Final Report

PB98-157522

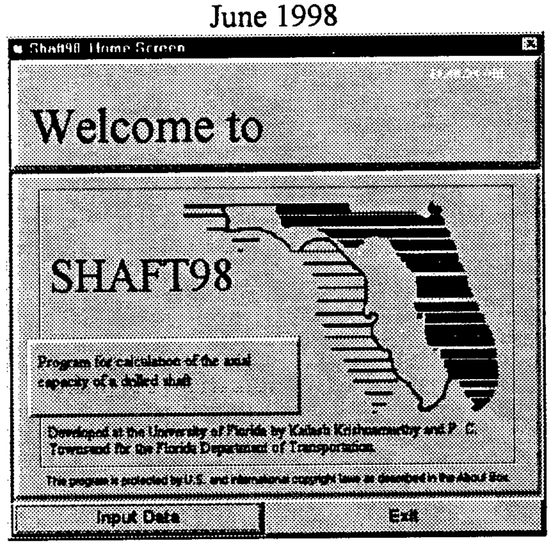
SHAFT98 - COMPUTER DESIGN PROGRAM FOR AXIALLY LOADED DRILLED SHAFTS

SUBMITTED TO

FLORIDA DEPARTMENT OF TRANSPORTATION

State Project No: 99700-3322-119
UF Project No: 49104504526
WPI No: 0510739
Contract No: B-9906

June 1998



Principal Investigator: Dr. F.C. Townsend
Post-Doctorate: Dr. Kailash Krishnamurthy

Department of Civil Engineering
College of Engineering
University of Florida
Gainesville, Florida 32611-6580

Technical Report Documentation Page

1. Report No.	2. Government Accession No.	3. Recipient's Catalog No.
WPI # 0510739		

4. Title and Subtitle	5. Report Date
Shaft98 - Computer Design Program for Axially Loaded Drilled Shafts	June 1998
	6. Performing Organization Code 49104504526-12
7. Author(s) F.C. Townsend, K. Krishnamurthy	8. Performing Organization Report No.
9. Performing Organization Name and Address University of Florida Department of Civil Engineering 345 Weil Hall / P. O. Box 116580 Gainesville, FL 32611-6580	10. Work Unit No. (TRAIS) 99700-3322-119 11. Contract or Grant No. B-9906
12. Sponsoring Agency Name and Address Florida Department of Transportation Research Management Center 605 Suwannee Street, MS 30 Tallahassee, FL 32301-8064	13. Type of Report and Period Covered Final Report 8/15/95 - 6/30/98 14. Sponsoring Agency Code

15. Supplementary Notes

Prepared in cooperation with the Federal Highway Administration

16. Abstract

Shaft98 is a Visual Basic program based upon the FHWA reports: (a) Drilled Shafts: Construction Procedure and Design Methods (1988) by L.C. Reese and M.W. O'Neill, and (b) Load Transfer for Drilled Shafts in Intermediate Geomaterials (1996) by M.W. O'Neill et al. The program calculates axial capacities and settlements for clays, sands and soft rock [unconfined compression strength (q_u) between 0.5 and 5.0 MPa].

17. Key Words	18. Distribution Statement
Drilled Shafts, Deep Foundations, Computer Program, Geotechnical Engineering	No restrictions. This document is available to the public through the National Technical Information Service, Springfield, VA, 22161

19. Security Classif. (of this report)	20. Security Classif. (of this page)	21. No. of Pages	22. Price
Unclassified	Unclassified	55	

Form DOT F 1700.7 (8-72)

Reproduction of completed page authorized

SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
		AREA		
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
		VOLUME		
fl oz	fluid ounces	29.57	milliliters	ml
gal	gallons	3.785	liters	l
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3

NOTE: Volumes greater than 1000 l shall be shown in m^3.

Symbol	When You Know	Multiply By	To Find	Symbol
		MASS		
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams	Mg
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
		ILLUMINATION		
fc	foot-candles	10.76	lux	l
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
		FORCE and PRESSURE or STRESS		
lbf	poundforce	4.45	newtons	N
psi	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
		LENGTH		
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
		AREA		
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
		VOLUME		
ml	milliliters	0.034	fluid ounces	fl oz
l	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
		MASS		
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg	megagrams	1.103	short tons (2000 lb)	T
		TEMPERATURE (exact)		
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
		ILLUMINATION		
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
		FORCE and PRESSURE or STRESS		
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	psi

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised August 1992)

DISCLAIMER

"The opinions, findings, and conclusions expressed in this publication are those of the authors and not necessarily those of the Florida Department of Transportation or the U.S. Department of Transportation."

No Warranty, expressed or implied, is made by the Florida Department of Transportation as to the accuracy and the functioning of the program text or the results it produces, nor shall the fact of distribution constitute any such warranty, and no responsibility is assumed by Florida Department of Transportation in any connection therewith.

Prepared in cooperation with the State of Florida Department of Transportation and the U.S. Department of Transportation.

Table of Contents

List of Figures

List of Tables

SHAFT98 - COMPUTER DESIGN PROGRAM
FOR
AXIALLY LOADED DRILLED SHAFTS

Introduction

The SHAFT98 computer program is a *Windows* based program used to estimate the static axial capacity of drilled shafts. The methodology is based upon Federal Highway Administration reports: (a) Reese, L. and O'Neill, M. (1988) "Drilled Shafts: Construction Procedure and Design Methods", and (b) O'Neill, M.W. et al. (1996) "Load Transfer for Drilled Shafts in Intermediate Geomaterials". The former presents methods for estimating drilled shaft capacity in clays or sands, and provides settlement estimates. The latter addresses intermediate geomaterials, soft rock, q_u between 0.5 and 5.0 Mpa (1.7 to 17 tsf) and SPT blow counts of 50 - 100; and provides settlement analyses. Load transfer for rock socketed shafts in Florida limestone is based upon the methodology described in; (a) FDOT Final Report " An Evaluation of Design Methods for Drilled Shafts)" (1990), which is also found (b) McVay, M.C. et al. (1992).

SHAFT98 replaces earlier versions of SHAFTUF and SHAFT93.

Method of Analysis

The axial capacity of drilled shafts can be calculated as:

$$Q_t = Q_s + Q_b \qquad \text{(Eqn 1)}$$

where:

Q_t = Ultimate shaft capacity
Q_s = capacity in skin friction
Q_b = Capacity in end bearing

The computations of side resistance (skin friction) and end bearing are presented in separate sections for clay, sand, and intermediate geomaterial (soft rock). Settlement calculations are also presented. These three material types (clay, sand, and soft rock) are identified as follows to be compatible with FDOT's SPT94 program.

Table 1 Code for Soil Type

Description	SHAFT98	SPT94
Plastic Clay	2	1
Clay-Silt-Sand Mixtures	2	2
Clean Sand	3	3
Soft Rock	4	4

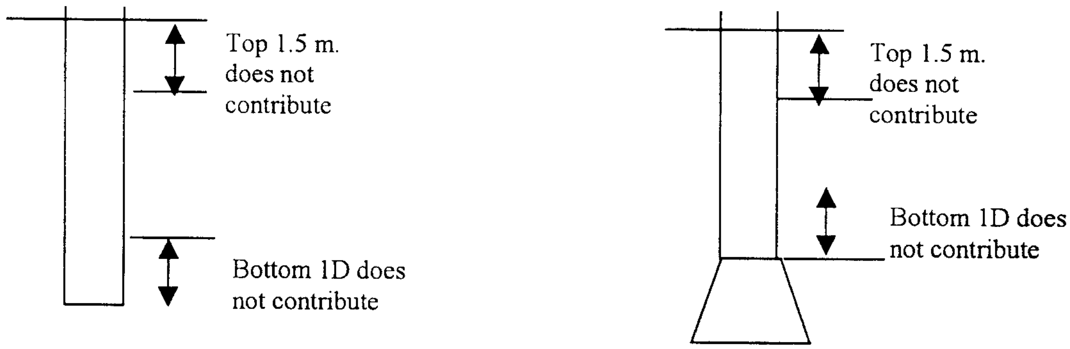


Figure 1 Portions of Drilled Shaft Non-Contributory in Friction

Table 2 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	
From ground surface to length of casing	0.0	
Bottom 1 diameter of shaft or 1 stem diameter above top of bell	0.0	
All other points along drilled shaft sides	0.55	2.75 tsf 275 kPa

Design for Clay

Shear Transfer - The load transfer in side resistance for drilled shafts in clay employs the Alpha (α) method. That is, the undrained shear strength C_u of clay is found from appropriate soil tests or correlations with insitu tests and the following equation used to compute the ultimate value if unit load transfer at the depth z below the ground surface.

$$f_{su} = \alpha\, C_u \qquad \text{(Eqn 2)}$$

where

f_{su} = ultimate unit load transfer in side resistance at depth z

α = empirical factor that varies with depth, (see Table 2 and Figure 1)and

C_u = undrained shear strength at depth z,

The total load Q_s in side resistance is now computed as:

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

where

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

and

L_1 and L_2 = penetration of drilled shaft below ground surface between two layers.

Figure 1 illustrates the zones where α is assumed to be zero. The setting of $\alpha = 0$ for a distance of 1 diameter above the base is from the work of Ellison et al. (1971), who 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. Consequently, the lateral stress at the base will be reduced causing a reduction in load transfer in skin friction for this zone. In cases where a clay layer is present above the base, the program takes the arithmetic average of those C_u values between the top and the bottom of the clay layer. For a belled shaft the C_u are averaged between the top of the clay layer and to one shaft diameter above the top of the bell (if the bottom of a clay layer is below the depth of one shaft diameter above the top of the bell). However, if the top of the clay layer falls within 5 ft (1.52m) below the ground surface, the C_u average starts from the bottom of 5 ft (1.52m). The user must provide at least one C_u value for each clay layer.

End Bearing - The end bearing resistance for drilled shafts in clay is derived from the work of Skempton (1951) as follows:

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

where:

q_b = unit end bearing for drilled shafts in clay

$N_c = 6.0[1 + 0.2(L/D)] \quad N_c < 9$

C_u = average undrained shear strength of clay for one diameter (1.0D) below the tip.

L = total embedment length of shaft
D = diameter of shaft base.

The limiting value of q_b shown in equation 4 is merely the largest value of end bearing that has been measured for clays and is not a theoretical limit (Engling and Reese, 1974)

SHAFT98 interpolates or extrapolates values of Cu at depths of one base diameter of the shaft, below the base. Interpolation and extrapolation depend on the depth of Cu values input by the user. For the calculation of an average Cu value, the program takes a weighted average of all the Cu values present in above described depth range. An example with hand calculations is shown in Appendix A.

In the case where the shaft base is at the top of a clay layer, SHAFT98 takes a weighted average of Cu values between the base and one base diameter below the base.

In those rare instances where the clay at the base is soft, the value of C_u may be reduced by one-third to account for local (high strain) bearing failure. Furthermore, when the base of the shaft has a diameter greater than 75 inches (1.9 m) consideration should be given to reducing q_b because the settlement required to obtain the ultimate value of q_b will be so great that application of safety factors in the usual range of 2 or 3 may result in excessive short term settlement. It is therefore recommended that for drilled shafts in stiff to hard clay, with D exceeding 75 inches (1.9 m), that the following expressions be used to reduce q_b to q_{br}, where q_{br} is the reduced ultimate end bearing stress, to which appropriate safety factors are applied.

$$q_{br} = F_r \, q_b \qquad \text{(Eqn 5)}$$

where:

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

in which

$a = 0.0071 + 0.0021 \, (L/D_b), \quad a < 0.015$

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

These expressions are based upon load tests of large under-reamed drilled shafts in very stiff clay (O'Neill and Sheikh, 1985) and restrict q_{br} to be the net bearing stress at a base settlement of 2.5 inches (6.35 cm). When half or more of the design load is carried in end bearing and a global factor of safety applied, the global safety factor should not be less than 2.5, unless site specific load tests deem otherwise.

