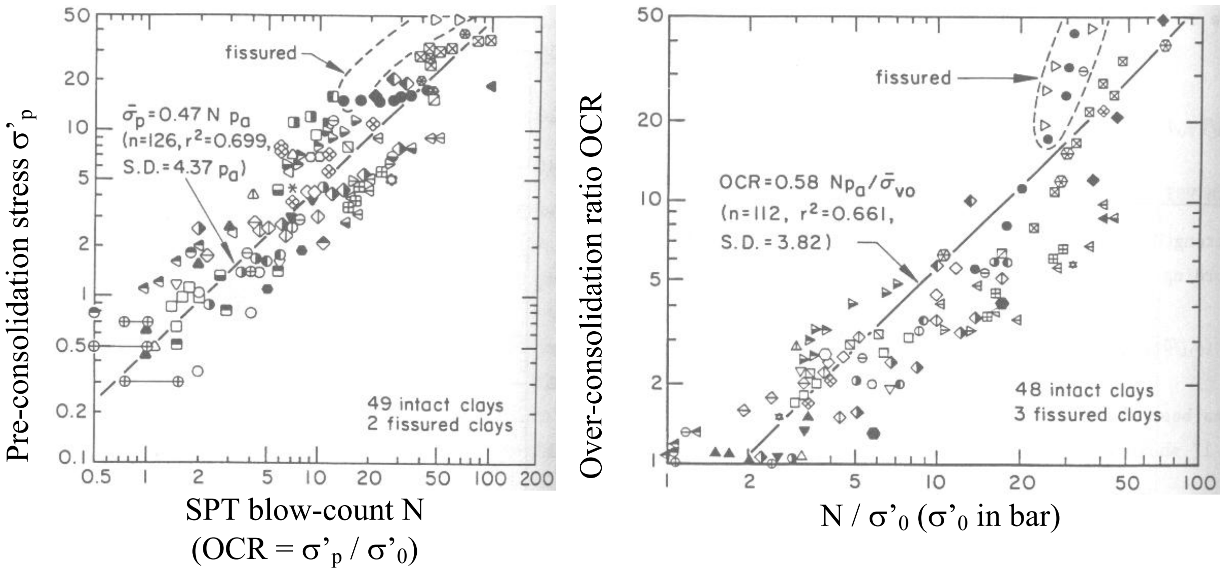


Figure 24: OCR--N Relationship (Kulhawy and Mayne, 1990)

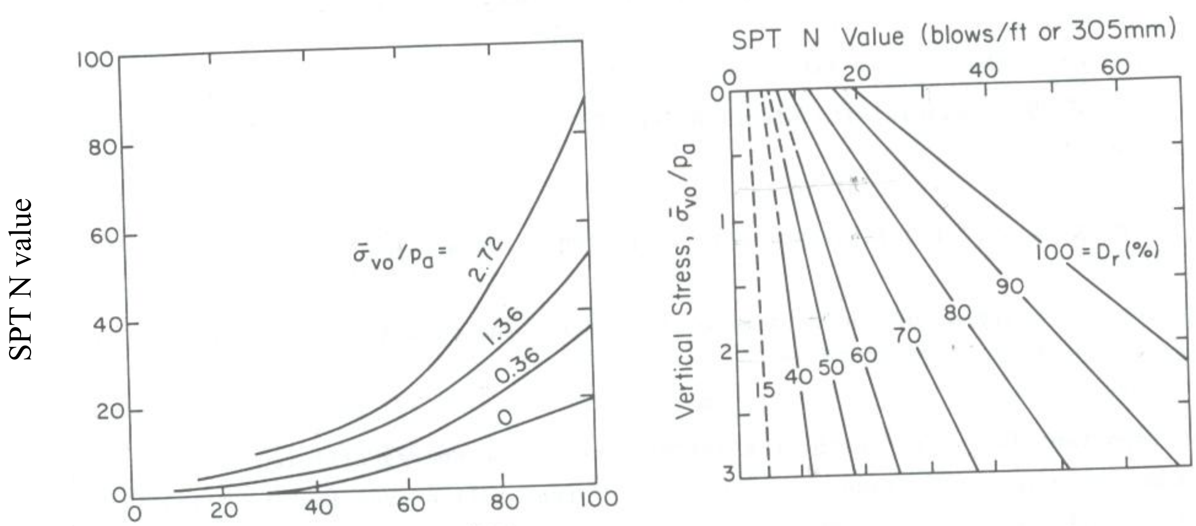


Figure 25: Relative Density--N--Stress Relationship (Kulhawy and Mayne, 1990)

