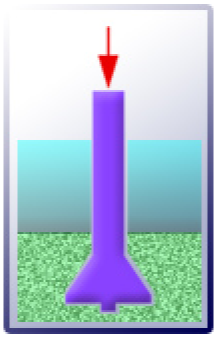
Welcome to FB-Deep



The FB-Deep computer program is a Windows based program used to estimate the static axial capacity of drilled shafts and driven piles. The drilled shaft methodology is based upon Federal Highway Administration reports. Driven pile methodology utilizes two types of analyses: SPT and CPT. SPT methodology is based on empirical correlations between cone penetrometer tests and standard penetration tests for typical Florida soil types. Unit end bearing resistance and unit skin friction resistance versus SPT N values are given in the FDOT research bulletin RB-121, for the different soil types. Driven pile capacity calculated using CPT data can be determined by three separate methods. The first method is the Schmertmann method proposed by Schmertmann in 1978 (AASHTO LRFD Bridge Design Manual). The second method is the LCPC method proposed by Bustamante and Gianeselli for the French Highway Department in 1982. The third method is the UF method proposed by Bloomquist, McVay and Hu for the FDOT in 2007.

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

Introduction

The FB-Deep computer program is a Windows based program used to estimate the static axial capacity of drilled shafts and driven piles.

The drilled shaft 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, qu 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).

Driven pile methodology utilizes two types of analyses: SPT and CPT. SPT methodology is based on empirical correlations between cone penetrometer tests and standard penetration tests for typical Florida soil types. Unit end bearing resistance and unit skin friction resistance versus SPT N values are given in the FDOT research bulletin RB-121, for the different soil types.

Driven pile capacity calculated using CPT data can be determined by three separate methods. The first method is the Schmertmann method proposed by Schmertmann in 1978 (AASHTO LRFD Bridge Design Manual). The second method is the LCPC method proposed by Bustamante and Gianeselli for the French Highway Department in 1982. The third method is the UF method proposed by Bloomquist, McVay and Hu for the Florida Department of Transportation in 2007.

FB-Deep replaces earlier versions of ShaftSPT97. ShaftSPT97 replaced SHAFTUF and SHAFT93 and SPT97.

1 Drilled Shafts

1.1 Shaft: Method of Analysis

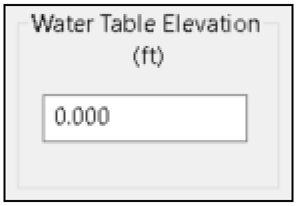
The axial capacity of drilled shafts can be calculated as:

Equation: 1.1.a
$Q_t = Q_s + Q_b$

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 SPT97 program. There is NO skin friction contribution along the length of the casing.

1.2 Water Table Elevation



The water table elevation is relative to mean sea level. For example, if the inputted ground surface elevation is 50 feet, and the inputted water table elevation is –10 feet, this would mean the water table elevation is 60 feet below the ground surface.

1.3 Design for Clay

1. Shear Transfer
2. End Bearing
3. Short-Term Settlement

1.3.1 Shaft: Clay Shear Transfer

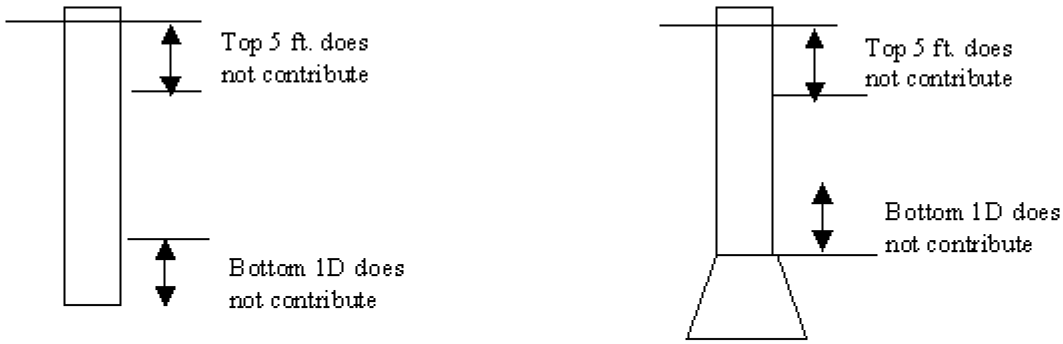


Figure: 1.3.a Portions of Drilled Shaft Non-Contributory in Friction

Location Along Drilled Shaft	Value of α	Maximum Value of fsu (tsf)
From ground surface to depth of 5 feet (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)

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 (Equation: 1.3.a) used to compute the ultimate value if unit load transfer at the depth z below the ground surface.

$f_{su} = \alpha C_u$

Equation: 1.3.a

where

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

α = empirical factor that varies with depth, (see above table and Figure: 1.3.a) and

C_u = undrained shear strength at depth z, The total load Q_s in side resistance is now computed as:

$$Q_s = \int_{L_1}^{L_2} f_{su} dA$$

Equation: 1.3.b

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: 1.3.a illustrates the zones where α is assumed to be zero. The setting of α = 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.

1.3.2 Shaft: Clay End Bearing

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

$q_b = N_c C_u$, $q_b < 40$ tsf (4000 kPa)

Equation: 1.3.c

where:

q_b = unit end bearing for drilled shafts in clay

$N_c = 6.0[1 + 0.2(L/B)]$ $N_c < 9$

C_u = average undrained shear strength of clay for 3.0 B below the tip

L = total embedment length of shaft

B = diameter of shaft base.

The limiting value of q_b shown in Equation 1.3c is merely the largest value of end bearing that has been measured for clays and is not a theoretical limit (Engling and Reese, 1974) .

FB-Deep 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.

For the calculation of an average Cu value, the program takes an 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, FB-Deep takes an area average of Cu values between the base and three diameter widths 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 B 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.

$q_{br} = F_r \, q_b$

Equation: 1.3.d

where:

$F_r = 2.5/[aB \text{ (inches)} + 2.5\, b]$ $F_r < 1.0$

in which

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

$b = 0.45\,(C_{ub})^{0.5}$, $0.5 < b < 1.5$ 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.

1.3.3 Shaft: Clay Short-Term Settlement

The reference curves are presented in Figure: 1.3.b. The marks represent the values proposed by Reese and O'Neill [FHWA (1988)] and the solid lines are the adopted curves. It should be observed that a considerable scatter is present around these 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 ≤ 0.12

f_s/f_{smax} = R/(0.095155+0.892937*R)   for R ≤ 0.74

f_s/f_{smax} = 0.978929-0.115817*(R-0.74)   for R ≤ 2.0

f_s/f_{smax} = 0.833   for R > 2.0

where R = 100 * Settlement / D

For end bearing mobilization the trendline is given as:

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

q_b/q_{bmax} = 0.98 for R > 6.5

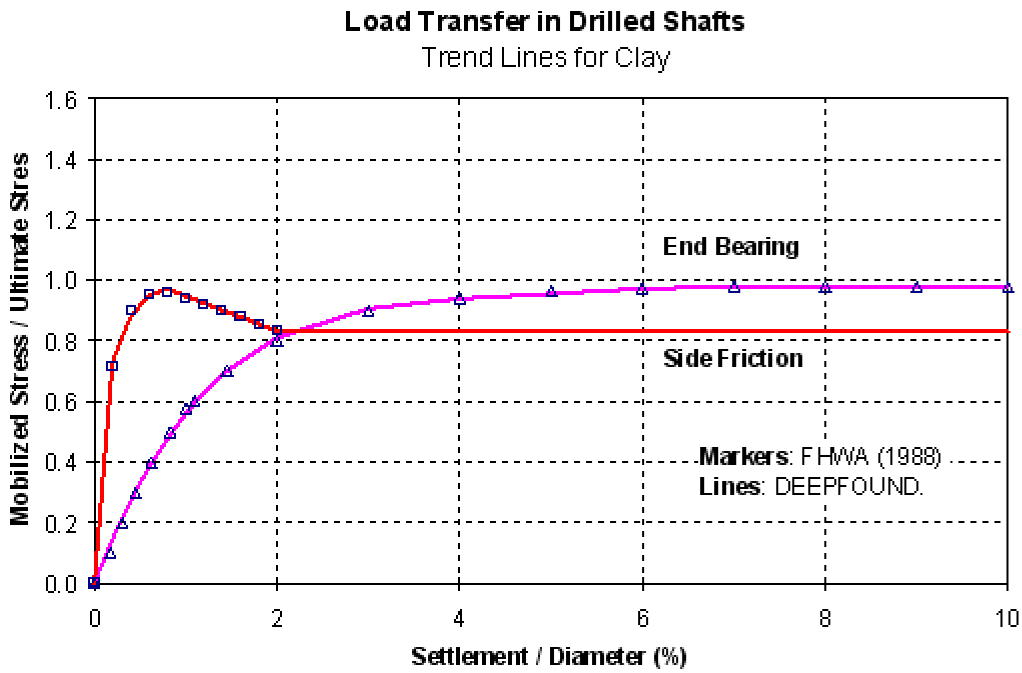


Figure: 1.3.b Trend lines for side friction and end bearing in clay

1.4 Design for Sand

1. Side Shear Resistance
2. End Bearing
3. Immediate Settlement

1.4.1 Sand: Side Shear Resistance

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, 15 cm.) 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, 23 cm.) 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 b method for calculating the unit side shear transfer is use with the following rationale:

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

Equation: 1.4.a

$$Q_s = \int_0^L K \, \sigma_z \, \tan \phi_c \, dA$$

Equation: 1.4.b

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

Equation: 1.4.a and Equation: 1.4.b can be used in computations, but simpler expressions can be developed by combining the terms for K and tan φ_c as β; resulting in:

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

Equation: 1.4.c

$$Q_s = \int \beta \sigma_z \, dA$$

Equation: 1.4.d

$$\beta = 1.5 - 0.135\sqrt{z}$$

$1.2 > \beta > 0.25$

Equation: 1.4.e

where

z = depth below ground surface, ft.

The factor β in Equation: 1.4.d 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. In sand layers with blowcounts of less than 15, an adjustment is made by dividing the blowcount by 15, and multiplying this value by β.

The limiting value of side resistance in Equation: 1.4.e 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.

1.4.2 Sand: End Bearing

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: 1.4.f.

$$q_{br} = 50 * \left(\frac{q_b}{B_b} \right);$$ B_b in inches

or

$$q_{br} = 1.3 * \left(\frac{q_b}{B_b} \right);$$ B_b in meters

Equation: 1.4.f

Recommend Unit End Bearing Values for Cohesionless Soils

N_{spt} Values (Blowcount)	**Value of q_b (tsf)**
0 to 50	(0.60 N_{spt})[60 N_{spt}]
Above 50 is treated as 50	(0.60 N_{spt})[60 N_{spt}]

In the case where the shaft base is in sand, FB-Deep uses the basic assumption that the soil 1.5B above and 2B 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 1.5B - 2B range is obtained via Equation: 1.4.g.

$$N_{spt} = \frac{\Sigma N_k L_k}{\Sigma L_k}$$

Equation: 1.4.g

FB-Deep 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 1.5 shaft diameters above the base and 2.0 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, FB-Deep 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.

1.4.3 Sand: Immediate Settlement

Immediate Settlements - The immediate settlements are computed using non-linear t-z and Q-z springs, with the shape presented in Figure: 1.4.a. 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 ≤ 0.908333

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

where R = 100 * Settlement / D

End bearing mobilization

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

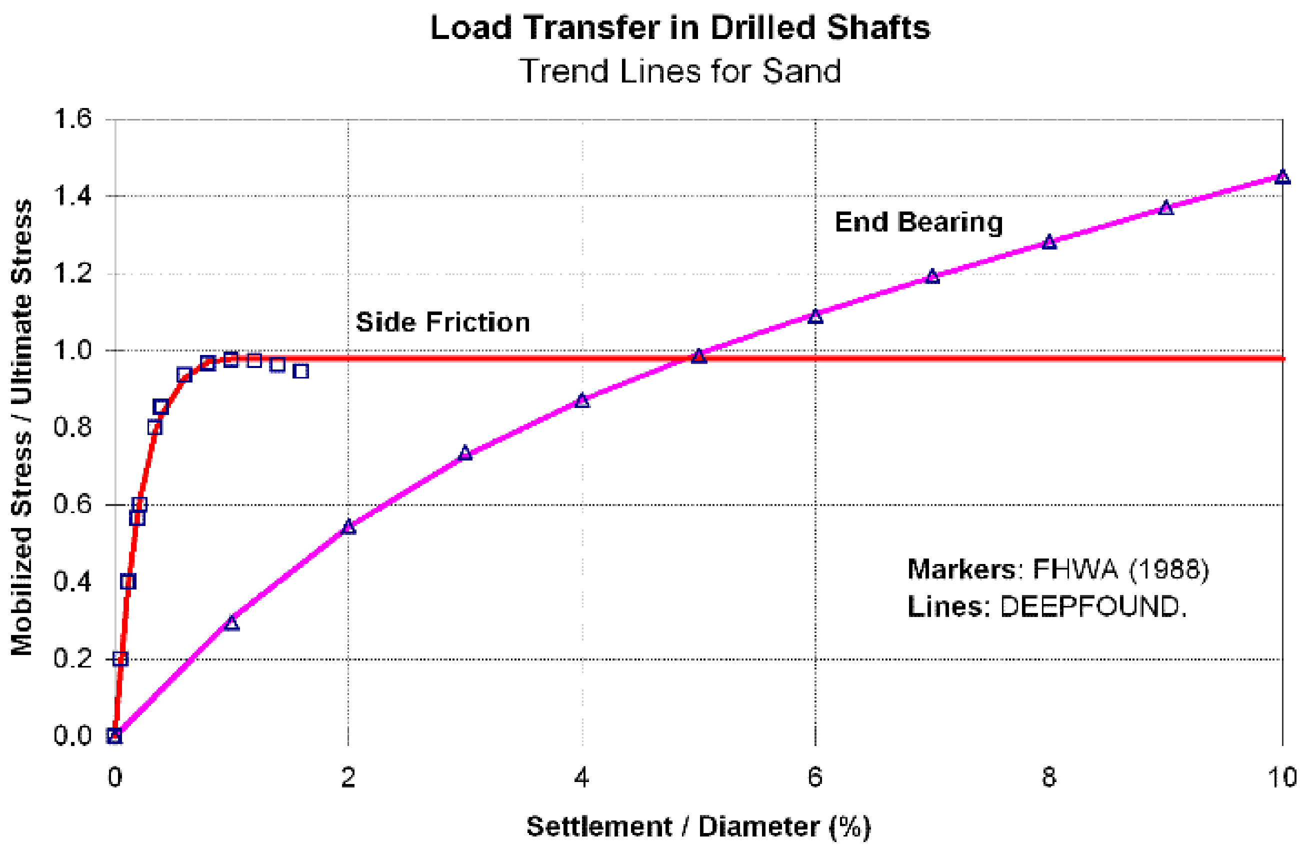


Figure: 1.4.a Trend lines for side friction and end bearing in sand

1.5 Design for Rock

1. Side Shear Resistance
2. End Bearing
3. Short-Term Settlement

1.5.1 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) and are typically based upon unconfined compression strengths, ***q_u***, (a values) or a combination of unconfined and split tensile strengths ($0.5\sqrt{q_u}\sqrt{q_t}$). These correlations listed below can be entered into FB-Deep as

(Note: 1 tsf = 95.8 kPa):

$$f_{su} = A\,q_u^{\,B}$$

Equation: 1.5.a

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

N value	10	15	20	25	30	>30

	f_{su}(tsf)	0.36	0.77	1.10	1.80	2.60	2.60
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.

The value of f_{su} is modified by the Rock Recovery percentage. The Rock Recovery percentage is inputted in the Boring Log as a value between 0.0 and 1.0. Note that for input files created in FB-Deep version 1.18 and previous, the value of f_{su} is modified by RQD, not Rock Recovery. To force an old input file (one that was created on FB-Deep version 1.18 or before) to use Rock Recovery instead of RQD, simply open the old input file in FB-Deep version 1.19, and resave it.

When entering values for socket roughness, 0 = smooth socket, and 1 = rough socket. If any rock layers in the boring log have a smooth designation, the Aq_u^B method will automatically become selected and the William's and McVay's methods will become disabled. The default values of 1.0 and 0.07 will then be assigned to A and B, respectively. (Note that for input files created in FB-Deep version 1.18 and previous, you can still use any of the three Rock Side Friction methods, even if a smooth socket is present. However, once you save the old input file on version 1.19, you will be forced to used the Aq_u^B method if any smooth sockets are present.

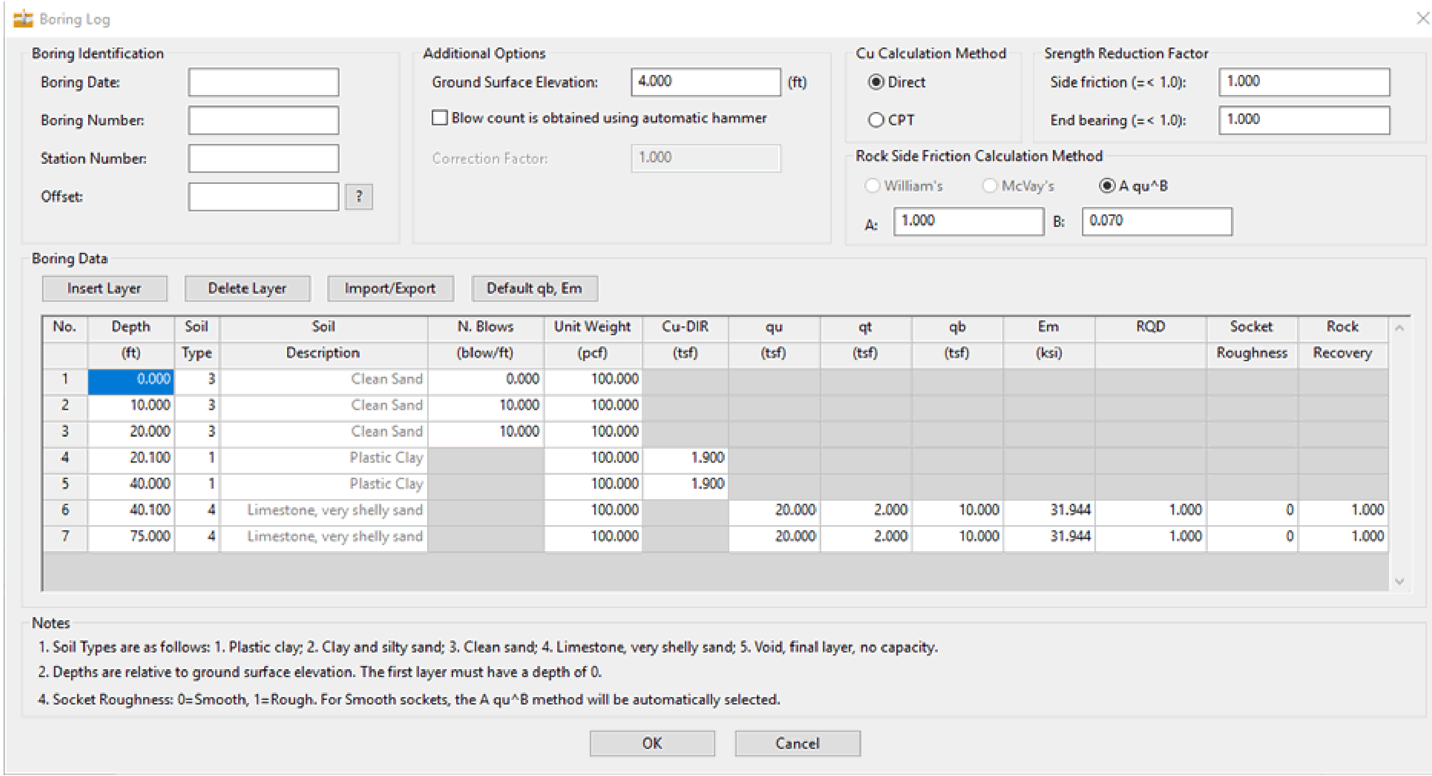


Figure: 1.5.a Boring Log Dialog

1.5.2 Rock: End Bearing

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

$Q_b = q_{bu} A_b$

Equation: 1.5.b

where:

Q_b = ultimate end bearing

q_{bu} = unit end bearing capacity, and

A_b = shaft base area

q_{bu} is user defined.

1.5.3 Rock: Short-Term Settlement

The short-term settlements in rock are estimated using the direct method of O'Neill, et al. (1996) (FHWA report: Load Transfer for Drilled Shafts in Intermediate Geomaterials) for rough sockets [IGM_Type = 1.0] or smooth [IGM_Type <> 1.0].

Step-by-step procedure for settlement capacity computation of Drilled Shaft IGM:

1. Find the total socketed length along the pile (L).

   $$L = \Sigma L_k$$

   where, k is rock layer number, L_k is k rock layer thickness

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

   $$E_{m\_avg} = \frac{\Sigma E_{mk} L_k}{\Sigma L_k}$$ where $E_m = 115\, q_u$

   $$f_{su\_avg} = \frac{\Sigma f_{su} L_k}{\Sigma L_k}$$ where f_{su} = side friction from Equation: 1.5.a.

3. Find n

   $$n = \frac{\sigma_n}{q_u}$$

   For "rough" sockets;
   where, σ_n = normal stress of concrete = $\gamma_c Z_c M$

   γ_c is the unit weight of the concrete and

Z_c is the distance from the top of the completed column of concrete to the point in the borehole at which σ_n is desired (usually the middle of the socket).

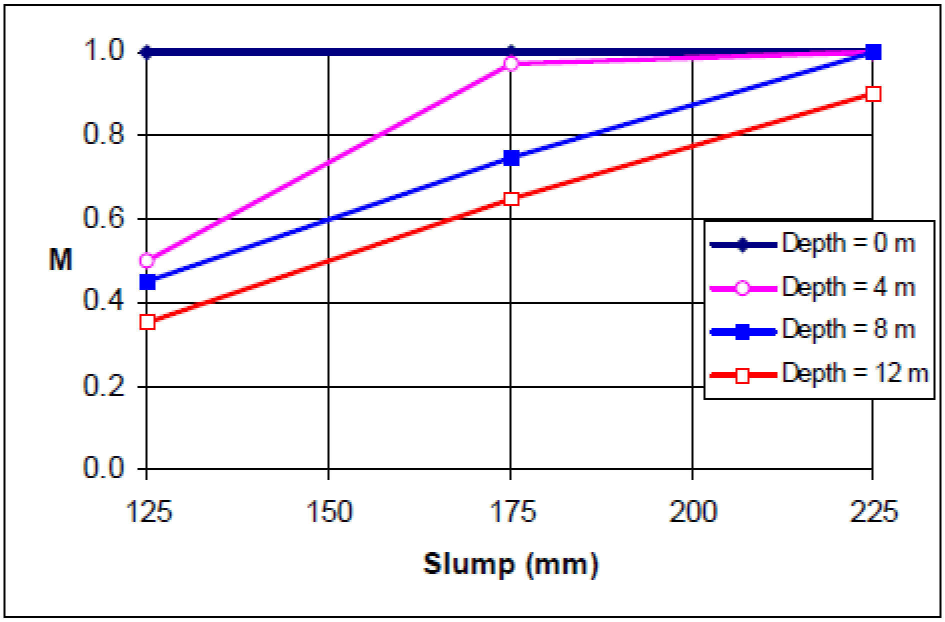


Table 4 Values of M

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 $\sigma_n = M \cdot [\gamma_c Z_w + (\gamma_c - \gamma_w) \cdot (Z_c - Z_w)]$, where Z_w = depth to water table.

For "smooth" sockets,
n is estimated (See Figure: 1.5.b).

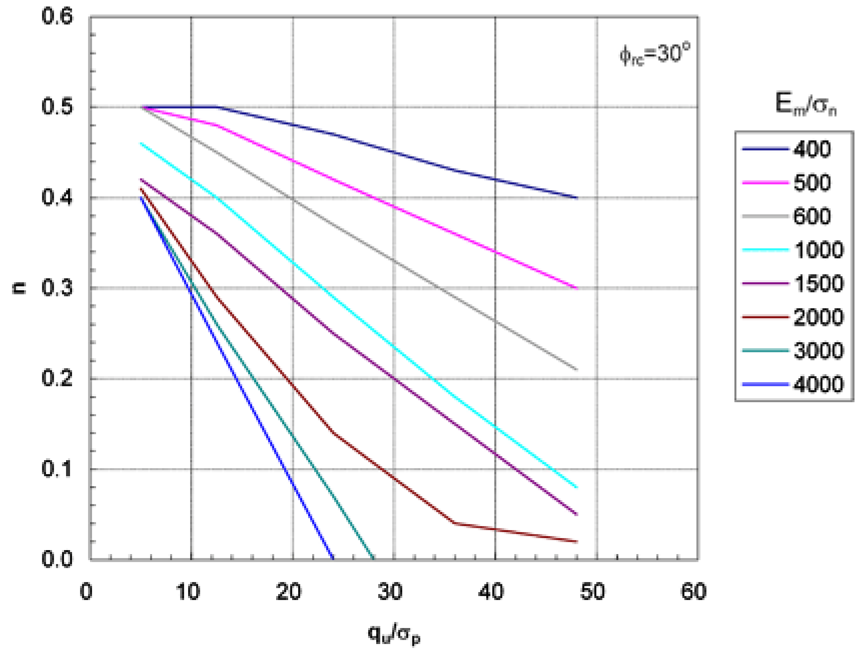


Figure: 1.5.b: N Factors for Smooth Sockets

Average n is calculated as $n_{avg} = \frac{\Sigma\, n_k L_k}{\Sigma\, L_k}$

Where, k is rock layer number, L_k is k layer thickness

4. Calculate **Ω**

$$\Omega = 1.14\left(\frac{L}{D}\right)^{0.5} - 0.05\left(\left(\frac{L}{D}\right)^{0.5} - 1\right)\log_{10}\left(\frac{E_c}{E_{m\_avg}}\right) - 0.44$$

where, L is the total socket length, E_c is user defined concrete elastic modulus, and $\frac{L}{D} \geq 2$

5. Calculate **Γ**

$$\Gamma = 0.37\left(\frac{L}{D}\right)^{0.5} - 0.15\left(\left(\frac{L}{D}\right)^{0.5} - 1\right)\log_{10}\left(\frac{E_c}{E_{m\_avg}}\right) + 0.13$$

1. Calculate Θ_f and K_f

$$\Theta_f = \frac{E_{m\_avg}\Omega}{\pi L \Gamma f_{su\_avg}} w_t$$

$$K_f = n_{avg} + \frac{\left(\Theta_f - n_{avg}\right)\left(1 - n_{avg}\right)}{\Theta_f - 2n_{avg} + 1} \quad < 1$$

where, w_t = Settlement at top of rock socket

2. Calculate the side shear load transfer - deformation as

$$Q_s = \pi D L \Theta_f f_{su\_avg} \qquad \Theta_f < n_{avg}$$
$$Q_s = \pi D L K_f f_{su\_avg} \qquad \Theta_f > n_{avg}$$

K_f = 1.0 when, $K_f < n_{avg}$

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

Find Q_b

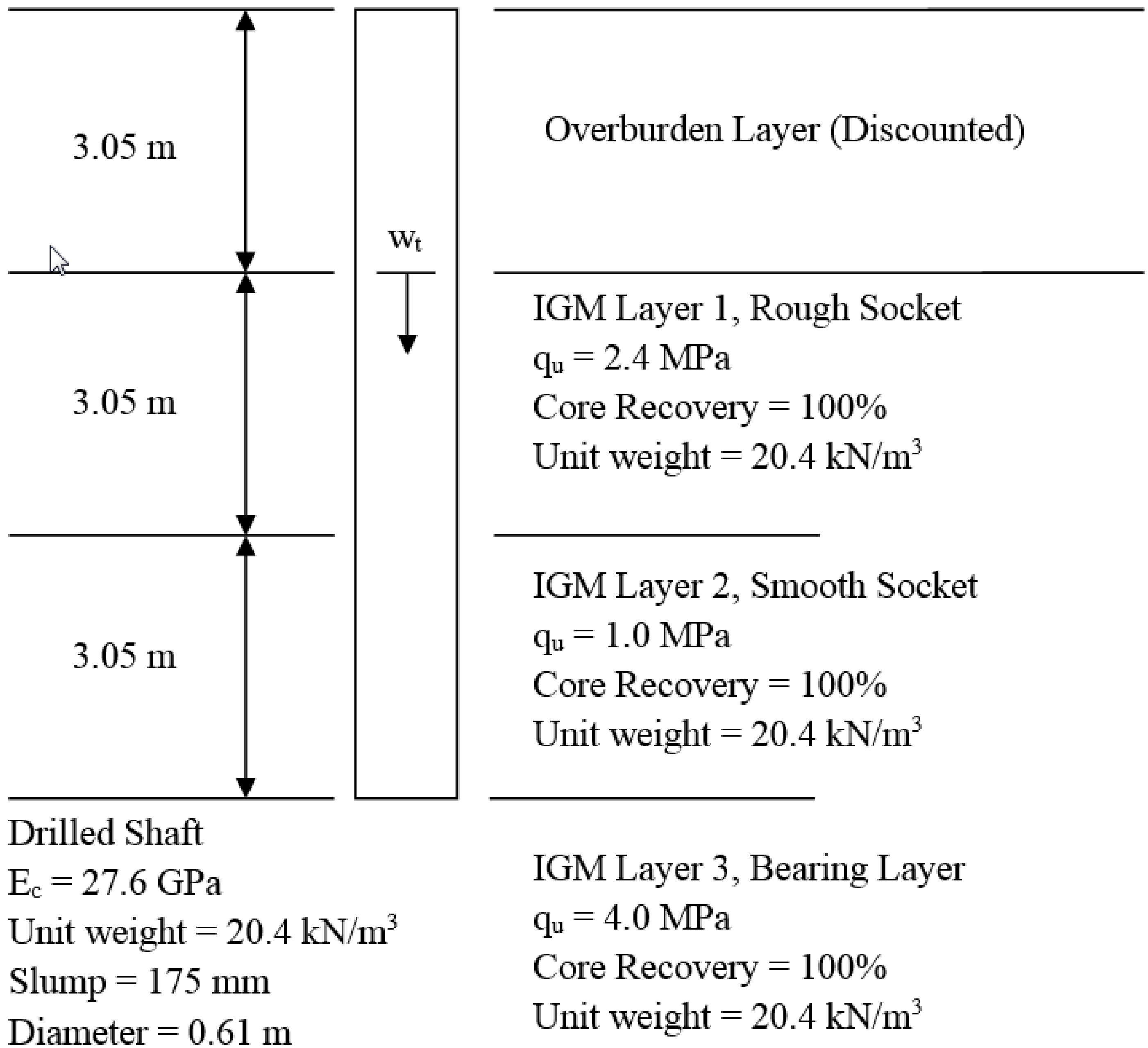
$$Q_b = \frac{\pi D^2}{4} q_b$$

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

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

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

Example:



1. Find the total socketed length along the pile (L).

L = 6.1 m

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

$$E_{m\_avg} = \frac{\Sigma E_{mk} L_k}{\Sigma L_k}$$ where $E_m = 115\, q_u$

E_{m_avg} = 195500 kPa

$$f_{su\_avg} = \frac{\Sigma f_{su} L_k}{\Sigma L_k}$$ where, $f_{su} = A \bullet q_u^B$ (selecting A = 0.4 and B = 1)

f_{su_avg} = 680 kPa

3. Find average n

IGM Layer 1, for "rough" sockets;

$$n = \frac{\sigma_n}{q_u}$$

where, $\sigma_n = \gamma_c Z_c M$

γ_c = 20.4 kN/m^3, M = 0.92 (for Z_c = 4.575 m and Slump = 175 mm)

σ_n = 85.86 kN/m^2

n(σ_n, q_u) = 0.036

IGM Layer 2, for "smooth" sockets, n is estimated using Figure: 1.5.b).

$\sigma_n = \gamma_c Z_c M$

γ_c = 20.4 kN/m^3, M = 0.769 (for Z_c = 7.625 m and Slump = 175 mm)

σ_n = 119.6 kN/m^2

$n(E_m/\sigma_n, q_u/\sigma_p) = 0.456$

Average n is calculated as

$n_{avg} = 0.246$

4. Calculate Ω

$$\Omega = 1.14\left(\frac{L}{D}\right)^{0.5} - 0.05\left(\left(\frac{L}{D}\right)^{0.5} - 1\right)\log_{10}\left(\frac{E_c}{E_{m\_avg}}\right) - 0.44$$

Ω = 2.93

5. Calculate Γ

$$\Gamma = 0.37\left(\frac{L}{D}\right)^{0.5} - 0.15\left(\left(\frac{L}{D}\right)^{0.5} - 1\right)\log_{10}\left(\frac{E_c}{E_{m\_avg}}\right) + 0.13$$

Γ = 0.602

6. Calculate Θ_f

$$\Theta_f = \frac{E_{m\_avg}\Omega}{\pi L \Gamma f_{su\_avg}} w_t$$

For 2mm settlement at top of rock socket (wt = 0.002m)

$\Theta_f = 0.146$

7. Calculate the side shear load capacity – settlement as

$$Q_s = \pi D L \Theta_f f_{su\_avg} \qquad \Theta_f < n_{avg}$$

$$Q_s = \pi D L K_f f_{su\_avg} \qquad \Theta_f > n_{avg}$$

For $\Theta_f < n_{avg}$

Q_s = 1160 kN

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

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

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

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

Λ = 71054 kN/m^2.

Q_b = 323 kN

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

Q_t =1160 + 323 = 1823 kN (for 2mm settlement)

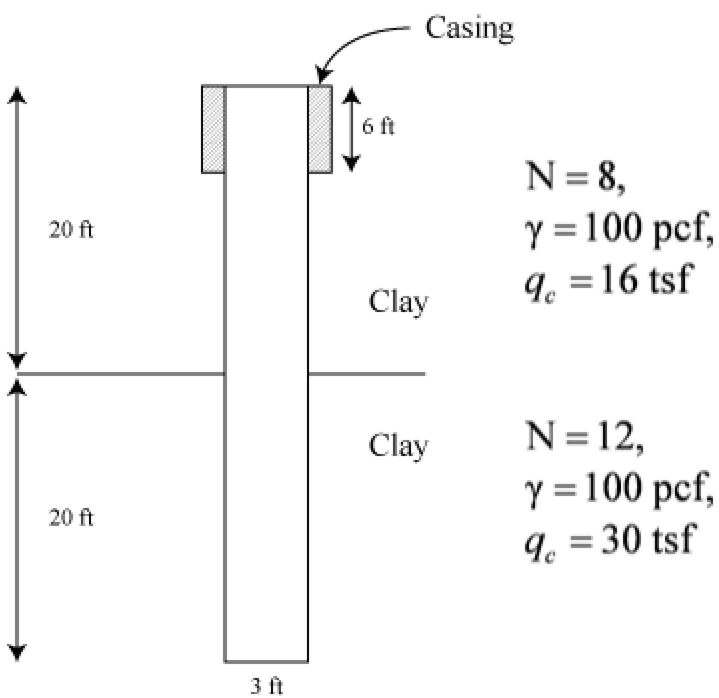
1.6 Shaft: Layered Soils

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.