Short-Term Settlement - The reference curves are presented in Figure 2. The marks represent the values proposed by Reese and O'Neill [FHWA (1988)] and the solid lines are the adopted curves. If the short-term settlements or differential settlements appear to be too great the applied loads can be adjusted accordingly. Normally, if the procedures for establishing ultimate loads are followed, short-term settlements should be restricted to less than one inch (2.54 cm.) when appropriate safety factors are applied.

Side friction mobilization

$f_s/f_{smax} = 0.593157*R/0.12$ for $R \leq 0.12$
$f_s/f_{smax} = R/(0.095155+0.892937*R)$ for $R \leq 0.74$
$f_s/f_{smax} = 0.978929-0.115817*(R-0.74)$ for $R \leq 2.0$
$f_s/f_{smax} = 0.833$ for $R > 2.0$

where $R = \frac{S}{D}*100$

For end bearing mobilization the trendline is given as:

$q_b/q_{bmax} = 1.1823E\text{-}4*R^5-3.7091E\text{-}3*R^4+4.4944E\text{-}2*R^3-0.26537*R^2+0.78436*R$ for $R \leq 6.5$
$q_b/q_{bmax} = 0.98$ for $R > 6.5$

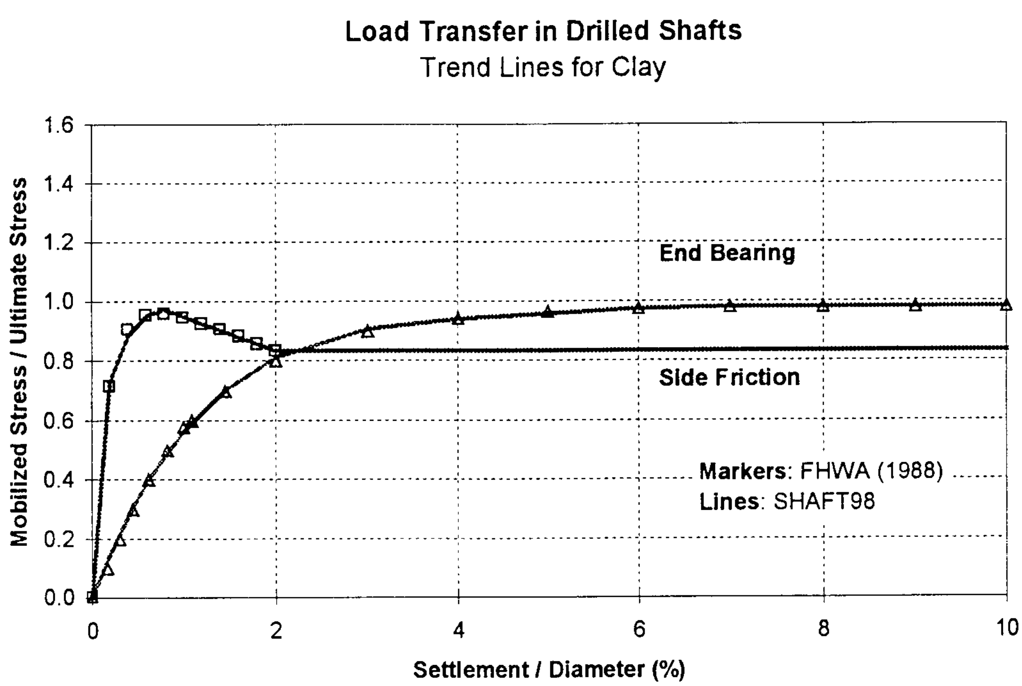


Figure 2 Trend lines for side friction and end bearing in clay

An example is presented in Appendix A for a drilled shaft in clay.

Design for Sand

Side Shear resistance - The unit side resistance, as the drilled shaft is pushed downward is equal to the normal effective stress times the tangent on the interface friction angle. The normal stress at the interface of the drilled shaft and soil will be relatively low when the excavation is completed. 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 a moderate slump (up to 6 inches, 150 mm.) acts hydrostatically over a depth of 10 to 15 ft. (3 to 4.5 m.) and there is a leveling off in the lateral stress at greater depths, probably due to arching (Bernal and Reese, 1983). Concrete with a high slump (about 9 inches, 230 mm.) acts 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. Furthermore, the friction angle of the soil-concrete interface will also be affected by construction details. Consequently, a β method for calculating the unit side shear transfer is use with the following rationale:

$$f_{sz} = K\sigma_z \tan\phi_c \qquad \text{(Eqn 6)}$$

$$Q_s = \int_0^L K\sigma_z \tan\phi_c \, dA \qquad \text{(Eqn 7)}$$

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

Equations 6 and 7can be used in computations, but simpler expressions can be developed by combining the terms for K and $\tan\phi_c$ as β; resulting in:

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

$$Q_s = \int \beta\sigma_z \, dA \qquad \text{(Eqn 8)}$$

$$\beta = 1.5 - 0.135\sqrt{z} \qquad 1.2 > \beta > 0.25 \qquad \text{(Eqn 9)}$$

where
z = depth below ground surface, ft.

The factor β in equation 9 is independent of ϕ (or N_{SPT}) because drilling plus stress relief produces high shearing strains in the sand at the borehole interface, and the friction angle ϕ is forced toward some common critical state value. Thus, the parameter β varies principally with the coefficient of lateral pressure K and experimental studies have shown that this coefficient both for soil and fresh concrete exhibits some decrease with depth.

The limiting value of side resistance in 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.

End Bearing - Because of stress relief when an excavation is drilled into sand, there is a tendency for the sand to loosen slightly at the bottom of the excavation. Also there appears to be some densification of the sand beneath the base of the drilled shaft as settlement occurs. The load-settlement curves that have been obtained by experiment for the base of drilled shafts are consistent with the above concepts. The load continued to increase for some tests to a settlement of more than 15 percent of the base diameter. Such a large settlement could not be tolerated for most structures; therefore, it was decided to limit the values of end bearing for drilled shafts in granular soils to that which would occur at a downward movement of 5 percent of the base diameter.

The values of q_b are tabulated as a function of N_{SPT} (uncorrected field values) in Table 3. However, these values may have to be reduced for large diameter shafts [D> 50 in. (1.3m)], as shown by equation 10.

$q_{br} = 50 * (q_b/D_b)$; D_b in inches

or $q_{br} = 1.3 * (q_b/D_b)$; D_b in meters (Eqn 10)

Table 3 Recommended Unit End Bearing Values for Cohesion's 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 suggests a limiting value of end bearing as 45 tsf (4500 kPa) at a settlement of 5 percent of the base diameter. A value of 58 tsf (5800 kPa) was measured at a settlement of 4 percent of the base diameter in Florida (Owens and Reese, 1982).

In the case where the shaft base is in sand, SHAFT98 uses the basic assumption that the soil 8D above and 3.5D below the shaft base contributes to the end bearing capacity. This assumption differs from O'Neill (1988) in which a single N value at the base characterizes the tip resistance. A weighted average in this 8D - 3.5D range is obtained via equation 11.

$$N_{spt} = \frac{\Sigma N_k L_k}{\Sigma L_k} \quad \text{(Eqn 11)}$$

SHAFT98 needs at least one value of SPT for each sand layer. It then calculates an area average of SPT values between the depth range of 8 shaft diameters above the base and 3.5 base diameters below the base, if no other layer except a sand layer is present in this depth range. If any other soil except sand is present in this range, then it calculates area average of SPT values between top of other layer (in other layer is present below the base), and bottom of other layer (if other layer is present above the base). If a sand layer is present above the base while the shaft is not tipped in sand, SHAFT98 asks for at least one value of SPT for each sand layer. However, SPT values are not required to calculate skin friction, but in case of editing the shaft data, this information may be required.

Immediate Settlements - The immediate settlements are computed using non-linear t-z and Q-z springs, with the shape presented in Figure 3. The equations are provided but is should be referred that there is a considerable scatter around these trend lines.

Side friction mobilization

$f_s/f_{smax} = -2.16{*}R^4+6.34{*}R^3-7.36{*}R^2+4.15{*}R$ for $R \leq 0.908333$

$f_s/f_{smax} = 0.978112$ for $R > 0.908333$

where $R = \frac{S}{D}*100$

End bearing mobilization

$q_b/q_{bmax} = -0.0001079{*}\,R^4+0.0035584{*}\,R^3-0.045115{*}\,R^2+0.34861{*}R$

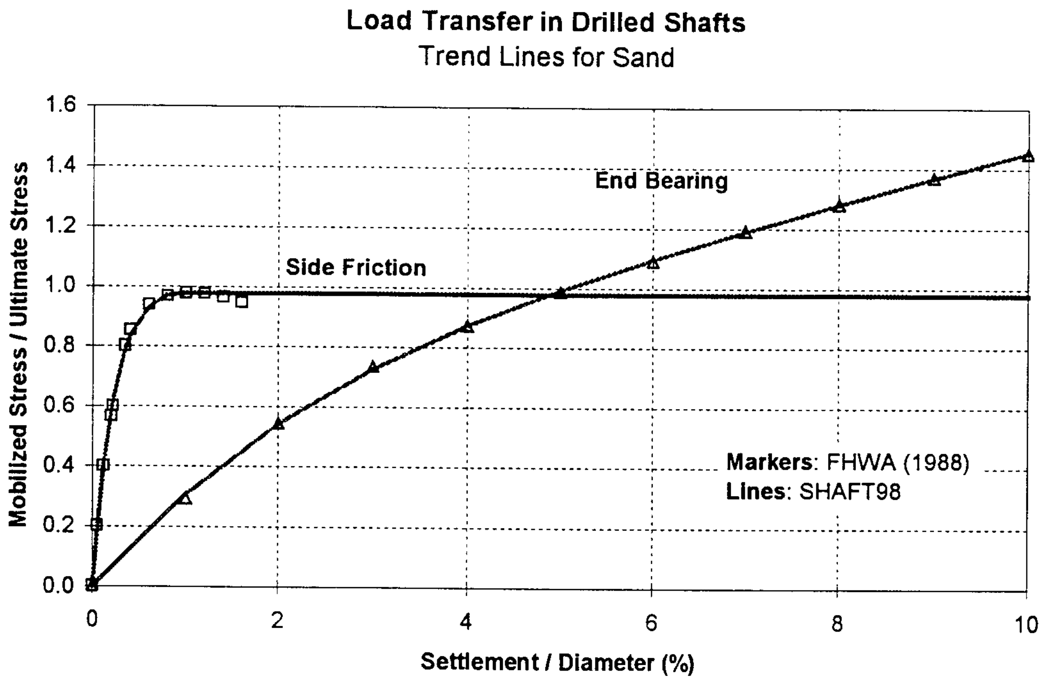


Figure 3 Trend lines for side friction and end bearing in sand

Design for Rock

Side shear resistance - Several equations have been suggested for estimating the ultimate side friction (f_{su}) for drilled shafts in rock. (McVay et al. 1992). They are typically based upon unconfined compression strengths, q_u (α values), or a combination of unconfined and split tensile strengths ($0.5\sqrt{q_u}\sqrt{q_t}$). These correlations listed below may be entered into SHAFT98 as (Note: 1 tsf = 95.8 kPa):