Table 7. Correlations of soil properties from CPT

Properties	From CPT	Figure	Reference	
ϕ	Robertson and Campanella: atan(0.1+0.38*Log(q_c/σ'))	25	Figure 4.14 and Equation 4.12	Kulhawy and Mayne, 1990
S_u (bar)	Theoretical: (q_c - σ_o) / Nk q_c and σ_o in bars.		Equation 4.61	
OCR for clay	Mayne: 0.29 q_c / σ'_o q_c and σ_o in bars.	27	Figure 3.10	
Dr	Jamiolkowski: 68 log(q_{cn}) – 68 $q_{cn} = \dfrac{q'_c}{\sqrt{P_a \sigma'_0}}$ (dimensionless) $q'_c = q_c / K_q$ $K_q = 0.9 + Dr/300$ q_c and σ'_o in bars.	26	Figure 2.24 and Equation 2.20	

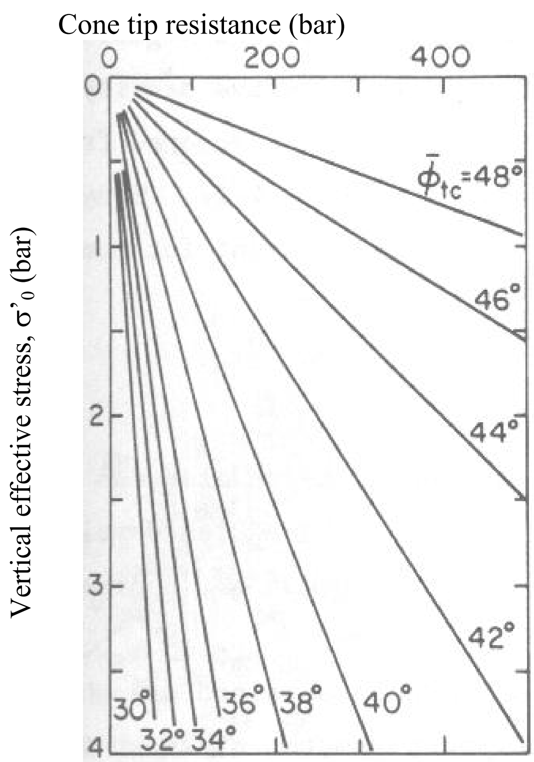


Figure 26: ϕ'_{tc} correlated from q_c for NC, uncemented quartz sands (Kulhawy and Mayne, 1990)

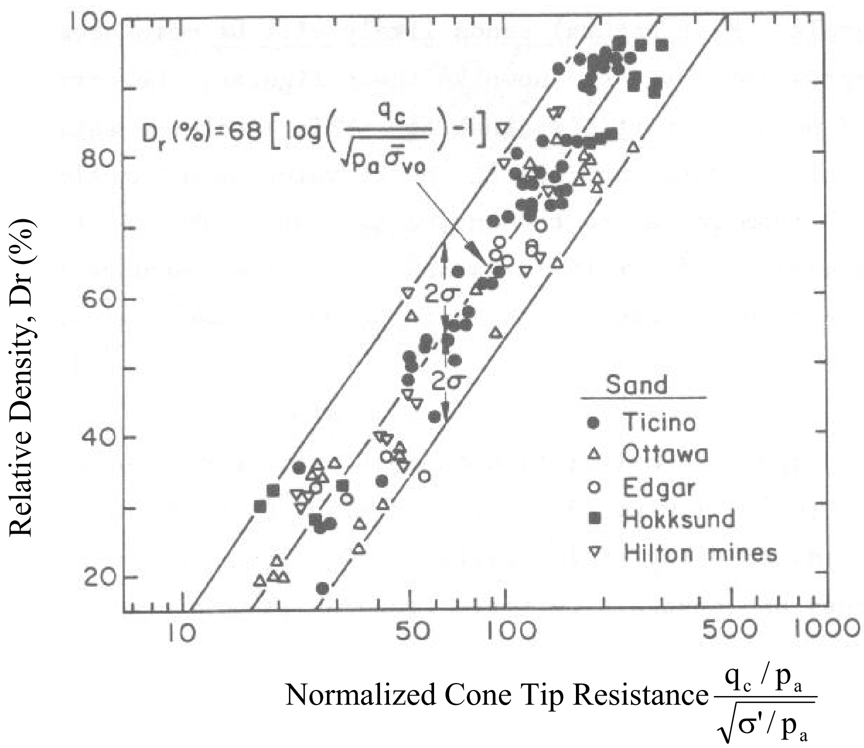


Figure 27: Correlation between Dr and q_c (uncorrected for boundary effect) (Kulhawy and Mayne, 1990)