1.7 Examples

1. Example 1
2. Example 2
3. Example 3
4. Example 4
5. Example 5
6. Example 6

1.7.1 Example 1: Multi Layer Clay with Casing



1- Undrained Shear Strength calculation from CPT results

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

σ_o: average stress; q_c: tip cone resistance

Clay Layer # 1 :

$$Cu = \frac{(16 \cdot 2000) - (10 \cdot 100)}{15} = 2,066.67\ psf\,(1.0333\ tsf)$$

Clay Layer # 2 :

$$Cu = \frac{(30 \cdot 2000) - (30 \cdot 100)}{15} = 3,800\ psf\,(1.90\ tsf)$$

2- Skin Friction:

Skin friction coefficient

$$f_{su} = \alpha \cdot Cu$$

Layer #1:

$$f_{su} = 0.55 \cdot (1.0333\ tsf) = 0.568\ tsf$$

Layer#2:

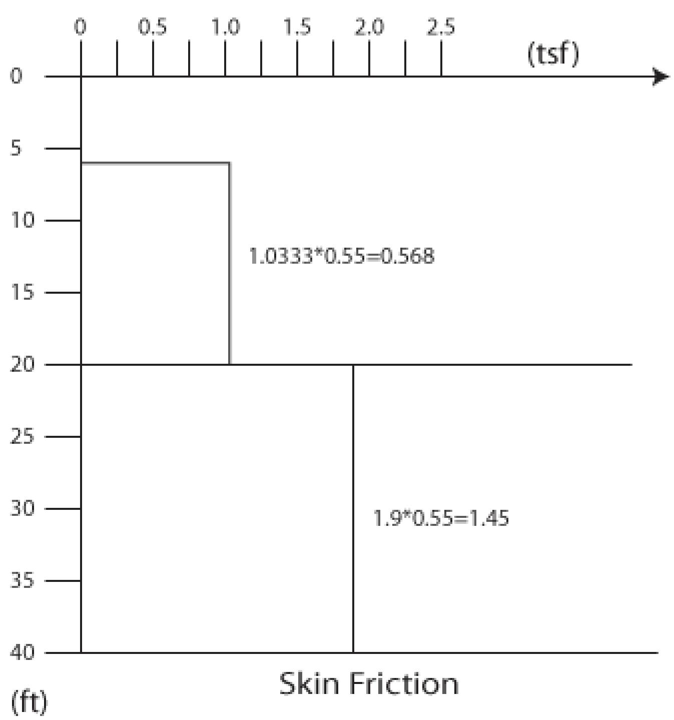
$$f_{su} = 0.55 \cdot (1.90\ tsf) = 1.045\ tsf$$

$$Q_s = \int_{L_1}^{L_f} f_{su} dA$$

$$Q_s = \int_{6'}^{20'} (0.568\ tsf) \cdot (\pi \cdot 3.0\, ft) + \int_{20'}^{37'} (1.045\ tsf) \cdot (\pi \cdot 3.0\, ft)$$

$$Q_s = \pi \cdot 3.0 \cdot \left[(20' - 6')(0.568) + (37' - 20')(1.045) \right]$$

$$\boxed{Q_s = 242.42\ Tons}$$



Skin Friction

3- End Bearing:

$$Q_b = q_b\left(\frac{\pi B^2}{4}\right) \qquad q_b = N_c \cdot \overline{C_u}$$

$$N_c = 6.0\cdot\left[1+0.2\left(\frac{40'}{3'}\right)\right] = 22 > 9 \quad (use\, 9) \quad \left[Skempton\ (1951)\right]$$

$$\overline{C_u} = \frac{q_c - \overline{\sigma_0}}{15}$$

Average undrained shear strength 1.5B above the tip

$$C_u = \frac{(30tsf\cdot 2000)-(100\,pcf)\cdot(40\,ft+\frac{1.5\cdot 3.0\,ft}{2})}{15} = 3,748.33\ psf\,(1.874\,tsf)$$

$$q_{b1} = 9(1.874\ tsf) = 16.870\ tsf$$

Average undrained shear strength 3B below the tip

$$C_u = \frac{(30tsf\cdot 2000)-(100\,pcf)\cdot(40\,ft+\frac{3\cdot 3.0\,ft}{2})}{15} = 3,703.00\ psf\,(1.852\,tsf)$$

$$q_{b2} = 9\cdot(1.852\ tsf) = 16.665\ tsf$$

Average Unit skin friction

$$q_{ave} = \frac{16.66+16.87}{2} = 16.77\ tsf$$

$$Q_b = (16.77\ tsf)\left(\frac{\pi\, 3^2}{4}\right)$$

$$\boxed{Q_b = 118.514\ Tons}$$

4- Total Capacity:

$$Q_T = Skin\ Friction\ +\ End\ Bearing$$

$$Q_T = 242.42 + 118.54$$

$$\boxed{Q_T = 360.94 Tons}$$

$$Q_s = 242.42\ Tons, \quad Q_b = 118.54\ Tons, \quad Q_T = 360.94\ Tons$$

5- Settlement:

a) Settlement = 0.3"

$$R\% = \frac{S}{B}\cdot 100 = \frac{0.3}{36}\cdot 100 = 0.833$$

Side friction mobilization: 0.74 < R% < 2.0

$$\frac{Q_{s(mob)}}{Q_s} = 0.978929 - 0.115817(R - 0.74)$$

$$Q_{s(mob)} = [0.978929 - 0.115817(0.833 - 0.74)]\cdot 242.42$$

$$\boxed{Q_{s(mob)} = 234.70\ Tons}$$

End bearing mobilization: R<6.5

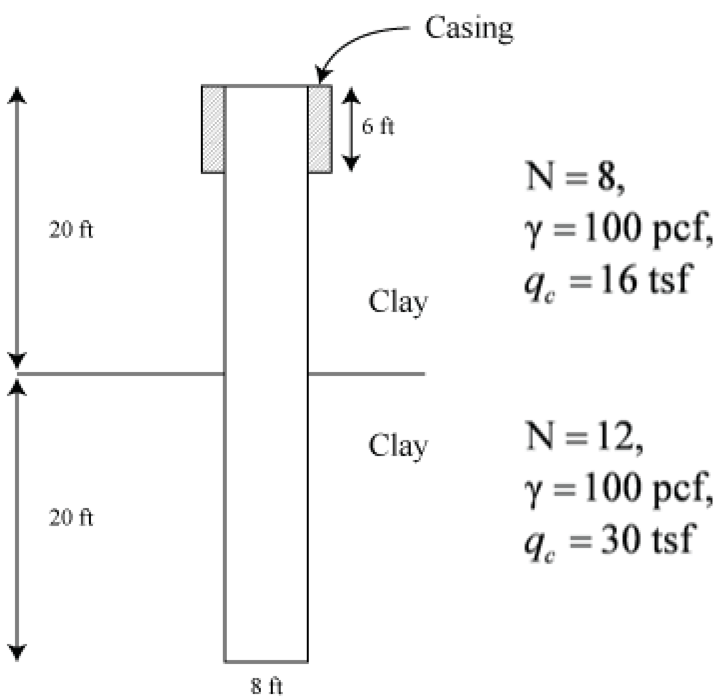
$$\frac{Q_{b(mob)}}{Q_b} = 1.1832\cdot 10^{-4}(R)^5 - 3.7091\cdot 10^{-3}(R)^4 + 4.4944\cdot 10^{-2}(R)^3 - 0.26537(R)^2 + 0.78436(R)$$

$$\frac{Q_{b(mob)}}{118.54} = 1.18\cdot 10^{-4}(0.83)^5 - 3.71\cdot 10^{-3}(0.83)^4 + 4.49\cdot 10^{-2}(0.83)^3 - 0.26(0.83)^2 + 0.78(0.83)$$

$$\boxed{Q_{b(mob)} = 58.5\ Tons}$$

$$Q_T\ @\ 0.3" = 234.70 + 58.5 = \boxed{293.2\ Tons}$$

1.7.2 Example 2: Multi Layer Clay with Casing, but B>75" (1.9m):



1- Undrained Shear Strength calculation from CPT results

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

σ_o: average stress q_c: tip cone resistance

Clay Layer # 1:

$$Cu = \frac{(16 \cdot 2000) - (10 \cdot 100)}{15} = 2,066.67 \ psf\,(1.0333\ tsf)$$

Clay Layer # 2:

$$Cu = \frac{(30 \cdot 2000) - (30 \cdot 100)}{15} = 3,800 \ psf\,(1.90\ tsf)$$

2- Skin Friction:

Skin friction coefficient

$$f_{su} = \alpha \cdot Cu$$

Layer #1:

$$f_{su} = 0.55 \cdot (1.0333\, tsf) = 0.568\, tsf$$

Layer#2:

$$f_{su} = 0.55 \cdot (1.90\ tsf) = 1.045\ tsf$$

$$Q_s = \int_{L_1}^{L_f} f_{su} dA$$

$$Q_s = \int_{6'}^{20'} (0.568\ tsf) \cdot (\pi \cdot 8.0\, ft) + \int_{20'}^{32'} (1.045\ tsf) \cdot (\pi \cdot 8.0\, ft)$$

$$Q_s = \pi \cdot 8.0 \cdot \left[(20' - 6')(0.568) + (32' - 20')(1.045) \right]$$

$$\boxed{Q_s = 515.14\ Tons}$$

3- End Bearing:

$$Q_b = q_b \left(\frac{\pi B^2}{4} \right) \qquad q_b = N_c \cdot \overline{C_u}$$

$$N_c = 6.0 \cdot \left[1 + 0.2 \left(\frac{40'}{8'} \right) \right] = 12 > 9 \quad (use\, 9) \quad \left[Skempton\ (1951) \right]$$

$$\overline{C_u} = \frac{q_c - \overline{\sigma_0}}{15}$$

Average undrained shear strength 1.5B above the tip

$$C_u = \frac{(30tsf \cdot 2000) - (100\, pcf) \cdot (40\, ft - \frac{1.5 \cdot 8.0\, ft}{2})}{15} = 3,773.33\ psf\, (1.887\ tsf)$$

$$q_{b1} = 9 \cdot (1.887\ tsf) = 16.98\ tsf$$

Average undrained shear strength 3B below the tip

$$C_u = \frac{(30tsf \cdot 2000) - (100\,pcf) \cdot (40\,ft + \frac{3 \cdot 8.0\,ft}{2})}{15} = 3{,}653.33\ psf\,(1.827\ tsf)$$

$$q_{b2} = 9 \cdot (1.827\ tsf) = 16.44\ tsf$$

Average Unit skin friction

$$q_{ave} = \frac{16.98 + 16.44}{2} = 16.71\ tsf$$

$$Q_b = (16.71\,tsf)\left(\frac{\pi\,8^2}{4}\right)$$

$$\boxed{Q_b = 839.94\ Tons}$$

4- Corrected end bearing If B > 75 ", then

$$q_{br} = F_r \cdot q_b$$

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

$$a = 0.0071 + 0.0021\left(\frac{L}{B_b}\right); \qquad b = 0.45\sqrt{C_u}\ , \quad C_u\ in\ ksf$$

Cu at the mid-level of bearing layer change and pile tip.

$$C_u = \frac{(30tsf \cdot 2000) - (100\,pcf) \cdot (40\,ft - \frac{(40-20)\,ft}{2})}{15} = 3{,}800.00\ psf\,(1.9\ tsf)$$

$$a = 0.0071 + 0.0021\left(\frac{40'}{8'}\right); \qquad b = 0.45\sqrt{1.9 \cdot 2.0}$$

$$a = 0.018, \text{ but } a \le 0.015; \qquad b = 0.8772,\ 0.5 < b < 1.5$$

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

$$Q_{br} = F_r \cdot Q_b$$

$$Q_{br} = (0.6881) \cdot (839.94)\,Tons$$

$$\boxed{Q_{br} = 579.96\ Tons}$$

5- Total Capacity:

$$Q_T = Skin\ Friction\ +\ Corrected\ End\ Bearing$$

$$Q_T = 515.14 + 577.96$$

$$\boxed{Q_T = 1093.10\ Tons}$$

Q_s =515.14 Tons, Q_{br} =577.96 Tons, Q_T =1093.10 Tons

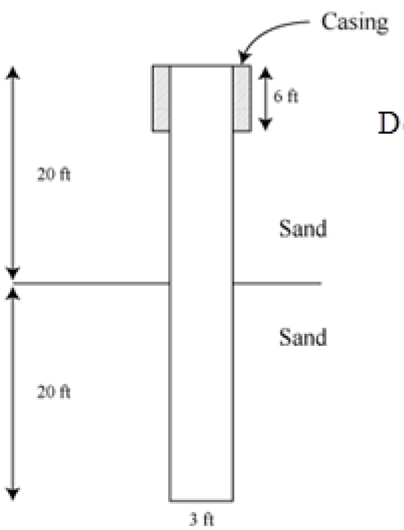
6- Settlement

a) Settlement = 3.0"

Side friction mobilization: R% > 2.0

End bearing mobilization: R<6.5

1.7.3 Example 3: Multilayer - Sand - Sand



Depth	N	γ_s pcf
0	10	100
10	10	100
20	10	100
25	15	100
60	15	100

1- Skin Friction:

In sand layers with blowcounts of less than 15, an adjustment is made by dividing the blowcount by 15, and multiplying this value by β.

$$\beta = 1.5 - 0.135 \cdot \sqrt{z}; \quad 0.25 < \beta < 1.2$$

$$Q_s = \pi \cdot B \cdot \left(\int_6^{25} \beta \cdot \frac{10}{15} \sigma_v dz + \cdot \int_{25}^{40} \beta \sigma_v dz \right)$$

$$Q_s = \pi \cdot 3 \cdot \left(\left(\frac{10}{15} \right) \int_6^{25} (1.5 - 0.135 \cdot \sqrt{z}) \cdot (\gamma \cdot z) \cdot dz + \int_{25}^{40} (1.5 - 0.135 \cdot \sqrt{z}) \cdot (\gamma \cdot z) \cdot dz \right)$$

$$Q_s = \pi \cdot 3 \cdot \left((0.667) \int_6^{25} \left(150 \cdot z - 13.5 \cdot z^{3/2} \right) dz + \int_{25}^{40} \left(150 \cdot z - 13.5 \cdot z^{3/2} \right) dz \right)$$

$$Q_s = \pi \cdot 3 \cdot \left[(0.667)\left(\frac{150 \cdot z^2}{2} - 13.5 \cdot z^{5/2} \cdot \frac{2}{5} \right)\Bigg|_6^{25} + \left(\frac{150 \cdot z^2}{2} - 13.5 \cdot z^{5/2} \cdot \frac{2}{5} \right)\Bigg|_{25}^{40} \right]$$

$$Q_s = \pi \cdot 3 \cdot \left[(0.667) \cdot (30,000 - 2,223.82) + (65,355.84 - 30,000) \right]$$

$$Q_s = 53,882.55 \cdot \frac{3 \cdot \pi}{2000}$$

$$\boxed{Q_s = 253.91\, Tons}$$

2- End Bearing:

Above 1.5B and Below 2B:

$$Above: \; 40.0 - 1.5B = 40.0 - 1.5 \cdot 3 = 35.5\, ft$$

$$Below: \; 40.0 + 2B = 40.0 + 2 \cdot 3 = 46\, ft$$

$$For: \; z = 35.5\, ft \rightarrow N_{spt} = 15\; blow/ft$$

$$z = 46\, ft \rightarrow N_{spt} = 15\; blow/ft$$

$$N_{spt(average)} = 15\; blow/ft$$

$$q_b = (0.60 \cdot N_{spt})$$

$$q_b = (0.60 \cdot 15) = 9.0 tsf$$

$$Q_b = q_b \cdot \left[\frac{\pi\, B^2}{4} \right]$$

$$Q_b = 9.0 \cdot \left[\frac{\pi\, 3^2}{4} \right] = \boxed{63.62\; Tons}$$

3- Total Capacity:

$$Q_T = Skin\ Friction\ +\ End\ Bearing$$

$$Q_T = 253.91 + 63.62$$

$$\boxed{Q_T = 317.53\ Tons}$$

$$Q_s = 253.91\, Tons, \quad Q_b = 63.62\, Tons, \quad Q_T = 317.53\, Tons$$

4- Settlement:

a) Settlement = 1.44 "

$$R\% = \frac{S}{B} \cdot 100 = \frac{1.44}{36} \cdot 100 = 4.0$$

Side friction mobilization: R% > 0.908333

$$\frac{Q_{s(mob)}}{Q_s} = 0.978112$$

$$Q_{s(mob)} = 253.91 \cdot 0.978112 \boxed{= 248.35\ Tons}$$

End bearing mobilization:

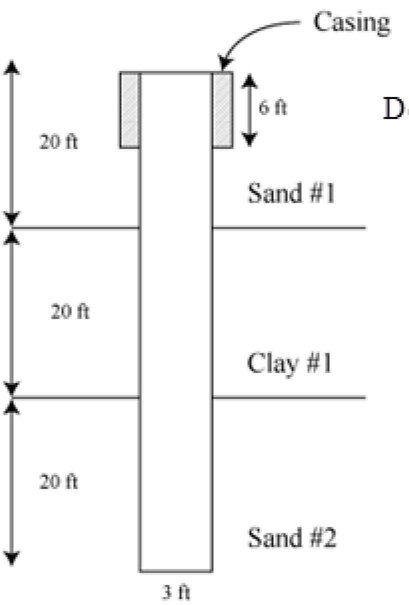
$$\frac{Q_{b(mob)}}{Q_b} = -0.0001079 \cdot (4.00)^4 + 0.0035584 \cdot (R)^3 - 0.045115 \cdot (R)^2 + 0.34861 \cdot (R)$$

$$\frac{Q_{b(mob)}}{63.62} = -0.00011 \cdot (4)^4 + 0.0036 \cdot (4)^3 - 0.045115 \cdot (4.00)^2 + 0.349 \cdot (4)$$

$$\boxed{Q_{b(mob)} = 55.52\ Tons}$$

$$Q_T\ @\ 1.44" = 248.35 + 55.52 = \boxed{303.87\ Tons}$$

1.7.4 Example 4: Multilayer - Sand - Clay - Sand:



Depth	N	γ, pcf	C_u, tsf
0	10	100	
10	10	100	
20	10	100	
20		100	1.9
40		100	1.9
40	15	100	
40	15	100	
70	15	100	

1- Skin Friction

Layer 1: Sand (6-20ft)

In sand layers with blowcounts of less than 15, an adjustment is made by dividing the blowcount by 15, and multiplying this value by β.

$$\beta = 1.5 - 0.135\cdot\sqrt{z};\quad 0.25 < \beta < 1.2$$

$$Q_s = \pi\cdot B\cdot\left(\int_{6'}^{20'} \beta\cdot\frac{N}{15}\cdot\sigma_v dz\right)$$

$$Q_s = \pi\cdot 3\cdot\left(\int_{6}^{20} (1.5 - 0.135\cdot\sqrt{z})\cdot\left(\frac{10}{15}\right)(\gamma\cdot z)\cdot dz\right)$$

$$Q_s = \frac{\pi\cdot 30}{15}\cdot\left(\left[\frac{150\cdot z^2}{2} - 13.5\cdot z^{5/2}\cdot\frac{2}{5}\right]_6^{20}\right)$$

$$Q_s = \frac{\pi \cdot 30}{15} \cdot \left(20,340.186 - 2,223.82\right)$$

$$Q_s = 18,116.37 \cdot \frac{30 \cdot \pi}{15} \cdot \frac{1}{2000}$$

$$\boxed{Q_{s1} = 56.91\,Tons}$$

Layer 2: Clay (20-40ft)

$$f_{su} = \alpha \cdot Cu$$

$$f_{su} = 0.55 \cdot (1.90\ tsf) = 1.045\ tsf$$

$$Q_s = \int_{L_s}^{L_f} f_{su} dA$$

$$Q_s = \int_{20'}^{40'} (1.045\ tsf) \cdot (\pi \cdot 3.0\,ft)$$

$$Q_s = \pi \cdot 3.0 \cdot \left[\left(40' - 20'\right)\left(1.045\right)\right]$$

$$\boxed{Q_{s2} = 196.98\,Tons}$$

Layer 3: Sand (40-60ft)

$$Q_s = \pi \cdot 3 \cdot \int_{40}^{60} (1.5 - 0.135 \cdot \sqrt{z}) \cdot (\gamma \cdot z) \cdot dz$$

$$Q_s = \pi \cdot 3 \cdot \int_{40}^{60} \left(150 \cdot z - 13.5 \cdot z^{3/2}\right) dz$$

$$Q_s = \pi \cdot 3 \cdot \left[\frac{150 \cdot z^2}{2} - 13.5 \cdot z^{5/2} \cdot \frac{2}{5}\right]_{40}^{60}$$

$$Q_s = \pi \cdot 3 \cdot (150,000 - 95,937.40)$$

$$Q_s = 54,062.6 \cdot \frac{3 \cdot \pi}{2000}$$

$$\boxed{Q_{s3} = 254.76\ Tons}$$

$$Q_s = Q_{s1} + Q_{s2} + Q_{s3}$$

$$Q_s = 56.91 + 196.98 + 254.76$$

$$\boxed{Q_s = 508.65\ Tons}$$

2- End Bearing:

Above 1.5B and Below 2B:

$$Above:\ 60.0 - 1.5B = 60.0 - 1.5 \cdot 3 = 55.5\,ft$$

$$Below:\ 60.0 + 2B = 60.0 + 2 \cdot 3 = 66\,ft$$

$$For:\ z = 55.5\,ft \rightarrow N_{spt} = 15\ blow/ft$$

$$z = 66\,ft \rightarrow N_{spt} = 15\ blow/ft$$

$$N_{spt(average)} = 15\ blow/ft$$

$$Q_b = q_b \cdot \left[\frac{\pi B^2}{4}\right]$$

$$q_b = (0.60 \cdot N_{spt})$$

$$q_b = (0.60 \cdot 15) = 9.0 tsf$$

$$Q_b = 9.0 \cdot \left[\frac{\pi 3^2}{4}\right] = \boxed{63.62\ Tons}$$

3- Total Capacity:

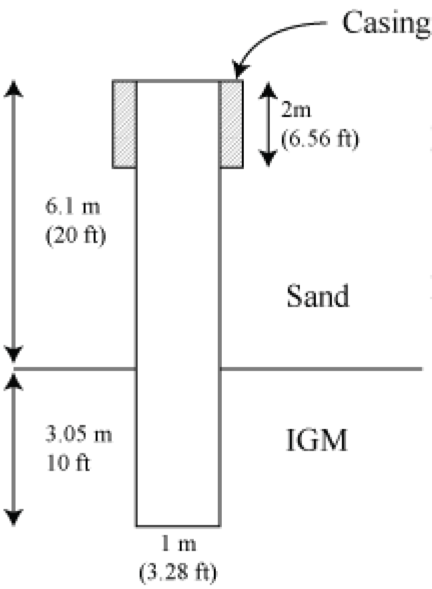
$$Q_T = Skin\ Friction\ +\ End\ Bearing$$

$$Q_T = 253.91 + 63.62$$

$$\boxed{Q_T = 317.53\ Tons}$$

$$Q_s = 253.91\,Tons, \quad Q_b = 63.62\,Tons, \quad Q_T = 317.53\,Tons$$

1.7.5 Example 5: IGM: (Sand & Limestone)



$\gamma = 100\,\text{pcf}\ (15.708\,\text{kN/m}^3)$

$\text{N} = 10$

$\text{Limestone}: q_u = 10\,\text{tsf}\ (957.6\,\text{kPa}, 0.96\,\text{MPa})$

$q_t = 1\,\text{tsf}\ (95.76\,\text{kPa}, 0.096\,\text{MPa})$

$\gamma = 135\,\text{pcf}\ (21.2\,\text{kN/m}^3), \gamma_c = 20.4\,\text{kN/m}^3$

$E_c = 57{,}000\sqrt{f'_y} = 57{,}000\sqrt{5000\,\text{psi}}$

$= 4.03E6\ \text{psi}\ \ (2.77E6\,\text{kPa})$

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

1. Skin Friction (Sand):

   In sand layers with blowcounts of less than 15, an adjustment is made by dividing the blowcount by 15, and multiplying this value by β.

$$Q_s = \frac{\pi D}{2000} \int_{6.56}^{20} \beta \bullet \left(\frac{N}{15}\right) \sigma_z \, dz$$

$$Q_s = \frac{3.28\pi}{2000} \bullet \left(\frac{10}{15}\right) \int_{6.56}^{20} \left(1.5 - 0.135\sqrt{Z}\right) \gamma \, z \, dz$$

$$= 0.003435\left[75 \bullet \left(20^2 - 6.56^2\right) - 5.4 \bullet \left(20^{5/2} - 6.56^{5/2}\right)\right]$$

Q_S = 60.82 tons (541.08 kN)

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

   Skin Friction (Rock)

   $$Q_s = \pi D L f_{su}$$

   $$f_{su} = 0.5\sqrt{q_u}\sqrt{q_t}$$

   Q_s = π (1 m) (3.05 m) (151.41 kN/m^2)

   Q_s = 1450.79 kN

   Total Skin Friction

   Σ Q_s = 541.08 + 1450.79 = 1992 kN

   End Bearing (Rock)

   Assuming qb = ½ qu

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

   Q_b = 376 kN

3. Total Capacity :

   Q_t = Σ Q_s + Q_b = 1992 + 376 = 2368 kN

Settlement Calculation

Sand Layer

Skin Friction -

$Q_s/Q_{smax} = -2.16*R^4+6.34*R^3-7.36*R^2+4.15*R$ For R ≤ 0.908333

Q_s/Q_{smax} = 0.978112 for R > 0.908333

where R = 100 * Settlement / D, and Q_{smax} = 541.08 kN (Check above)

Rock Layer:
Skin Friction -

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

$$E_{m\_avg} = \frac{\Sigma E_{mk} L_k}{\Sigma L_k}$$ where $$E_m = 115\, q_u$$

$$f_{su\_avg} = \frac{\Sigma f_{su} L_k}{\Sigma L_k}$$ where, $$f_{su} = A \bullet q_u^B$$ (selecting A = 0.4 and B = 1)

For "rough" sockets; $$n = \frac{\sigma_n}{q_u}$$

where, $\sigma_n = \gamma_c Z_c M$

Only one IGM layer is present.

$E_{m\_avg} = E_m$=115 q_u = 110400 kPa

$f_{su\_avg} = f_{su}$ =151.41 kPa

n_{avg} = n

γ_c = 20.4 kN/m^3, M = 0.627 (for Z_c = 7.625 m and Slump = 152.4 mm)

$\sigma_n = \gamma_c Z_c M$ =97.53 kN/m^2

n_{avg} = n(σ_n, q_u) = 0.102

Calculate Ω, and Γ

$$\Omega = 1.14\left(\frac{L}{D}\right)^{0.5} - 0.05\left(\left(\frac{L}{D}\right)^{0.5} - 1\right)\log_{10}\left(\frac{E_c}{E_{m\_avg}}\right) - 0.44$$

$$\Gamma = 0.37\left(\frac{L}{D}\right)^{0.5} - 0.15\left(\left(\frac{L}{D}\right)^{0.5} - 1\right)\log_{10}\left(\frac{E_c}{E_{m\_avg}}\right) + 0.13$$

Calculate Θ_f and K_f

$$\Theta_f = \frac{E_{m\_avg}\Omega}{\pi L \Gamma f_{su\_avg}} w_t$$

$$K_f = n_{avg} + \frac{\left(\Theta_f - n_{avg}\right)\left(1 - n_{avg}\right)}{\Theta_f - 2n_{avg} + 1} \quad < 1$$

where, w_t = Settlement at top of rock socket

Side shear load capacity – settlement

$$Q_s = \pi D L \Theta_f f_{su\_avg} \qquad \Theta_f < n_{avg}$$
$$Q_s = \pi D L K_f f_{su\_avg} \qquad \Theta_f > n_{avg}$$

K_f = 1.0 when, $K_f < n_{avg}$

End bearing –settlement (Rock)

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

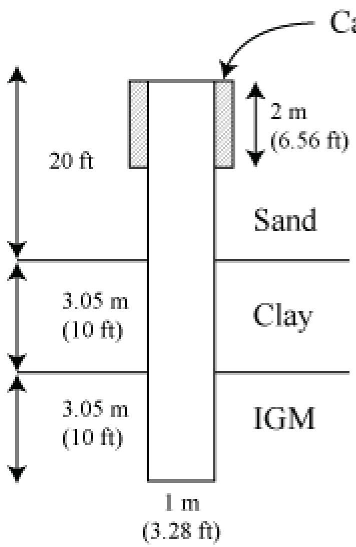
where $q_b = \Lambda W_t^{0.67}$, and

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

For 2mm and 5mm settlement at top of rock socket (w_t = 0.002m and w_t = 0.005m)

Settlement	**2mm**	**5mm**
E_{m_avg}	110400 kPa	110400 kPa
f_{su_avg}	151.41 kPa	151.41 kPa
n_{avg}	0.102	0.102
Omega, Ω	1.462	1.462
Lamda, Γ	0.507	0.507
Λ	14769 kN/m^2	14769 kN/m^2
Θ_f	0.437	1.093
K_f	0.346	0.573
R	0.2	0.5
Side Friction (Q_s, Sand)	315.65 kN	484.50 kN
Side Friction (Q_s, Rock)	502.00 kN	831.3 kN
End bearing (Q_b, Rock)	180.3 kN	333.2 kN
Total Capacity (Q_t)	998 kN	1649 kN

1.7.6 Example 6: IGM: (Sand, Clay & Limestone)



$\gamma = 100\,\text{pcf}\ (15.708\,\text{kN/m}^3)$
$N = 10$

$\gamma = 100\,\text{pcf}\ (15.708\,\text{kN/m}^3)$
$c = 1.9\,\text{tsf}\ (181.94\,\text{kPa})$

$$\text{Limestone}: q_u = 10\,\text{tsf}\ (957.6\,\text{kPa}, 0.96\,\text{MPa})$$
$$q_t = 1\,\text{tsf}\ (95.76\,\text{kPa}, 0.096\,\text{MPa})$$
$$\gamma = 135\,\text{pcf}\ (21.2\,\text{kN/m}^3), \gamma_c = 20.4\,\text{kN/m}^3$$
$$E_c = 57{,}000\sqrt{f'_y} = 57{,}000\sqrt{5000\,\text{psi}}$$
$$= 4.03E6\ \text{psi}\ \ (2.77E6\,\text{kPa})$$
$$f_{su} = \frac{1}{2}\sqrt{q_u}\sqrt{q_t} = 151.41\,\text{kPa}$$
Smooth socket IGM type = 2.0

1. Skin Friction (Sand):

   In sand layers with blowcounts of less than 15, an adjustment is made by dividing the blowcount by 15, and multiplying this value by β.

$$Q_s = \frac{\pi D}{2000}\int_{6.56}^{20} \beta \bullet \left(\frac{N}{15}\right)\sigma_z\, dz$$

$$Q_s = \frac{3.28\pi}{2000} \bullet \left(\frac{10}{15}\right)\int_{6.56}^{20}\left(1.5 - 0.135\sqrt{Z}\right)\gamma\, z\, dz$$

$$= 0.003435\left[75 \bullet \left(20^2 - 6.56^2\right) - 5.4 \bullet \left(20^{5/2} - 6.56^{5/2}\right)\right]$$

Q_s = 60.82 tons (541.08 kN)

2. Skin Friction (Clay):

$$Q_s = \pi\ D\ L\ a\ C_u$$

Q_s = π (1 m) (3.05 m) (0.55 · 181.94)

Q_s = 958.85 kN (107.78 tons)

3. Skin Friction (Rock):

$$Q_s = \pi\ D\ L\ f_{su}$$

$$f_{su} = 0.5\sqrt{q_u}\sqrt{q_t}$$

Q_s = π (1 m) (3.05 m) (151.41 kN/m^2)

Q_s = 1450.79 kN (163.075 tons)

Total Skin Friction

Σ Q_s = 541.08 + 958.85 kN + 1450.79kN = 2950.72 kN (331.67 tons)

4. End Bearing (Rock):

Assuming qb = ½ qu

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

Q_b = 376 kN (42.27 tons)

5. Total Capacity :

   $Q_t = \Sigma Q_s + Q_b$ = 2950.72 + 376 = 3326.72 kN (373.94 tons)

Settlement Calculation

Sand Layer

Skin Friction -

$Q_s/Q_{smax} = -2.16*R^4+6.34*R^3-7.36*R^2+4.15*R$ For $R \le 0.908333$

$Q_s/Q_{smax} = 0.978112$ for $R > 0.908333$

where R = 100 * Settlement / D, and Q_{smax} = 541.08 kN (Check above)

Clay Layer

Skin Friction -

$Q_s/Q_{smax} = 0.593157*R/0.12$ for $R \le 0.12$

$Q_s/Q_{smax} = R/(0.095155+0.892937*R)$ for $R \le 0.74$

$Q_s/Q_{smax} = 0.978929-0.115817*(R-0.74)$ for $R \le 2.0$

$Q_s/Q_{smax} = 0.833$ for $R > 2.0$

where R = 100 * Settlement / D, and Q_{smax} = 958.85 kN (Check above)

Rock Layer:

Skin Friction -

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

$$E_{m\_avg} = \frac{\Sigma E_{mk} L_k}{\Sigma L_k}$$ where $E_m = 115\, q_u$

$$f_{su\_avg} = \frac{\Sigma f_{su} L_k}{\Sigma L_k}$$ where, $f_{su} = A \bullet q_u^B$ (selecting A = 0.4 and B = 1)

For "rough" sockets; $$n = \frac{\sigma_n}{q_u}$$

where, $\sigma_n = \gamma_c Z_c M$

Only one IGM layer is present.