$f_{su} = a\,q_u^b$, a and b are emperical parameters used by authors based upon their experiences (Eqn 12)

1. Williams, et.al. (1980): $f_{su} = 1.842 q_u^{0.367}$
2. Rowe and Armitage (1987): f_{su} (tsf)= 1.45 $\sqrt{q_u}$
   for clean sockets, and f_{su} (tsf) = 1.94 $\sqrt{q_u}$ for rough sockets;
3. Horvath and Kenney (1979): f_{su} (tsf) = 0.67 $\sqrt{q_u}$
4. Carter and Kulhawy (1988): f_{su} (tsf) = 0.63$\sqrt{q_u}$
5. Reynolds and Kaderabek (1980): f_{su} (tsf) = 0.3 (q_u);
6. Gupton and Logan (1984): f_{su} (tsf) = 0.2 (q_u);
7. Reese and O'Neill (1988): f_{su} (tsf) = 0.15 (q_u);
8. Crapps (1986): f_{su} = 0.01N (tsf) or f = -5.54+0.41N (tsf)
9. CIRIA (Hobbs and Healy, 1979)

N value	10	15	20	25	30	>30
f_{su}(tsf)	.36	.77	1.1	1.8	2.6	2.6

10. McMahan (1988)

N Range	10 - 20	20 - 50	50 - 50/3"	>50/3"
f_{su}(tsf)	1.5	2.5	3.8	5

An examination of these methods reveals that in the case of #5, #6 and #7, skin friction is a simple constant times q_u, whereas #1, #2, #3, and #4 use a power curve relationship.

End Bearing - The ultimate end bearing resistance in rock can be calculated as:

$$Q_b = q_{bu} A_b \qquad \text{(Eqn 13)}$$

where

Q_b = ultimate end bearing
q_{bu} = unit end bearing capacity, and
A_b = shaft base area

SHAFT98 uses the Canadian Foundation Manual method of equation 14 to estimate end bearing in rocks. However, sinkhole potential of Florida's karstic terrain, and questions concerning cleanliness of the shaft base if wet hole construction is used, have led some designers to neglect conservatively end bearing of drilled shafts in Florida (i.e., assume $q_{bt} = 0$).

$$q_{bt\,max} = 3\,\Delta\,K_{sp}\,[\,q_u \text{ (beneath base)}] \qquad \text{(Eqn 14)}$$

where:

Δ = depth factor = 1 + 0.4 (L/D) < 3.4, and

$$K_{sp} = (3 + {}^{S_d}\!/_{D}) / (10\sqrt{1 + 300\,{}^{t_d}\!/_{S_d}}$$

in which:

s_d = vertical spacing of horizontal joints beneath base
t_d = thickness of these horizontal joints

s_d = vertical spacing of horizontal joints beneath base
t_d = thickness of these horizontal joints

The application of equation 14 is limited to $0.05 < S_d / D < 2$; $t_d / D < 0.02$ and $D > 0.3$m. If these limiting values of $S_d / D = 0.05$ and $t_d / D = 0.02$ are assumed, then,

$K_{sp} = 0.115$. and

$$q_{bt\,max} = 0.346\,(1 + 0.4\,L/D)\,q_u \qquad \text{(Eqn 15)}$$

Equation 15 is programmed into SHAFT98. However. this value of $q_{bt\,max}$ is limited to $< 2.5\, q_u$.

Short - term settlements in rock - The short-term settlements in rock are estimated using the direct method of O'Neill. et al. (1996) for rough sockets [IGM_Type = 1.0] or smooth [IGM_Type ≠ 1.0].

For side shear resistance:

1. Find the average E_m and f_{su} along the side of the rock socket.

$E_m = \Sigma E_{mk} L_k / \Sigma L_k$ where $E_{mk} = 115\, q_{uk}$

$f_{su} = \Sigma f_{su} L_k / \Sigma L_k$ where f_{su} = side friction from equation 12. Note 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 ($0.5\sqrt{q_u}\sqrt{q_t}$).

2. Calculate Ω

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

where $E_c(\Psi) = 57.000\sqrt{q_{uc}}$

3. Calculate Γ

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

4. Find n

For "rough" sockets;
$n = \sigma / q_u$ where σ = normal stress of concrete = $\gamma_c Z_c M$

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

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.

For "smooth" sockets, n is estimated from Figure 4.

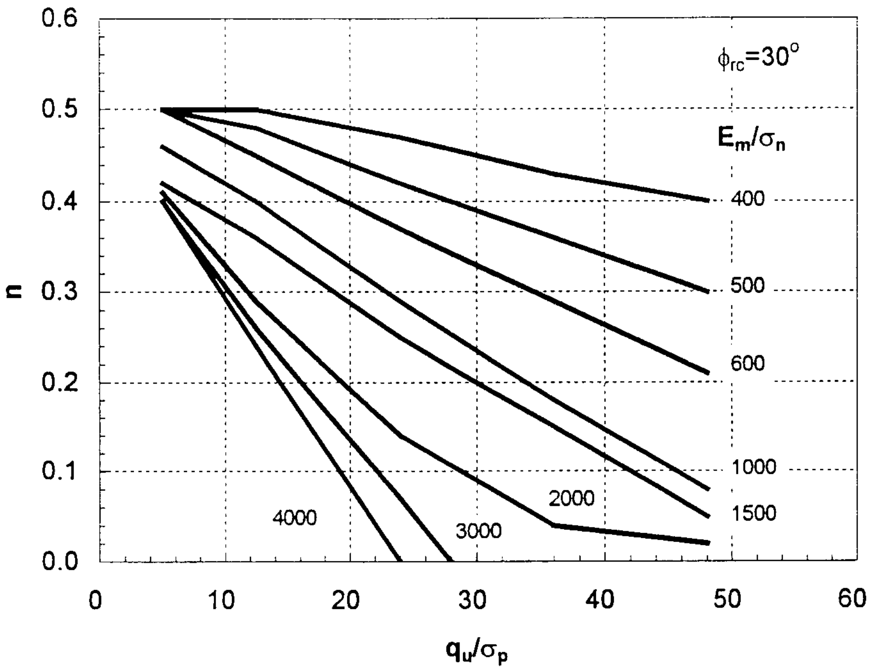


Figure 4 N Factors for Smooth Sockets

5. Calculate Θ_f and K_f

$$\Theta_f = \frac{E_m \Omega}{\pi L \Gamma f_{su}} W_t$$

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

$$\Theta_f = \frac{E_m \Omega}{\pi L \Gamma f_{su}} W_t$$

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

where

W_t = deflection at top of rock socket

6. Calculate the side shear load transfer - deformation as

$$Q_s = \pi D L \Theta_f f_{su} \quad \Theta_f < n$$
$$Q_s = \pi D L K_f f_{su} \quad \Theta_f > n$$

For end bearing short-term settlements in rock sockets, the O'Neill et al. (1996) procedure follows as:

Find $Q_b = \frac{\pi D^2}{4} q_b$

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

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

The total settlement (Qt) for a rock socket would be the sum of $Q_s + Q_b$.

An example for a rock-socketed shaft is presented in Appendix A.

In rare cases where IGM is at the ground surface the first layer should be fictitiously thin, i.e., 0.1 m.

Layered Soils

In the case of alternating layers of clay, sand, or rock, the side resistance is calculated by summing the incremental resistances for each layer. Obviously, the end bearing depends upon the layer in which the base is tipped.

USER'S GUIDE

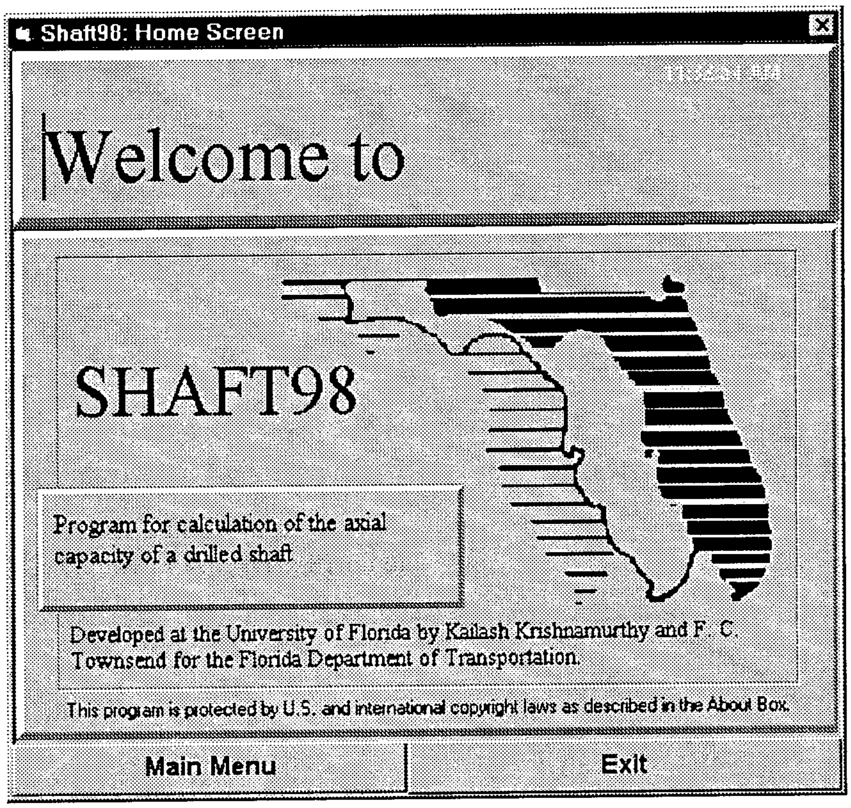


Figure 5

Go to Start Button in Windows 95/NT, Programs, Shaft98. On Clicking Shaft98 Figure 1 pops up. Click on Main Menu. In Main Menu window, click on File, Open and then choose the appropriate input file.