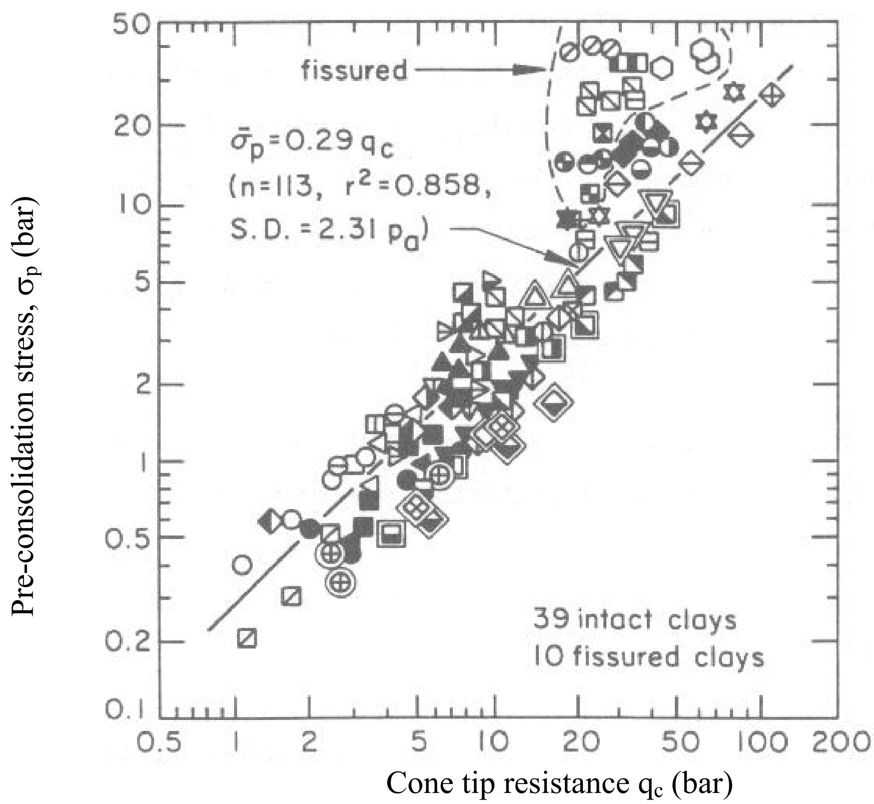


Figure 28: σ_p correlated with q_c (OCR = σ_p / σ'_0) (Kulhawy and Mayne, 1990)

Example 1. Newbury Test Site

1.1 Static Analysis Test Pile #2 – Closed End Steel Pipe Pile

Pile Geometry:

Pile dimension – 12.75” OD

Wall Thickness – 0.375”

Penetration Depth – 80ft

$$A_p = \frac{\pi\left(\frac{12.75}{12}\right)^2}{4} = 0.39\text{ft}^2 \quad \text{(Area)}$$

$$C_p = \pi\left(\frac{12.75}{12}\right) = 3.34\text{ft} \quad \text{(Circumference)}$$

Subsurface Conditions:

Layer #	Layer Description	**Depth to Top of Layer (ft)**	**Depth to Bottom of Layer (ft)**	**ϕ' (Deg)**	**γ_t (pcf)**	**D_r (%)**	**S_u (tsf)**
1	Misc. Fill	0.0 =ele. + 18.30′	8.0	30	125	35	-
2	OC Clay	8.0	18.0	33	120	-	1
3	NC Clay	18.0	54.0	-	112.5	-	0.3
4	Interbedded Silt, Sand & Clay	54.0	86.0	34	122.5	35	-

GWT @ 5ft Below ground surface.

Design Methods for Pipe Piles Driven in Mixed Soil:

- α-Tomlinson/Nordlund/Thurman
- β-Method/Thurman

Skin Friction:

(Calculations according to Paikowsky & Hajduk, 1999)

Layer #1 – Miscellaneous Fill

a) Nordlund Method

$$f_s = K_\delta\, C_f\, \sigma'_v \sin_\delta$$

$V = A_p(\text{ft/ft}) = 0.89\ \text{ft}^3/\text{ft}$

$K_\delta = 0.85$

$\sigma'_v = 0.25$ tsf

$\delta/\phi = 0.62 \qquad \therefore\ \delta = 0.62*30° = 18.6°$

$C_f = 0.85$

$$f_s = (0.85)(0.85)(0.25)(\sin 18.6°)$$

$$f_s = 0.06\ \text{tsf}$$

b) β-Bushan Method

$$f_s = (0.18 + 0.0065\ D_r)\ \sigma'_v$$

$D_r = 35\%$
$\sigma'_v = 0.25$ tsf

$$f_s = (0.18 + (0.0065)(35))\ 0.25$$
$$f_s = 0.10 \text{ tsf}$$

Layer #2 - OC Clay

a) α-Tomlinson Method

$$f_s = \alpha S_u$$

$S_u = 1.0$ tsf
D/B = 13'/(12.75"/12) = 12.2 $\quad\quad \therefore\ L \cong 10\ B$
$\alpha = 0.9$