$E_{m\_avg} = E_m = 115\, q_u$ = 110400 kPa

$f_{su\_avg} = f_{su}$ =151.41 kPa

n_{avg} = n

γ_c = 20.4 kN/m^3, M = 0.548 (for Z_c = 10.675 m and Slump = 152.4 mm)

$\sigma_n = \gamma_c Z_c M$ =119.34 kN/m^2

$n_{avg} = n(\sigma_n, q_u)$ = 0.1246

Calculate Ω, and Γ

$$\Omega = 1.14\left(\frac{L}{D}\right)^{0.5} - 0.05\left(\left(\frac{L}{D}\right)^{0.5} - 1\right)\log_{10}\left(\frac{E_c}{E_{m\_avg}}\right) - 0.44$$

$$\Gamma = 0.37\left(\frac{L}{D}\right)^{0.5} - 0.15\left(\left(\frac{L}{D}\right)^{0.5} - 1\right)\log_{10}\left(\frac{E_c}{E_{m\_avg}}\right) + 0.13$$

Calculate Θ_f and K_f

$$\Theta_f = \frac{E_{m\_avg}\Omega}{\pi L \Gamma f_{su\_avg}} w_t$$

$$K_f = n_{avg} + \frac{(\Theta_f - n_{avg})(1 - n_{avg})}{\Theta_f - 2n_{avg} + 1} \quad < 1$$

where, w_t = Settlement at top of rock socket

Side shear load capacity – settlement

$$Q_s = \pi D L \Theta_f f_{su\_avg} \qquad \Theta_f < n_{avg}$$

$$Q_s = \pi D L K_f f_{su\_avg} \qquad \Theta_f > n_{avg}$$

K_f = 1.0 when, $K_f < n_{avg}$

End bearing –settlement (Rock)

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

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

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

For 2mm and 5mm settlement at top of rock socket (w_t = 0.002m and w_t = 0.005m)

Settlement	**2mm**	**5mm**
E_{m_avg}	110400 kPa	110400 kPa
f_{su_avg}	151.41 kPa	151.41 kPa
n_{avg}	0.1246	0.1246
Omega, Ω	1.462	1.462
Lamda, Γ	0.507	0.507
Λ	14769 kN/m^2	14769 kN/m^2
Θ_f	0.437	1.093
Kf	0.355	0.608
R	0.2	0.5
Side Friction (Q_s, Sand)	315.65 kN	482.91 kN
Side Friction (Q_s, Clay)	700 kN	884.84 kN
Side Friction (Q_s, Rock)	514.81 kN	847.55 kN
End bearing (Q_b, Rock)	180.3 kN	333.2 kN
Total Capacity (Q_t)	1710.76 kN	2548.50 kN

2 Driven Piles

1. SPT
2. CPT

2.1 SPT

1. Concrete Piles
2. Steel Pipe Piles
3. Steel H Piles
4. Concrete Cylinder Piles
5. Methodology

2.1.1 Piles: Precast Concrete Piles (PCP)

In FB-Deep, there are two types of precast concrete pile (PCP) section types: square, and round. The unit skin friction and unit end bearing formulas are given in the two tables immediately below.

SIDE FRICTION

Unit side friction at a given depth is also based on the type of soil and the corresponding SPT blowcount. The following table shows the empirically derived equations for ultimate unit side friction versus blowcount for the four soil types.

Soil Type	Soil Description	Unit Skin Friction (tsf)	SPT Blow Count Range
1	Plastic Clay	$f_s = 2 \cdot N \cdot (110 - N) / 4006.6$	$3 \leq N \leq 60$
2	Clay-silt-sand mixtures	$f_s = 2 \cdot N \cdot (110 - N) / 4583.3$	$3 \leq N \leq 60$
3	Clean Sands	$f_s = 0.019 \cdot N$	$3 \leq N \leq 60$
4	Soft limestone, very shelly sand	$f_s = 0.01 \cdot N$	$3 \leq N \leq 100$
5	Void	$f_s = 0.0$	n/a

To convert TSF to kPa multiply the f_s by 95.76.

END BEARING

Unit end bearing at a given depth is based on the type of soil and the corresponding SPT blowcount. The following table shows the empirically derived equations for mobilized unit end bearing versus blowcount for the four soil types.

Soil Type	**Soil Description**	**Unit End Bearing (tsf)**	**SPT Blow Count Range**
1	Plastic Clay	$q_t = 0.7 \cdot N / 3$	$3 \leq N \leq 60$
2	Clay-silt-sand mixtures	$q_t = 1.6 \cdot N / 3$	$3 \leq N \leq 60$
3	Clean Sands	$q_t = 3.2 \cdot N / 3$	$3 \leq N \leq 60$
4	Soft limestone, very shelly sand	$q_t = 0.7873 \cdot N + 0.0026 \cdot N^2 + 1 \cdot 10^{-5} \cdot N^3$	$3 \leq N \leq 100$
5	Void	$q_t = 0.0$	n/a

To convert TSF to kPa multiply the q_t by 95.76.

The methodology used to calculate the end bearing capacity for a given depth includes the critical depth correction, and the end bearing contribution zone of 8B above and 3.5B below the pile tip is considered. Exception is when the PCP tip is in Limestone, the end bearing contribution zone of 4B below the pile tip only is considered and critical depth correction in not performed.

Database research at the University of Florida has indicated that, using the RB-121 methodology, it requires excessive pile movement to mobilize the calculated ultimate pile capacities. To better match the measured load test capacity values (using Davisson failure criteria) with calculated values, the "Mobilized End Bearing" has been set equal to 1/3 of the RB-121 ultimate end bearing value. The Davisson capacity equals the ultimate side friction plus the mobilized end bearing. The allowable pile capacity is taken as 1/2 the Davisson capacity. The ultimate capacity is then equal to the ultimate side friction plus 3 times the mobilized end bearing.

2.1.2 Piles: Steel Pipe Piles

SIDE FRICTION:

The following table shows the ultimate side friction versus SPT blowcounts for the five soil types, for steel pipe piles.

Soil Type	Soil Description	Ultimate Unit Side Friction (tsf)	SPT Blow Count Range
1	Plastic Clay	$f_s = 0.4236 \cdot ln(N) - 0.5404$	$3 \leq N \leq 100$
2	Clay-silt-sand mixtures, Very silty sand, silts and marls	$f_s = 0.029 + 0.045 \cdot N - 8.98 \cdot 10^{-4} \cdot N^2 + 6.371 \cdot 10^{-6} \cdot N^3$	$3 \leq N < 40$
		$f_s = 0.799944 + 0.00362 \cdot (N - 40)$	$40 \leq N \leq 100$
3	Clean Sands	$f_s = -0.026 + 0.023 \cdot N - 1.435 \cdot 10^{-4} \cdot N^2 - 6.527 \cdot 10^{-7} \cdot N^3$	$3 \leq N < 40$
		$f_s = 0.622627 + 0.003689 \cdot (N - 40)$	$40 \leq N \leq 100$
4	Soft limestone, very shelly sand	$f_s = 0.01 \cdot N$	$3 \leq N \leq 100$
5	Void	$f_s = 0.0$	n/a

END BEARING:

For end bearing capacity of steel pipe piles there are two groups, based on pile diameter: steel pipe piles with a diameter of 36 inches or less, and those with a diameter greater than 36 inches.

For **Diameter ≤ 36" (914.4mm)**

The following table provides the plots of mobilized unit end bearing capacity versus SPT blow count for the five soil types, for steel pipe piles with a diameter ≤ 36" (914.4mm).

Soil Type	Soil Description	Unit End Friction (tsf)	SPT Blow Count Range
1	Plastic Clay	$q = \frac{0.7 * N}{3}$	3 ≤ *N* ≤ 100
2	Clay-silt-sand mixtures, Very silty sand, silts and marls	$q = \frac{1.6 * N}{3}$	3 ≤ *N* ≤ 100
3	Clean Sands	$q = \frac{3.2 * N}{3}$	3 ≤ *N* ≤ 30
		$q = 32.0 + \frac{4.0 * (N - 30.0)}{30.0}$	30 < *N* ≤ 100
4	Soft limestone, very shelly sand	$q = 1.2 * N$	3 ≤ *N* ≤ 30
		$q = 36.0 + \frac{7.0 * (N - 30.0)}{30.0}$	30 < *N* ≤100
5	Void	$q = 0.0$	n/a

For **Diameter > 36" (914.4mm)**

The formulas for large diameter steel pipe piles can be seen in the following tables, which are based on the work of M. C. McVay, D. Badri, and Z. Hu, from the report "Determination of Axial Pile Capacity of Prestressed Concrete Cylinder Piles", 2004, Table 8-3, page 95.

Soil Type	Soil Description	Unit End Friction (tsf)	SPT Blow Count Range

1	Plastic Clay	$q_t = .2226N$	3 ≤ *N* ≤ 100
2	Clay-silt-sand mixtures	$q_t = .4101N$	3 ≤ *N* ≤ 100
3	Clean Sands	$q_t = .5676N$	3 ≤ *N* ≤ 100
4	Soft limestone, very shelly sand	$q = 1.2 * N$	3 ≤ *N* ≤ 30
		$q = 36.0 + \frac{7.0 * (N - 30.0)}{30.0}$	30 < *N* ≤ 100
5	Void	$q = 0.0$	n/a

Note: for input files created on FB-Deep, version 1.18 and previous, the end bearing formulas will NOT be used for steel pipe piles with a diameter greater than 36" (914.4). Instead, the formulas for diameter lesser than 36" (914.4) will be used. This is for the sake of backwards compatibility. If you want an old file to use these new large diameter formulas, simply open the file in FB-Deep v201 or later, and resave it.

N is blowcount

f_s is unit skin friction

q_t is unit end bearing

The methodology used to calculate the end bearing capacity for a given depth includes the critical depth correction, the end bearing contribution zone of 8B above and 3.5B below the pile tip is considered.

*Pipe pile can be assigned open-ended or closed-ended by the user. For open-ended condition, the program internally checks the pile to be plugged or unplugged. The capacity is then selected as per the lower prediction between the sum of “outer” and “inner” skin friction and end bearing on the annulus (unplugged condition), and sum of “outer” skin friction and end bearing of the closed-end cross section (plugged condition).

The corrected mobilized end bearing capacity is then computed as per plugged or unplugged condition.

PILE CAPACITY

Davisson's criteria is not used for this pile type because of the uncertainty of computing elastic settlement for a plugged pile. Instead the mobilized ultimate pile capacity is defined as an applied load at a settlement equal to three percent (3%) the diameter of the pile. The allowable pile capacity is equal to 1/2 the mobilized ultimate pile capacity. The ultimate pile capacity is determined as the ultimate side friction plus three (3) times the mobilized end bearing.

2.1.3 Piles: Steel H Piles

SIDE FRICTION

The empirical equations for the ultimate side friction versus SPT blowcount, which are derived from the data-base, are as follows:

Soil Type	Soil Description	Unit Skin Friction (tsf)	SPT Blow Count Range
1	Plastic Clay	$f_s = 2 \cdot N \cdot (110 - N) / 5335.9$	$3 \le N \le 60$
2	Clay-silt-sand mixtures	$f_s = -0.0227 + 0.033 \cdot N - 4.576 \cdot 10^{-4} \cdot N^2 + 2.465 \cdot 10^{-6} \cdot N^3$	$3 \le N < 75$
3	Clean Sands	$f_s = 0.0112 \cdot N$	$3 \le N \le 60$
4	Soft limestone, very shelly sand	$f_s = 0.0076 \cdot N$	$3 \le N < 100$
5	Void	$f_s = 0.0$	n/a

The H-Pile circumference used for the skin friction calculation in all soil types (soil type 1, 2, 3, and 4, namely, clay, silt, sand , and limestone) is considering 50% plugged condition. Circumference = (3 ·Width + 2 ·Depth)

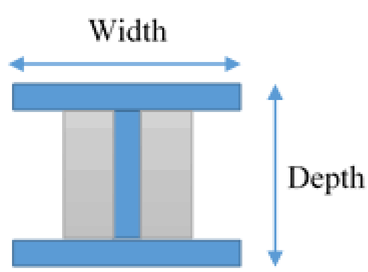


Figure: 2.1.a 50% plugged condition

The 'Width', and 'Depth' field can be seen in the 'Pile Geometry' table on the program's main screen.

END BEARING

The empirical equations (for the five soil types) for the plots of the mobilized unit end bearing capacity versus SPT blow count are presented as follows:

Soil Type	**Soil Description**	**Unit End Bearing (tsf)**	**SPT Blow Count Range**
1	Plastic Clay	$q_t = 0.7 \cdot N / 3$	$3 \le N \le 60$
2	Clay-silt-sand mixtures	$q_t = 1.6 \cdot N / 3$	$3 \le N < 60$
3	Clean Sands	$q_t = 3.2 \cdot N / 3$	$3 \le N \le 60$
4	Soft limestone, very shelly sand	$q_t = 3.6 \cdot N / 3$	$3 \le N < 100$
5	Void	$q_t = 0.0$	n/a

The methodology used to calculate the end bearing capacity for a given depth includes no critical depth correction, and the end bearing contribution zone of 4B below the pile tip only.

For H-Pile end bearing calculations in all soil types (soil type 1, 2, 3, and 4, namely, clay, silt, sand , and limestone), 50% plugged condition is considered. Half of the product of the user-inputted 'Width' and 'Depth' is used to calculate the pile tip area (Pile Tip Area = 0.5 x Width x Depth).

The 'Width', and 'Depth' field can be seen in the 'Pile Geometry' table on the program's main screen.

PILE CAPACITY

Davisson's criteria is used for this pile type. The Davisson capacity equals the ultimate side friction plus the mobilized end bearing. The mobilized end bearing capacity is defined as 1/3 of the calculated ultimate end bearing using the RB-121 methodology. The allowable pile capacity is taken as 1/2 the Davisson capacity. The ultimate capacity is then equal to the ultimate side friction plus 3 times the mobilized end bearing, with the following exception: for H-Piles tipped in sand or limestone, the ultimate pile capacity is the ultimate side friction plus 2 times the mobilized end bearing.

2.1.4 Concrete Cylinder Piles

SIDE FRICTION:

The following table shows the ultimate side friction versus SPT blowcounts for the five soil types, for concrete cylinder piles.

Soil Type	Soil Description	Ultimate Unit Side Friction (tsf)	SPT Blow Count Range
1	Plastic Clay	$f_s = 0.4236 \cdot ln(N) - 0.5404$	$3 \le N \le 100$
2	Clay-silt-sand mixtures, Very silty sand, silts and marls	$f_s = 0.029 + 0.045 \cdot N - 8.98 \cdot 10^{-4} \cdot N^2 + 6.371 \cdot 10^{-6} \cdot N^3$	$3 \le N < 40$
		$f_s = 0.799944 + 0.00362 \cdot (N - 40)$	$40 \le N \le 100$
3	Clean Sands	$f_s = -0.026 + 0.023 \cdot N - 1.435 \cdot 10^{-4} \cdot N^2 - 6.527 \cdot 10^{-7} \cdot N^3$	$3 \le N < 40$
		$f_s = 0.622627 + 0.003689 \cdot (N - 40)$	$40 \le N \le 100$
4	Soft limestone, very shelly sand	$f_s = 0.01 \cdot N$	$3 \le N \le 100$
5	Void	$f_s = 0.0$	n/a

END BEARING:

For end bearing capacity of steel pipe piles there are two groups, based on pile diameter: steel pipe piles with a diameter of 36 inches or less, and those with a diameter greater than 36 inches.

For **Diameter ≤ 36" (914.4mm)**

The following table provides the plots of mobilized unit end bearing capacity versus SPT blow count for the five soil types, for concrete cylinder piles with a diameter ≤ 36" (914.4mm).

Soil Type	**Soil Description**	**Unit End Bearing (tsf)**	**SPT Blow Count Range**
1	Plastic Clay	$q_t = 0.7 \cdot N / 3$	$3 \leq N \leq 100$
2	Clay-silt-sand mixtures	$q_t = 1.6 \cdot N / 3$	$3 \leq N \leq 100$
3	Clean Sands	$q_t = 3.2 \cdot N / 3$	$3 \leq N \leq 100$
4	Soft limestone, very shelly sand	$q_t = 3.6 \cdot N / 3$	$3 \leq N \leq 100$
5	Void	$q_t = 0.0$	n/a

For **Diameter > 36" (914.4mm)**

Large diameter cylinder piles have a diameter greater than 36 inches (914.4mm). The formulas for these piles can be seen in the following table, which are based on the work of M.C. McVay, D. Badri, and Z.Hu, from the report "Determination of Axial Pile Capacity of Prestressed Concrete Cylinder Piles", 2004, Table 8-3, page 95.

Soil Type	**Soil Description**	**Unit End Bearing (tsf)**	**SPT Blow Count Range**
1	Plastic Clay	$q_t = 0.2226 \cdot N$	$3 \leq N \leq 100$
2	Clay-silt-sand mixtures	$q_t = 0.4101 \cdot N$	$3 \leq N \leq 100$
3	Clean Sands	$q_t = 0.5676 \cdot N$	$3 \leq N \leq 100$
4	Soft limestone, very shelly sand	$q_t = 3.6 \cdot N / 3$	$3 \leq N \leq 100$
5	Void	$q_t = 0.0$	n/a

*N is blowcount

f_s is unit skin friction

q_t is unit end bearing

Note: for input files created on FB-Deep, version 1.18 and previous, the above formulas will NOT be used for cylinder piles with a diameter greater than 36" (914.4). Instead, the formulas for diameter lesser than 36" (914.4) will be used. This is for the sake of backwards compatibility. If you want an old file to use the new large diameter formulas, simply open the input file in FB-Deep v2.01 or later, and resave it.

The methodology used to calculate the end bearing capacity for a given depth includes the critical depth correction, the end bearing contribution zone of 8B above and 3.5B below the pile tip is considered.

*Concrete Cylinder pile can be assigned open-ended or closed-ended by the user. For open-ended condition, the program internally checks the pile to be plugged or unplugged. The capacity is then selected as per the lower prediction between the sum of “outer” and “inner” skin friction and end bearing on the annulus (unplugged condition), and sum of “outer” skin friction and end bearing of the closed-end cross section (plugged condition).

The corrected mobilized end bearing capacity is then computed as per plugged or unplugged condition.

2.1.5 Methodology

The methodology is based on empirical correlations between cone penetrometer tests and standard penetration tests for typical Florida soil types. Unit end bearing resistance and unit skin friction resistance versus SPT N values are given in RB-121 for the different soil types. The program recognizes five soil types:

1.Plastic Clay
2.Clay and Silty Sand
3.Clean Sand
4.Limestone, Very Shelly Sand
5.Void Layer

Per soil classification for mixture of soil and limestone aggregates, types of soil (either Type 2 or 3) is recommended.

In order to define the soil profile, the user inputs the depth, N value, and soil type of each SPT sample. A layer change is established at the elevation where a new soil type is input. The unit end bearing and unit side friction values at the interface correspond to the soil type below the layer change. Typical blowcount spacings are every 2.5 feet (.762 meters). Results based on spacings of greater than 5.00 feet (1.524 meters) can become inaccurate.

END BEARING

One of the basic assumptions of this program is that the soil 3.5B below and 8.0B above the pile tip contributes to the end bearing capacity. The value B refers to the diameter or width of the pile. An exception occurs when the bearing layer is weaker than the overlying layer. If this is the case, the upper limit terminates at the layer change rather than 8.0B, to prevent a "punching" type end bearing failure. Individual unit end bearing capacities are computed at each depth within the above ranges. When the pile tip, upper and lower limits of the range do not correspond to an actual sample depth, the capacity is determined by interpolating between the samples just above and below the desired depth. The average unit end bearing in each layer or range is determined by taking a weighted average of the end bearing values within the range.

The average unit end bearing above the pile tip is added to the average unit end bearing below the pile tip. The sum is divided by 2.0 to yield the average unit end bearing value. The uncorrected end bearing capacity is the average unit end bearing times the pile tip cross sectional area. Correction factors for the final end bearing capacity are discussed in the subsequent section CRITICAL DEPTH CORRECTIONS. (Also see section on AVERAGING UNIT SKIN FRICTION AND END BEARING OVER A LAYER below).

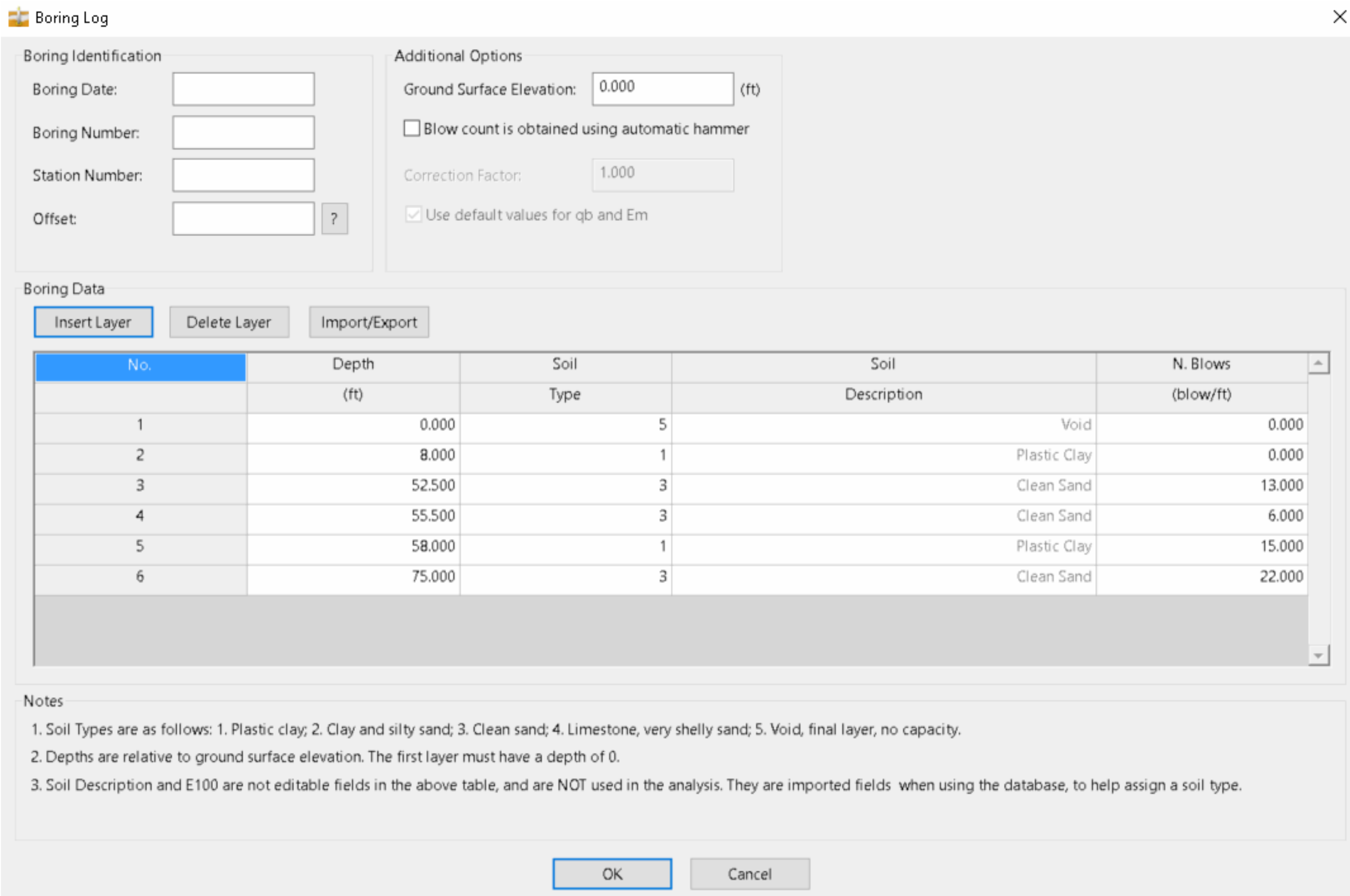
Exception is when the prestressed concrete piles (PCP) is in Limestone (soil type 4) or H-Pile in all soil types, the end bearing contribution zone of 4B below the pile tip only is considered and critical depth correction is not performed.

SIDE FRICTION

The ultimate side friction resistance in the layers above the bearing layer and in the bearing layer are determined separately. As with the unit end bearing, a weighted average technique is used to establish the ultimate unit skin friction in each layer or range. The unit side friction is multiplied by the corresponding surface area in each layer to calculate a cumulative ultimate skin friction. A correction factor for the ultimate side friction capacity of the bearing layer is required, and is discussed below. (Also see section on AVERAGE BLOWCOUNT below).

AVERAGING UNIT SKIN FRICTION AND END BEARING OVER A LAYER

To determine the unit skin friction and unit end bearing contribution for a soil layer, an averaging technique is used. The average unit skin friction for a soil layer is computed as follows: the user-inputted blowcount for the top of a soil layer is applied to the unit skin friction formula for the layer. The next entry in the Boring Log is then inspected, and based on its blowcount and soil type, another unit skin friction calculation is made. These two unit skin frictions are then averaged. See Figure: 2.1.b and Figure: 2.1.c titled "Unit Skin Friction Averaging, A" and "Unit Skin Friction Averaging, B").



Boring Log

Boring Identification

Boring Date:

Boring Number:

Station Number:

Offset: ?

Additional Options

Ground Surface Elevation: 0.000 (ft)

Blow count is obtained using automatic hammer

Correction Factor: 1.000

Use default values for qb and Em

Boring Data

Insert Layer | Delete Layer | Import/Export

No.	Depth (ft)	Soil Type	Soil Description	N. Blows (blow/ft)
1	0.000	5	Void	0.000
2	8.000	1	Plastic Clay	0.000
3	52.500	3	Clean Sand	13.000
4	55.500	3	Clean Sand	6.000
5	58.000	1	Plastic Clay	15.000
6	75.000	3	Clean Sand	22.000

Notes

1. Soil Types are as follows: 1. Plastic clay; 2. Clay and silty sand; 3. Clean sand; 4. Limestone, very shelly sand; 5. Void, final layer, no capacity.
2. Depths are relative to ground surface elevation. The first layer must have a depth of 0.
3. Soil Description and E100 are not editable fields in the above table, and are NOT used in the analysis. They are imported fields when using the database, to help assign a soil type.

OK | Cancel

Figure: 2.1.b Unit Skin Friction Averaging, A

In Figure: 2.1.b, "Unit Skin Friction Averaging: A", a clay soil layer begins at a depth of 8.00, with a blowcount of 0 blows/ft. This layer extends to a depth of 52.5 feet, where a sand layer begins, with a blowcount of 13 blows/ft. The average unit skin friction for this clay layer is computed as thusly:

Use soil type 1 (clay) formula for the depth at the top of the clay layer (8.00 ft):

(2.0 * N)(110 – N) / 4006.6 =

(2.0 * 0.0)(110 – 0.0) / 4006.6 =

(0.0)(110) / 4006.6 =

0.0 tsf

Use soil type 3 (sand) formula for the depth at the top of the next layer (52.5 feet):

0.019 * N =

0.019 * (13) =

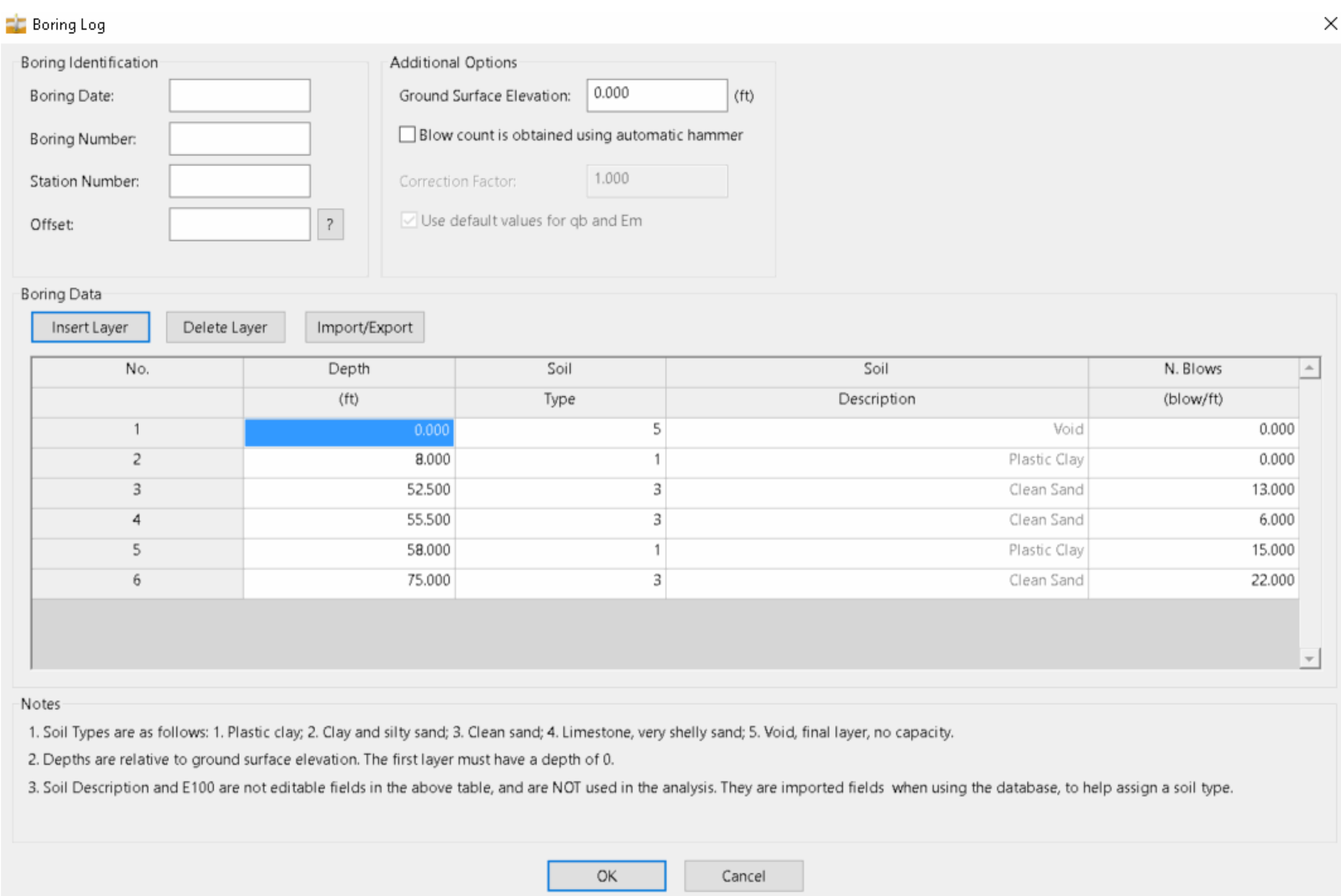
0.247 tsf

Average the two unit skin frictions:

(0.0 + 0.247) / 2 =

0.1235 tsf is the unit skin friction for the clay layer

Identical logic is used to compute an average unit end bearing in a layer, the only difference being the unit end bearing formulas are used in place of the skin friction formulas. Note, in the above example, if the user's intention was for the clay layer to have a blowcount of 0 blows/ft throughout the layer, then an additional entry could be made to the boring log for a clay layer with a depth of 52.5 feet, and a blowcount of 0.0; Then, the average unit skin friction would be be (0.0 + 0.0) / 2 = 0.0 tsf. This additional layer entry at 52.5 feet can be seen in Figure: 2.1.c "Unit Skin Friction Averaging, B".


Boring Log

Boring Identification

Boring Date:

Boring Number:

Station Number:

Offset: ?

Additional Options

Ground Surface Elevation: 0.000 (ft)

Blow count is obtained using automatic hammer

Correction Factor: 1.000

Use default values for qb and Em

Boring Data

Insert Layer | Delete Layer | Import/Export

No.	Depth (ft)	Soil Type	Soil Description	N. Blows (blow/ft)
1	0.000	5	Void	0.000
2	8.000	1	Plastic Clay	0.000
3	52.500	3	Clean Sand	13.000
4	55.500	3	Clean Sand	6.000
5	58.000	1	Plastic Clay	15.000
6	75.000	3	Clean Sand	22.000

Notes

1. Soil Types are as follows: 1. Plastic clay; 2. Clay and silty sand; 3. Clean sand; 4. Limestone, very shelly sand; 5. Void, final layer, no capacity.
2. Depths are relative to ground surface elevation. The first layer must have a depth of 0.
3. Soil Description and E100 are not editable fields in the above table, and are NOT used in the analysis. They are imported fields when using the database, to help assign a soil type.

OK | Cancel

Figure: 2.1.c Unit Skin Friction Averaging, B

FICTITIOUS LAYERS

A fictitious layer is an imaginary soil layer with a blowcount of less than 3, which will be rounded to a 0. Its role is to break up a thick soil layer whose blowcount varies greatly over the layer depth, to help ensure more accurate results. And because of the zeroed blowcount, fictitious layers do NOT make a skin friction or end bearing contribution of their own. Fictitious layers also help to clarify the contributions of each section of the thick soil layer, at various depths. (See Figure: 2.1.d and Figure: 2.1.e).