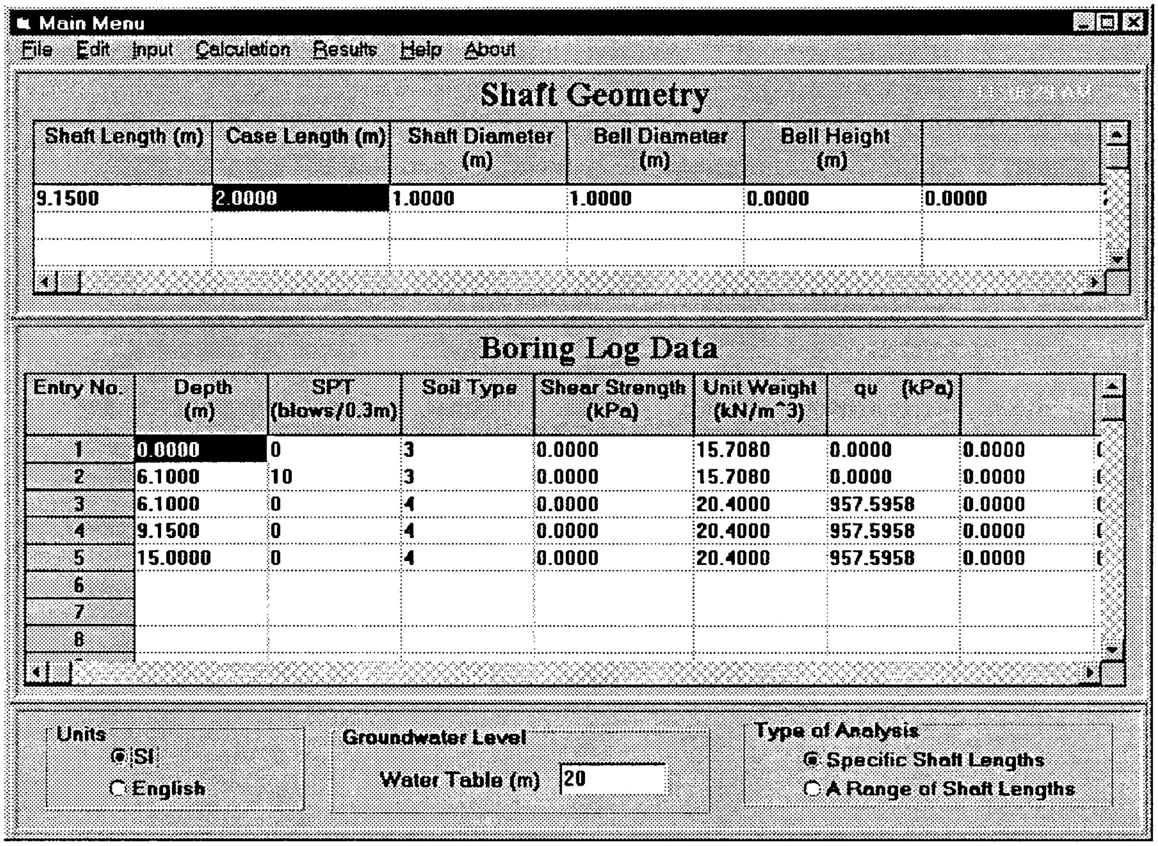


Figure 6

Prior to inputting data, select "Units" in lower left-hand box. Enter "Groundwater location" (depth). Select "Type of Analysis" for either single or range of shaft lengths. Continuing in "Input", enter data pertaining to shaft geometry, and soil properties. "Soil Type"

SHAFT '98 Soil Type	SPT 97 Soil Type	Description
2	1	Plastic Clay
2	2	Clay-silt-sand mixtures
3	3	Clean Sand
4	4	Soft limestone
	5	Void

For SHAFT '98 "Soil Types" relating to cohesionless soils (sands) Type = 3 SPT values **must** be input. For cohesive soils (clays) TYPE = 1 or 2, C_u values are needed. However, these maybe obtained via: (1) direct input, (2) correlations with SPT, or (3) correlations from CPT. For intermediate geomaterials, TYPE = 4, unconfined / compression (q_u) strengths are required.

In order to do the analysis, click on Calculation, Start. Once calculation is done an information dialog box, with OK button shows up. Click the OK button for the next screen (Figure 7)

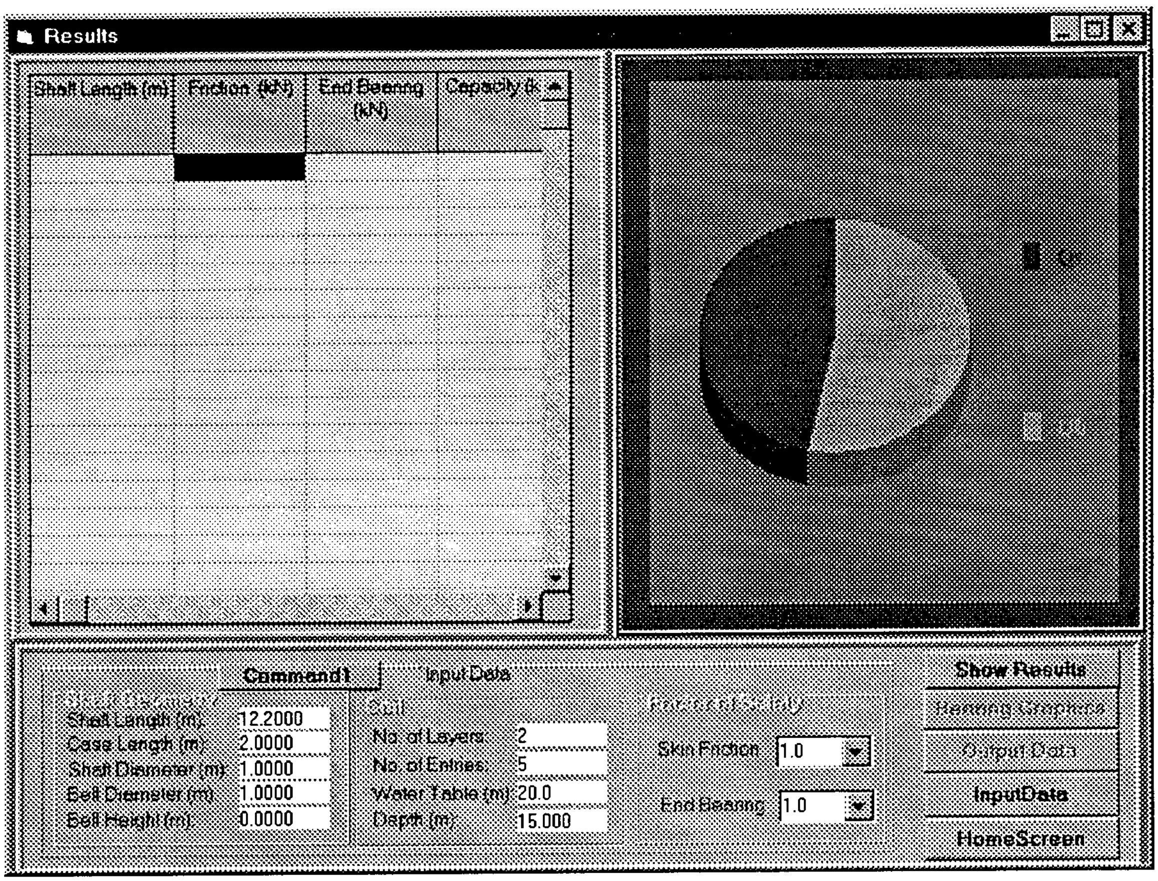


Figure 7

This window displays the shaft geometry, the information of soil layers, and water table. In order to view results, click on Show Results command button. In order to go back to input data or Home Screen windows click on the respective buttons. Once the Show Results button is clicked, Figure 7 looks like Figure 8.

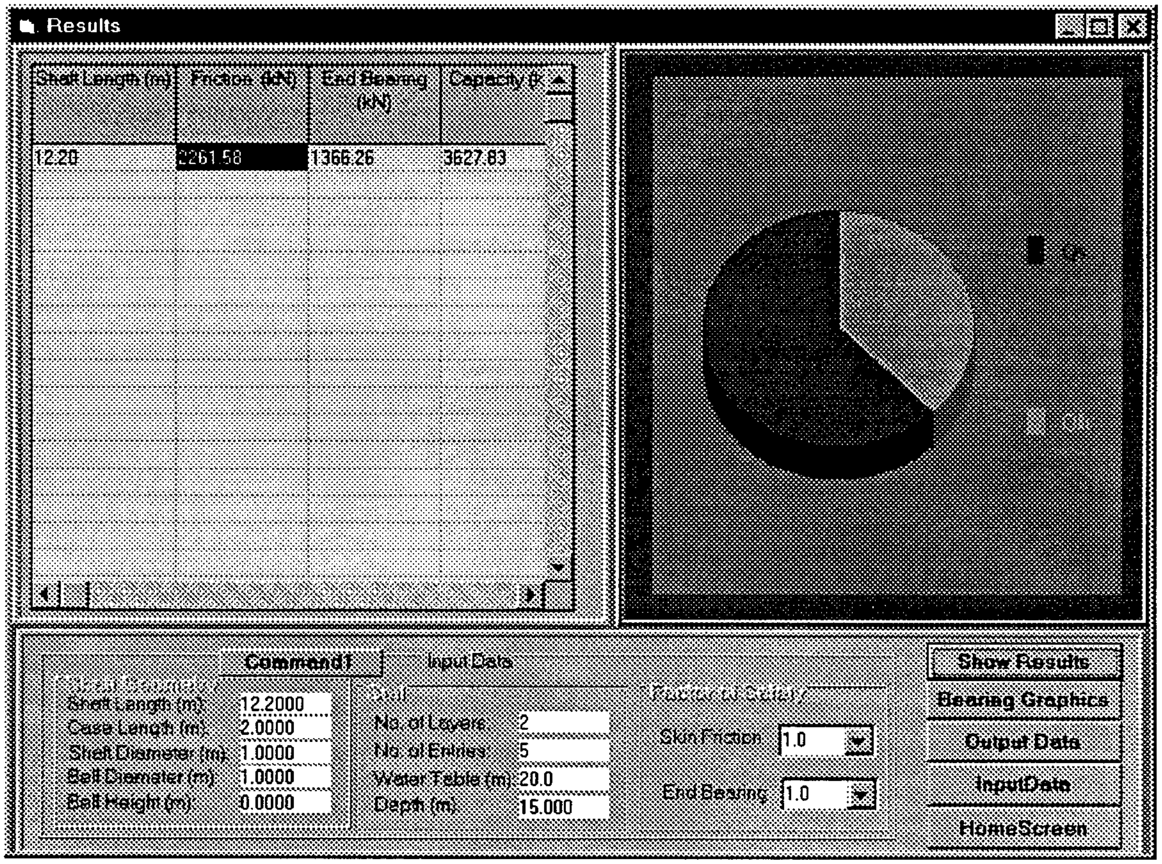


Figure 8

This screen can be used to set "Factors of Safety" on skin friction and/or end bearing. The Bearing Graphics button will provide the graphics illustrated in Figure 9. To obtain the settlement data, use the "input data" button to return to the input data screen (Figure 6), and click results. From there you will be prompted for settlement.

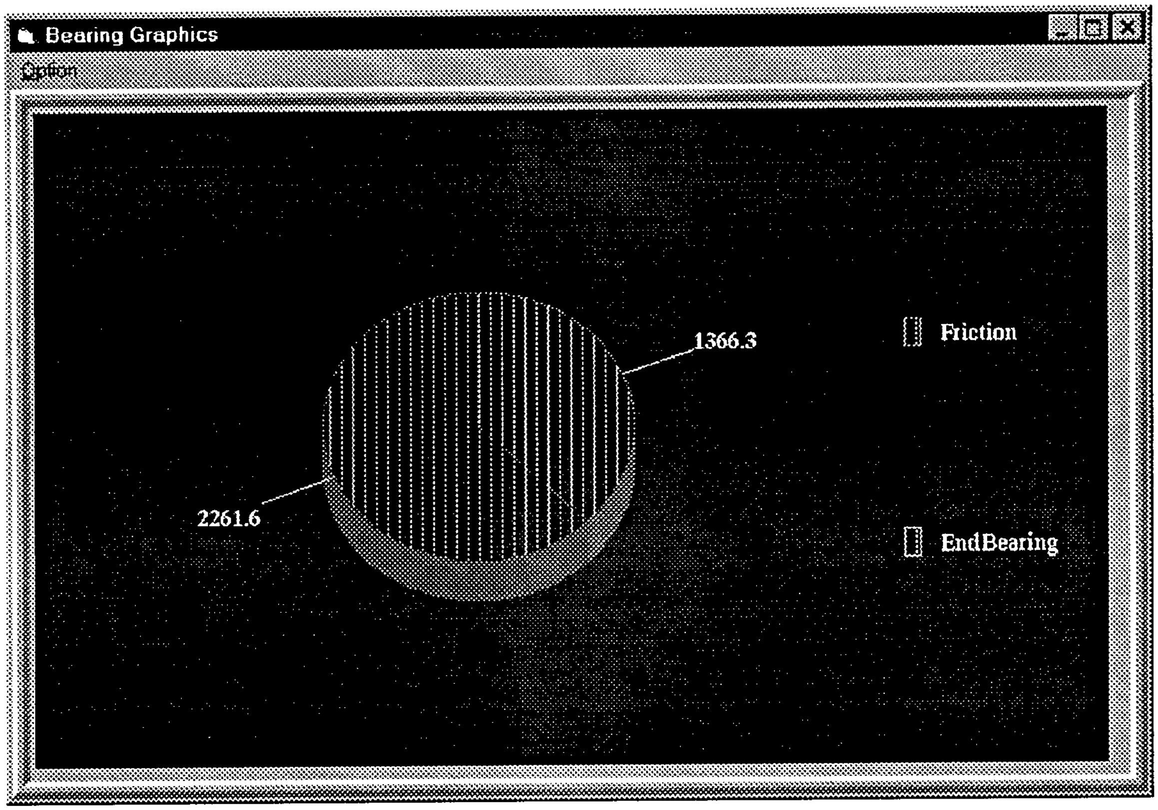


Figure 9

This screen presents the bearing graph showing skin friction and tip resistance percentages. Clicking "options" (upper left) provides a menu for other screens.