$$f_s = (0.9)(1.0)$$
$$f_s = 0.9 \text{ tsf}$$

b) β-Burland Method

$$f_s = \beta\ \sigma'_v$$

$\sigma'_v = 0.55$ tsf
β (OCR≅10) = 1.5

$$f_s = (1.5)(0.55)$$
$$f_s = 0.83 \text{ tsf}$$

Layer #3 - NC Clay

a) α-Tomlinson Method

$$f_s = \alpha S_u$$

$S_u = 0.3$ tsf
D/B = 36'/(12.75"/12) = 33.9
$\alpha = 1.0$

$$f_s = (1.0)(0.3)$$
$$f_s = 0.3 \text{ tsf}$$

b) β-Burland Method

$$f_s = \beta\ \sigma'_v$$

$\sigma'_v = 1.15$ tsf
β (OCR≅1) = 0.3

$$f_s = (0.3)(1.15)$$
$$f_s = 0.345 \text{ tsf}$$

Layer #4 - Interbedded Silt, Sand, and Clay

a) Nordlund Method

$$f_s = K_\delta \, C_f \, \sigma'_v \, \sin_\delta$$

$V = 0.89\ ft^3/ft$

$K_\delta = 0.85$

$\sigma'_v = 1.99$ tsf

$\delta/\phi = 0.62$ $\quad\therefore\ \delta = 0.62*34° = 21.1°$

$C_f = 0.85$

$$f_s = (0.85)(0.85)(1.99)(\sin 21.1°)$$

$$f_s = 0.52 \text{ tsf}$$

b). β-Bushan Method

$$f_s = (0.18 + 0.0065\ D_r)\ \sigma'_v$$

$D_r = 35\%$

$\sigma'_v = 1.99$ tsf

$$f_s = (0.18 + (0.0065)(35))\ 1.99$$

$$f_s = 0.81 \text{ tsf}$$

Tip Resistance

Layer #4 – Interbedded Silt, Sand, and Clay

Thurman Method

$$q_p = \alpha_t\ N'_q\ \sigma'_v < q_{limit}$$

α_t (D/b > 45, ϕ=34°) = 0.6

$N'_q = 55$

$\sigma'_v = 2.38$ tsf

q_{limit} = 3,750 kPa = 39.1 tsf

$$q_p = (0.6)(55)(2.38) = 78.5 \text{ tsf} > 39.1 \text{ tsf}$$

$$q_p = q_L = 39.1 \text{ tsf}$$

Newbury Test Site - TP#2 (Closed End Steel Pipe Pile)
Summary Table

Layer #	Layer Thickness (ft)	α-Tomlinson/Nordlund		β-Method		Thurmon Method	
		Skin Friction (tsf)	Friction Capacity (tons)	Skin Friction (tsf)	Friction Capacity (tons)	Tip Resistance (tsf)	End Bearing Capacity (tons)
1	8	0.06	1.6	0.10	2.7	-	-
2	2	0.90	3.0	0.83	2.8	-	-
	8	0.90	27.1	0.83	24.9	-	-
3	36	0.30	36.1	0.345	41.5	-	-
4	26	0.52	45.2	0.81	70.3	39.5	35.2
		Q_a =	108.4		136.7	Q_p =	35.2

Note: Upper 10 feet were cased prior to driving.

Total Pile Capacity

α-Tomlinson/Nordlund/Thurman Q_T = 144 tons
β-Method/Thurman Q_T = 172 tons

1.2 Resistance Factors - Static Analysis

Using Table 25 for pipe piles in mixed soils the resistance factor for both combination of methods is ϕ = 0.25 for redundant piles.

Factored pile capacity for static analysis

a) α-Tomlinson/Nordlund/Thurman

$$R_r = \phi R_n = 0.25 \text{ x } 144 = 36 \text{ tons} = 320 \text{ kN}$$

b) β-Method/Thurman

$$R_r = \phi R_n = 0.25 \text{ x } 172 = 43 \text{ tons} = 380\text{kN}$$

1.3 Dynamic Analyses

The following table describes the results of the different dynamic analyses, using the appropriate resistance factors based on Table 27 for redundant piles.