In this example, the sand layer is 30 feet thick, with the blowcounts varying within the layer as follows:

Depth (ft)	Blowcount (blows/ft)
0.0	7.0
7.5	38.0
19.0	51.0
30.0	15.0

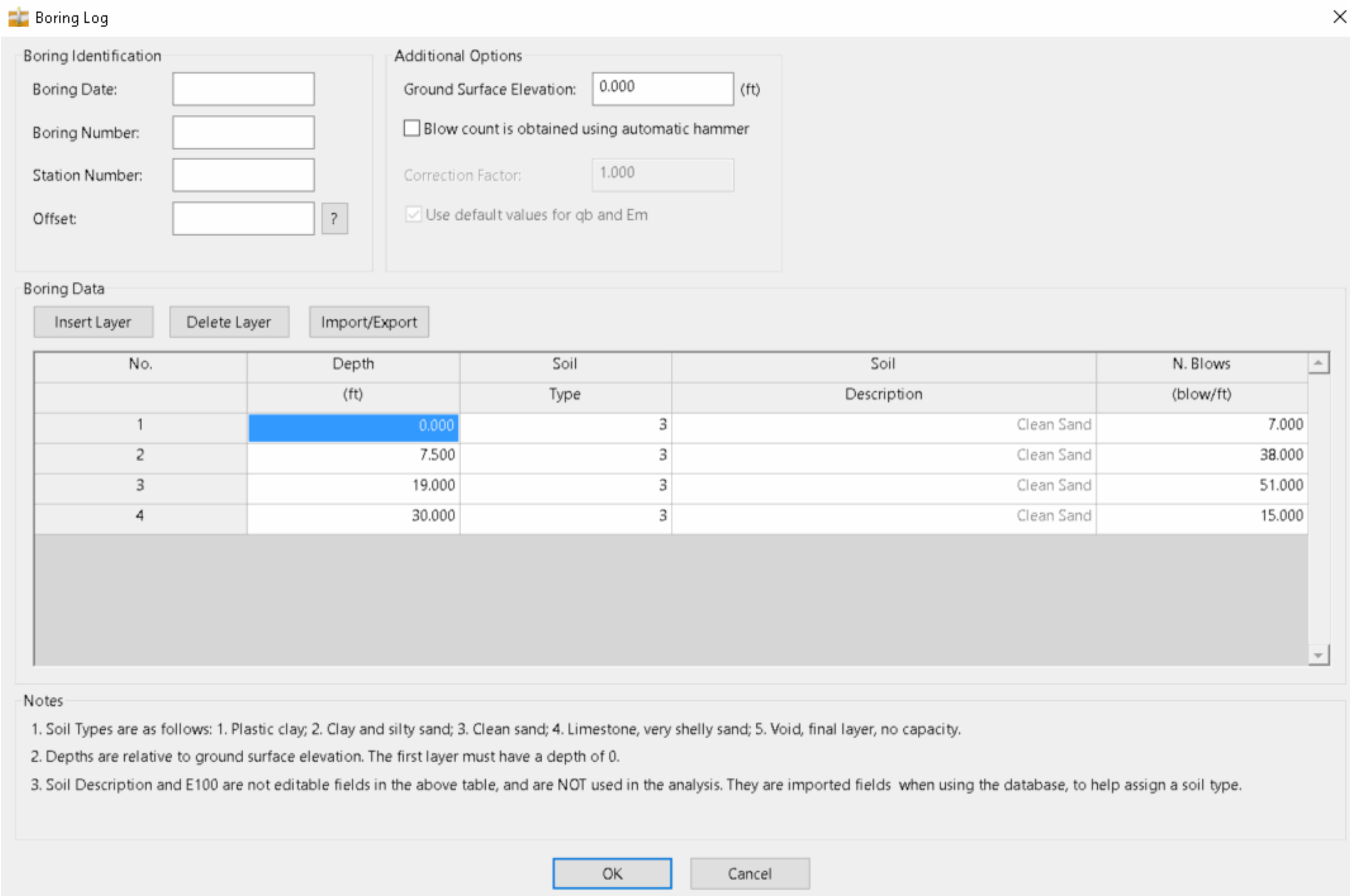
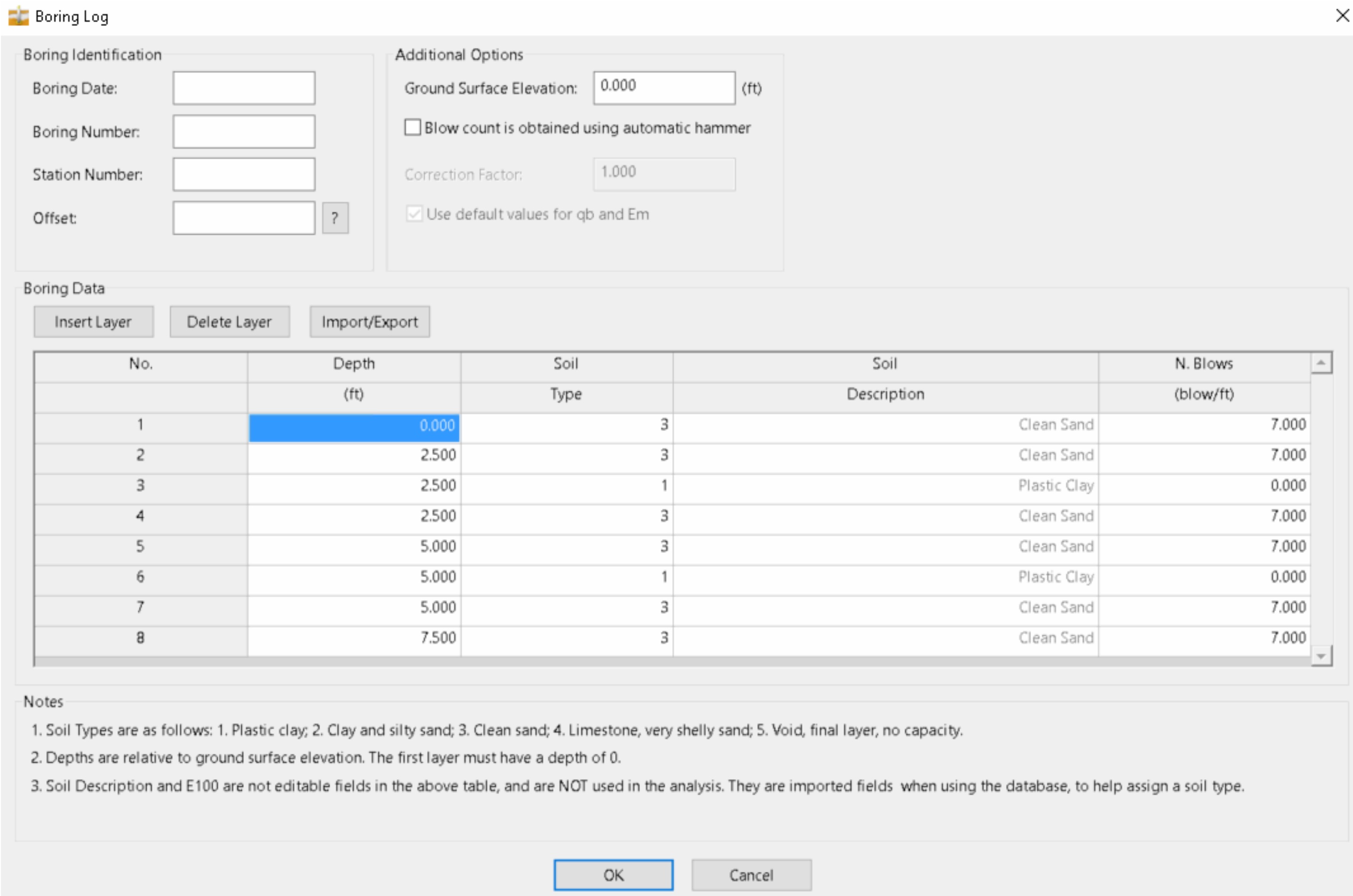


Figure: 2.1.d "Fictitious Layer, A"

No fictitious layers exist in Figure: 2.1.d. Instead, the entire boring log is one single layer, with four different blowcounts, depending on the depth. The first blowcount change occurs at a depth of 7.5 feet, where the blowcount changes from 7 to 38. To insert fictitious layers, this 7.5 foot deep section is divided into 3 smaller sections, each 2.5 feet in depth. Each 2.5 foot section has a blowcount of 7. Between each 2.5 foot section, a soil layer with a different type is inserted, with a width of 0.00 feet, and a blowcount of 0.00. A fictitious layer is inserted at depths of 2.5 feet and 5.0 feet. Because the soil type of the given layer is sand, a clay layer is inserted as the fictitious layer.


Boring Log

Boring Identification

Boring Date:

Boring Number:

Station Number:

Offset: ?

Additional Options

Ground Surface Elevation: 0.000 (ft)

Blow count is obtained using automatic hammer

Correction Factor: 1.000

Use default values for qb and Em

Boring Data

Insert Layer | Delete Layer | Import/Export

No.	Depth (ft)	Soil Type	Soil Description	N. Blows (blow/ft)
1	0.000	3	Clean Sand	7.000
2	2.500	3	Clean Sand	7.000
3	2.500	1	Plastic Clay	0.000
4	2.500	3	Clean Sand	7.000
5	5.000	3	Clean Sand	7.000
6	5.000	1	Plastic Clay	0.000
7	5.000	3	Clean Sand	7.000
8	7.500	3	Clean Sand	7.000

Notes

1. Soil Types are as follows: 1. Plastic clay; 2. Clay and silty sand; 3. Clean sand; 4. Limestone, very shelly sand; 5. Void, final layer, no capacity.
2. Depths are relative to ground surface elevation. The first layer must have a depth of 0.
3. Soil Description and E100 are not editable fields in the above table, and are NOT used in the analysis. They are imported fields when using the database, to help assign a soil type.

OK | Cancel

Figure: 2.1.e "Fictitious Layer, B"

This process is repeated for the other sections of the sand layer, between 7.5 and 19 feet, and between 19 and 30 feet.

CRITICAL DEPTH CORRECTIONS

The concept of critical depth is used in both the end bearing and side friction calculations. The ultimate end bearing capacity of a pile cannot be fully mobilized with the bearing layer until the pile tip reaches a critical depth/width ratio (D/B), where D is the depth of embedment in the bearing layer and B is the pile width. The bearing layer is the soil layer in which the pile tip is embedded. The following D/B ratios are used in FB-Deep; these have been modified from those shown in the original RB-121 study.

Soil Type	Soil Description	Critical Depth Ratio (D/B)
1	Plastic Clay	2
2	Clay and Silty Sand	3
3	Clean Sands (N=12 or less) (N=30 or less) (N>30)	 6 9 12
4	Limestone, Very Shelly Sand	6

When the actual depth of embedment is less than the critical depth and when the bearing layer is stronger than the overlying layer, a correction (reduction) is applied to the unit end bearing capacity. The corrected unit end bearing value is determined by interpolating between the bearing capacity at the top of the bearing layer and the bearing capacity at the pile tip according to the following equation:

$$q = q_{LC} + \frac{D_A}{D_C}(q_T - q_{LC})$$

where:

q = Corrected unit end bearing @ pile tip

q_{LC} = Unit end bearing at layer change

q_T = Uncorrected unit end bearing @ pile tip

D_A = Actual embedment in bearing layer

D_C = Critical depth of embedment+

If the pile tip embedment in the bearing layer is less than the critical depth and the overlying layer is weaker than the bearing layer, the side friction in the bearing layer is corrected (reduced) in accordance with the following equation:

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

where:

CSFBL = Corrected side friction in the bearing layer

SFBL = Uncorrected side friction in the bearing layer

q_T, q_{LC}, D_A, D_C as previously defined

If the pile tip embedment in the bearing layer is greater than the critical depth and when the overlying layer is weaker than the bearing layer, the skin friction between the top of the bearing layer and the critical depth is corrected (reduced) according to the following equation:

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

where:

CSFACD = Corrected side friction in the bearing layer from the top of the bearing layer to the critical depth

USFACD = Uncorrected side friction from the top of the bearing layer to the critical depth

q_{CD} = Unit end bearing at critical depth

q_{LC} = as previously defined

No corrections are applied when the overlying layer is stronger than the bearing layer.

2.2 CPT

1. Schmertmann
2. UF
3. LCPC
4. CPT Modeling

2.2.1 Schmertmann

This method was first proposed by Schmertmann in 1978. It uses both tip resistance and sleeve friction to predict pile capacity. The pile's unit tip capacity is calculated by the minimum path rule shown in Figure: 2.2.a. Schmertmann set an upper limit of 150 tsf for the unit tip capacity.

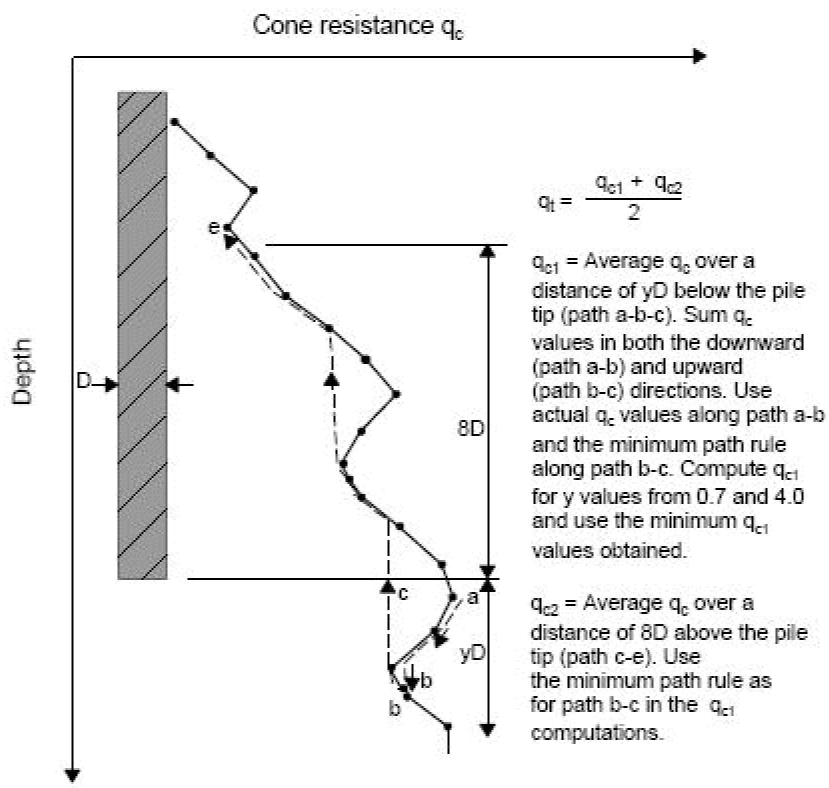


Figure: 2.2.a Calculation of average tip resistance using the Minimum Path Rule in the Schmertmann method

The pile's unit skin friction:

$f_s = \alpha_c \cdot f_{sa} \leq 1.2$ tsf

In clay:

where: α_c is a function of f_{sa} and pile material, as shown in Figure: 2.2.b, and Figure: 2.2.c.

$f_{sa} \leq 1.2$ tsf

$$Q_s := \alpha_s \cdot \left(\sum_{y=0}^{8D} \frac{y}{8D} \cdot f_{sa} \cdot A_s + \sum_{y=8D}^{L} f_{sa} \cdot A_s \right)$$

In sand:

where: α_s is a function of pile depth to width ratio and pile material as shown in Figure: 2.2.d, and Figure: 2.2.e.

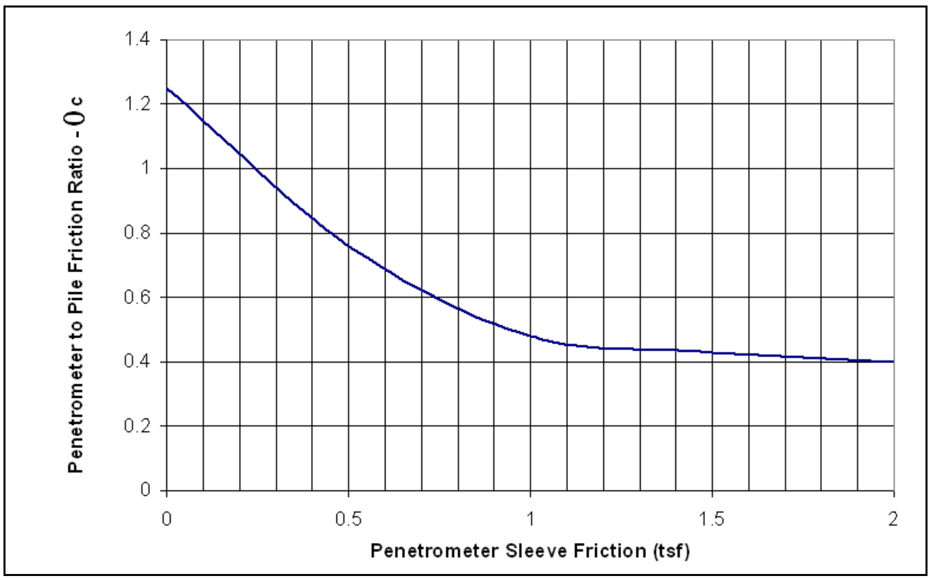


Figure: 2.2.b Design curve for concrete pile side friction in clay (Schmertmann method)

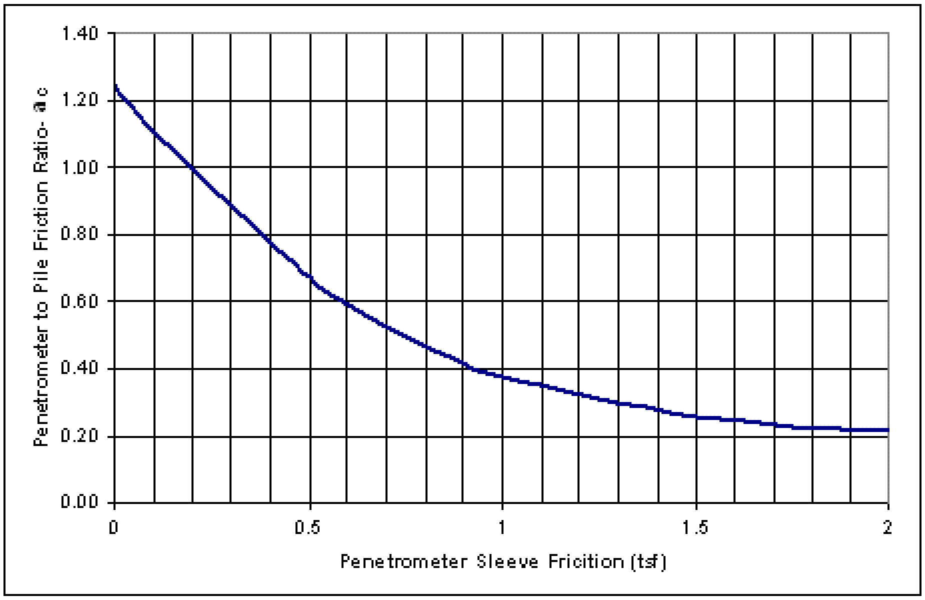


Figure: 2.2.c Design curve for steel pile side friction in clay (Schmertmann method)

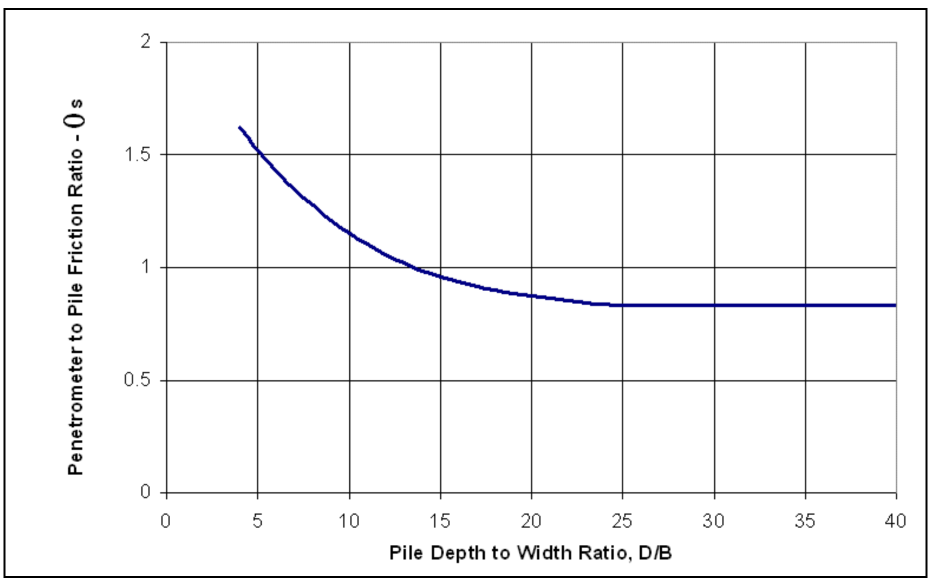


Figure: 2.2.d Design curve for concrete pile side friction in sand (Schmertmann method)

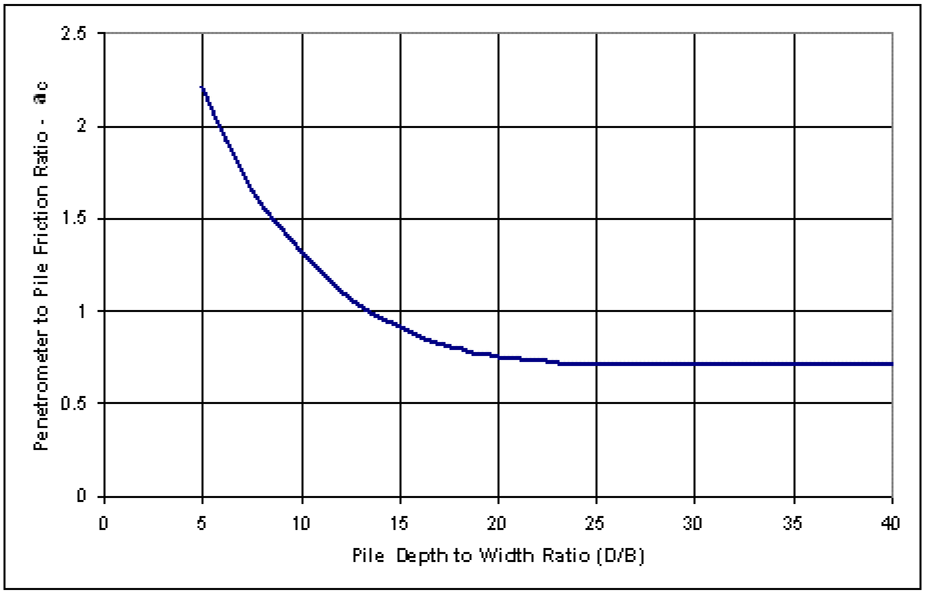


Figure: 2.2.e Design curve for steel pile side friction in sand (Schmertmann method)

2.2.2 UF

The UF method uses the following equation to estimate the ultimate pile unit tip resistance, q_t, from the CPT tip resistance, q_c:

$q_t = k_b \cdot q_{ca}$ (tip) = 150 tsf

where,

k_b is a factor that depends on the soil type as shown in Table: 2.2.a. The soil type was determined by simplified soil classification chart for standard electronic friction cone (Robertson et al, 1986) using CPT tip resistance and sleeve friction. Soil cementation was determined by SPT samples, DTP tip2/tip1 ratio or SPT qc/N ratio (>10).

q_{ca} (tip): the average CPT tip resistance, which is calculated as follows:

q_{ca} (tip) = $(q_{ca\ above} + q_{ca\ below}) / 2$

$q_{ca\ above}$: average q_c measured from the tip to 8•D above the tip;

$q_{ca\ below}$: average q_c measured from the tip to 3•D below the tip for sand or 1•D below the tip for clay;

Impose the condition: $q_{ca\ above} = q_{ca\ below}$, which means if $q_{ca\ above} = q_{ca\ below}$, let q_{ca}(tip) be equal to $q_{ca\ below}$.

The UF method uses the following equation to estimate the ultimate skin friction resistance of the pile, f_s, from the CPT tip resistance, q_c:

$f_s = q_{ca}$(side) • 1.25 / F_s = 1.2 tsf

where,

F_s: friction factor that depends on the soil type as shown in Table: 2.2.b. Figure: 2.2.f was used to determine the relative density of sand and the following criterion was used to determine the sand state: loose sand (R.D. < 40 %), medium dense sand (40 % < R.D. < 70 %), and dense sand (R.D. >70 %).

q_{ca}(side) : the average q_c within the calculating soil layers along the pile.

Table: 2.2.a Ultimate unit tip resistance factor k_b

Well Cemented Sand	Lightly Cemented Sand	Gravel	Sand	Silt	Clay
0.1	0.15	0.35	0.4	0.45	1.0

Table: 2.2.b Ultimate unit skin friction empirical factor, F_s

Well Cemented Sand	Lightly Cemented Sand	Gravel and Dense Sand	Medium Dense Sand	Loose Sand	Silt, Sandy Clay, Clayey Sand	Clay
300	250	200	150	100	60	50

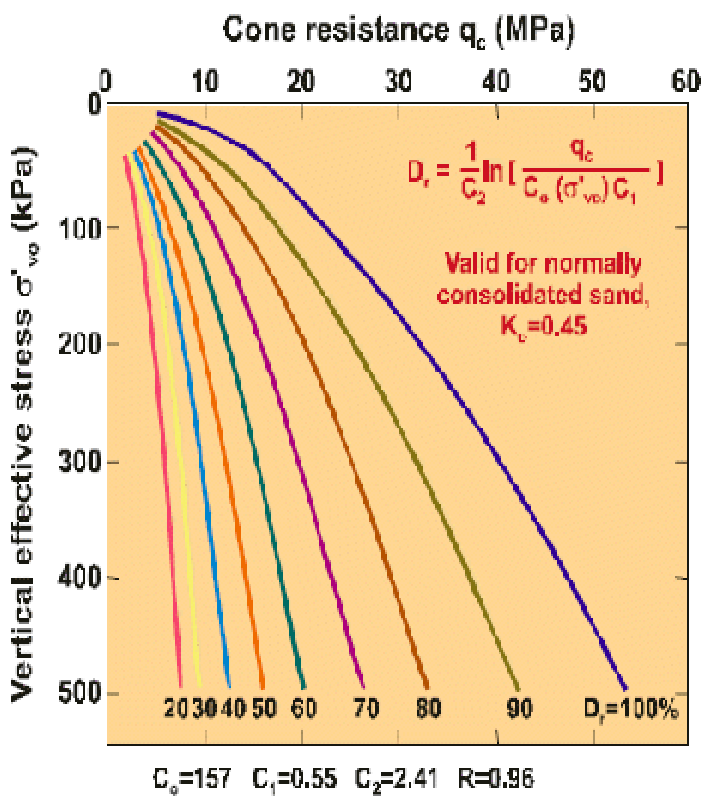


Figure: 2.2.f Relative density relationship for N.C. moderately compressible, uncemented, unaged quartz sands (after Baldi et al, 1986)

2.2.3 LCPC

The LCPC method (1982) only uses cone tip resistance for predicting axial pile capacity. It was proposed by Bustamante and Gianeselli for the French Highway Department after the study of 197 piles in Europe. It is also called the French method.

The pile's unit tip capacity:

$q_t = k_b \cdot q_{eq}$ (tip)

where,

q_{eq} (tip) is the average of tip resistance within 1.5•D above and 1.5•D below the pile tip after eliminating abnormal data (out of the range of ±30% of the average value);

k_b used by the program is a function of soil and pile type and can be found from Table: 2.2.c. The pile's unit skin friction is obtained by first noting pile type (Table: 2.2.d), then determining the Curve No. from Table: 2.2.e, Table: 2.2.f and Table: 2.2.g, and finally looking at Figure: 2.2.g, Figure: 2.2.h, and Figure: 2.2.i.

Table: 2.2.c Bearing factors k_b for the LCPC Method

Soil Type	**Bored Piles**	**Driven Piles**
Clay-Silt	0.375	0.600
Sand-Gravel	0.150	0.375
Chalk	0.200	0.400

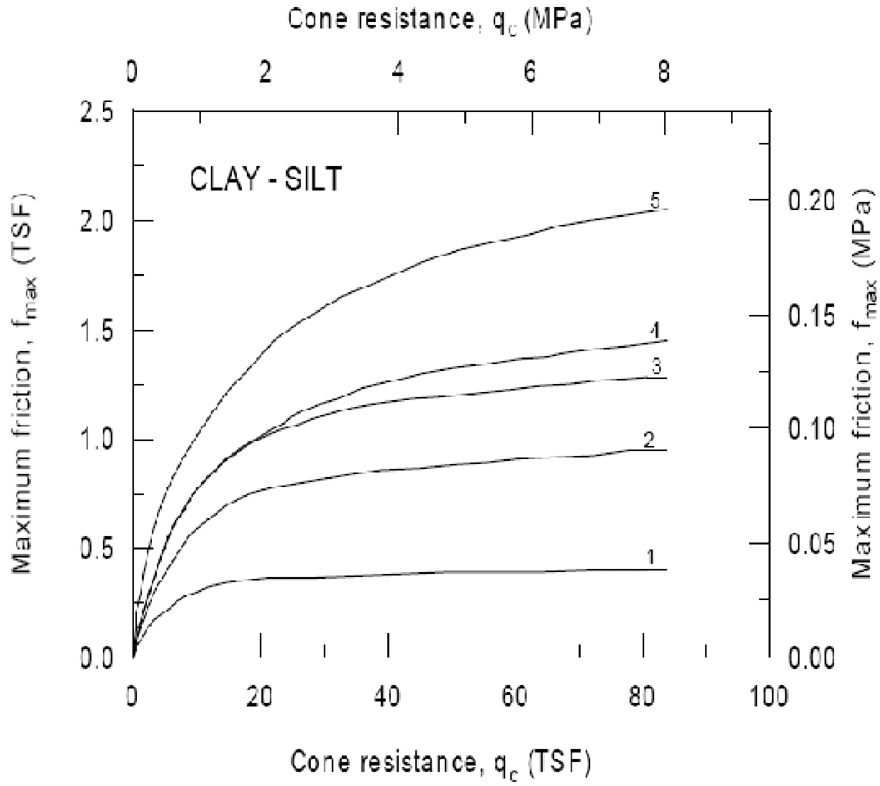


Figure: 2.2.g Ultimate skin friction curves for clay and silt from the LCPC method

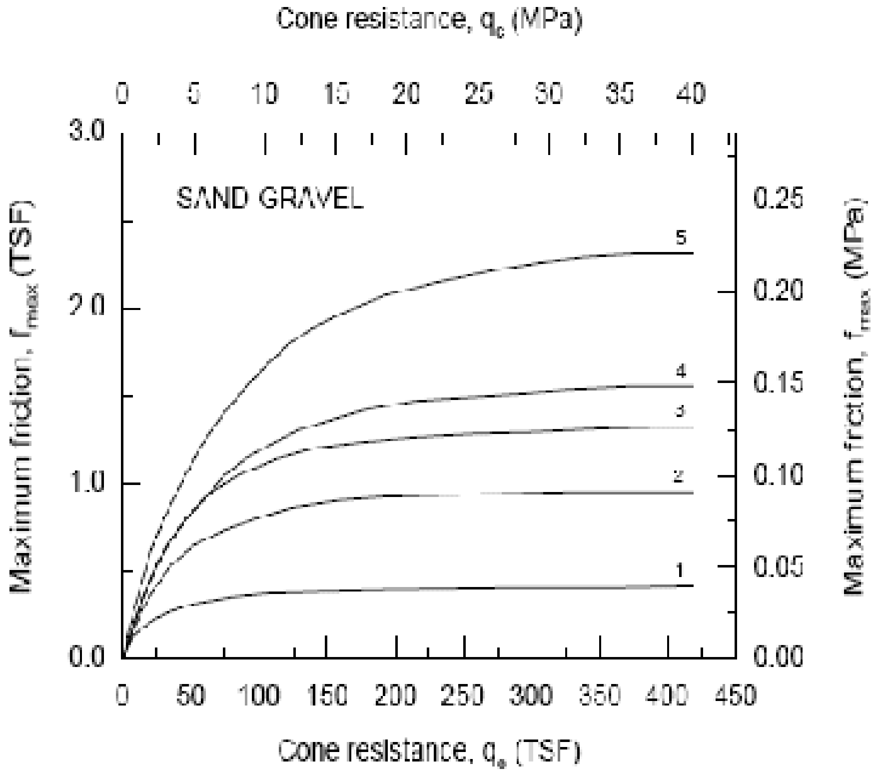


Figure: 2.2.h Ultimate skin friction curves for sand and gravel from the LCPC method

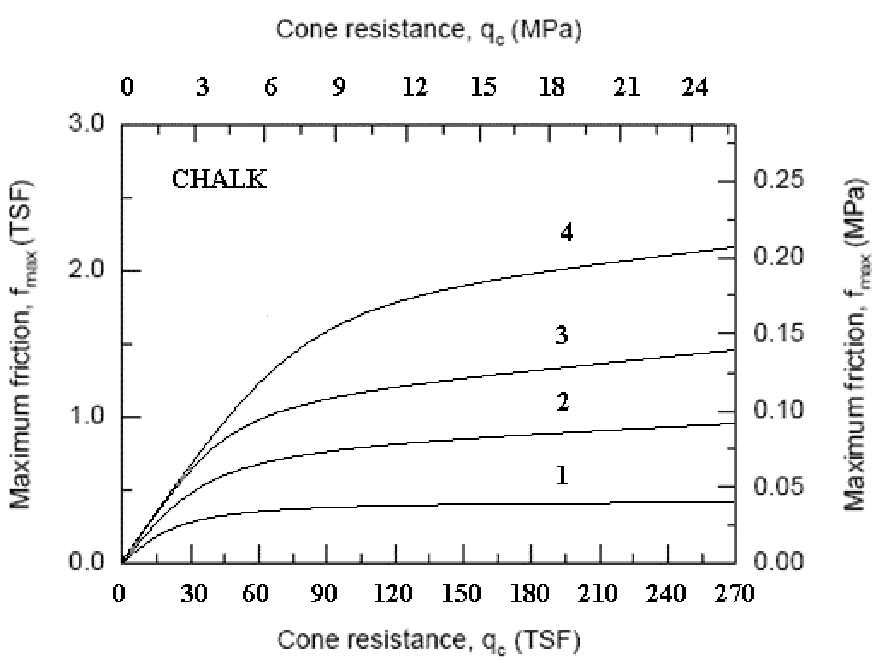


Figure: 2.2.i Ultimate skin friction curves for chalk from the LCPC method

Table: 2.2.d Pile type from the LCPC Method

Pile type	Descriptions
1. FS drilled shaft with no drilling mud	Installed without supporting the soil with drilling mud. Applicable only for cohesive soils above the water table.
2. FB drilled shaft with drilling mud	Installed using mud to support the sides of the hole. Concrete is poured from the bottom up, displacing the mud.
3. FT drilled shaft with casting (FTU)	Drilled within the confinement of a steel casing. As the casing is retrieved, concrete is poured in the hole.
4. FTC drilled shaft, hollow auger (auger cast piles)	Installed using a hollow stem continuous auger having a length at least equal to the proposed pile length. The auger is extracted without turning while, simultaneously, concrete is injected through the auger stem.
5. FPU Pier	Hand excavated foundations. The drilling method requires the presence of workers at the bottom of the excavation. The sides are supported with retaining elements or casing.
6. FIG Micropile type I (BIG)	Drilled pile with casing. Diameter less than 250 mm (10 in). After the casing has been filled with concrete, the top of the casing is plugged. Pressure is applied inside the casing between the concrete and the plug. The casing is recovered by maintaining the pressure against the concrete.
7. VMO screwed-in piles	Not applicable for cohesionless or soils below water table. A screw type tool is placed in front of a corrugated pipe which is pushed and screwed in place. The rotation is reversed for pulling out the casing while concrete is poured.
8. BE driven piles, concrete coated	- Pile piles 150 mm (6 in) to 500 mm (20 in) external diameter. - H piles. - Caissons made of 2, 3, or 4 sheet pile sections. The pile is driven with an oversized protecting shoe. As driving proceeds, concrete is injected through a hose near the oversized shoe producing a coating around the pile.
9. BBA driven prefabricated piles	Reinforced or prestressed concrete piles installed by driving or vibrodriving.
10. BM steel driven piles	Piles made of steel only and driven in place. - H piles. - Pipe piles. - Any shape obtained by welding sheet-pile sections.
11. BPR prestressed tube pile	Made of hollow cylinder elements of lightly reinforced concrete assembled together by prestressing before driving. Each element is generally 1.5 to 3 m (4-9 ft) long and 0.7 to 0.9 m (2-3 ft) in diameter. The thickness is approximately 0.15 m (6 in). The piles are driven open ended.
12. BFR driven pile, bottom concrete plug	Driving is achieved through the bottom concrete plug. The casing is pulled out while low slump concrete is compacted in it.