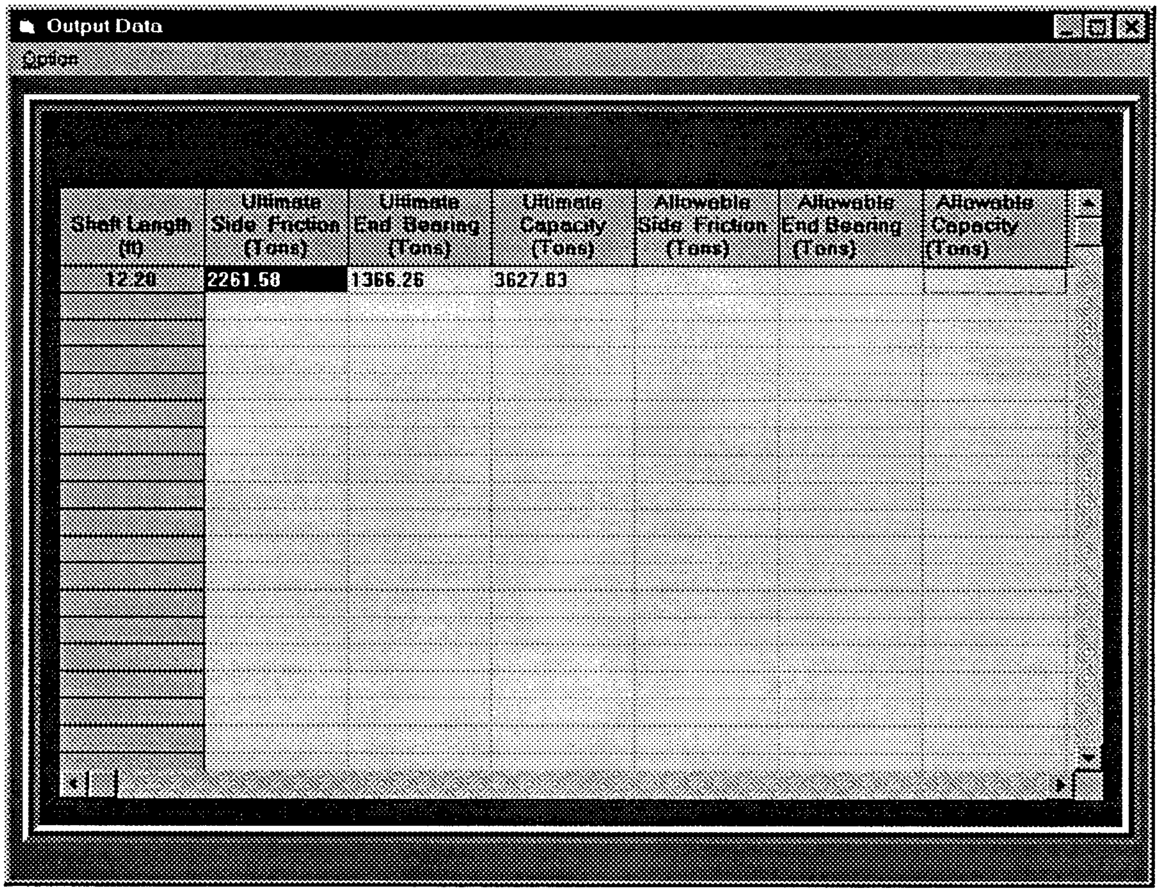


Figure 10

This screen presents the ultimate and allowable, depending upon Factor of Safety values used, friction and end bearing values.

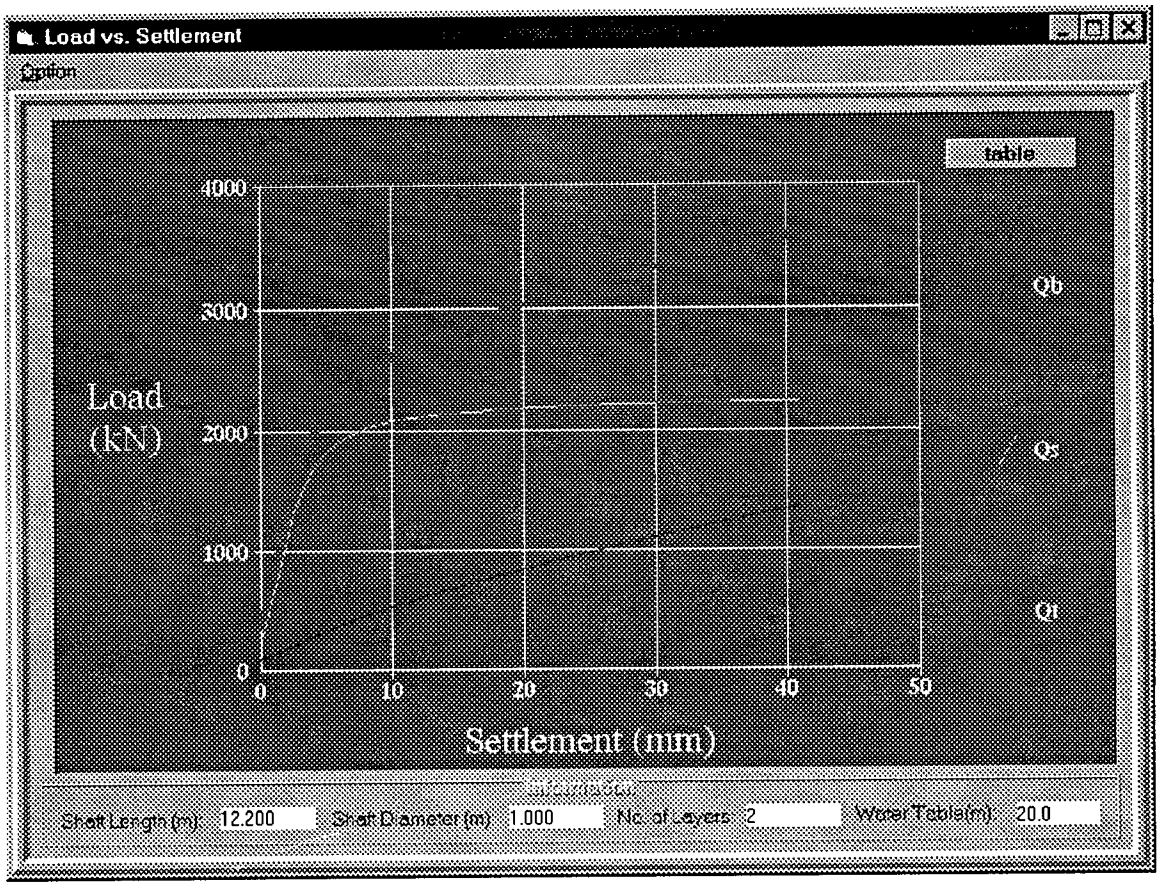
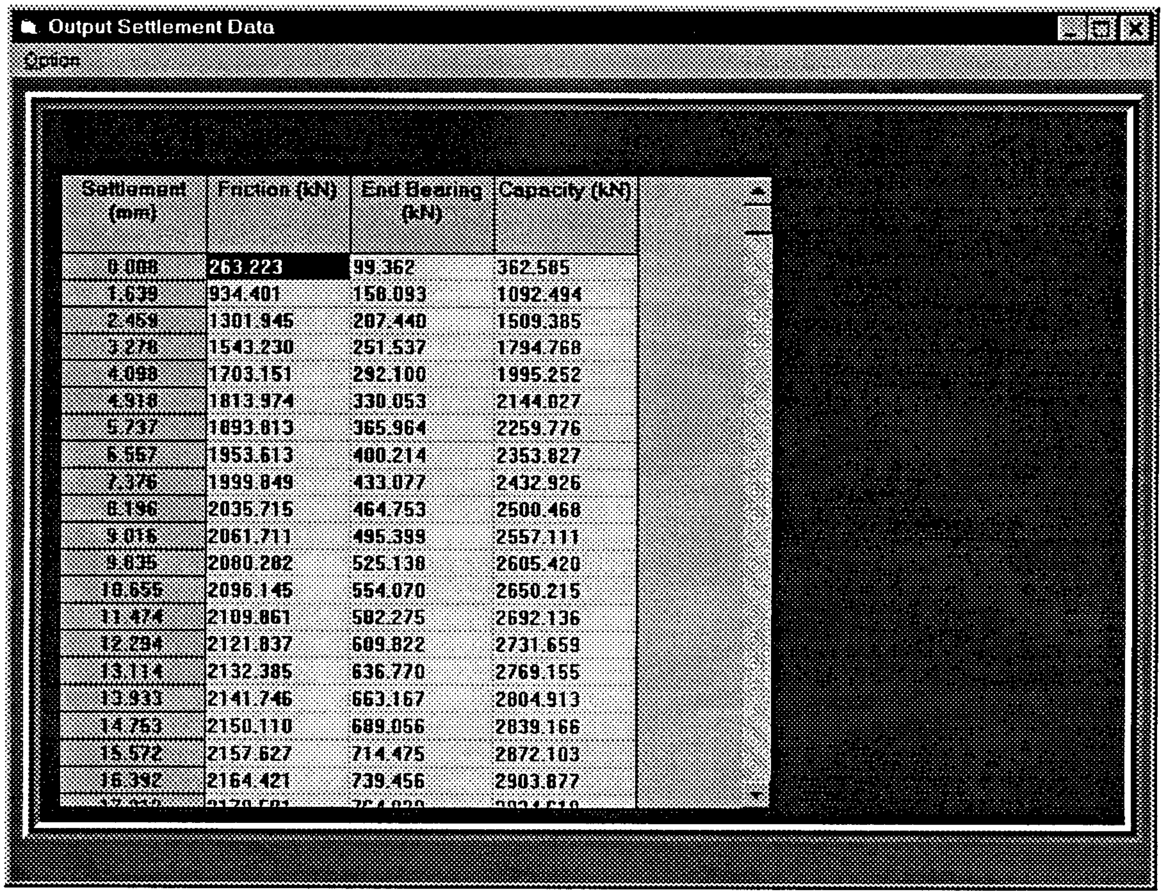


Figure 11

This screen presents the settlement graphics. Clicking the "Table" button will provide the tabular data for these curves as shown in Figure 12.