Predictions of Pile Capacity Based on the Dynamic Methods for TP#2

Type of Analysis	**Method**	**Q_{ult} (kN)**	**ϕ (Table 25)**	**R_r (kN)**
CAPWAP Analyses	CAPWAP EOD (A_R<350 BPI<4)	418	0.40	167
	CAPWAP BORL	1112	0.65	723
Simplified Methods	EA EOD	792	0.55	436
Dynamic Equations	ENR w/FS = 6, EOD	347	0.25	87
	Gates EOD	721	0.75	541
	FHWA EOD	1157	0.40	463
Static Load Test Results		Q_{ult}= 658	0.80	526

1) 1 ton = 8.896 kN
2) Blow Count EOD for TP#2 is 1 to 2 BPI < 4 BPI
3) A_R = 70ft x 3.34ft/0.89ft^2 = 263 < 350 (large displacement)
4) The BORL represents the last restrike, which in the case of TP#2 was carried out 3 months after EOD.

1.4 Static Load Test Results

The following table analyzes the site variability based on the coefficient of variation for the standard penetration test across the same identifiable layers. Overall, it can be judged that the site is of low variability. Based on Table 30, the resistance factor for a single load test in a low variability site is ϕ = 0.8.

$$R_r = 658 \text{ x } 0.8 = 525\text{kN}$$

Newbury Test Site - Site Variability Summary Table

Layer #	Layer Description	Layer Upper Ele. (ft)	Layer Lower Ele. (ft)	Mean Value of SPT (M_x)	Standard Deviation in SPT Values (δ_x)	COV (%)	Layer Variability
1	Misc. Fill	+17.9	+9.9	100.0	0.00	0.0	Low*
2	OC Clay	+9.9	-0.1	21.0	1.41	6.7	Low
3	NC Clay	-0.1	-36.1	WOR	0.00	0.0	Low
4-1	Interbedded (#1)	-36.1	-43.0	11.8	0.35	3.0	Low
4-2	Interbedded (#2)	-43.0	-48.0	1.5	2.10	140.0	High**
4-3	Interbedded Silty Sand (#3)	-48.0	-58.0	15.8	3.20	20.2	Low
4-4	Interbedded (#4)	-58.0	-68.4	14.0	9.90	70.7	High
5	Fine to Medium Sand	-68.4	-73.0	23	0.00	0.0	Low*

* Information only from boring NB1, therefore, no variability (low).

** Soft Layer (WOR in NB1 and 3 blows/ft in NB4). High variability value due to low SPT values. Low variability in this layer is more realistic.

Example 2. RI Rt. I-195/I-95 Test Site
PPC 2-1 / PPC 2-2

2.1 Static Capacity Evaluation
Pile Geometry:
Pile dimension – 14" square precast, prestressed concrete (PPC)
Penetration Depth – 90ft (PPC 2-1)
110ft (PPC 2-2)

A_p = (14/12)/(14/12) = 1.36 ft^2 (area)
C_p = 14/12 = 4.67 ft(circumference)

Subsurface Conditions:

Layer #	Layer Description	**Depth to Top of Layer (ft)**	**Depth to Bottom of Layer (ft)**	**ϕ' (Deg)**	**γ_t (pcf)**	**D_r (%)**	**S_u (tsf)**
1	Fill	0.0 (+ 32.5)	4.5	40	127.3	80	-
2	Silty Sand	4.5	37.2	34	119.5	40	-
3	Varied Silt	37.2	51.5	36	122.9	-	5600
4	Soft Varied Silt	51.5	57.8	-	115.1	-	4800
5	Varied Silt	57.8	105.6	36	122.9	-	4800
6	Till	105.6	112.0	43	123.4	100	-
7	Weathered Bedrock	112.0	120.1	-	-	-	-
8	Bedrock	120.1	-	-	-	-	-

GWT @ 15.5ft below ground surface.

Design Methods for PPC Piles Driven in Mixed Soil:
- β-Method/Thurman
- α-Tomlinson/Nordlund/Thurman

Skin Friction:

Layer #1 – Fill
a) β-Bushan Method

$$f_s = (0.18 + 0.0065\ D_r)\ \sigma'_v$$

D_r = 80%
σ'_v = 0.143 tsf

$$f_s = (0.18 + (0.0065 \times 80))\ 0.143$$
$$f_s = 0.10 \text{ tsf}$$

b) Nordlund Method

$$f_s = K_\delta \, C_f \, \sigma'_v \, \sin_\delta$$

$K_\delta = 3.15$
$\sigma'_v = 0.143$ tsf
$\delta/\phi = 0.83$ $\quad\quad \therefore\ \delta = 0.83*40.1° = 33.3°$
$C_f = 0.92$

$$f_s = (3.15)(0.92)(0.143)(\sin 33.3°)$$
$$f_s = 0.235 \text{ tsf}$$

Layer #2 - Silty Sand

a) β-Bushan Method

$$f_s = (0.18 + 0.0065\, D_r)\, \sigma'_v$$

$D_r = 40\%$
$\sigma'_v = 0.95$ tsf

$$f_s = (0.18 + (0.0065 \text{ x } 40))\ 0.95$$
$$f_s = 0.42 \text{ tsf}$$

b) Nordlund Method

$$f_s = K_\delta \, C_f \, \sigma'_v \, \sin_\delta$$

$K_\delta = 1.70$
$\sigma'_v = 0.95$ tsf
$\delta/\phi = 0.83$ $\quad\quad \therefore\ \delta = 0.83*33.8° = 28.0°$
$C_f = 0.95$