13. BMO driven piles, molded	A plugged tube is driven until the final position is reached. The tube is filled with medium slump concrete to the top and the tube is extracted.
14. VBA concrete piles, pushed-in	Pile is made of cylindrical concrete elements prefabricated or cast-in-place, 0.5 to 2.5 m (1.5 to 8 ft) long and 30 to 60 cm (1 to 2 ft) in diameter. The elements are pushed in by a hydraulic jack.
15. VME steel piles, pushed-in	Piles made of steel only are pushed in by a hydraulic jack.
16. FIP micropile type II	Drilled pile < 250 mm (10 in) in diameter. The reinforcing cage is placed in the hole and concrete placed from bottom up.
17. BIP high pressure injected pile, large diameter	Diameter > 250 mm (10 in). The injection system should be able to produce high pressures.

Table: 2.2.e Curve No. for clay and silt from the LCPC Method

Curve number	qc (ksf)	Pile type	Comments on insertion procedure
1	< 14.6 > 14.6	1-17 1, 2	- Very probable values when using tools without teeth or with oversized blades and where a remolded layer of material can be deposited along the sides of the drilled hole. Use these values also for deep holes below the water table where the hole must be cleaned several times. Use these values also for cases when the relaxation of the sides of the hole is allowed due to incidents slowing or stopping the pouring of concrete. For all the previous conditions, experience shows, however, that q_s can be between curve 1 and 2; use an intermediate value of q_s if such value is warranted by a load test.
2	> 25.1	4, 5, 8, 9, 10, 11, 13, 14, 15	- For all steel piles, experience shows that in plastic soils, q_s is often as low as curve 1; therefore, use curve 1 when no previous load test is available. For all driven concrete piles use curve 3 in low plasticity soils with sand or sand and gravel layers or containing boulders and when q_c > 52.2 ksf.
	> 25.1	7	- Use these values for soils where q_c < 52.2 ksf and the rate of penetration is slow; otherwise use curve 1. Also for slow penetration, when q_c > 93.9 ksf, use curve 3.
	> 25.1	6	- Use curve 3 based on previous load test.
	> 25.1	1, 2	- Use these values when careful method of drilling with an auger equipped with teeth and immediate concrete pouring is used. In the case of constant supervision with cleaning and grooving of the borehole walls followed by immediate concrete pouring, for soils of q_c > 93.9 ksf, curve 3 can be used.
	> 25.1	3	- For dry holes. It is recommended to vibrate the concrete after taking out the casing. In the case of work below the water table, where pumping is required and frequent movement of the casing is necessary, use curve 1 unless load test results are available.
3	> 25.1 < 41.8	12	- Usual conditions of execution as described in DTP 13.2
5	> 14.8	16, 17	- In the case of injection done selectively and repetitively at low flow rate it will be possible to use curve 5, if it is justified by previous load test.

Table: 2.2.f Curve No. for Sand and Gravel from LCPC Method

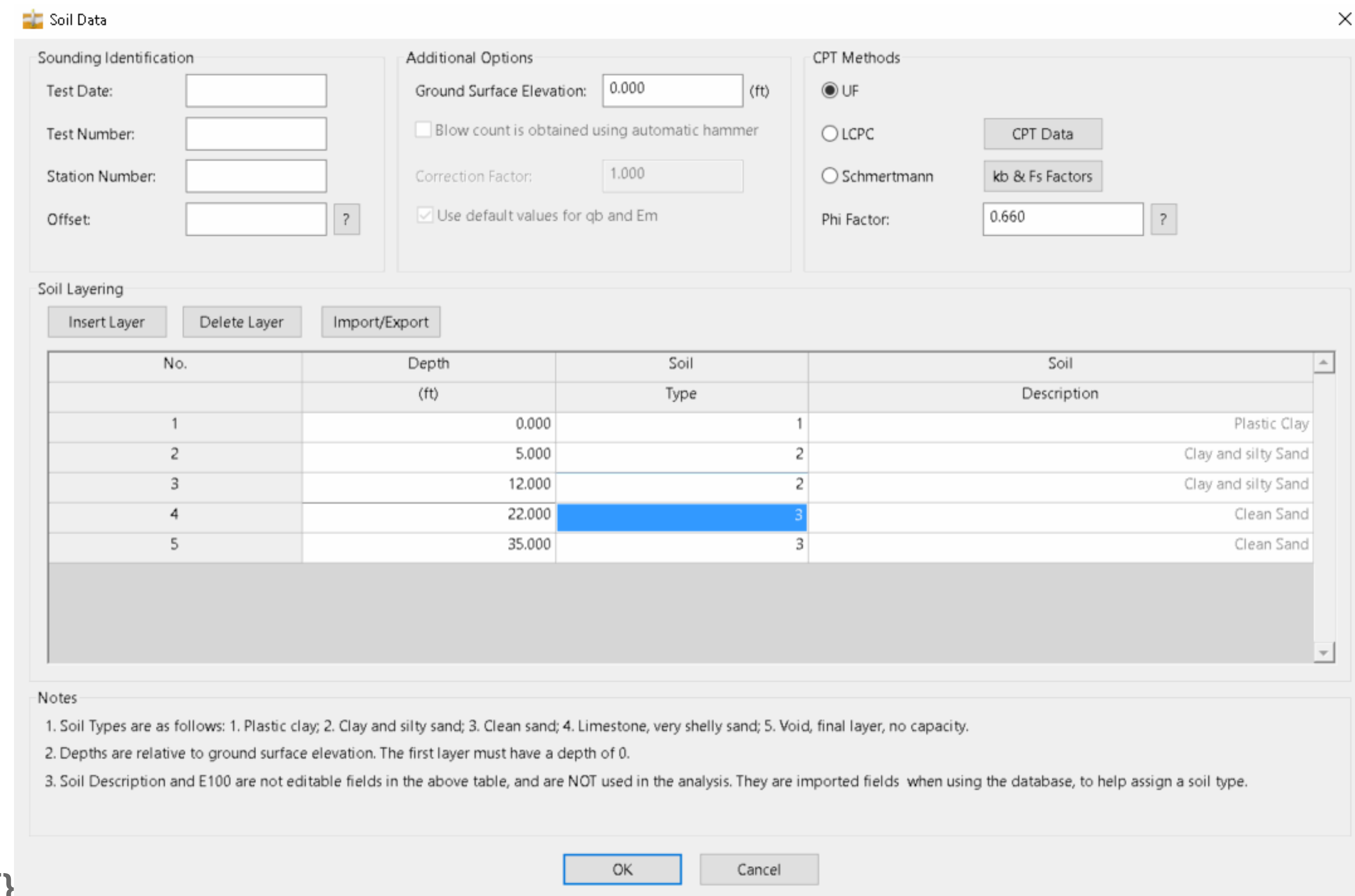
Curve number	qc (ksf)	Pile type	Comments on insertion procedure
1	< 73.1	2 - 4, 6 -15	
2	> 73.1	6, 7, 9 -15	- For fine sands. Since steel piles can lead to very small values of q_s in such soils, use curve 1 unless higher values can be based on load test results. For concrete piles, use curve 2 for fine sands of q_c > 156.6 ksf.
	> 104.4	2, 3	- Only for fine sands and bored piles which are less than 30m (100 ft) long. For piles longer than 30 m (100 ft) in fine sand, q_s may vary between curves 1 and 2. Where no load test data is available, use curve 1.
	> 104.4	4	- Reserved for sands exhibiting some cohesion.
3	> 156.6	6, 7, 9 - 11, 13 - 15, 17	- For coarse gravelly sand or gravel only. For concrete piles, use curve 4 if it can be justified by a load test.
	> 156.6	2, 3	- For coarse gravelly sand or gravel and bored piles less than 30 m (100 ft) long. - For gravel where q_c > 83.5 ksf, use curve 4.
4	> 156.6	8, 12	- For coarse gravelly sand and gravel only.
5	> 104.4	16, 17	- Use of values higher than curve 5 is acceptable if based on load test.

Table: 2.2.g Curve No. for chalk from LCPC Method

Curve number	q_c (ksf)	Pile type	Comments on insertion procedure
1	<62.6	1, 2, 3, 4, 6, 7, 8, 9, 10, 11, 12, 13, 14, 15	
3	>62.6 >93.9 >93.9	7, 8, 9, 10, 11, 13, 14, 15 6, 8 1, 2, 3, 5, 7	- Experience shows that in some chalks where q_c< 146.1 ksf, below water table, steel or smooth concrete piles may exhibit q_s values as low as those of curve 2. When no load test is available, use curve 2 for q_c < 146.1 ksf. For chalk of q_c > 250.5 ksf use curve 4 based on a load tests. - For bored piles above the water table and concrete poured immediately after boring. For type 7 piles, use a slow penetration thus creating corrugations along the hole walls. Also for chalk above the water table and for q_c > 250.5 ksf use curve 4 if based on a load test. - Below the water table and with tools producing a smooth wall or when a deposit of remolded chalk is left on the walls of the hole, experience shows that q_s can drop to values given by curve 2. Use higher values only on the basis of load tests.
4	>93.9 >93.9	12 16, 17	- Higher values than curve 4 can be used if based on a load test.

2.2.4 CPT Modeling

The CPT analysis options for driven piles are available on the boring log, near the top of the screen. There are three options: CPT – UF, CPT – LCPC, and CPT – Schmertmann. Note these analysis types are only available for driven pile analysis. If drilled shaft analysis is selected, these options will become hidden, and the SPT analysis type will automatically be used.



{}

Figure: 2.2.j Soil Data Window

When a CPT method is selected, the Phi Factor will change to the default Phi Factor for the selected CPT method. However, this value can be adjusted manually by entering a value in the Phi Factor textbox. A table displaying the default values can be seen by clicking the Details button closest to the Phi Factor textbox.

To import CPT reading data, click the "Import/Export" button. Then select the menu item "Import CPT Data from File".

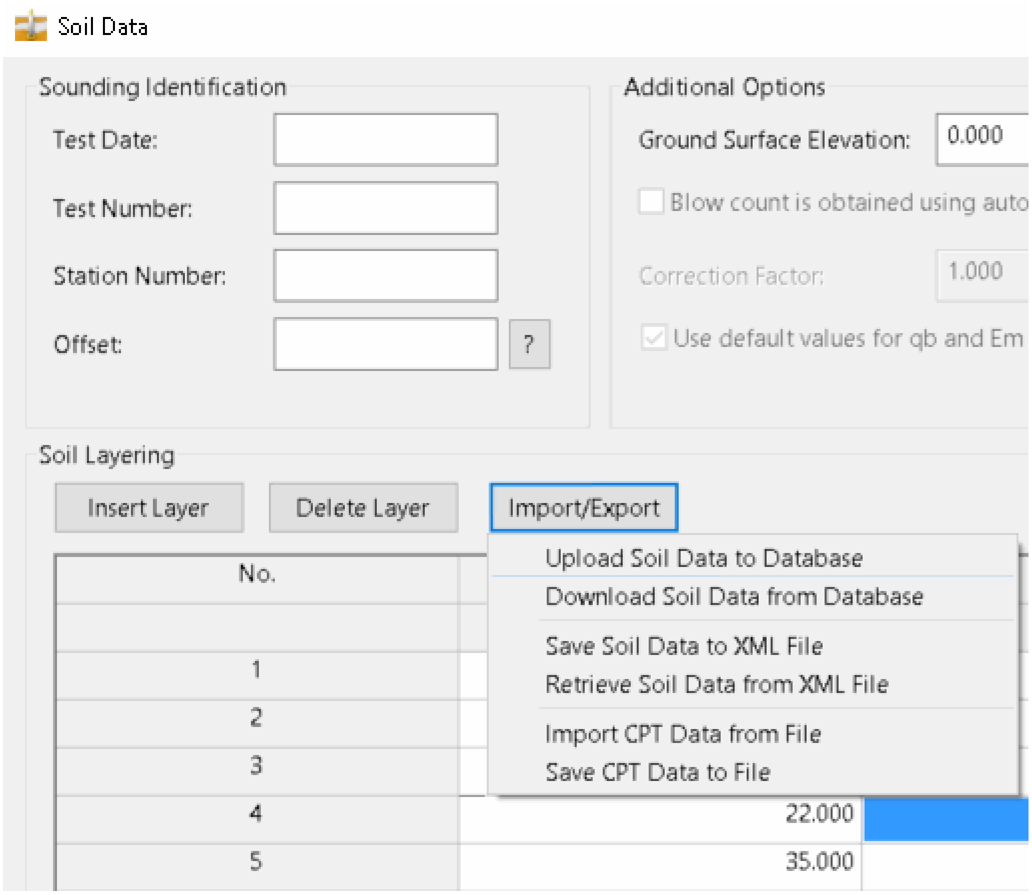


Figure: 2.2.k Import CPT Data

Imported data can be of two forms: text files (.txt extension), and excel files (.xls extension). It is important to note that the imported data need be in the correct units used by FB-Deep. The program's English units are as follows: Depth – feet, qt (tsf), fs (tsf); The program's Metric units are as follows: Depth (meters), qt (MN/m2), fs (kN/m2). FB-Deep requires that corrected tip resistance, qt, be inputted for piezometric cones.

Imported data must be in the correct format to import successfully. Text File format is as follows: four columns of data, in the following order: Depth, qt, fs, and Friction Ratio. Column headings are optional. Readings must be on consecutive lines. Comment lines and blank lines are allowed, above the lines of reading data. Here is an example of the text file format:

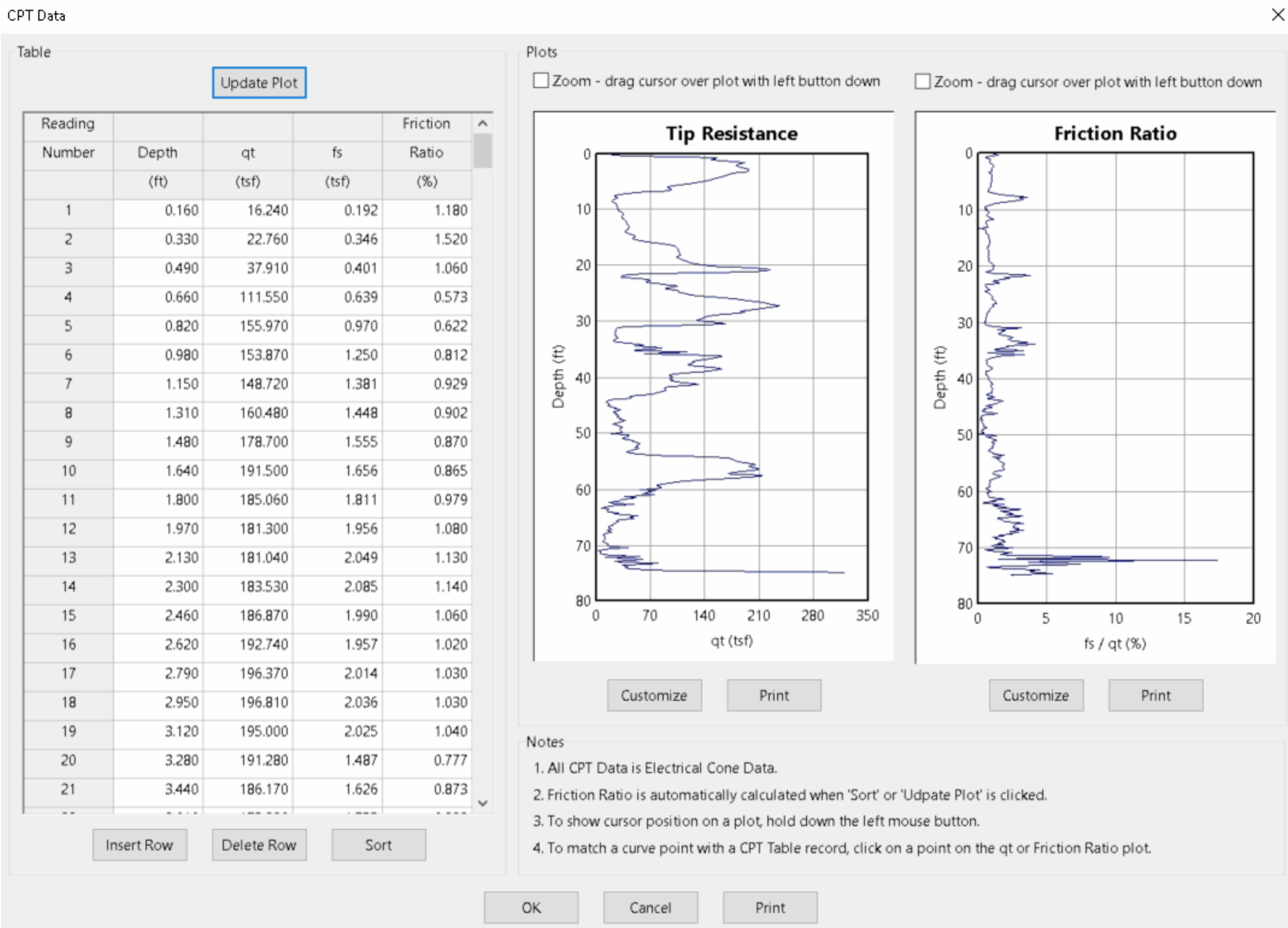
Depth	qt	fs	Friction Ratio
0.6560	15.3720	0.3280	2.1300
1.3120	23.5590	0.4510	1.9100
1.9690	25.6170	0.4100	1.6000
2.6250	24.5830	0.4300	1.7500
3.2810	28.6870	0.5330	1.8600
3.9370	27.6630	0.4410	1.5900
4.5930	20.4890	0.3070	1.5000
5.2490	23.5590	0.3790	1.6100
5.9060	32.7810	0.5740	1.7500
6.5620	30.7340	0.6150	2.0000
7.2180	26.6400	0.3380	1.2700
7.8740	32.7810	0.4710	1.4400
8.5300	23.5590	0.3180	1.3500
9.1860	30.7340	0.3280	1.0700
9.8430	28.6870	0.4410	1.5400
10.4990	33.8040	0.4610	1.3600
11.1550	29.7100	0.4510	1.5200
11.8110	26.6400	0.3280	1.2300
12.4670	30.7340	0.3380	1.1000
13.1230	35.8610	0.3480	0.9700
13.7800	37.9080	0.3280	0.8650
14.4360	34.8270	0.5530	1.5900
15.0920	15.3720	0.5330	3.4700
15.7480	9.2210	0.5020	5.4400
16.4040	5.1170	0.2660	5.2000
17.0600	4.0940	0.2560	6.2500
17.7170	3.5820	0.2150	6.0000
18.3730	4.0940	0.2360	5.7600
19.0290	4.6050	0.2870	6.2300
19.6850	6.6630	0.4000	6.0000
20.3410	5.6390	0.3070	5.4400
20.9970	5.6390	0.2660	4.7200
21.6540	5.1170	0.3070	6.0000
22.3100	7.1740	0.4410	6.1500
22.9660	7.1740	0.4510	6.2900
23.6220	6.1510	0.3590	5.8400
24.2780	8.7090	0.5220	5.9900
24.9340	7.1740	0.3890	5.4200

The Excel spreadsheet format is as follows: four columns of data, in the following order: Depth, qt, fs, and Friction Ratio. The following column headings MUST be used: Depth, qt, fs, and FR. Readings must be on consecutive lines. The worksheet must be named "CPTdata". Here is an example of the spreadsheet (.xls) file format:

	A	B	C	D
1	DEPTH	qt	fs	FR
2				
3	0.05	3.50	17.00	0.48
4	0.10	3.91	22.00	0.55
5	0.15	4.33	27.00	0.61
6	0.20	4.55	30.00	0.65
7	0.25	4.69	39.00	0.83
8	0.30	4.82	55.00	1.14
9	0.35	5.28	53.00	1.01
10	0.40	5.85	50.00	0.85
11	0.45	5.54	47.00	0.84
12	0.50	4.88	35.00	0.72
13	0.55	4.62	37.00	0.80
14	0.60	3.90	34.00	0.87
15	0.65	3.59	33.00	0.91
16	0.70	3.37	31.00	0.90
17	0.75	2.99	29.00	0.97
18	0.80	2.65	29.00	1.08
19	0.85	2.40	28.00	1.16
20	0.90	2.07	27.00	1.32
21	0.95	1.84	26.00	1.42
22	1.00	1.77	24.00	1.38
23	1.05	1.75	20.00	1.15
24	1.10	1.70	20.00	1.15
25	1.15	1.63	20.00	1.21
26	1.20	1.57	20.00	1.26
27	1.25	1.57	19.00	1.23
28	1.30	1.56	20.00	1.27
29	1.35	1.62	20.00	1.20
30	1.40	1.76	22.00	1.25
31	1.45	1.94	24.00	1.24
32	1.50	2.06	24.00	1.16
33	1.55	2.46	24.00	1.39
34	1.60	2.55	28.00	1.10
35	1.65	2.25	25.00	1.09
36	1.70	2.27	24.00	1.05
37	1.75	2.23	23.00	1.02

CPTdata / Sheet2 / Sheet3

To view the imported data, click the "CPT Data" button on the Soil Data screen. This launches the CPT Data screen (shown here).



Reading Number	Depth (ft)	qt (tsf)	fs (tsf)	Friction Ratio (%)
1	0.160	16.240	0.192	1.180
2	0.330	22.760	0.346	1.520
3	0.490	37.910	0.401	1.060
4	0.660	111.550	0.639	0.573
5	0.820	155.970	0.970	0.622
6	0.980	153.870	1.250	0.812
7	1.150	148.720	1.381	0.929
8	1.310	160.480	1.448	0.902
9	1.480	178.700	1.555	0.870
10	1.640	191.500	1.656	0.865
11	1.800	185.060	1.811	0.979
12	1.970	181.300	1.956	1.080
13	2.130	181.040	2.049	1.130
14	2.300	183.530	2.085	1.140
15	2.460	186.870	1.990	1.060
16	2.620	192.740	1.957	1.020
17	2.790	196.370	2.014	1.030
18	2.950	196.810	2.036	1.030
19	3.120	195.000	2.025	1.040
20	3.280	191.280	1.487	0.777
21	3.440	186.170	1.626	0.873

Figure: 2.2.l CPT Data Window

Once imported, the data can be adjusted in the table. Clicking the "Sort" button will arrange the readings according to depth. Clicking "Update Plot" will redraw the plot windows using the table data. Additional rows can be added to the table by clicking "Insert Row". The inserted row will be positioned immediately after the row that is currently selected (highlighted) in the table. Clicking "Print" will print a screenshot of the entire CPT screen. Because readings often number in the hundreds, most readings will not be visible without scrolling. To quickly find a reading in the table, simply click on the desired depth in either of the plot windows. The associated reading will automatically be scrolled to, and become selected. When saving the input file (.spc), any existing CPT readings will be saved to a text file (.txt) of the same name. This is done so the readings can be easily viewed.

CPT Factors

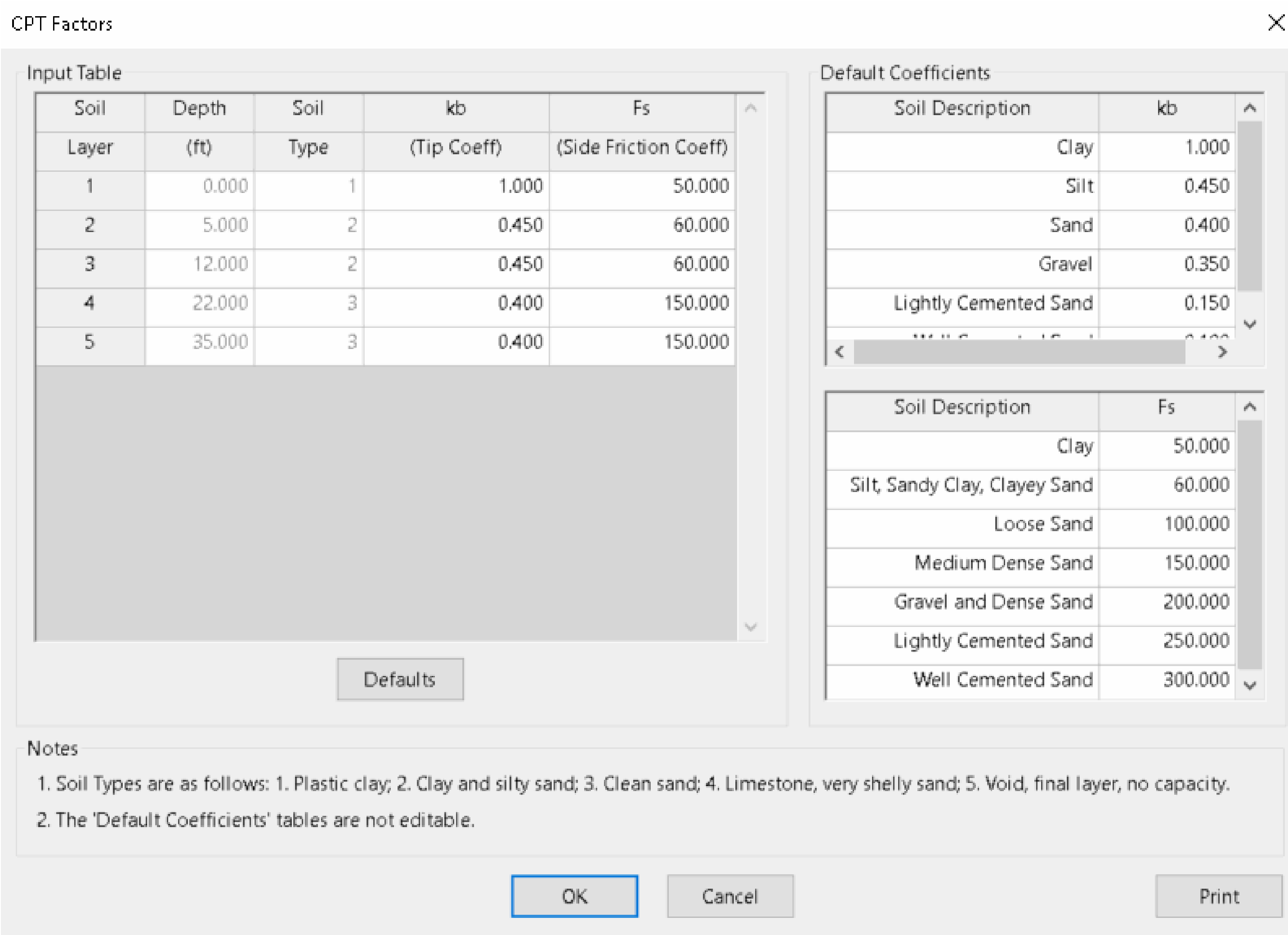


Figure: 2.2.m CPT Factors Window

The CPT Factors Dialog is used to input kb and Fs data for each soil layer. It is launched via the "Edit Factors" button on the CPT Data dialog. Both kb and Fs values are unit-less coefficients. Only kb and Fs data may be edited on the dialog. Soil Depths and Soil Types are shown here, but can only be changed on the CPT Soil Dialog.

Click the "Defaults" button to change all kb and Fs values to default values. If the user does not use this dialog, the default values for each soil layer will automatically be assigned.

The default kb values are as follows:

Soil Type 1, Plastic Clay: 1.0

Soil Type 2, Clay and Silty Sand: 0.45

Soil Type 3, Clean Sand: 0.40

Soil Type 4, Limestone, Very Shelly Sand: 0.35

Soil Type 5, Void: 0.0

The default Fs values are as follows:

Soil Type 1, Plastic Clay: 50.0

Soil Type 2, Clay and Silty Sand: 60.0

Soil Type 3, Clean Sand: 150.0

Soil Type 4, Limestone, Very Shelly Sand: 200.0

Soil Type 5, Void: 1000000

The three CPT analysis types are limited by the pile's section type. The table below shows what pile sections are available for each of the three CPT methods.

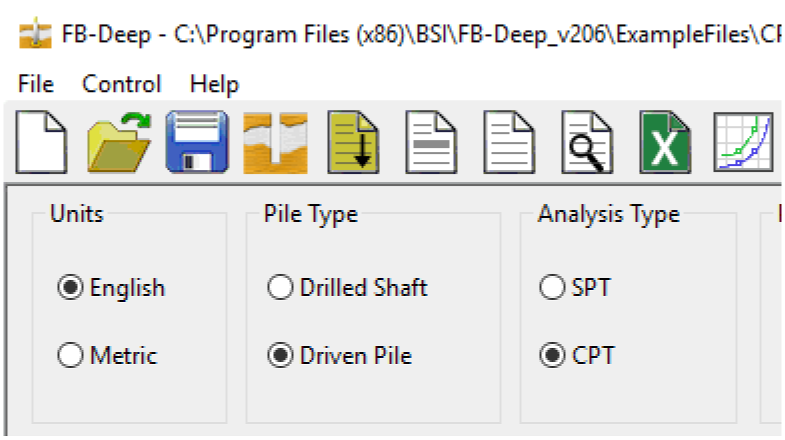
Analysis	Square	Round	Cylinder	Pipe	H-Section
Schmertmann	Yes	X	X	Yes	X
UF	Yes	Yes	X	X	X
LCPC	Yes	Yes	Yes	Yes	Yes

X- Method does not calculate capacity for that type of section

3 User's Guide

1. Units
2. Import SPT94 file
3. Boring Log/Soil Data Screen
4. Drilled Shafts
5. Driven Piles
6. Database
7. Graphical Output
8. Backwards Compatibility

3.1 Units



The user can select to work with either the English or the Metric units systems.

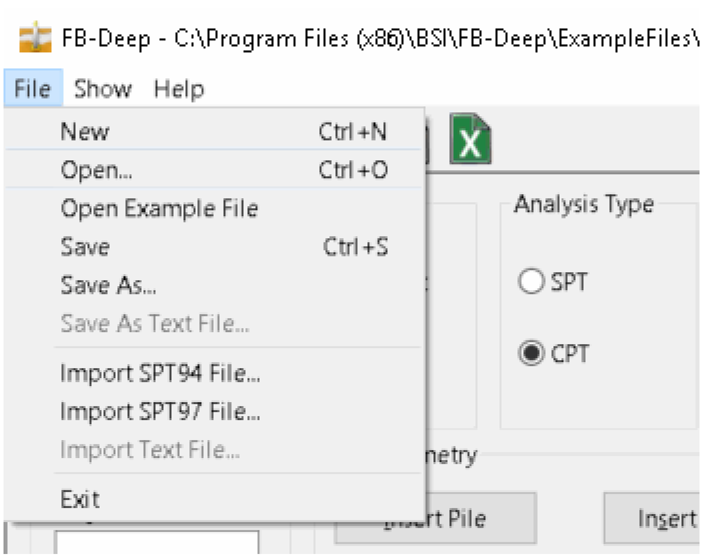
For English units:

- Forces are in tons (tons).
- Lengths are in feet (ft).
- Dimensions are in inches (in).
- Unit weights are in pound per cubic foot (pcf).
- Soil stresses are in pound per square foot (psf).
- Pile stresses are in kilo-pound per square inch (ksi).

For Metric units:

- Forces are in kilo-Newton (kN).
- Lengths are in meters (m).
- Dimensions are in millimeters (mm).
- Unit weights are in kilo-Newton per cubic meters (kN/m^3).
- Soil stresses are in kilo-Pascal (kPa).
- Pile stresses are in Mega-Pascal (MPa).

3.2 Import SPT94/SPT97 file



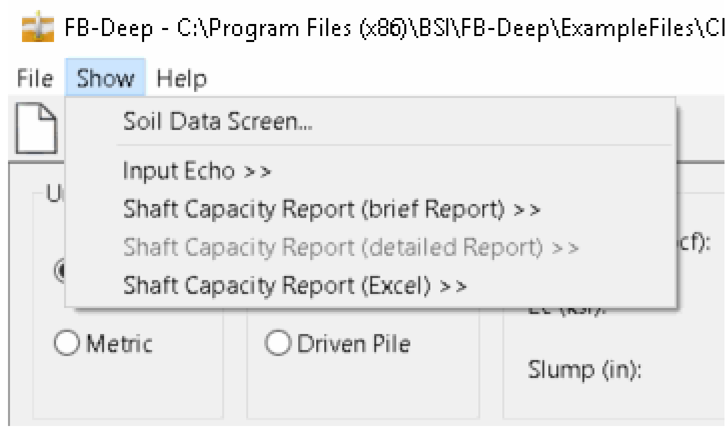
FB-Deep allows the user to use old files created for the SPT94/SPT97 program. To do so, the user selects <File-Import SPT94 File> or <File-Import SPT97 File>from the menu. The user will be prompted to select the SPT 94/SPT 97 file.

Once the old file is selected, the program will copy the data to the latest FB-Deep program input format.

3.3 Boring Log Soil Data Screen

1. Showing Screen
2. Strength Reduction Factor
3. Soil Type
4. Hammer Type
5. Phi Factor
6. Station Number and Offset

3.3.1 Boring Log Dialog / Soil Data Screen



For SPT Analysis, the soil input screen is called the Boring Log. For CPT Analysis, the soil input screen is called the Soil Data Screen.

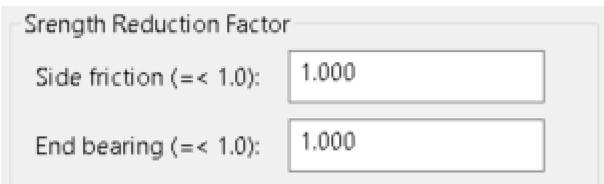
To display the Boring Log / Soil Data Screen, select the <Show-Boring Log> (or <Show-Soil Data Screen>) menu item or click on the corresponding on the tool bar button.

For driven-pile analysis, only the blows count is needed for the calculation for all soil types. For drilled-shaft analysis, every soil type requires different soil properties.

3.3.2 Boring Log / Soil Data Screen: Strength Reduction Factors

The strength reduction factors should be less than 1. The default values are 1.

The ultimate strengths of drilled shafts are reduced by the strength reduction factor. However, the load-settlement curves are not reduced.