Settlement (mm)	Friction (kN)	End Bearing (kN)	Capacity (kN)
0.008	263.223	99.362	362.585
1.639	934.401	158.093	1092.494
2.459	1301.945	207.440	1509.385
3.278	1543.230	251.537	1794.768
4.098	1703.151	292.100	1995.252
4.918	1813.974	330.053	2144.027
5.737	1893.813	365.964	2259.776
6.557	1953.613	400.214	2353.827
7.376	1999.849	433.077	2432.926
8.196	2035.715	464.753	2500.468
9.016	2061.711	495.399	2557.111
9.835	2080.282	525.138	2605.420
10.655	2096.145	554.070	2650.215
11.474	2109.861	582.275	2692.136
12.294	2121.837	609.822	2731.659
13.114	2132.385	636.770	2769.155
13.933	2141.746	663.167	2804.913
14.753	2150.110	689.056	2839.166
15.572	2157.627	714.475	2872.103
16.392	2164.421	739.456	2903.877

Figure 12

This screen presents the settlement data in tabular form. It can be obtained by clicking the Table" button upper right (See Figure 11).

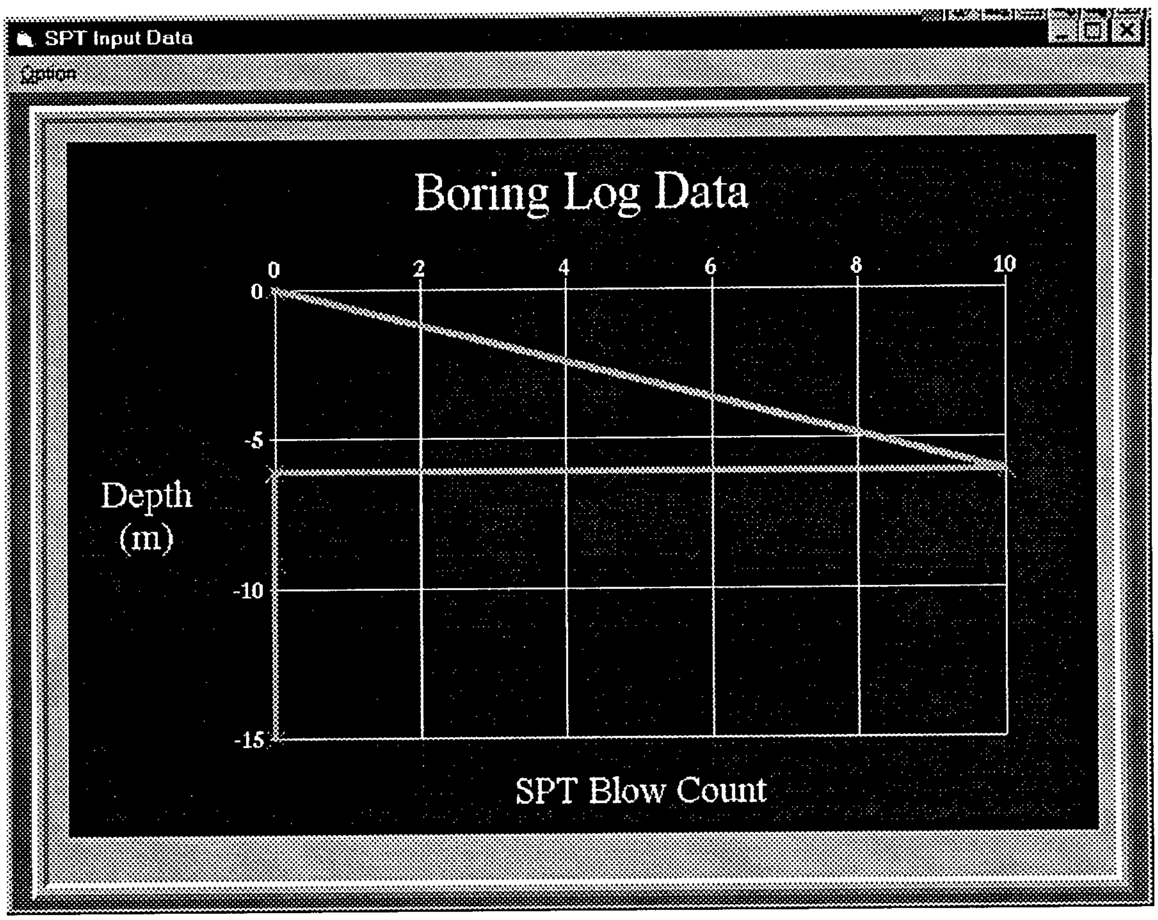


Figure 13

This screen presents the SPT blow count graphics. It can be accessed from the Input Data screen and clicking "Results".

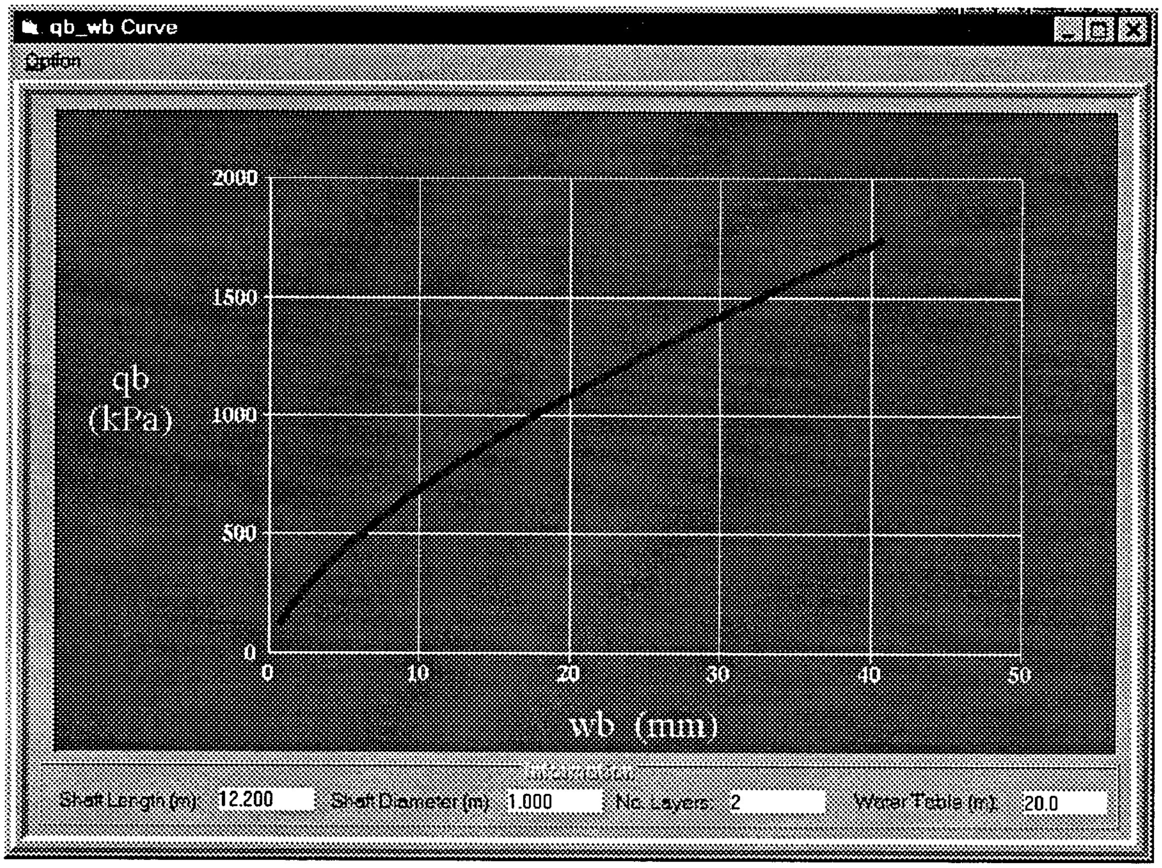


Figure 14

This screen presents the tip resistance VS displacement graphics. It can be accessed from the Input Data screen and clicking "Results".

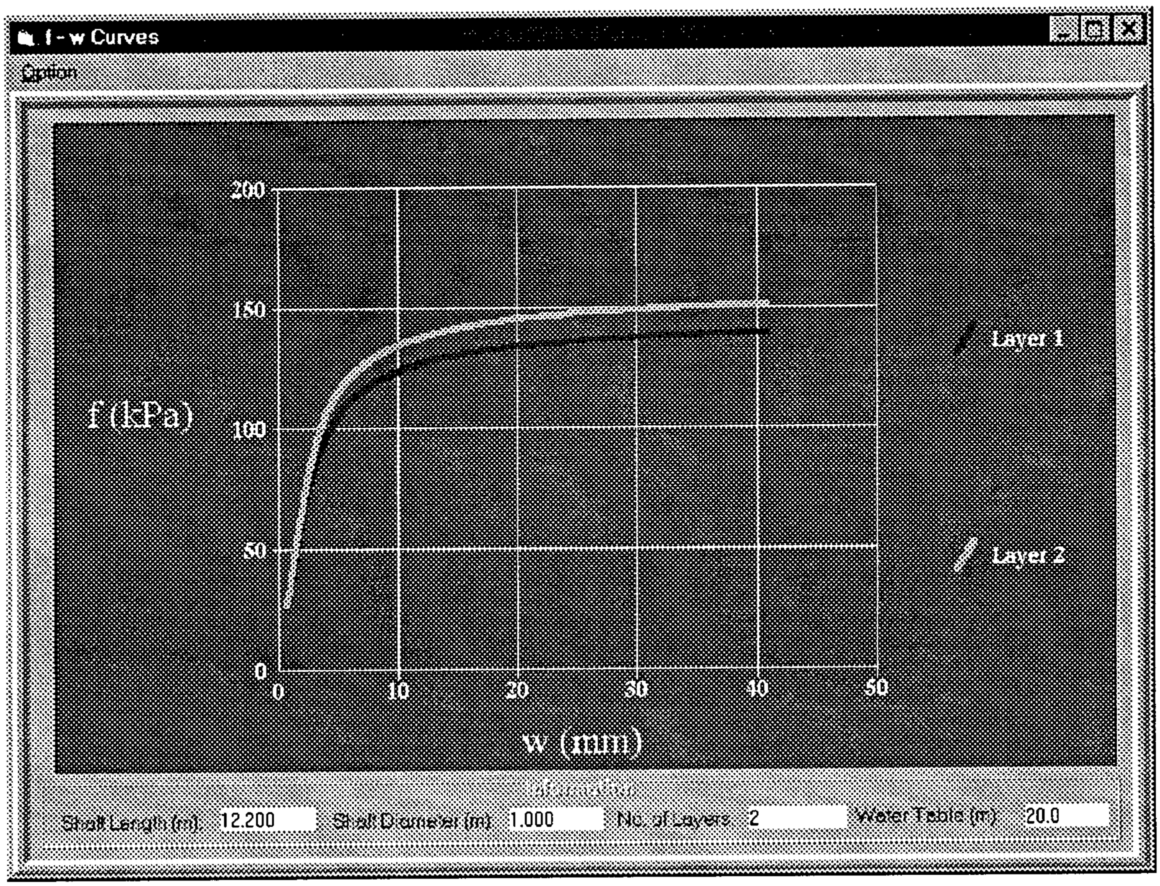


Figure 15

This screen presents the friction VS displacement (t-z) graphics. It can be accessed from the Input Data screen and clicking "Results".