$$f_s = (1.70)(0.95)(0.95)(\sin 28.0°)$$
$$f_s = 0.72 \text{ tsf}$$

Layer #3 - Upper Varied Silt

a) α-Tomlinson Method

$$f_s = \alpha S_u$$

$S_u = 2.8$ tsf
$\alpha = 0.4$

$$f_s = (0.4)(2.8)$$
$$f_s = 1.12 \text{ tsf}$$

b) β-Burland Method

$$f_s = \beta\, \sigma'_v$$

$\sigma'_v = 1.78$ tsf
β (OCR = 4) = 0.7 (OCR from consolidation tests)

$$f_s = (0.7)(1.78)$$
$$f_s = 1.25 \text{ tsf}$$

Layers # 4 & 5 - Lower Varied Silt

a) α-Tomlinson Method

$$f_s = \alpha S_u$$

$S_u = 2.4$ tsf

$\alpha = 0.4$

$$f_s = (0.4)(2.4)$$

$$f_s = 0.96 \text{ tsf}$$

b) β-Burland Method

$$f_s = \beta\, \sigma'_v$$

$\sigma'_v = 2.78$ tsf

β (OCR = 1) = 0.3 (OCR from consolidation tests)

$$f_s = (0.3)(2.78)$$

$$f_s = 0.837 \text{ tsf}$$

Layer #6 - Till

a) β-Bushan Method

$$f_s = (0.18 + 0.0065\, D_r)\, \sigma'_v$$

$D_r = 100\%$

$\sigma'_v = 3.6$ tsf

$$f_s = (0.18 + (0.0065 \text{ x } 100))\ 3.6$$

$$f_s = 3.0 \text{ tsf}$$

b) Nordlund Method

$$f_s = K_\delta\, C_f\, \sigma'_v \sin_\delta$$

$K_\delta = 3.2$

$\sigma'_v = 3.6$ tsf

$\delta/\phi = 0.83$ $\therefore\ \delta = 0.83 * 43.2° = 35.9°$

$C_f = 0.90$

$$f_s = (3.2)(0.90)(3.6)(\sin 35.9°)$$

$$f_s = 6.08 \text{ tsf}$$

Tip Resistance:

Layer #5 - Varied Silt

Thurman Method

$$q_p = \alpha_t\, N'_q\, \sigma'_v < q_{limit}$$

$\alpha_t = 0.7$

$N'_q = 75$

$\sigma'_v = 3.2$ tsf

$q_{limit} = 7{,}500$ kPa = 78.3 tsf

$$q_p = 0.7 \text{ x } 75 \text{ x } 3.2 = 168 \text{ tsf} > 78.3 \text{ tsf}$$

$$q_p = q_L = 78.3 \text{ tsf}$$

Layer #6 - Till
Thurman Method

$$q_p = \alpha_t N'_q \sigma'_v < q_{limit}$$

$\alpha_t = 0.77$
$N'_q = 110$
$\sigma'_v = 3.5$ tsf
$q_{limit} = 26{,}000$ kPa = 271.4 tsf

$$q_p = 0.77 \times 110 \times 3.5 = 296.5 \text{ tsf} > 271.4 \text{ tsf}$$

$$q_p = q_L = 271.4 \text{ tsf}$$

RI I-195/I-95 Test Site, PPC 2-1, 2-2 - Static Analysis Summary Table

Layer #	Layer Thickness (ft)	β-Method		α-Tomlinson/Nordlund		Thurmon Method	
		Skin Friction (tsf)	Friction Capacity (tons)	Skin Friction (tsf)	Friction Capacity (tons)	Tip Resistance (tsf)	End Bearing Capacity (tons)
1	4.5	0.10	2.1	0.235	4.9	-	-
2	32.7	0.42	64.1	0.72	110.0	-	-
3	14.3	1.25	83.5	1.12	74.8	-	-
4	6.3	0.837	24.6	0.96	28.2	-	-
5	32.2 (PPC 2-1)	0.837	125.9	0.96	144.4	78.3	106.5
	47.8 (PPC 2-2)	0.837	186.8	0.96	214.3		
6	4.4	3.0	61.6	6.08	124.9	271.4	369.1
	PPC 2-1:	Q_s =	300.2		362.3	Q_p =	106.5
	PPC 2-2:	Q_s =	422.7		557.1	Q_p =	369.1