3.3.3 Soil Type

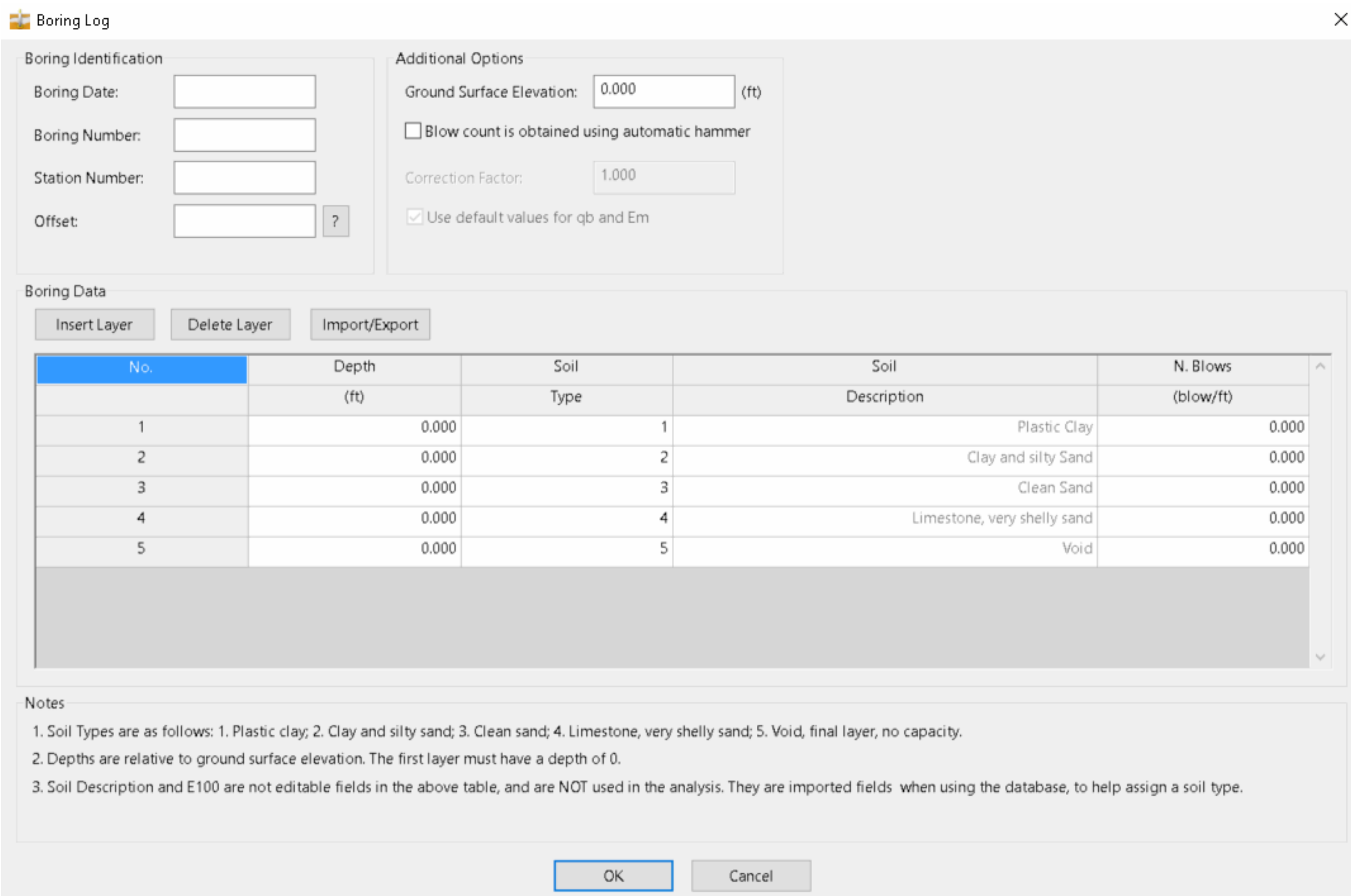
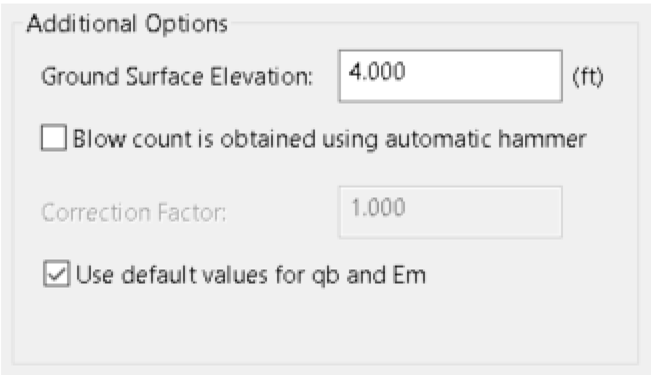


Figure: 3.3.a Boring log Dialog

FB-Deep allows 5 soil types:

1. Plastic clay.
2. Clay and silty sand.
3. Clean sand.
4. Limestone and very shelly sand.
5. Void.

3.3.4 Hammer Type



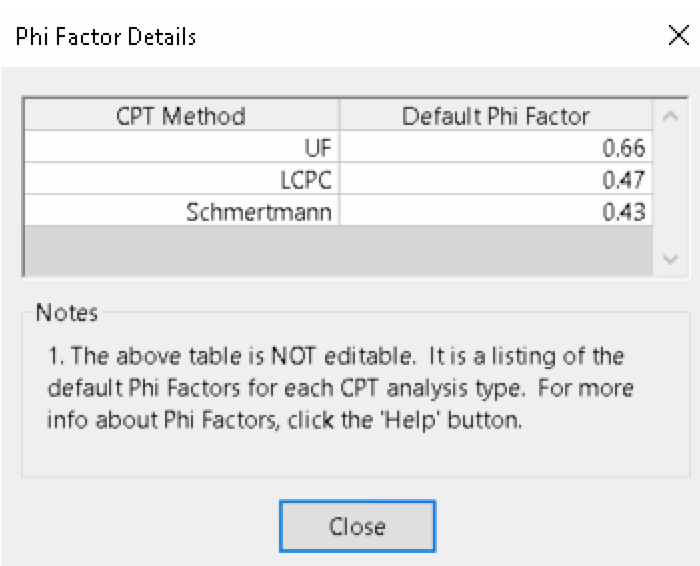
Calculations for driven-pile capacity and drilled-shafts capacity are based on blow count readings from safety hammer. If automatic hammer is used, the user is allowed to specify a correction factor as shown in the figure.

3.3.5 Phi Factor

The three CPT methods used in FB-Deep all use the LRFD design methodology. This results in the use of phi factor which is multiplied by the nominal resistance to give a design resistance. The default phi factors used by FB-Deep were determined by research done at the University of Florida. For more information on these phi factors click on the link below or review the AASHTO LRFD specifications.

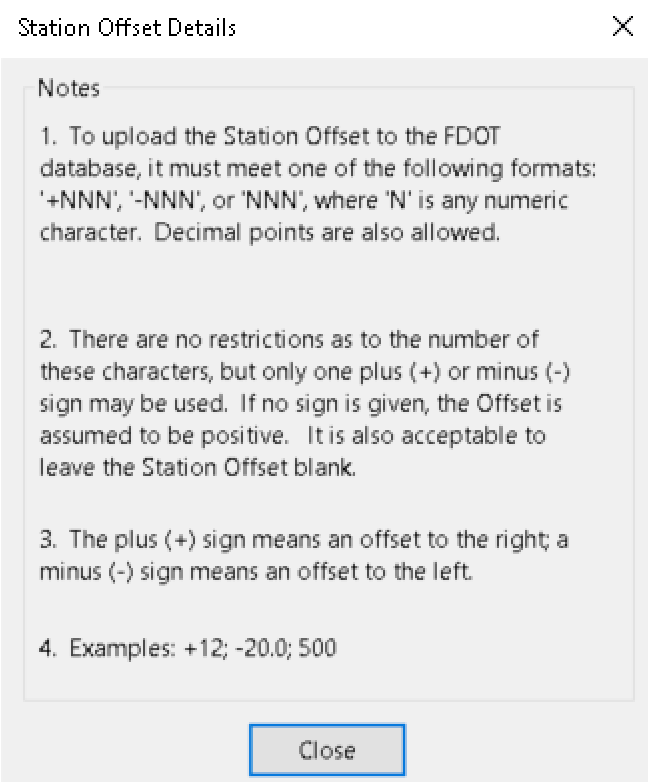
http://www.dot.state.fl.us/research-center/Completed_Proj/Summary_SMO/FDOT_BD545_43_rpt.pdf

Each time a CPT method is selected, the corresponding default value will display in the Phi Factor editbox. This value can be adjusted. To view the default Phi Factor table, click the "Details" button on the Soil Data Screen.



3.3.6 Station Number and Offset

Prior to version 2.01, the Station Number and Offset were input in a single editbox. Starting in version 2.01, the station number and offset are input separately. The Station Number can have any value, including letters, numbers or characters. The offset must be numeric, to comply with the FDOT database format. The Station Offset Dialog (pictures below) explains the format of the offset.



3.4 Drilled Shafts

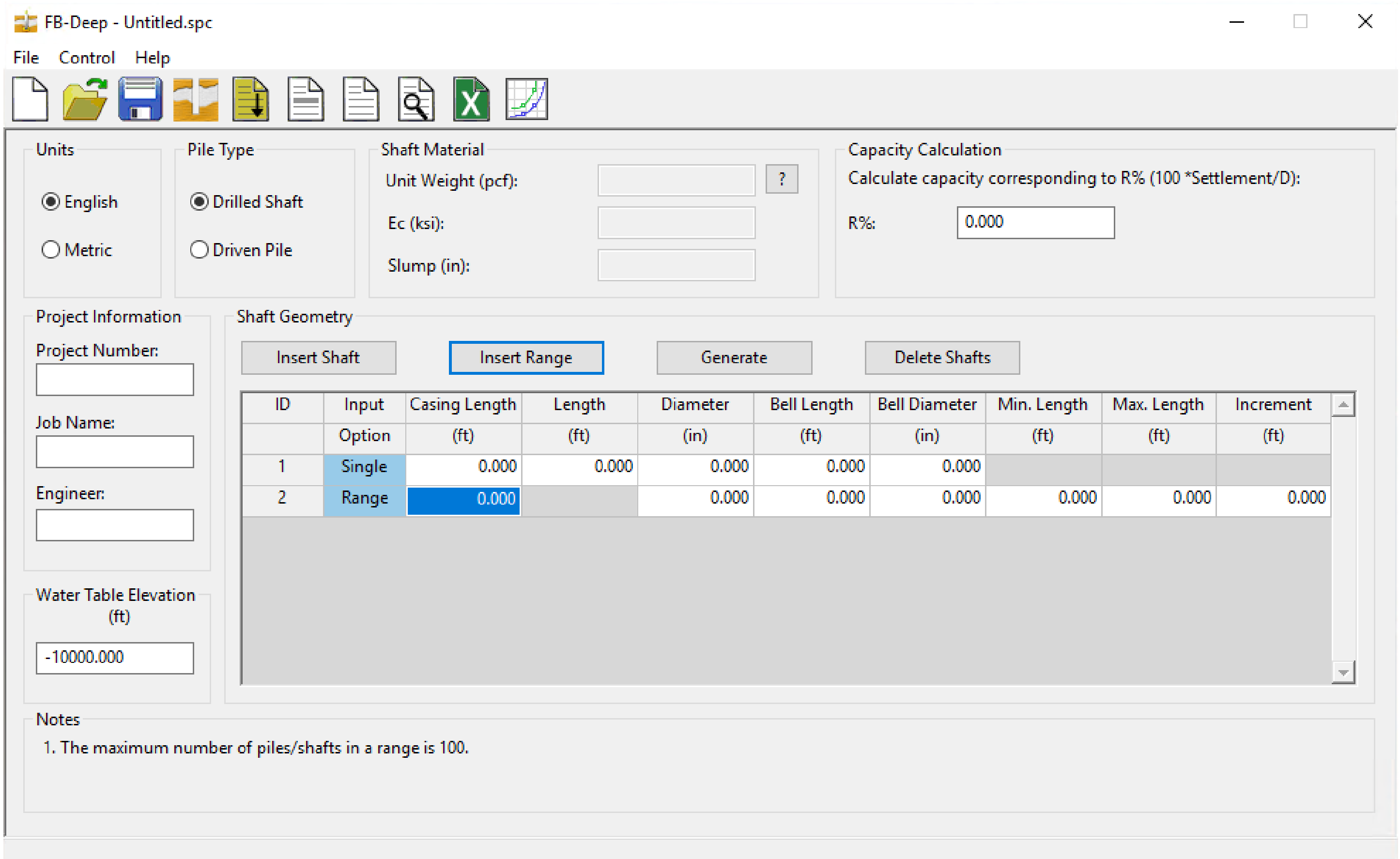


Figure: 3.4.a Drilled Shaft Model

Shaft Material

Unit weight, concrete slump and Ec are used for the calculation of the contribution of the Rock layers to the Load-Settlement curve. The parameters are available only if the boring log contains rock entries

Section Type

The user can choose a section type from among six options: Concrete Square, Concrete Round, Concrete Cylinder, Steel H Section, Steel Pipe Pile. Depending on the selected section type, the pile geometry grid changes to indicate the parameters required for the selected section type.

Insert Pile/Range

The user has the option to either specify a single shaft or a range of shaft lengths that have the same section properties.

For shaft range, the user specifies the maximum length, the minimum length and the increment. The program generates number of shafts with lengths starting with the minimum length and not exceeding the maximum length. When, the user enters a value for the maximum length or the minimum length, the program checks that the minimum length is not greater than the maximum length.

The number of shafts can be generated by a range is 100. As the user enters the maximum length and the minimum length, the program updates the increment, if the existing increment value results in the generation of more than 100 shafts. In this case, the increment is modified to satisfy two conditions. One is that the number of generated shafts is 100. The second is that the maximum length generated by the program is the closest to the user-defined maximum length.

If the user enters a value of the increment that generates more than 100 shafts, the program rejects this value and the user is prompted to enter a new value.

Generate

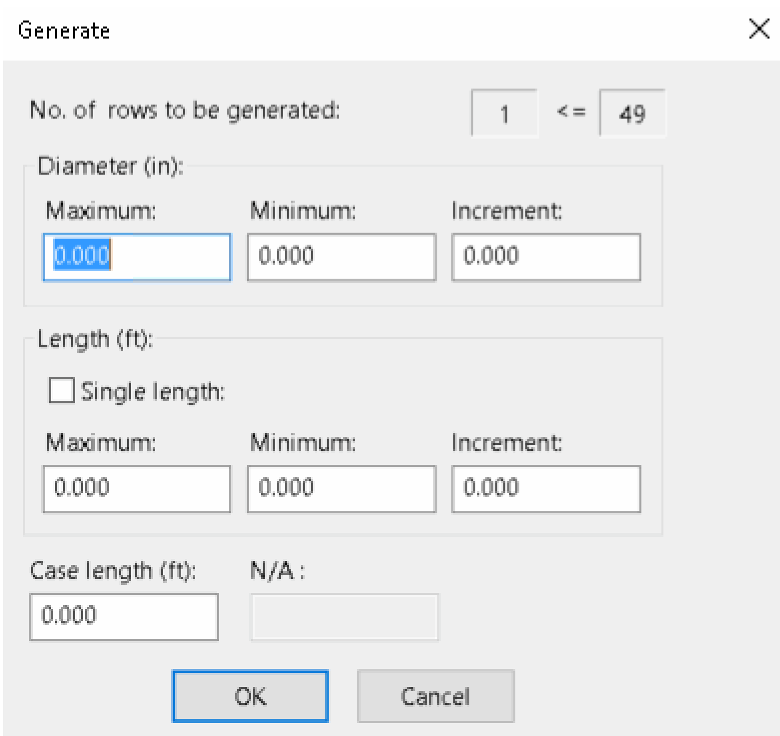


Figure: 3.4.b Generate Shafts Dialog

The Generate dialog box aids the user to generate multiple shaft records with different width. The width is set to change linearly between the first and last generated records. The user can generate multiple records of either single or range input option.

The user specifies the maximum width, the minimum width and the increment. The program generates number of records with widths starting with the minimum width and not exceeding the maximum width. When, the user enters a value for the maximum width or the minimum width, the program checks that the minimum width is not greater than the maximum width.

The maximum number of records can be generated depends on how many records already exist in the shaft geometry grid. The total number of records that can be entered in the shaft grid is 48. As the user enters the maximum width and the minimum width, the program updates the increment, if the existing increment value results in the generation of more than the maximum. In this case, the increment is modified to satisfy two conditions. One is that the number of generated shafts is equal to the maximum. The second is that the maximum width generated by the program is the closest to the user-defined maximum width.

3.5 Driven Piles

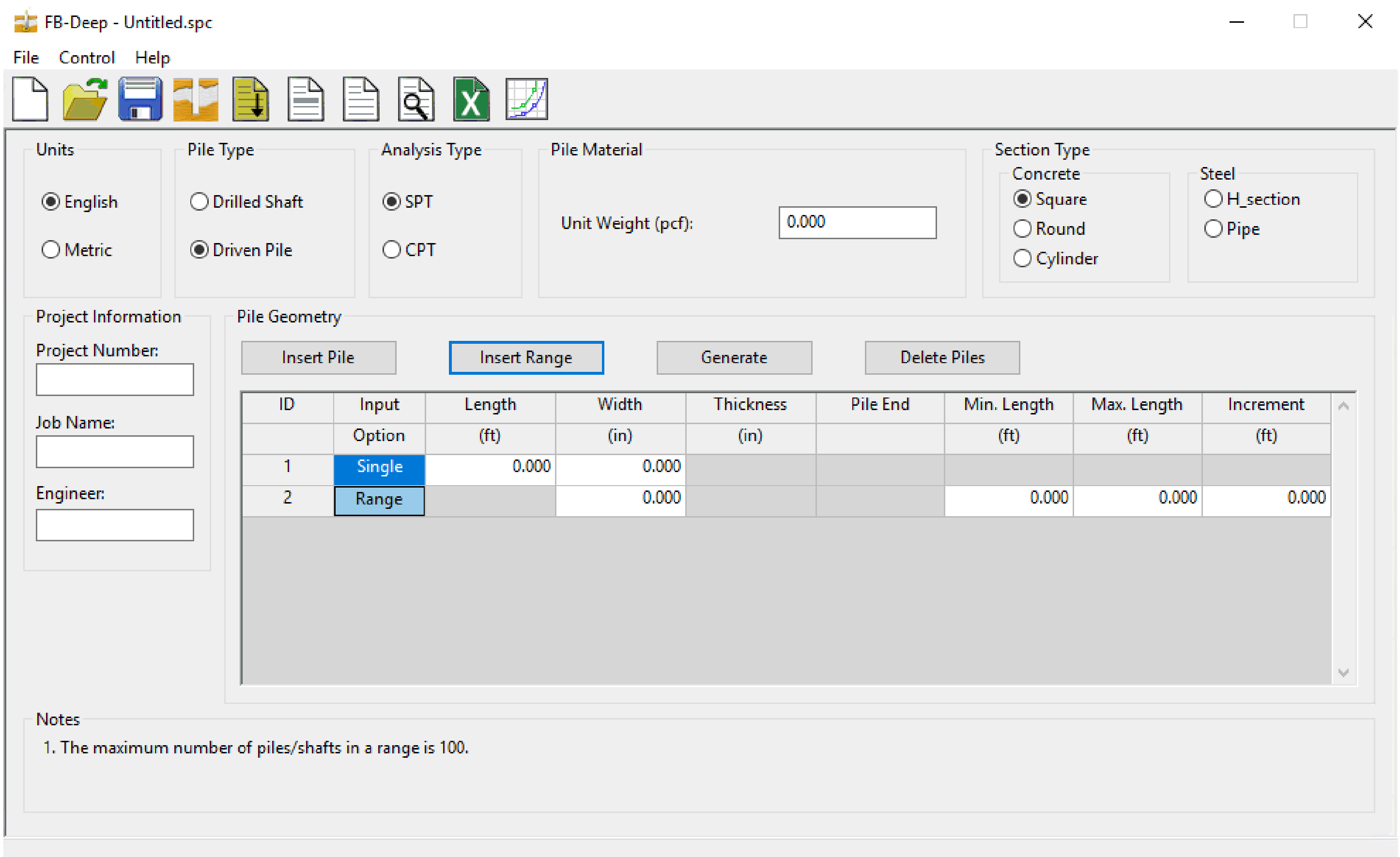


Figure: 3.5.a Driven Pile Model

Analysis Type

SPT or CPT analysis method can be defined in Driven Pile model.

Section Type

The user can choose a section type from among six options: Concrete Square, Concrete Round, Concrete Cylinder, Steel H Section, Steel Pipe Pile. Depending on the selected section type, the pile geometry grid changes to indicate the parameters required for the selected section type.

Insert Pile/Range

The user has the option to either specify a single pile ("**Insert Pile**") or a range ("**Insert Range**") of pile lengths that have the same section properties.

For pile range, the user specifies the maximum length, the minimum length and the increment. The program generates number of piles with lengths starting with the minimum length and not exceeding the maximum length. When, the user enters a value for the maximum length or the minimum length, the program checks that the minimum length is not greater than the maximum length.

The number of piles can be generated by a range is 100. As the user enters the maximum length and the minimum length, the program updates the increment, if the existing increment value results in the generation of more than 100 piles. In this case, the increment is modified to satisfy two conditions. One is that the number of generated piles is 100. The second is that the maximum length generated by the program is the closest to the user-defined maximum length.

If the user enters a value of the increment that generates more than 100 piles, the program rejects this value and the user is prompted to enter a new value.

Generate

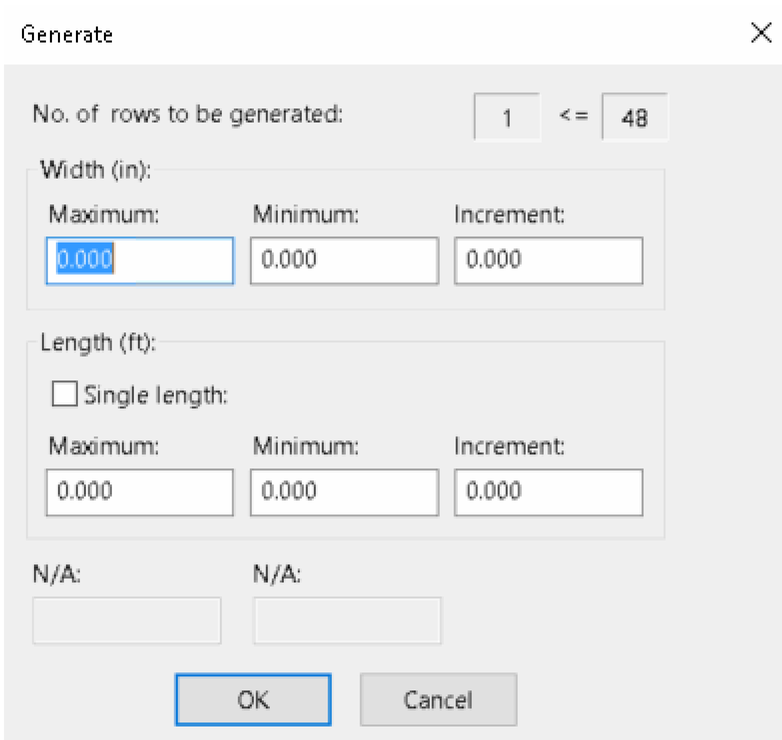


Figure: 3.5.b Generate Piles Dialog

The Generate dialog box aids the user to generate multiple pile records with different width. The width is set to change linearly between the first and last generated records. The user can generate multiple records of either single or range input option.

The user specifies the maximum width, the minimum width and the increment. The program generates number of records with widths starting with the minimum width and not exceeding the maximum width. When, the user enters a value for the maximum width or the minimum width, the program checks that the minimum width is not greater than the maximum width.

The maximum number of records can be generated depends on how many records already exist in the pile geometry grid. The total number of records that can be entered in the pile grid is 48. As the user enters the maximum width and the minimum width, the program updates the increment, if the existing increment value results in the generation of more than the maximum. In this case, the increment is modified to satisfy two conditions. One is that the number of generated piles is equal to the maximum. The second is that the maximum width generated by the program is the closest to the user-defined maximum width.

3.6 Database

1. Introduction to Database
2. Downloading Walkthrough
3. Uploading Walkthrough
4. XML Files
5. Troubleshooting

3.6.1 Introduction to Database

The Florida Department of Transportation (FDOT), in conjunction with the University of Florida, has developed a Geotechnical Database. This database contains geotechnical laboratory data, as well as construction as-built in-situ data. FB-Deep features connectivity to this database.

A login account is required to use the database. Contact your company's technical support department to have your account created. To learn more, log on to http://fdot.ce.ufl.edu/

Currently, FB-Deep can upload and download soil data. FB-Deep cannot upload or download pile/shaft data (pile length, diameter, shape, etc), though this enhancement will be added in a future version.

3.6.2 Downloading Walkthrough

Downloading (importing) from the database can be accomplished through the following steps:

Go to the Boring Log screen, either by clicking the Boring Log icon, or by using the 'Boring Log' menu item under the 'Show' menu (Figure: 3.6.a)

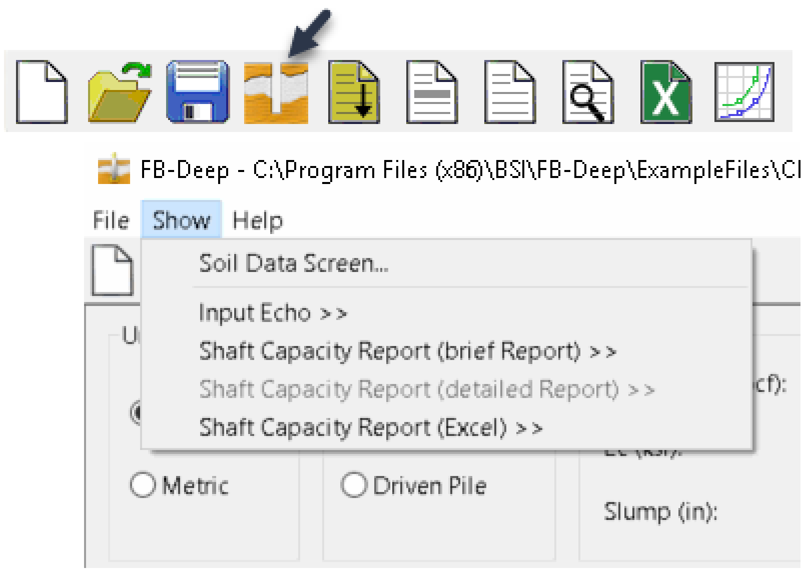


Figure: 3.6.a Opening Boring Log

When the Boring Log appears, click the 'Import/Export' button (Figure: 3.6.b). The database menu will appear. Select 'Download Soil Data from Database'.

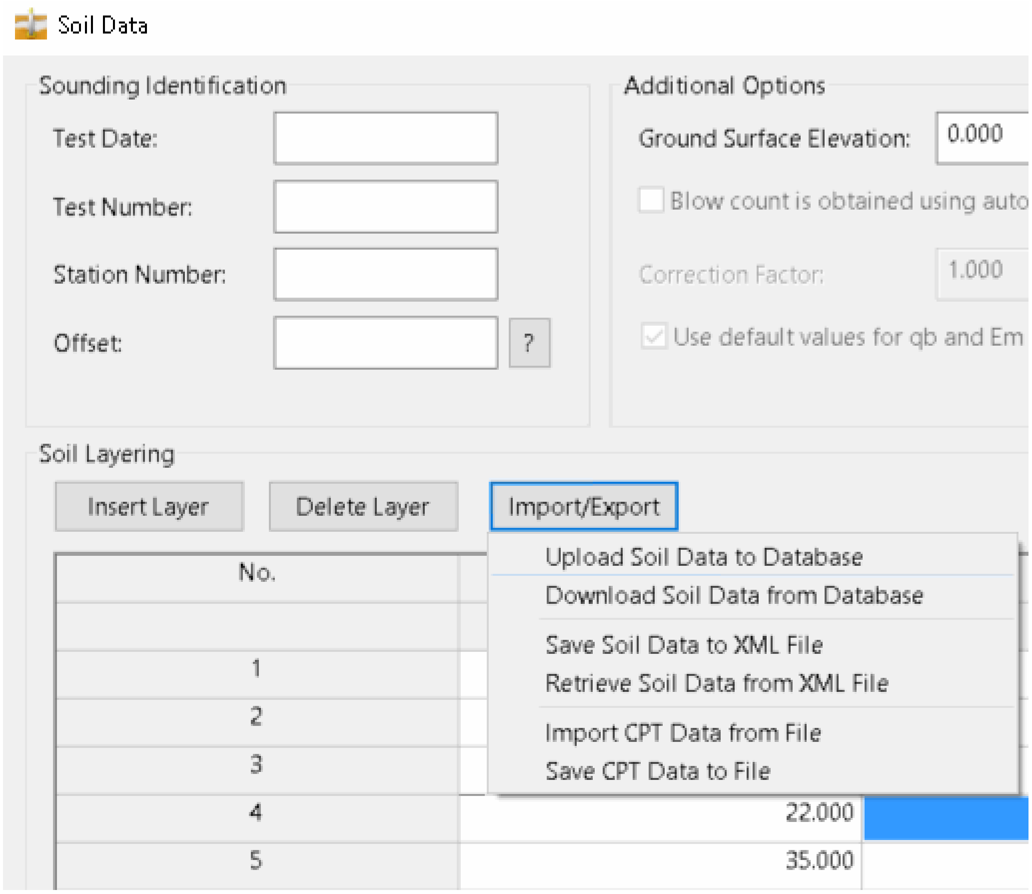


Figure: 3.6.b Import/Export Soil Data

If soil data is currently in the soil table, this data will be deleted. It is therefore recommended to save the current data before beginning the download process. Click Yes.

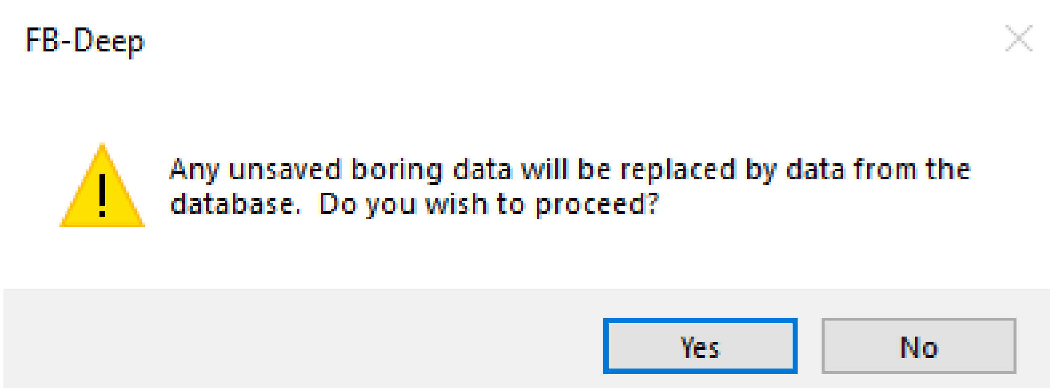


Figure: 3.6.c Warning Message

Database Connectivity message is be displayed. Click OK.

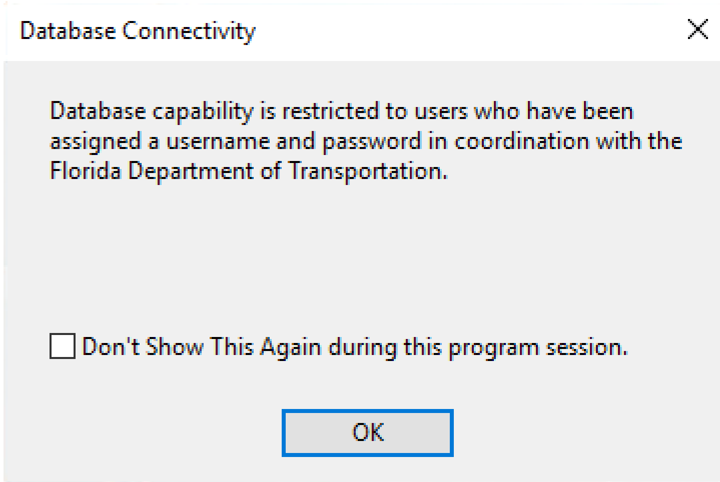


Figure: 3.6.d Database Connectivity Message

A few seconds after choosing 'Download Soil Data from Database', a Login screen will appear (Figure: 3.6.e).

Figure: 3.6.e Database Login Dialog

Enter a User Name and Password, and click the 'OK' button. (An account is required to use the database. If you do not have an account, please contact your company's technical support). The database tree will display (Figure: 3.6.f). This can take several seconds.