REFERENCES

Bernal, J.B., and Reese. L.C. "Study of the Lateral Pressure of Fresh Concrete as Related to the Design of Drilled Shafts", Research Report 308-1F, Center for Transportation Research, University of Texas, Austin, TX 1983

Carter, J.P., and Kulhawy, F.H., "Analysis and Design of Drilled Shaft Foundations Socketed into Rock", EPRI Report EL-5918, Electric Power Research Institute, Palo Alto, California, 1988.

Crapps, D.K., "Design, Construction & Inspection of Drilled Shafts in Limerock and Limestone", Annual Meeting of Florida Section ASCE, 1986.

Ellison, R.D., D'Appolinia, E., and Theirs, G.R., "Load-Deformation Mechanism for Bored Piles", Journal of Soil Mechanics, ASCE, Vol. 97, No. SM4, April 1971, pp. 661-678.

Engleing, D., Reese, L.C., "Behavior of Three Instrumented Drilled Shafts under Short Term Axial Loading", Research Report 176-3, Conducted at the Center for Highway Research, University of Texas, Austin TX, for FHWA and Texas Highway Department, May 1974, 116 Pages.

Gupton, C., and Logan T., "Design Guidelines for Drilled Shafts in Weak Rocks of South Florida", South Florida Annual ASCE Meeting , Miami, 1984.

Hobbs, N.B., and Healy, P.R., "Piling in Chalk", Doe and CIRIA Piling Development Group, PSA, London, 1979.

Horvath, R.G., and Kenney, T.C., "Shaft Resistance of Rock Socketed Drilled Piers", Symposium on Deep Foundations, ASCE National Convention, Atlanta, Georgia, 1979.

McMahan, B., "Drilled Shaft Design and Construction in Florida", Department of Civil Engineering, University of Florida, Florida, 1988.

McVay, M.C., Townsend, F.C., and Williams, R.C., "Design of Socketed Drilled Shafts in Limestone", ASCE Journal of Geotechnical Engineering, Vol. 118, No. 10, pp. 1626-1637, 1992.

O'Neill, M.W., and Sheikh, S.A., "Geotechnical Behavior of Underreams in Pleistocene Clay, Drilled Piers and Caissons II, ASCE, May 1985 pp. 57-75.

O'Neill, M. W., Townsend, F.C., Hassan, K.M., Buller, A., and Chan, P.S., "Load Transfer for Drilled Shafts in Intermediate Geomaterials" FHWA - RD-95-172. 1996.

Owens, M.J. and Reese, L.C., "The Influence of a Steel Casing of the Axial Capacity of a Drilled Shaft" Research Report 255-1F, Report to the Texas State Highway Department, Center of Transportation Research, University of Texas, Austin, TX. July 1982, 204 pages.

Parra, F., Townsend, F.C., McVay, M.C., and Martinez, R.E., "Design Guidelines for Shaft Foundations, Final Report", submitted by the Department of Civil Engineering, University of Florida to the Florida Department of Transportation, July 1990.

Reese, L.C., and O'Neill, M.W., "Drilled Shafts: Construction and Procedures and Design Methods", Report No. FHWA-HI-88-042, Federal Highway Administration, 1988.

Reynolds, R.T., and Kaderabek, T.J., "Miami Limestone Foundation Design and Construction", ASCE, Florida Convention, Preprint No. 80-546, 1980.

Rowe, R.K., and Armitage, H.H., "A Design Method for Drilled Piers in Soft Rock", Canadian Geotechnical Journal, Vol. 24, 1987a.

Rowe, R.K., and Armitage, H.H., "Theoretical Solutions for Axial Deformation of Drilled Shafts in Rock", Canadian Geotechnical Journal, Vol. 24, 1987b.

Skempton, A.W., "The Bearing Capacity of Clay", Proceedings Bld. Research Congress, Div. I, Bld Res. Cong., London, 1951.

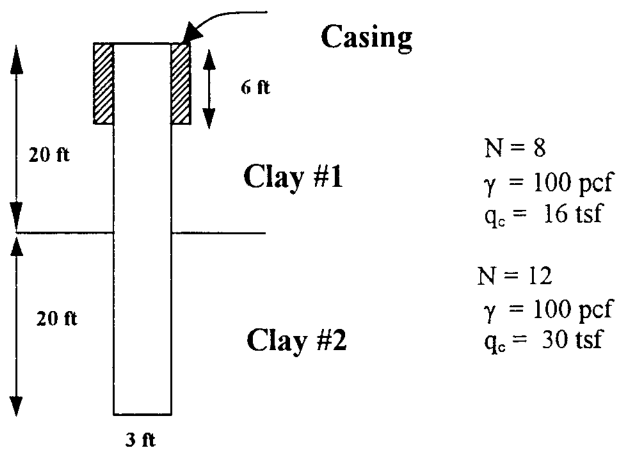
Williams, A.F., "Principles of Resistance Development in Rock Socketed Piles", Foundations on the Melbourne Mudstone - Rock Socketed Piles, Department of Civil Engineering, Monash University, Melbourne, Australia, February 1980.

APPENDIX A - Examples

CLAYS:
Example File: Clay1.dat

1. Multi Layer Clay with Casing

2. Multi Layer Clay with Casing D > 75 ”



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

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

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

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

a) Skin Friction:

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

b) End Bearing:

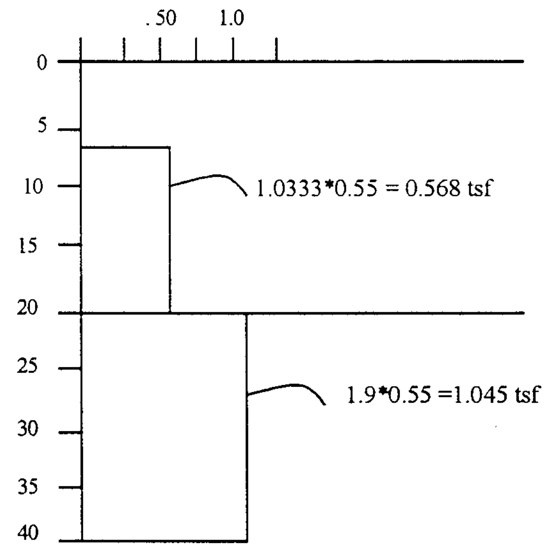
$$Q_b = q_b . \frac{\pi \; b^2}{4},$$
$$q_b = N_c C_u,$$

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

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

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

d) Calculation of Skin Friction:



e) Settlement: S = (i) 0.3 " and (ii) S = 3.0 "

$$Q_s = 242.42^T, \ Q_b = 120.87^T, \ Q_T = 363.24^T$$

(i)

$$\frac{S*100}{D} = \frac{0.3*100}{36} = 0.833 > 0.74, \quad q_{st} = 0.978929 - 0.115817(0.833 - 0.74) * 242.42 = 234.70$$

$$\frac{q_{br}}{Q_b} = 1.1832E - 04 * (0.833)^5 - 3.7091E - 03(0.833)^4 + 4.4944E - 02(0.833)^3 - 0.26537(0.833)$$

$$0.78436(0.833)$$

$$= 0.4935 * 120.87 = 59.65 \ T$$

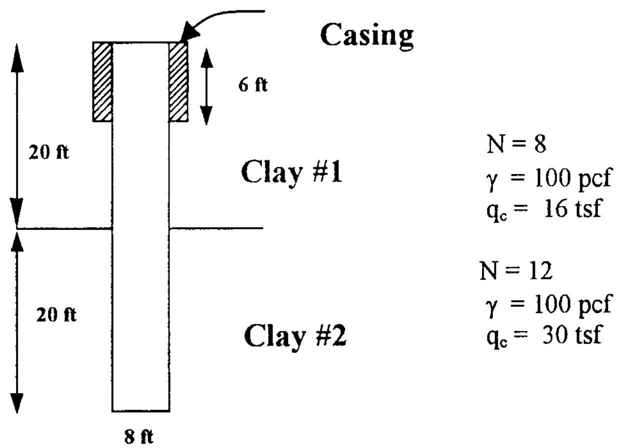
Q_T @0.3" = 234.70 + 59.65 = 294.35 T

(ii) $\dfrac{S*100}{D} = \dfrac{3.0*100}{36} = 8.33 > 0.74$, $q_{st} = 0.833*242.42 = 201.93^{T}$

$\dfrac{q_{br}}{Q_b} = 0.98*120.87 = 118.45T$

Q_T @ 3.0" = 201.93 + 118.45 = 320.38 Tons

2. Multi Layer Clay with Casing, but D>75" (1.9m):
Example File: Clay2.dat



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

$= 25.1327*[7.9567+12.5]$

$= 515.14\ Tons$

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

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

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

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

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

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

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

$Q_t = 515.14 + 591.48 = 1106.62$ *Tons*

e) Settlement: S = (i) 0.3 " and (ii) S = 3.0 "

$Q_s = 515.14^T$, $Q_b = 591.48^T$, $Q_T = 1106.62^T$

(i) $$\frac{S * 100}{D} = \frac{0.3 * 100}{96} = 0.3125$$

$$\frac{q_{st}}{Q_s} = \left(\frac{100 * S}{D}\right) * \left(\frac{1}{0.095155 + 0.892937 * \left[\frac{100 * S}{D}\right]}\right)$$

$$\frac{q_{st}}{Q_s} = (0.3125) * \left(\frac{1}{0.095155 + 0.892937 * [0.3125]}\right)$$

$$\therefore q_{st} = 0.8325 * 515.14 = 428.85^T$$

$$\frac{q_{br}}{Q_b} = 1.1832\mathrm{E} - 04 * (0.3125)^5 - 3.7091\mathrm{E} - 03(0.3125)^4 + 4.4944\mathrm{E} - 02(0.3125)^3 - 0.26537(0.3125)^2 + 0.78436(0.3125)$$

$= 0.2205 * 591.48 = 130.44\ T$

Q_T@ 0.3" = 428.85 + 130.44 = 559.29 Tons

(ii) $$\frac{S * 100}{D} = \frac{3.0 * 100}{96} = 3.125$$

$$\frac{q_{st}}{Q_s} = 0.833, \quad \therefore q_{st} = 0.833 * 515.14 = 429.11^T$$

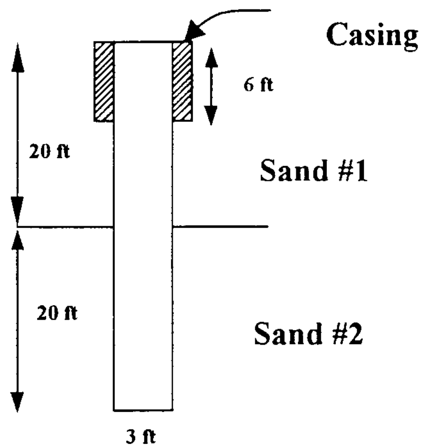
$$\frac{q_{br}}{Q_b} = 1.1832E - 04 * (3.125)^5 - 3.7091E - 03(3.125)^4 + 4.4944E - 02(3.125)^3 - 0.26537(3.125)^2 + 0.78436(3.125)$$

$= 0.9127 * 591.48 = 539.85\ T$

Q_T@3.0" = 429.11+539.85 = 986.96 Tons

SANDS:

Example file: Sand1.dat



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

1. Skin Friction:

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

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

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

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

2. End Bearing: above 8*D and below 3.5*D ,

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

below: 40.0 + 3.5*D = 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

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

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

$Q_T = 297.5 + 59.62 = 357.12$

2. Check settlements: (a) S = 0.3'' and (b) S = 1.44"

R=S * 100/D = 0.3*100/36 = 0.833 & R = S*100/D = 1.44*100/36 = 4.00

a. For R = 0.833

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

$= -2.16*(0.833)^4 + 6.34*(0.833)^3 - 7.36*(0.833)^2 + 4.15*(0.833)$
$= 0.9745$

$Q_s = 297.5 * 0.9745 = 289.91^T$

$q_{bt} / Q_b = -0.0001079*(.833)^4 + 0.0035584*(.833)^3 - 0.045115*(.833)^2 + 0.34861*(.833)$
$= 0.2796$

$\therefore Q_b = 0.2796*59.62 = 16.67^T$

b. For R = 4.00

$q_{st} / Q_s = 0.978112$

$Q_s = 297.5 * 0.978112$

$= 290.99^T$

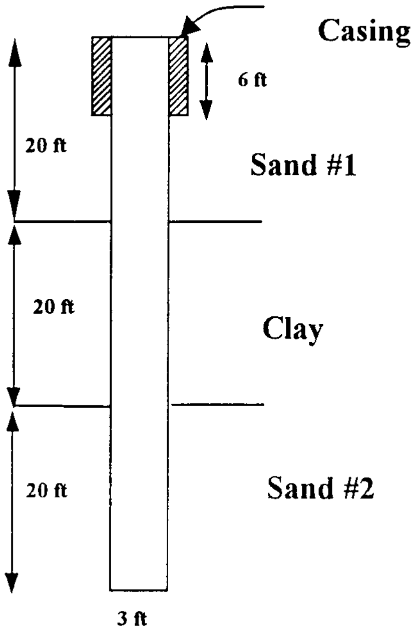
$q_{bt} / Q_b = -0.0001079*(4.00)^4 + 0.0035584*(4.00)^3 - 0.045115*(4.00)^2 +$

$0.34861*(4.00)$
$= 0.8727$

$\therefore Q_b = 0.8727*59.62 = 52.03^T$

MULTILAYER- SAND-CLAY-SAND:

Example File: Sand_c.dat



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	

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

$$= 0.0047\left[\frac{150*z^2}{2} - 13.5*z^{5/2}*\frac{5}{2}\right]_6^{20}$$

$$=0.0047\left[75*(20^2-6^2)-5.4*(20^{5/2}-6^{5/2})\right]$$

$$= 0.0047[27,300-9,183.6]$$

$$= 85.371^{T}$$

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

$$= \quad 196.978^{T}$$

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

$$= 0.0047\left[\frac{150*z^2}{2} - 13.5*z^{5/2}*\frac{5}{2}\right]_{40}^{60}$$

$$= 0.0047\left(75*(60^2-40^2)-5.4*(60^{5/2}-40^{5/2})\right)$$

$$= 0.0047[150,000-95,937.4]$$

$$= 254.764^{T}$$

$\Sigma\ Q_s = 85.371+196.978+254.764 = 537.11$ tons

4. Tip Resistance : above 8*D and below 3.5*D,

Above: 60.0 - 8*D = 60.0 - 8*(3)= 36 ft ;
Below: 60.0 + 3.5*D = 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, $\therefore$ stop @ 40ft.

5. Settlement : (a) S = 0.3 inches

R=S*100/D = 0.3*100/36 = 0.833

Sand (0-20 ft) : $q_{st} / Q_s = -2.16{*}R^4 + 6.34{*}R^3 - 7.36{*}R^2 + 4.15{*}R$

$= -2.16{*}(0.833)^4 + 6.34{*}(0.833)^3 - 7.36{*}(0.833)^2 + 4.15{*}(0.833)$

$= 0.9745$

$q_s = 0.9745 * 85.371$

$= 83.197^T$

Clay (20-40 ft): $q_{st} / Q_s = 0.978929-0.115817(R-0.74)$

$q_{st}= 0.978929-0.115817(0.833-0.74) * 196.978 = 190.706^T$

Sand (40-60 ft): $q_s = 0.9745 * 255.117$

$= 248.611^T$

$q_{bt} / Q_b = -0.0001079{*}(.833)^4 + 0.0035584{*}(.833)^3 - 0.045115{*}(.833)^2 +$

$0.34861{*}(.833)$

$= 0.2796$

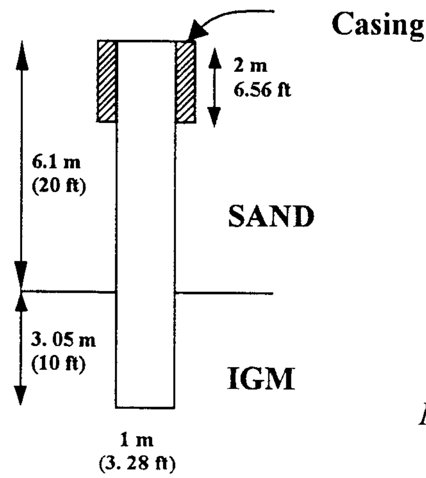
$\therefore\ Q_b = 0.2796*63.617 = 17.787^{T}$

$\therefore$ When S = 0.3 inches,

$Q_s = 83.197+190.706+248.611=522.51 + Q_t = 17.787^{T}$

$= 540.29$ tons.

IGM: (Sand & Limestone)
File: IGM_S.dat



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

3. Analysis of Rock resistance has 2 design methodologies:

(a) UF method: Skin Friction defined as

(1) Williams $f_{su} = 1.842q_u^{\ 0.367}$

(2) McVay $f_{su}(tsf) = \dfrac{1}{2}\sqrt{q_u}\sqrt{q_t}$

(3) User defined.

and Tip resistance is user defined, typically $q_b = q_u / 2$. No settlements can be calculated using this method.

(b) O'Neill (FHWA) intermediary geo-materials method, this method is deformation based.

4. UF method (Rock): (Note: Must enter values for q_u and q_t)

$$Q_s = \pi\ d\ L\ f_{su} = \pi\ (1m)\ (3.05m)\ (0.5\sqrt{95.76}\sqrt{95.76}) = 1450.79\ kN$$

$\Sigma Q_s = 811.66 + 1450.79 = 2262.45$ kPa

Assuming $q_b = ½\ q_u$: $Q_t = \dfrac{\pi(1)^2}{4}(0.5 * 957.6) = 376kN$

Then, $Q_s + Q_t = 2638.45$kN

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

   a. $E_m = 115\ q_u = 115\ (0.96\ \text{MPa}) = 110.4$ MPa.

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

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

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

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

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

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

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

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

   $$= 1.1141[4.7757]^{0.67}$$

   $$= 3.159\ \text{MPa m}^{-0.67}$$

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

$$= 1.1141[0.1316]$$

$$= 146.65 \text{ kPa mm}^{-0.67}$$

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

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

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

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

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

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,

$\therefore \theta$ = 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,

$\therefore \theta$ = 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. R = (s*100/D);

   @ 2mm R=(0.2cm*100/100cm) = 0.2

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

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

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

   = 0.5829 for w = 2mm

   q_s= 0.5829 * (811.66 kN)

   = 473.1 kN for 2 mm

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

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

   = 0.892 for w = 5mm

3. q_s = 0.892 * (811.66 kN)

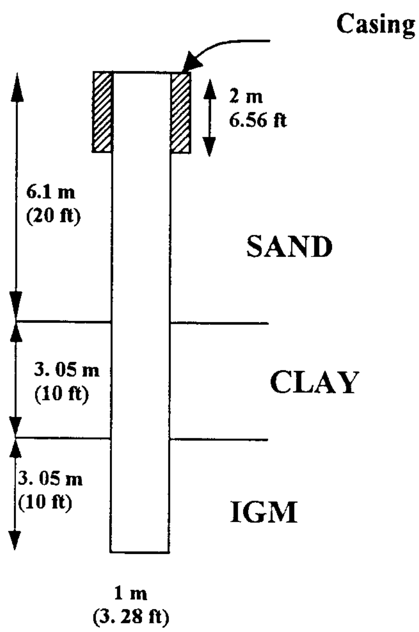
   = 724.4 kN for 5 mm

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

IGM: (Sand, Clay & Limestone)
File: S_C_LIMEROCK.DAT



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

$$E_c = 57{,}000\sqrt{f_y'} = 57{,}000\sqrt{5000 psi}$$

$$= 4.03E6psi \ \ (27.77E6 \ kPa)$$

$$f_{su} = \tfrac{1}{2}\sqrt{q_u}\sqrt{q_t} = 151.41 \ kPa$$

Smooth socket IGM_type = 2.0

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

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

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

$= 0.00515[26{,}772.5\text{-}9064.6]$

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

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

= 958.85 kN (107.78^T)

3. Skin Friction (Rock): (Note: UF method needs q_u and q_t)

$Q_s = \pi$ D L f_{su} = π (1) (3.05) (151.41 kPa) = 1450.8 kN (163.075^T)

4. End Bearing (Rock):

$$Q_b = \frac{\pi\ d^2}{4} q_b; \quad Let \quad q_b = 0.5\ q_u$$

$$= \pi * (1) * (0.25) * (0.5 * 957.6\ kPa)$$

$$= 376.05\ kN \quad (42.27^T)$$

5. Summary: Q_T $= 811.66 + 958.85 + 376.05 = 3{,}597.4$ kN (404.36^T)

No settlement in this method.

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

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

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

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

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

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

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},

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

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

$$= 636.85 + 182.8$$

$$= 819.2\ kPa$$

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

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

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

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

$$= 1084.6 + 335.9$$

$$= 1420.5\ kPa$$

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

1. R = (s*100/D);

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

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

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

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

$= 0.5829$ for w = 2mm

3. q_s = 0.5829 * (811.66 kN)

= 473.1 kN for 2 mm

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

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

$= 0.892$ for w = 5mm

3. q_s = 0.892 * (811.66 kN)

= 724.4 kN for 5 mm

4. Clay: R = s*100/D; @ 2 mm R=0.2 & 0.5 @ 5 mm 0.12 < R < 0.74

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

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

q_s = 0.7310*958.85 = 700.55 kN @ 2 mm

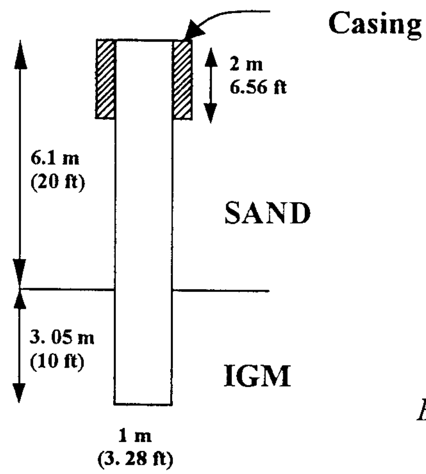
q_s = 0.9232*958.85 = 885.16 kN @ 5 mm

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

IGM: (Sand & Limestone) Consider "Rough" Socket:



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

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

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

=0.00515[26,772.5-9064.6]

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

2. 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" $n = \sigma_n / q_u$

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

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

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

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. R = (s*100/D); @ 2mm R=(0.2cm*100/100cm) = 0.2

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

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

   = 0.5829 for w = 2mm

3. q_s = 0.5829 * (811.66 kN)

   = 473.1 kN for 2 mm

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