Summary of Static Pile Capacity:

Static Analysis - Pile Capacity (tons)

Pile	β-Method/Thurman	α-Tomlinson/Nordlund/Thurman
PPC 2-1	407	469
PPC 2-2	792	926

2.2 Resistance Factors

Using Table 25 for concrete piles in mixed soils for redundant piles;

$\phi = 0.40$ for both methods

Rr - Factored Pile Capacity (tons)

Pile	β-Method/Thurman	α-Tomlinson/Nordlund/Thurman
PPC 2-1	165	190
PPC 2-2	315	370

Using the efficiency factors one can estimate that the β-Method/Thurman is more efficient than the α-Tomlinson/Nordlund/Thurman ($\phi/\lambda = 0.51$ vs. 0.41), and hence should prefer the use of these combination of methods.

2.3 Static Load Test

Eight static load tests are expected to be carried out at the site. The ultimate design load (see section 3.4.7 item 3) and the load test results should consider this factor.

Examination of the site variability.

RI - Site Variability - Test Area #2
Summary Table

Layer #	Layer Descriptio n	Layer Upper Ele. (ft)	Layer Lower Ele. (ft)	Mean Value of SPT (M_x)	Standard Deviation in SPT Values (δ_x)	COV (%)	Layer Variability
1	Fill	+32.0	+28.0	15.3	1.38	9.0	Low
2	Silty Sand	+28.0	-4.7	22.9	3.10	13.5	Low
3	Upper V. Silt	-4.7	-19.0	17.6	5.66	32.0	Medium
4	Soft V. Silt	-19.0	-25.3	5.30	1.15	21.6	Low
5 (up)	Lower V. Silt	-25.3	-72.0	27.3	4.32	16.0	Low
5 (mid.)	Lower V. Silt	-42.0	-57.0	18.0	2.30	12.7	Low
5 (bot.)	Lower V. Silt	-57.0	-73.1	12.8	4.10	32.0	Medium
6	Till	-73.1	-79.5	60.4	15.20	25.0	Low-Medium*

* For the calculation of average values of SPT in the Till layer, refusal was defined as 100 blows/ft even if actual numbers were higher.

Layers 2, 3, 5, and 6 contribute most to the calculated pile capacity, where layers 5 and 6 serve for end bearing calculations for PPC 2-1 and PPC 2-2, respectively. Judgment suggests the use of low site variability, however, one may chose medium variability.

Using Table 30 for 8 load-tests, both site variabilities result with the same resistance factor $\phi = 0.90$.

<u>Example 3 - Newbury Test Site, Test Pile #3</u>
14" Square PPC

3.1 Static Analysis

Pile Geometry:
Pile dimension – 14” Square Precast, Prestressed Concrete (PPC)
Penetration Depth – 80ft

$A_p = (14 \text{ x } 14)/12^2 = 1.36 \text{ ft}^2$ (Area)
$C_p = (14 \text{ x } 4)/12 = 4.67 \text{ ft}$ (Circumference)

Subsurface Conditions: See example 1 for soil conditions.

Design Methods for PPC Piles Driven in Mixed Soil:
- β-Method/Thurman
- α-Tomlinson/Nordlund/Thurman

Skin Friction:

Layer #1 - Miscellaneous Fill
a) β-Bushan Method

$$f_s = (0.18 + 0.0065\, D_r)\, \sigma'_v$$

$D_r = 35\%$
$\sigma'_v = 0.25$ tsf

$$f_s = (0.18 + (0.0065 \text{ x } 35))\, 0.25$$
$$f_s = 0.10 \text{ tsf}$$

b) Nordlund Method

$$f_s = K_\delta\, C_f\, \sigma'_v\, \sin_\delta$$

$V = A_p(\text{ft/ft}) = 1.36 \text{ ft}^3/\text{ft}$
$K_\delta = 1.15$
$\sigma'_v = 0.25$ tsf
$\delta/\phi = 0.85$ $\therefore\ \delta = 0.85*30° = 25.5°$
$C_f = 0.95$