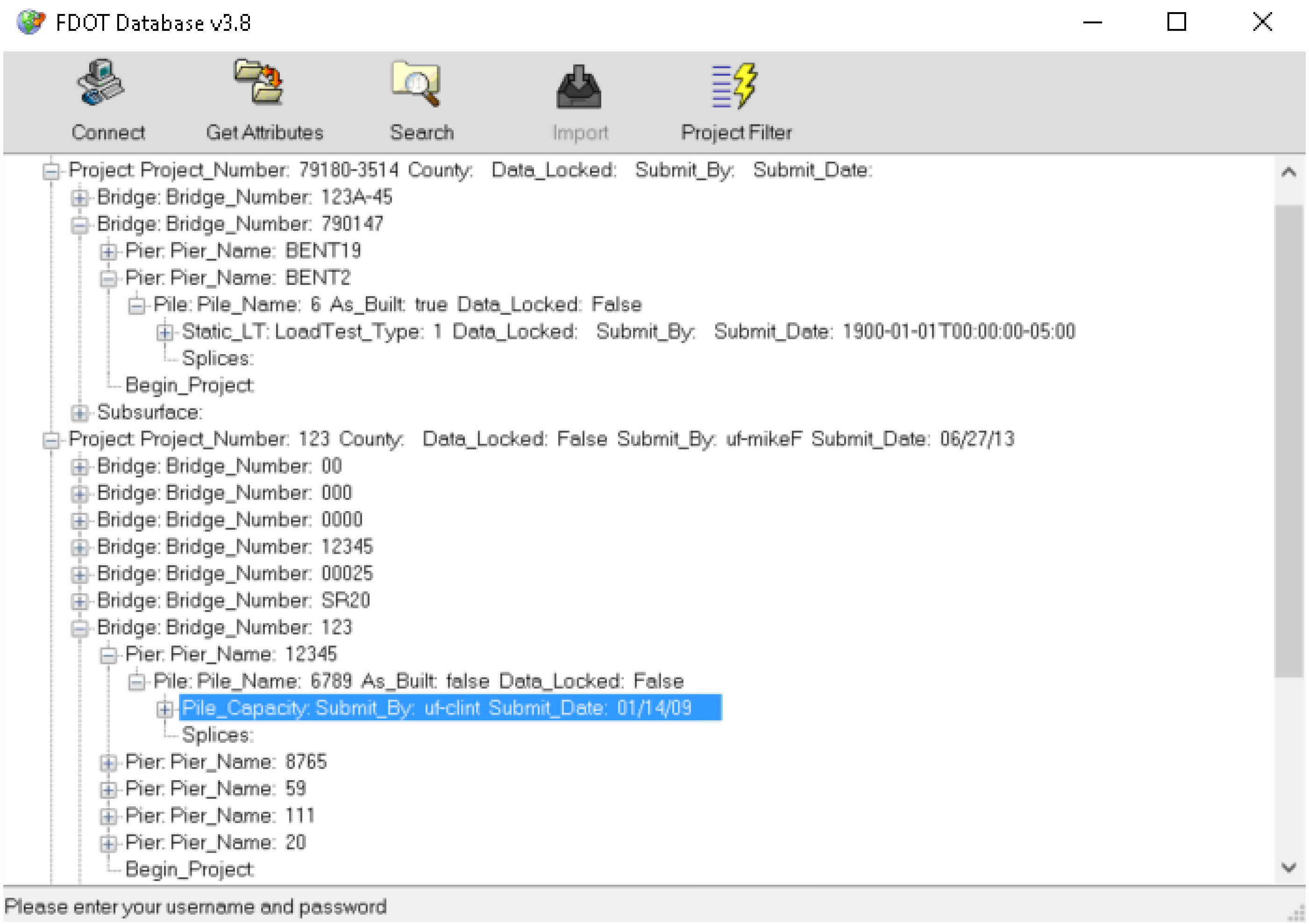


Figure: 3.6.f Database Tree Display

Once the tree is visible, search for the desired soil data. The pile type and analysis type will dictate what type of soil data should be downloaded. The following describes the types and location of downloadable soil data that are currently available, given a pile type and analysis type:

Drilled Shaft, SPT Analysis: a) Shaft soil data, available under 'GML → Bridge → Pier → Shaft → Shaft_Capacity → Soil_Data'. This dataset includes the following fields for each soil layer: depth, soil type, blowcount, total unit weight, undrained shear strength, unconfined compressive strength, tensile strength, end bearing, mass modulus, RQD reduction modification, shaft socket roughness, and rock recovery. All of these fields are supported by FB-Deep. The following data is also downloaded: Water Table Elevation, Boring Number, and Ground Surface Elevation. b) Rock Specimen soil data, available under 'GML → Subsurface → Hole → Lab_Rock → Core → Rock_Specimen'. This dataset includes the following fields which are supported by FB-Deep, for each soil layer: starting elevation (which is converted to a depth with respect to ground surface elevation), dry unit weight (which is currently downloaded into the total unit weight field in FB-Deep; this will result in a conservative calculation for skin friction of sand layers below the rock layers; these rock unit weight values can be adjusted as seen fit by the user), soil description, unconfined compressive strength, tensile strength, and E100 (intact modulus). The following data is also downloaded: Station and Offset, RQD reduction modification, and rock recovery.

Driven Pile, SPT Analysis: a) Pile soil data, available under 'GML → Bridge → Pier → Pile → Pile_Capacity → Soil_Data'. This dataset includes the following fields which are supported by FB-Deep, for each soil layer: depth, soil type, and blowcount. The following data is also downloaded: Boring Number, and Ground Surface Elevation.   b) Soil Specimen soil data, available under 'GML → Subsurface → Hole → Lab_Soil → Tube → Soil_Specimen'. This dataset includes the following fields which are supported by FB-Deep, for each soil layer: starting elevation (which is converted to a depth with respect to ground surface elevation), soil description, undrained shear strength, total unit weight, and E100. The following data is also downloaded: Station and Offset.

Driven Pile, CPT Analysis, Schmertmann or LCPC Methods: a) SPT Soil Layer Data, available under 'GML → Subsurface → Hole → CPT → CPT_Layer_Data. This dataset includes the following fields for each soil layer, which are supported by FB-Deep: elevation (which is converted to a depth with respect to ground surface elevation) and soil type. The following data is also downloaded: Cone Sounding Number, Station and Offset, and Ground Surface Elevation. b) SPT Data, available under 'GML → Subsurface → Hole → CPT → CPT_Data. This dataset includes the following fields for each reading, which are supported by FB-Deep: elevation (which is converted to a depth with respect to ground surface elevation), qc, and fs.

Driven Pile, CPT Analysis, UF Method: a) CPT Soil Layer Data, available under 'GML → Subsurface → Hole → CPT → CPT_Layer_Data. This dataset includes the following fields for each soil layer, which are supported by FB-Deep: elevation (which is converted to a depth with respect to ground surface elevation), soil type, kb, and Fs. . The following data is also downloaded: Cone Sounding Number, Station and Offset, and Ground Surface Elevation. b) CPT Data, available under 'GML → Subsurface → Hole → CPT → CPT_Data. This dataset includes the following fields for each reading, which are supported by FB-Deep: elevation (which is converted to a depth with respect to ground surface elevation), qc, and fs.

After selecting the desired soil branch, such as 'Shaft_Capacity' or 'Pile_Capacity', click the 'Get Attributes' button (Figure: 3.6.g).   This action isolates the soil layer data.

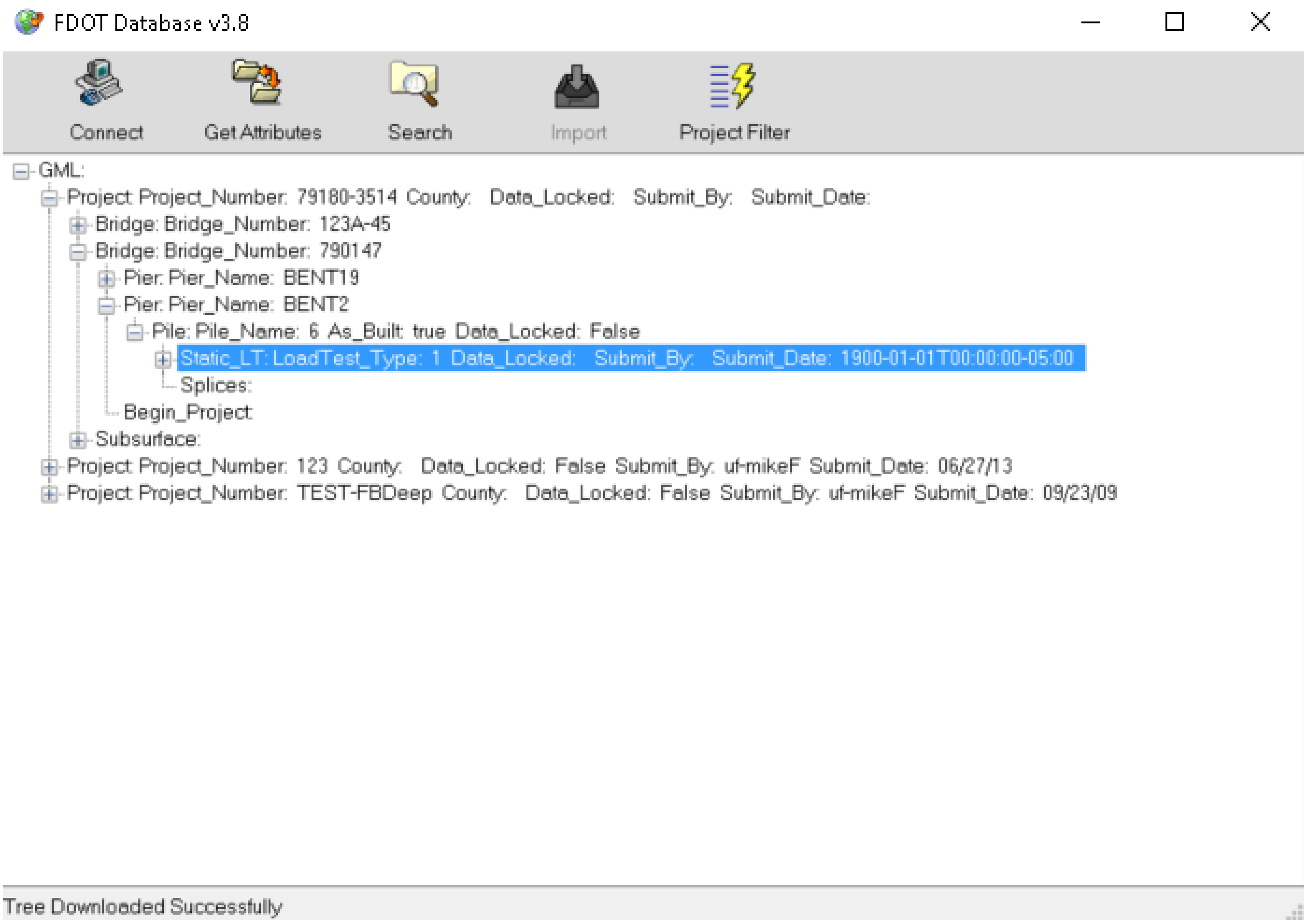


Figure: 3.6.g Isolate the required soil boring log.

Next, click the 'Import' button.

The imported soil data will then appear in the Soil Data Table in FB-Deep (Figure: 3.6.h)

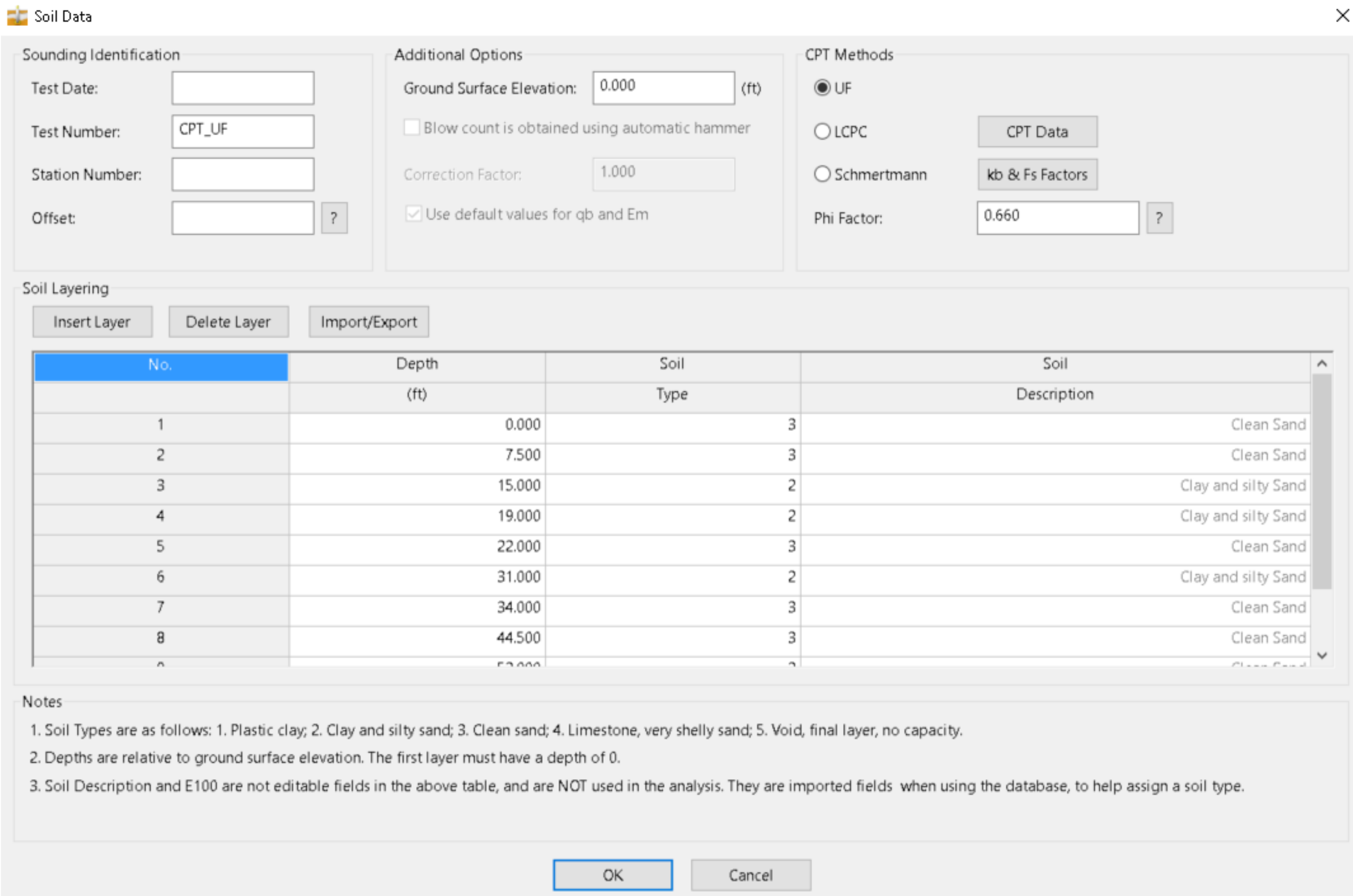


Figure: 3.6.h Soil Data Imported onto the Soil Data Dialog

Note that the Soil Description field is disabled. This field is not editable in FB-Deep. Certain soil data in the database will NOT have a soil type number, and will have a soil description that does not match one of the available soil types in FB-Deep. This soil description (along with other soil properties such as blowcount, unit weight, etc), can help the FB-Deep user decide which of FB-Deep 5 soil types to assign after the soil is imported.

It is important to remember that the database soil records may not have been originally uploaded from FB-Deep. Thus, when these soil records are downloaded, they may lack certain data that FB-Deep requires to run an analysis. It is therefore recommended after each download to double check the soil data within FB-Deep to ensure it is complete. The units of the downloaded data will automatically be converted, if necessary, to the current units in FB-Deep.

3.6.3 Uploading Walkthrough

Uploading to the Geotechnical Database can be accomplished through the following steps:

Go to the Boring Log screen, either by clicking the Boring Log icon, or using the 'Boring Log' menu item under the 'Show' menu (Figure: 3.6.i).

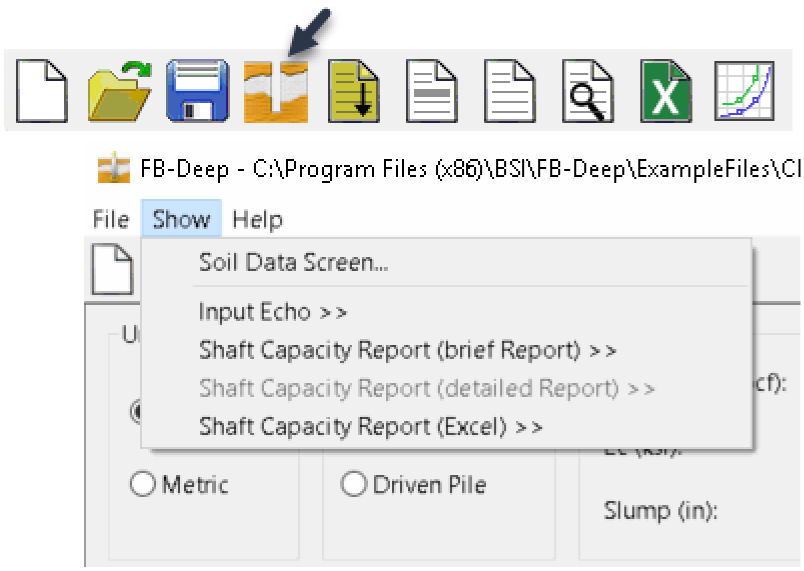


Figure: 3.6.i Opening Boring Log

When the Boring Log appears, enter desired Soil Data in the Soil Table. Then click the "Import/Export Soil Data' button (Figure: 3.6.j)

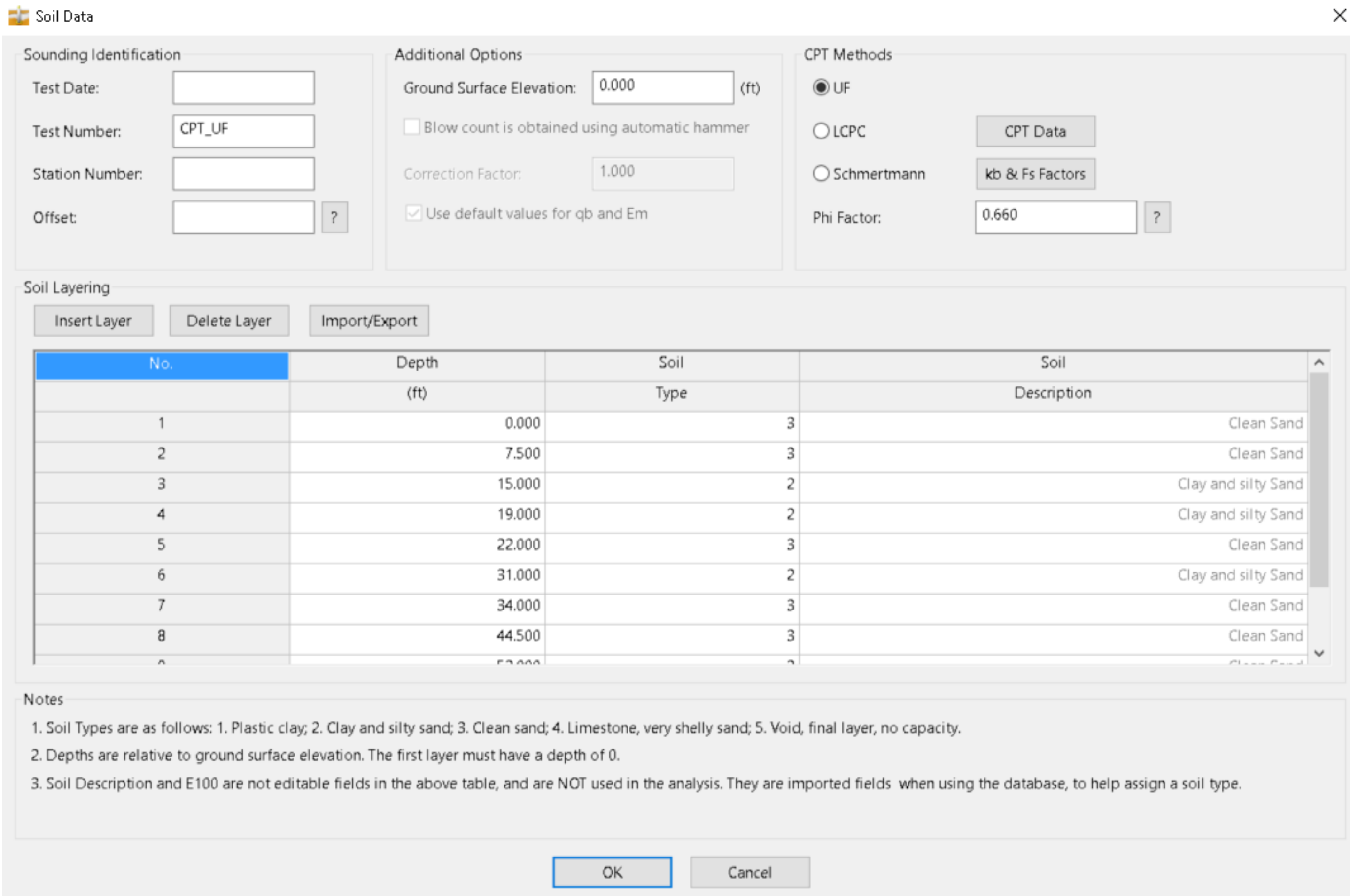


Figure: 3.6.j Soil Data Dialog

A menu (Figure: 3.6.k) will appear, with the following choices: 'Upload Soil Data to Database', 'Download Soil Data from Database', 'Save Soil Data to XML File', and 'Retrieve Soil Data from XML File'. Select 'Upload Soil Data from Database'.

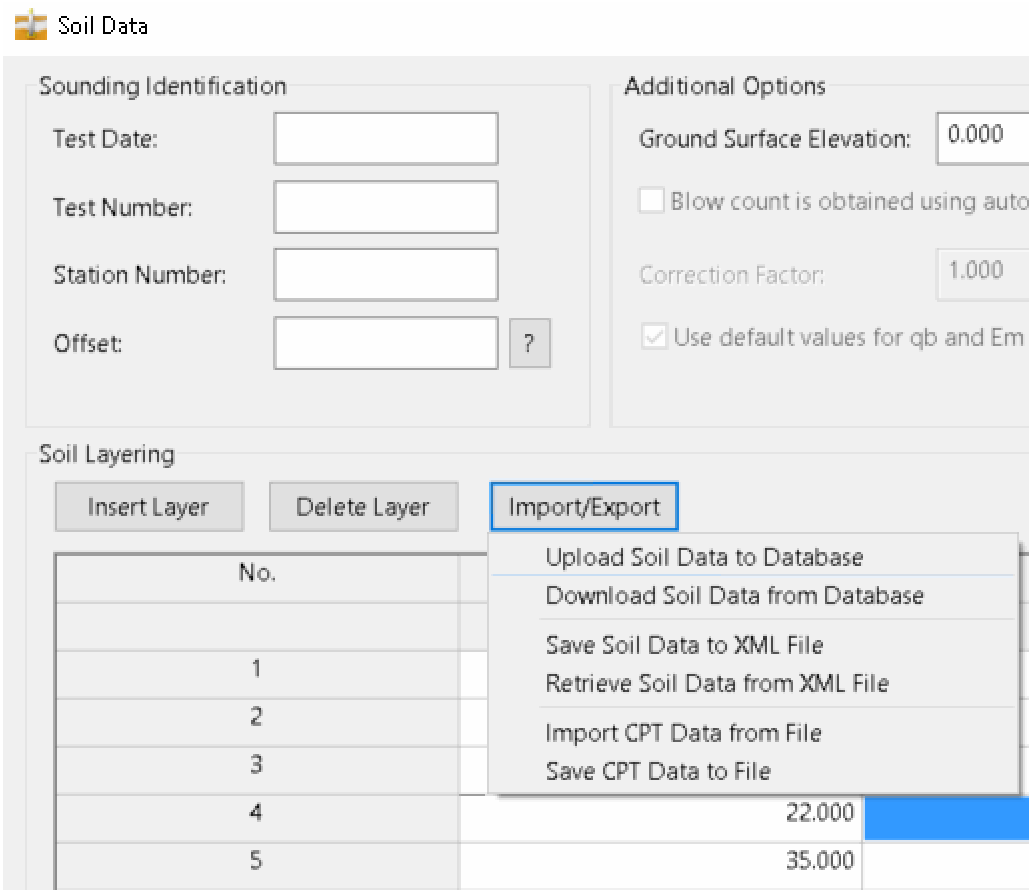


Figure: 3.6.k Import/Export Soil Data

A database log-in screen will display (Figure: 3.6.l), prompting for required info that will be used to position the uploaded data in the database.

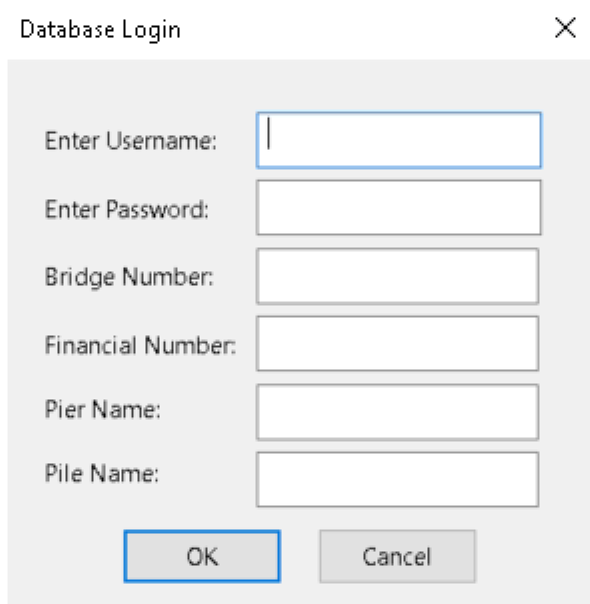


Figure: 3.6.l Database Upload Login Window

This required info varies, depending on the Analysis Type. For a SPT Analysis, the required log-in data include a) user name, b) password, c) Bridge Number, d) Financial Number, e) Pier Name, and f) Pile/Shaft Name. A Project Number is also required. However the Project Number is input on the Pile/Shaft input screen in FB-Deep, NOT in the log-in screen. For a CPT Analysis, the required log-in data includes a) user name, and b) password. The Cone Sounding Number is also required, but is input on the Boring Log (Soil Data screen), NOT in the log-in screen. After completing the log-in info, click the 'OK' button to send the data to the database. This can take several seconds, but generally will take should take less much less than one minute. When complete, the Upload Log screen will appear (Figure: 3.6.l)

Figure: 3.6.m Upload Log Window

To view the results of the Upload, click 'View Your Upload Log'. The Upload Log shows the status of each type of data that uploaded. For example, the result 'Pile_Soil_Data inserted' displays seven times, one for each uploaded soil layer (Figure: 3.6.n).

```
Your DLL Version: 3.5

Projects- Project_Number='123'
    Not Updated
Bridges- Bridge_Number='789'
    Updated
Piers- Pier_Name='20'
    Updated
Piles- Pile_Name='12'***AsBuilt=0
    Updated
Pile_Capacity
    Updated
Pile_Soil_Data - Deleted Original Records
    Inserted
Pile_Soil_Data
    Inserted
Pile_Soil_Data
    Inserted
Pile_Soil_Data
    Inserted
Pile_Soil_Data
    Inserted
Pile_Soil_Data
    Inserted
Pile_Soil_Data
    Inserted
Holes- Hole_Name='123-a.1'
    Inserted
```

Figure: 3.6.n Pile Soil Data Inserted

If an error kept some or all of the data from uploading successfully, the error will display in the Upload Log. For assistance troubleshooting database errors, please email the Bridge Software Institute at bsi@ce.ufl.edu

The following is a complete list of data that is uploaded to the database. As is the case with downloading, the pile type (pile or shaft) and analysis type (SPT or CPT) dictates the type of data that is uploaded, as well as the location in which the uploaded data resides.

Drilled Shaft, SPT Analysis: this data is uploaded to the following location: 'GML → Bridge → Pier → Shaft → Shaft_Capacity → Soil_Data'. This dataset includes the following fields for each soil layer: depth, soil type, blowcount, total unit weight, undrained shear strength, unconfined compressive strength, tensile strength, end bearing, mass modulus, RQD reduction modification, shaft socket roughness, and rock recovery. The following data from the Pile/Shaft screen is also uploaded: Project Name, Project Number, Water Table Elevation, and Units. The following data from the Boring Log screen is also uploaded: Boring Number, and Ground Elevation. The following data from the Login screen is also uploaded: Bridge Number, Bridge Financial Number, Pier Name, and Shaft Name.

Driven Pile, SPT Analysis: this data is uploaded to the following location: 'GML → Bridge → Pier → Pile → Pile_Capacity → Soil_Data'. This dataset includes the following fields for each soil layer: depth, soil type, and blowcount. The following data from the Pile/Shaft screen is also uploaded: Project Name, Project Number, Water Table Elevation, and Units. The following data from the Boring Log screen is also uploaded: Boring Number, and Ground Elevation. The following data from the Login screen is also uploaded: Bridge Number, Bridge Financial Number, Pier Name, and Pile Name.

Driven Pile, CPT Analysis, with Schmertmann or LCPC method: the soil layer data is uploaded to the following location: 'GML → Subsurface → Hole → CPT → CPT_Layer_Data'. This dataset includes the following fields for each soil layer: depth and soil type. The reading data is uploaded to the following location: 'GML → Subsurface → Subsurface → Hole → CPT → CPT_Data'. This dataset includes the following fields for each reading: depth, qc, and fs. The following data from the Pile/Shaft screen are also uploaded: Project Name, Water Table Elevation, and Units. The following data from the Boring Log (Soil Data) screen are also uploaded: Sounding Date, Cone Sounding Number, Station and Offset, and Ground Elevation.

Driven Pile, CPT Analysis, UF method: the soil layer data is uploaded to the following location: 'GML → Subsurface → Hole → CPT → CPT_Layer_Data'. This dataset includes the following fields for each soil layer: depth, soil type, kb, and Fs. The reading data is uploaded to the following location: 'GML → Subsurface → Hole → CPT → CPT_Data'. This dataset includes the following fields for each reading: depth, qc and fs The following data from the Pile/Shaft screen are also uploaded: Project Name, Water Table Elevation, and Units. The following data from the Boring Log (Soil Data) screen are also uploaded: Sounding Date, Cone Sounding Number, Station and Offset, and Ground Elevation.

3.6.4 XML Files

Related to the database connectivity is the ability to save and read soil data to and from XML files. To save to an XML file, click the ‘Import/Export’ button on the Boring Log. Then choose the menu item ‘Save Soil Data to an XML File’. This action saves the soil data to an XML file. This file differs from a "full" FB-Deep input file (.spc), in that this XML file does not contain any pile or shaft data. It contains only soil data (ie, layer depth, soil type, reading data, etc). This file can later be opened from the Boring Log screen, by clicking the ‘Import/Export Soil Data’ button, and choosing the menu item ‘Retrieve Soil Data from XML File’.

3.6.5 Troubleshooting

Incorrect Time on System Clock – If the system clock on the computer that has FB-Deep installed does not match the time on the database server, there is potential for an unsuccessful database connection. The degree of accuracy with which the two clocks must match is approximately 1 minute. The server updates its clock automatically via an on-line source. Therefore, if a connection to the database cannot be established, check the system clock on your machine. (This generally in not a problem on newer operating systems, such as XP or Vista).

Multiple Soil Sets – A soil set is defined as a group of soil layer data; for example, one boring log of data (see the section on the Soil Set Selector Window below for further explanation). FB-Deep can only store and display ONE set of soil data at a time. Thus, if an attempt is made to download multiple sets of soil data from the database in one download, an "Invalid Data" message will display. The same is true for attempting to import multiple soil sets from an XML file.

FB-Deep will prompt the user to select only one of these Soil Sets. In so doing, the Soil Set Selector window will display (Figure: 3.6.o).

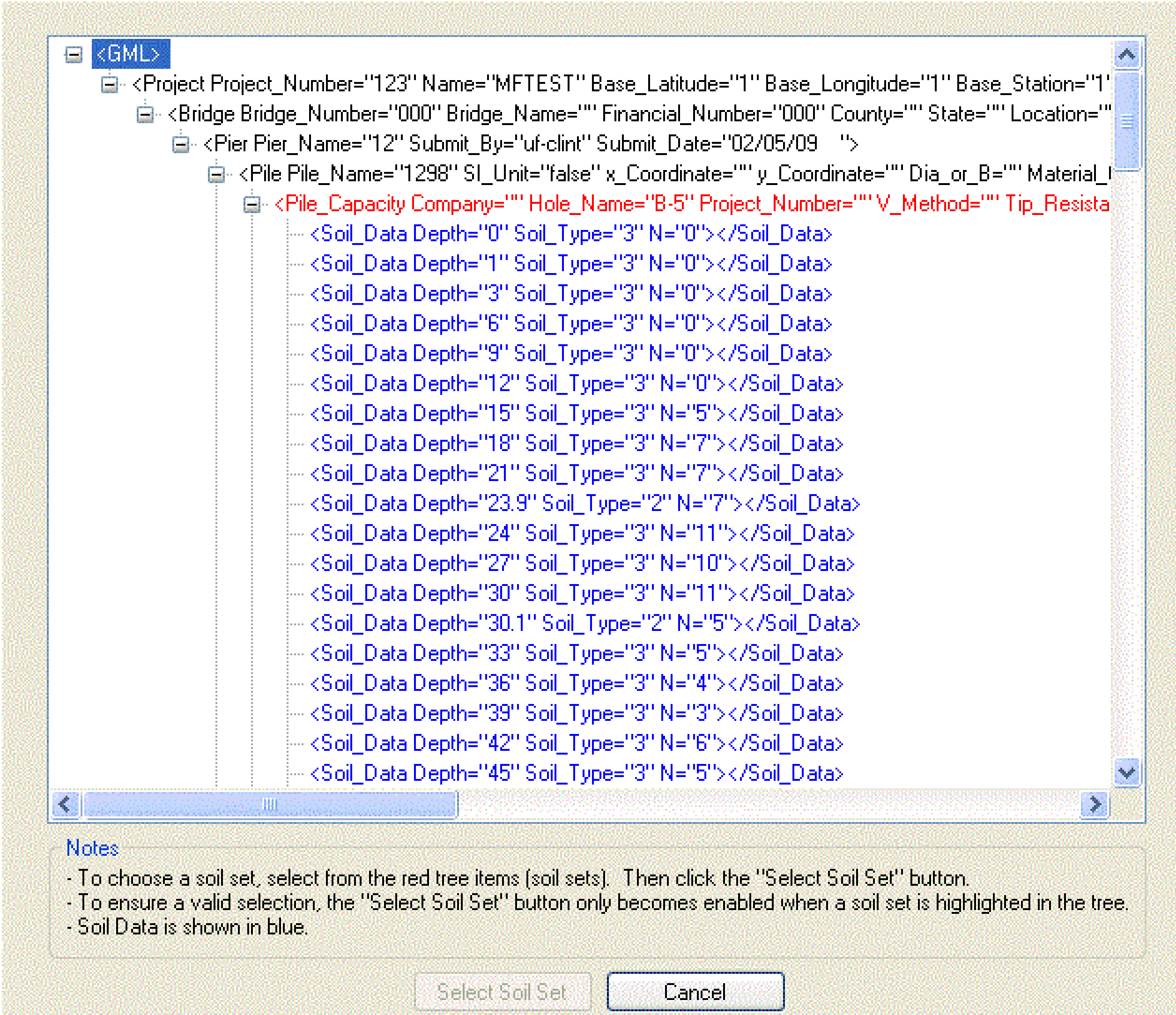


Figure: 3.6.o Soil Set Selector Window

The Soil Set Selector window displays an XML string in tree form. In the tree, each line of Soil Set data will begin with one of the following tags: "Soil_Data, "CPT_Layer_Data", "Rock_Specimen", or "Soil_Specimen". These lines of soil data are colored in blue, to make them easier to locate in the tree.

Each Soil Set will have a Soil Set parent tree node located immediately above the Soil Set. These parent nodes are colored in red. Each parent will begin with one of the following tags: Pile_Capacity, Shaft_Capacity, Core, Tube, or CPT.

To select a Soil Set (colored in blue), click on a soil set parent node (colored in red). These soil set parent nodes are positioned immediately above soil sets. For example, in Figure 2, the soil set parent node is <Pile_ Capacity Company="" Hole_Name-"B-5"...... And the Soil Set is the series of blue lines immediately below this line, beginning with <Soil_Data Depth="0" Soil_Type="3" N="0"..... When a soil set parent node is selected, the "Select Soil Set" button becomes enabled (Figure: 3.6.p).