$$f_s = (1.15)(0.95)(0.25)(\sin 25.5°)$$
$$f_s = 0.12 \text{ tsf}$$

Layer #2 - OC Clay
a) α-Tomlinson Method

$$f_s = \alpha S_u$$

$S_u = 1.0$ tsf
$\alpha = 0.9$

$$f_s = (0.9)(1.0)$$
$$f_s = 0.9 \text{ tsf}$$

b) β-Burland Method

$$f_s = \beta\ \sigma'_v$$

$\sigma'_v = 0.55$ tsf

β (OCR≅10) = 1.5

$$f_s = (1.5)(0.55)$$
$$f_s = 0.83 \text{ tsf}$$

Layer #3 - NC Clay

a) α-Tomlinson Method

$$f_s = \alpha S_u$$

$S_u = 0.3$ tsf

$\alpha = 1.0$

$$f_s = (1.0)(0.3)$$
$$f_s = 0.3 \text{ tsf}$$

b) β-Burland Method

$$f_s = \beta\ \sigma'_v$$

$\sigma'_v = 1.15$ tsf

β (OCR=1) = 0.3

$$f_s = (0.3)(1.15)$$
$$f_s = 0.345 \text{ tsf}$$

Layer #4 - Interbedded Silt, Sand, and Clay

a) β-Bushan Method

$$f_s = (0.18 + 0.0065\ D_r)\ \sigma'_v$$

$D_r = 35\%$

$\sigma'_v = 1.99$ tsf

$$f_s = (0.18 + (0.0065 \text{ x } 35))1.99$$
$$f_s = 0.81 \text{ tsf}$$

b) Nordlund Method

$$f_s = K_\delta\ C_f\ \sigma'_v\ \sin_\delta$$

$V = A_p(\text{ft/ft}) = 1.36\ \text{ft}^3/\text{ft}$

$K_\delta = 1.15$

$\sigma'_v = 1.99$ tsf

$\delta/\phi = 0.85$ $\quad\quad \therefore\ \delta = 0.85*34° = 28.9°$

$C_f = 0.95$

$$f_s = (1.15)(0.95)(1.99)(\sin 28.9°)$$
$$f_s = 1.05 \text{ tsf}$$

Tip Resistance:
Thurman Method

$$q_p = \alpha_t N'_q \sigma'_v < q_{limit}$$

α_t (D/b > 45, ϕ=34°) = 0.6
$N'_q = 55$
$\sigma'_v = 2.38$ tsf
$q_{limit} = 3{,}750$ kPa = 39.1 tsf

$$q_p = (0.6)(55)(2.38) = 78.5 \text{ tsf} > 39.1 \text{ tsf}$$
$$q_p = q_L = 39.1 \text{ tsf}$$

Newbury Test Site - TP#3 (14" Square PPC)
Summary Table

Layer #	Layer Thickness (ft)	α-Tomlinson/Nordlund		β-Method		Thurmon Method	
		Skin Friction (tsf)	Friction Capacity (tons)	Skin Friction (tsf)	Friction Capacity (tons)	Tip Resistance (tsf)	End Bearing Capacity (tons)
1	8	0.12	4.5	0.100	3.7	-	-
2	2	0.90	8.4	0.830	7.8	-	-
	8		33.6		31.0	-	-
3	36	0.30	50.4	0.345	58.0	-	-
4	26	1.05	127.5	0.810	98.4	39.5	53.7
		Q_a =	211.5		187.4	Q_p =	53.7

Note: Upper 10 feet were cased prior to driving.

Total Pile Capacity
α-Tomlinson/Nordlund/Thurman Q_T = 265 tons = 2357 kN
β-Method/Thurman Q_T = 241 tons = 2144 kN

3.2 Resistance Factors - Static Analysis
Using Table 25 for concrete piles in mixed soils the resistance factors for both combinations of methods is ϕ = 40 for redundant piles.

Factored pile capacity for static analysis
a) α-Tomlinson/Nordlund/Thurman

$$R_r = \phi R_n = 0.40 \times 265 = 106 \text{ tons} = 945 \text{ kN}$$

b) β-Method/Thurman

$$R_r = \phi R_n = 0.40 \times 241 = 96 \text{ tons} = 860 \text{ kN}$$

3.3 Dynamic Analyses

The following table describes the results of the different dynamic analyses.

Predictions of Pile Capacity Based on the Dynamic Methods for TP#3

Type of Analysis	Method	Q_{ult} (kN)	ϕ (Table 25)	R_r (kN)
CAPWAP Analyses	CAPWAP EOD	738	0.40	295
	CAPWAP BORL	1228	0.65	798
Simplified Methods	EA EOD	1228	0.55	675
Dynamic Equations	ENR w/FS = 6, EOD	409	0.25	102
	Gates EOD	783	0.75	587
	FHWA EOD	1299	0.40	520
Static Load Test Results		Q_{ult}= 872	0.80	698

1) 1 ton = 8.896 kN
2) Blow Count EOD for TP#3 is approximately 1 BPI
3) A_R = 70ft x 4.67ft/1.36ft^2 = 240 < 350
4) The BORL represents the last restrike, which in the case of TP#3 was carried out 8 months after EOD.

3.4 Static Load Test Results

Referring to the site variability analysis presented in Example 1, the resistance factor for the static load test is ϕ = 0.8, and the factored static load test resistance is:

$$R_r = 872 \text{ x } 0.80 = 700 \text{ kN}$$

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