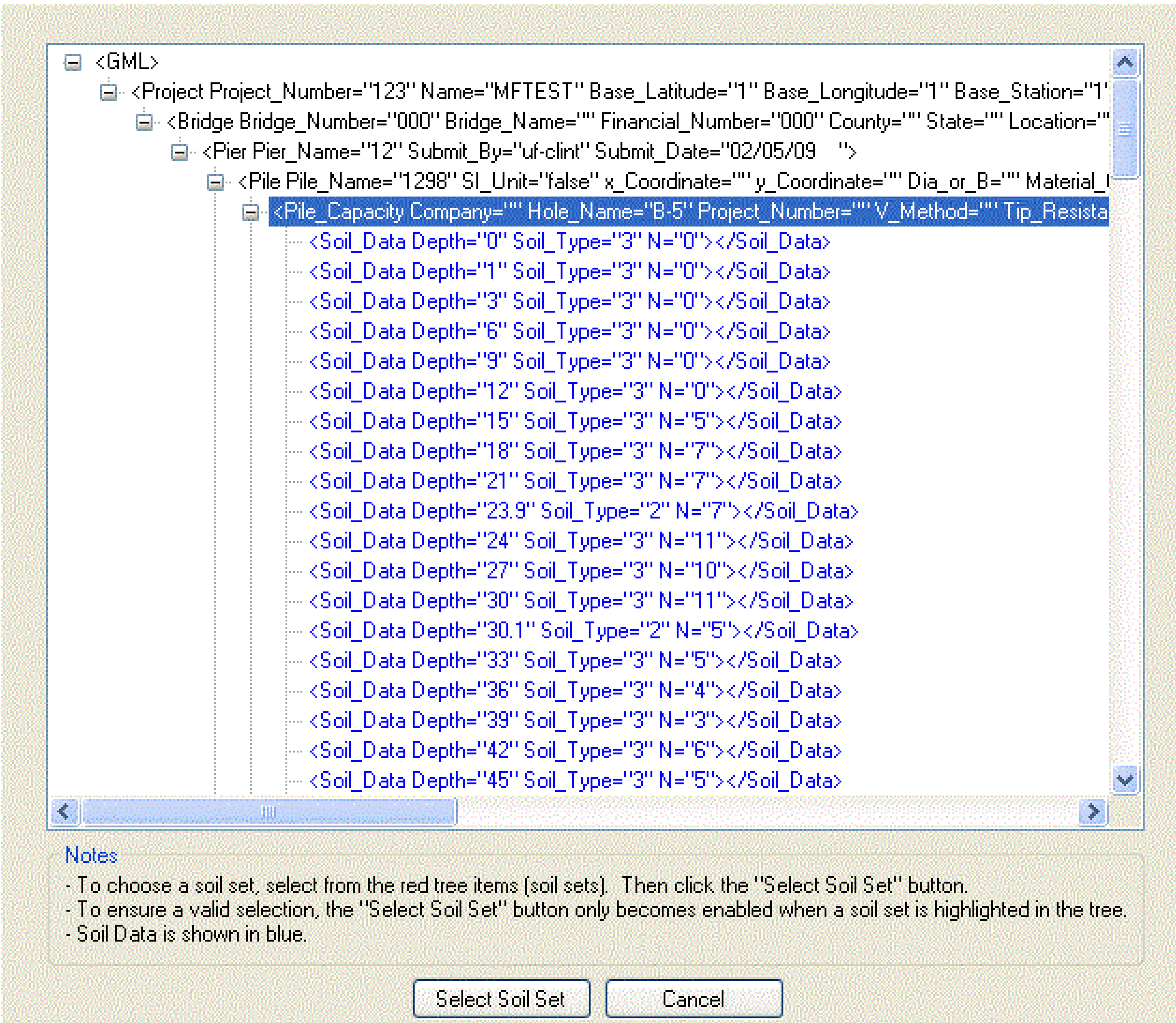


Figure: 3.6.p Soil Set Selector Window with Soil Set parent node selected (highlighted)

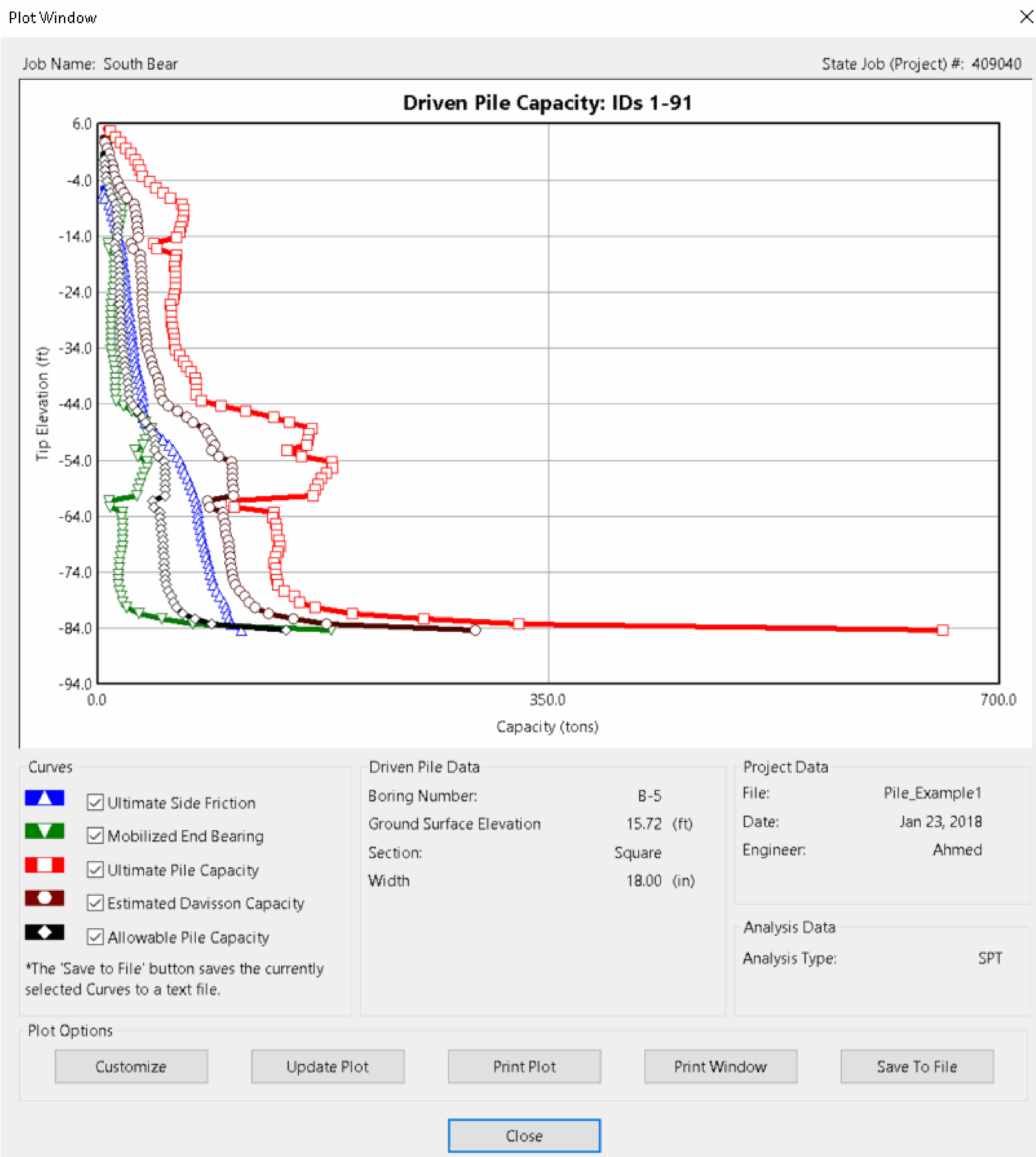
Click the "Select Soil Set" button to import the soil set into the Boring Data/Soil Layering Table.

After the selection has been made, the Soil Set data displays in the Boring Log/Soil Data window.

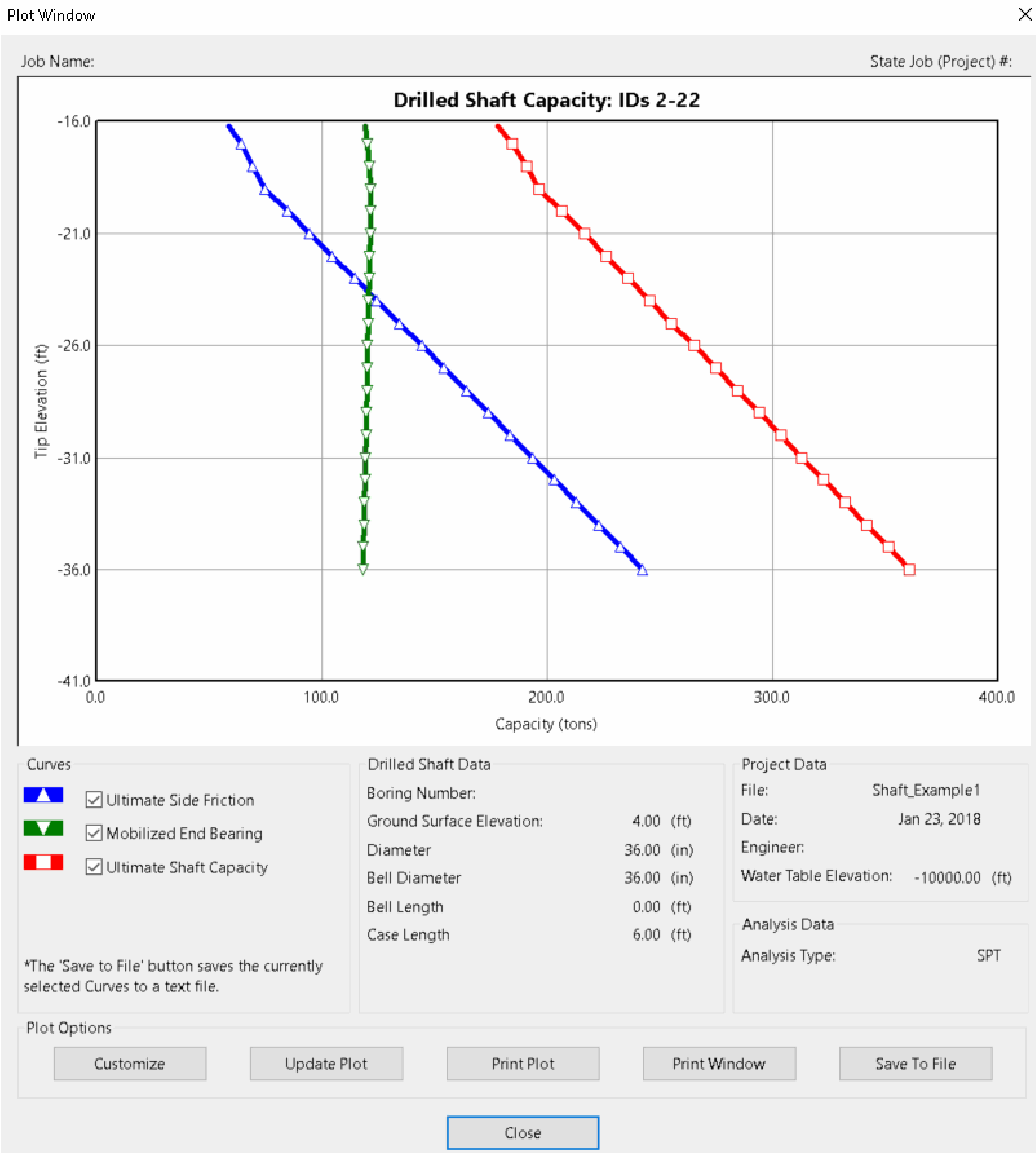
3.7 Graphical Output

FB-Deep produces several types of graphical results, all of which are shown in the Plot Window.

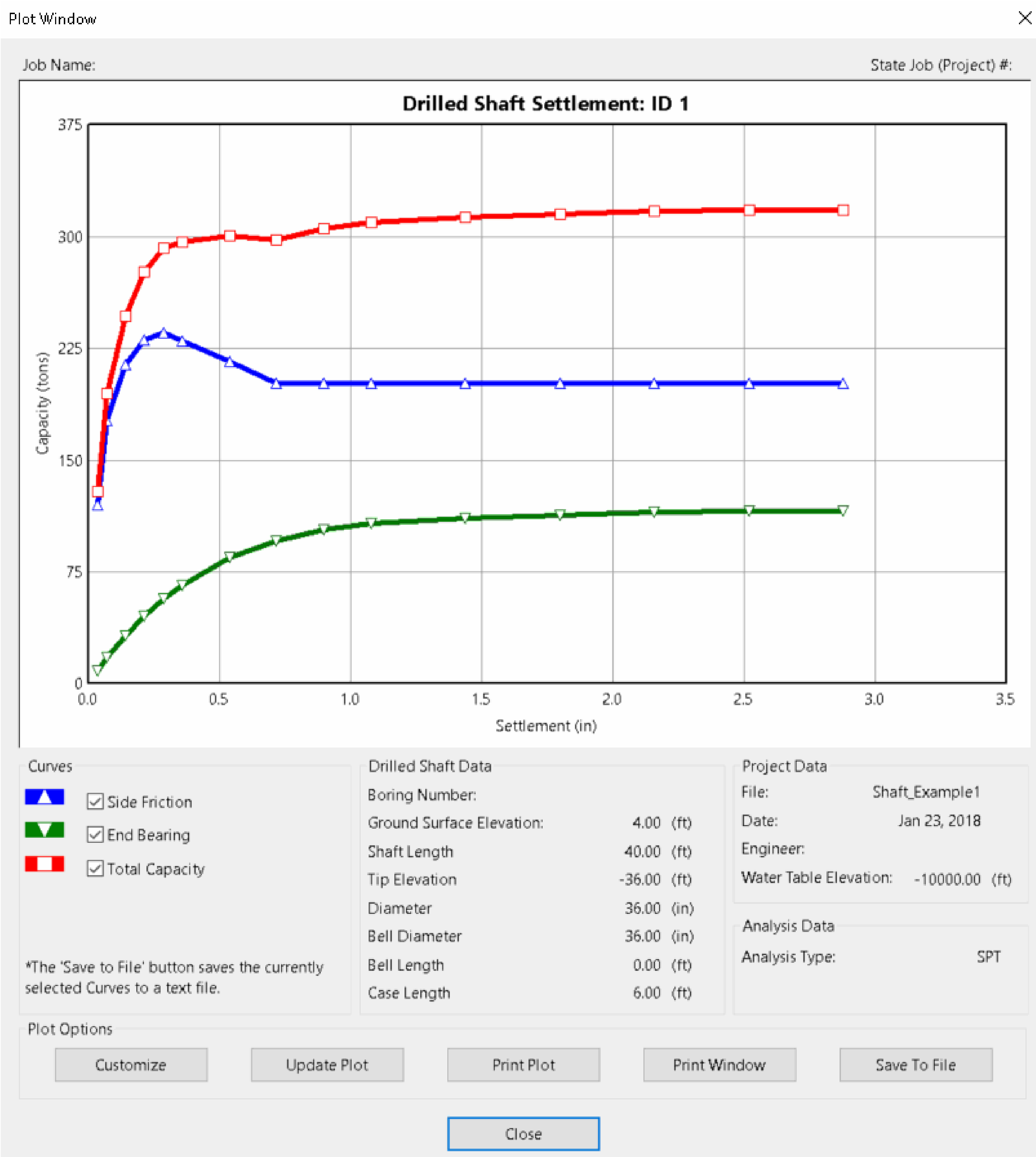
Driven Pile Capacity for a range of piles:


Plot Window
Job Name: South Bear
State Job (Project) #: 409040
Driven Pile Capacity: IDs 1-91
Tip Elevation (ft)
6.0
-4.0
-14.0
-24.0
-34.0
-44.0
-54.0
-64.0
-74.0
-84.0
-94.0
0.0
350.0
700.0
Capacity (tons)
Curves
Ultimate Side Friction
Mobilized End Bearing
Ultimate Pile Capacity
Estimated Davisson Capacity
Allowable Pile Capacity
*The 'Save to File' button saves the currently selected Curves to a text file.
Driven Pile Data
Boring Number: B-5
Ground Surface Elevation 15.72 (ft)
Section: Square
Width 18.00 (in)
Project Data
File: Pile_Example1
Date: Jan 23, 2018
Engineer: Ahmed
Analysis Data
Analysis Type: SPT
Plot Options
Customize
Update Plot
Print Plot
Print Window
Save To File
Close

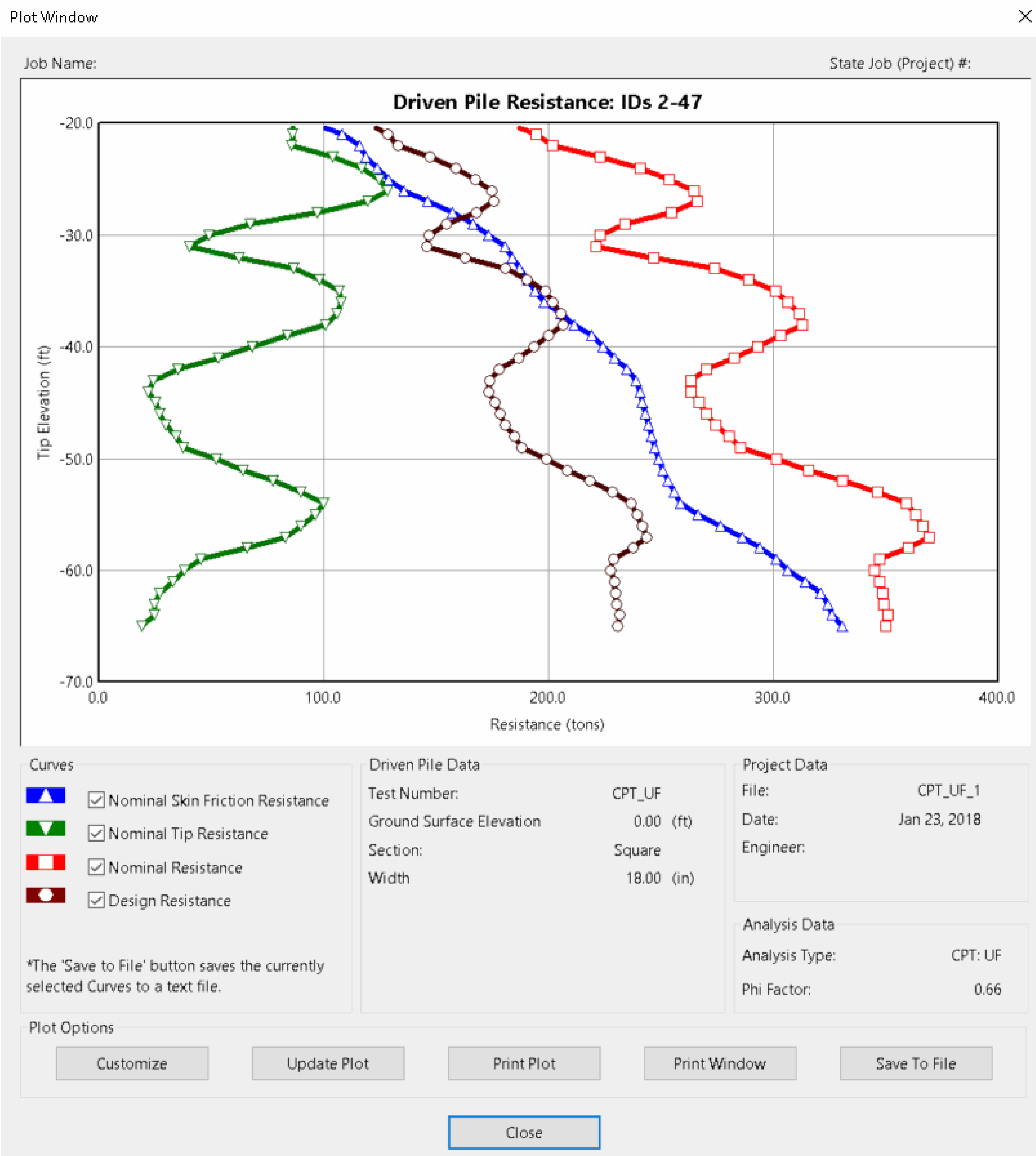
Drilled Shaft Capacity for a range of shafts:



Drilled Shaft Settlement for a single shaft:

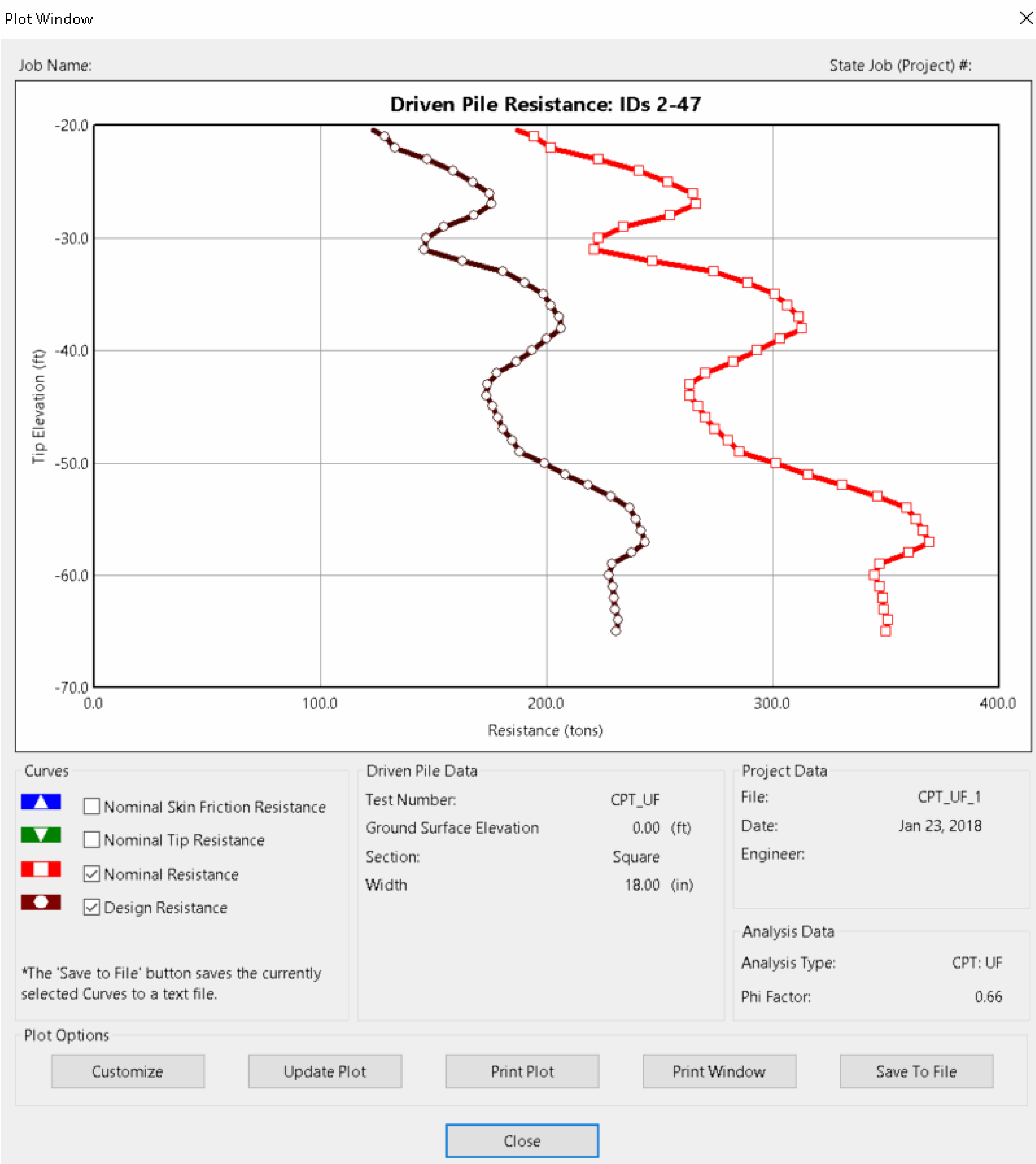

Plot Window
Job Name:
State Job (Project) #:
Drilled Shaft Settlement: ID 1
Capacity (tons)
375
300
225
150
75
0
0.0
0.5
1.0
1.5
2.0
2.5
3.0
3.5
Settlement (in)
Curves
Side Friction
End Bearing
Total Capacity
*The 'Save to File' button saves the currently selected Curves to a text file.
Drilled Shaft Data
Boring Number:
Ground Surface Elevation: 4.00 (ft)
Shaft Length 40.00 (ft)
Tip Elevation -36.00 (ft)
Diameter 36.00 (in)
Bell Diameter 36.00 (in)
Bell Length 0.00 (ft)
Case Length 6.00 (ft)
Project Data
File: Shaft_Example1
Date: Jan 23, 2018
Engineer:
Water Table Elevation: -10000.00 (ft)
Analysis Data
Analysis Type: SPT
Plot Options
Customize
Update Plot
Print Plot
Print Window
Save To File
Close

Driven Pile Resistance for a range of piles, using CPT analysis:



The Plot Window's graph area is customizable. Plot line size, color, fonts, x and y axes range, and the number of tick marks and grid lines can all be adjusted. The plot area can also be zoomed and panned. To access these customizable options, click the "Custom" button at the bottom of this dialog.

By default, all curve types are displaced. Curves can be hidden by unchecking them in the "Curves" frame, as pictured below. Because the x and y axis ranges are based on the currently displayed curves, hiding certain curves sometimes readjusts the plot perspective.


Plot Window
Job Name:
State Job (Project) #:
Driven Pile Resistance: IDs 2-47
Tip Elevation (ft)
-20.0
-30.0
-40.0
-50.0
-60.0
-70.0
0.0
100.0
200.0
300.0
400.0
Resistance (tons)
Curves
Nominal Skin Friction Resistance
Nominal Tip Resistance
Nominal Resistance
Design Resistance
*The 'Save to File' button saves the currently selected Curves to a text file.
Driven Pile Data
Test Number: CPT_UF
Ground Surface Elevation 0.00 (ft)
Section: Square
Width 18.00 (in)
Project Data
File: CPT_UF_1
Date: Jan 23, 2018
Engineer:
Analysis Data
Analysis Type: CPT: UF
Phi Factor: 0.66
Plot Options
Customize
Update Plot
Print Plot
Print Window
Save To File
Close

Graphical data can be saved to a text (.txt) file. The data saved in the text file matches the curves that are currently selected in the "Curves" frame on the Plot Window. This saved data can be used in other software, such as EXCEL. Here is an example of the file format:

Test Pile Length (ft)	Pile Width (in)	Nominal Skin Friction Resistance (tons)	Nominal Tip Resistance (tons)	Nominal Resistance (tons)	Design Resistance (tons)	Phi Factor
20.00	18.0	101.06	86.30	187.36	123.66	0.660
21.00	18.0	108.68	86.25	194.93	128.65	0.660
22.00	18.0	116.30	85.93	202.23	133.47	0.660
23.00	18.0	119.26	104.13	223.39	147.44	0.660
24.00	18.0	123.93	117.22	241.15	159.16	0.660
25.00	18.0	128.99	124.84	253.83	167.53	0.660
26.00	18.0	136.10	128.88	264.98	174.89	0.660
27.00	18.0	146.51	120.00	266.51	175.90	0.660
28.00	18.0	157.77	97.27	255.04	168.32	0.660
29.00	18.0	166.74	67.51	234.25	154.61	0.660
30.00	18.0	173.61	49.40	223.01	147.18	0.660
31.00	18.0	180.74	40.65	221.39	146.12	0.660
32.00	18.0	184.22	62.72	246.93	162.98	0.660
33.00	18.0	187.47	86.73	274.21	180.98	0.660
34.00	18.0	190.93	98.18	289.11	190.81	0.660
35.00	18.0	194.19	107.10	301.29	198.85	0.660
36.00	18.0	198.23	108.21	306.45	202.25	0.660
37.00	18.0	205.69	106.09	311.79	205.78	0.660
38.00	18.0	211.84	101.08	312.92	206.53	0.660
39.00	18.0	219.32	84.33	303.65	200.41	0.660
40.00	18.0	224.61	68.74	293.35	193.61	0.660

To save curve data to a text file, use the following steps:

1) On the Plot Window, select the desired curves in the "Curves" frame, by checking and/or unchecking the appropriate checkboxes.

2) Click the "Save to File" button.

3) By default, the directory for the text file will match the name of the current input file's directory. This path can be changed if desired, by browsing to a new directory using the "Save As" file dialog.

4) By default, the file name for the text file will match the name of the current input file, plus the ID(s) of the plotted shaft(s) or pile(s), plus a series of two-letter curve-type codes. These curve-type codes represent the currently selected curves in the "Curves" frame on the "Plot Window". For example, "SF" is the code for "Skin Friction", and "EB" is the code for "End Bearing". If all curves are selected, the curve-type code "ALL" will be used. For a complete list of codes, see the "Curve-Type Codes" list below. The file name can be changed by typing a new file name in the "Save As" file dialog.

Curve-type Codes

1. Drilled Shafts
    a. Single Shaft (Settlement Curve)
        i. "SF" - Side Friction
        ii. "EB" - End Bearing
        iii. "TC" - Total Capacity
        iv. "ALL" - All three of the above curves are selected
    b. Range of Shafts (Capacity Curve)
        i. "SF" - Ultimate Side Friction
        ii. "EB" - Mobilized End Bearing
        iii. "UC" - Ultimate Shaft Capacity
        iv. "ALL" - All three of the above curves are selected
2. Driven Piles
    a. SPT Analysis
        i. Range of Piles (Capacity Curve)
            I. "SF" - Ultimate Side Friction
            II. "EB" - Mobilized End Bearing
            III. "UC" - Ultimate Pile Capacity
            IV. "DC" - Estimated Davisson Capacity

V. "AC" - Allowable Pile Capacity

VI. "ALL" - All five of the above curves are selected

b. CPT Analysis

i. Range of Piles (Resistance Curve)

I. "SR" - Nominal Skin Friction Resistance

II. "TR" - Nominal Tip Resistance

III. "NR" - Nominal Resistance

IV. "DR" - Design Resistance

V. "ALL" - All four of the above curves are selected

Example File Names assigned by FB-Deep

Example 1:
Given the name of the input file is "MyInputFile.spc".
Given the curve type is a Drilled Shaft Settlement Curve.
Given the selected curves are Side Friction and End Bearing.
Given the plotted shaft ID is 1.
The default file name for the text file of curve values will be "MyInputFile_ID_1-SF-EB.txt".

Example 2:
Given the name of the input file is "MyInputFile.spc".
Given the curve type is a Drilled Shaft Settlement Curve.
Given the selected curves are Side Friction, End Bearing and Total Capacity.
Given the plotted shaft ID is 1.
The default file name for the text file of curve values will be "MyInputFile_ID_1-ALL.txt".

Example 3:
Given the name of the input file is "MyInputFile.spc".
Given the curve type is a Drilled Shaft Capacity Curve for a Range of Shafts.

Given the selected curves are Ultimate Side Friction.
Given the plotted shaft IDs are is 1 thru 5.
The default file name for the text file of curve values will be "MyInputFile_ID_1-5-SF.txt".

Example 4:
Given the name of the input file is "MyInputFile.spc".
Given the curve type is a Drilled Shaft Capacity Curve for a Range of Shafts.
Given the selected curves are Mobilized End Bearing and Ultimate Shaft Capacity.
Given the plotted shaft IDs are is 1 thru 5.
The default file name for the text file of curve values will be "MyInputFile_ID_1-5-EB-UC.txt".

Example 5:
Given the name of the input file is "MyInputFile.spc".
Given the curve type is a Driven Pile Capacity Curve for a Range of Piles, using an SPT analysis.
Given the selected curves are Ultimate Side Friction, Estimated Davisson
Capacity, and Allowable Pile Capacity.
Given the plotted pile IDs are is 1 thru 5.
The default file name for the text file of curve values will be "MyInputFile_ID_1-5-SF-DC-AC.txt".

Example 6:
Given the name of the input file is "MyInputFile.spc".
Given the curve type is a Driven Pile Resistance Curve for a Range of Piles, using an CPT analysis.
Given the selected curves are Nominal Skin Friction Resistance, Nominal Tip
Resistance, and Design Resistance.
Given the plotted pile IDs are is 1 thru 5.
The default file name for the text file of curve values will be "MyInputFile_ID_1-5-SR-TR-DR.txt".

3.8 Backwards Compatibility

Several changes were implemented in FB-Deep, version 1.19. These changes included the following: new unit skin friction and unit end bearing formulas for concrete cylinder piles with a diameter of greater than 36" (914.4mm); new unit skin friction and unit end bearing formulas for steel pipe piles with a diameter of greater than 36" (914.4mm); requiring the use of the Rock Side Friction method A qu ^ B if there are any smooth sockets present in the boring log; use of the new variable Rock Recovery to modify the skin friction in rock (formerly, RQD Modification has been used. Now, RQD is used only in the settlement calculations).

To allow backwards compatibility, if a user has an old input file (an input file that was created on FB-Deep version 1.18 or previous), the user can still use the same formulas and methodology that was available when these older input files were created. For example, if the user has an old input file, with a 40" diameter cylinder pile, the original skin friction and end bearing formulas will be used, instead of the new large diameter cylinder pile formulas.

To force an old input file to adopt the newest formulas and methodology, simply open the input file in the current version of FB-Deep. Then resave the file. You will be prompted with a message box asking if you want to adopt the new formulas and methods. If you click "Yes", then the input file will be saved as a NEW file, and the latest formulas and methods will be used. If you click "No", then the input file will be saved and an OLD input file, so all of the original methods will be used. If you do chose to save the file, you might want to resave the file under a different file name, so that you still have a copy of the old input file, which would still have access to the older methods, for example using RQD modification for skin friction calculations.

4 User Walkthrough

4.1 Walkthrough Introduction

User’s Guide for FB-Deep (Formerly SHAFTSPT)
F.C. Townsend

The FB-Deep computer program is a Windows based program used to estimate the static axial capacity of drilled shafts and driven piles.

The drilled shaft 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, qu 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).

Driven pile methodology utilizes two types of analyses: SPT and CPT. SPT methodology is based on empirical correlations between cone penetrometer tests and standard penetration tests for typical Florida soil types. Unit end bearing resistance and unit skin friction resistance versus SPT N values are given in the FDOT research bulletin RB-121, for the different soil types.

Driven pile capacity calculated using CPT data can be determined by three separate methods. The first method is the Schmertmann method proposed by Schmertmann in 1978 (AASHTO LRFD Bridge Design Manual). The second method is the LCPC method proposed by Bustamante and Gianeselli for the French Highway Department in 1982. The third method is the UF method proposed by Bloomquist, McVay and Hu for the Florida Department of Transportation in 2007.

FB-Deep replaces earlier versions of ShaftSPT97. ShaftSPT97 replaced SHAFTUF and SHAFT93 and SPT97.

4.2 Driven Piles

4.2.1 SPT Method

Prestressed Square Concrete Pile

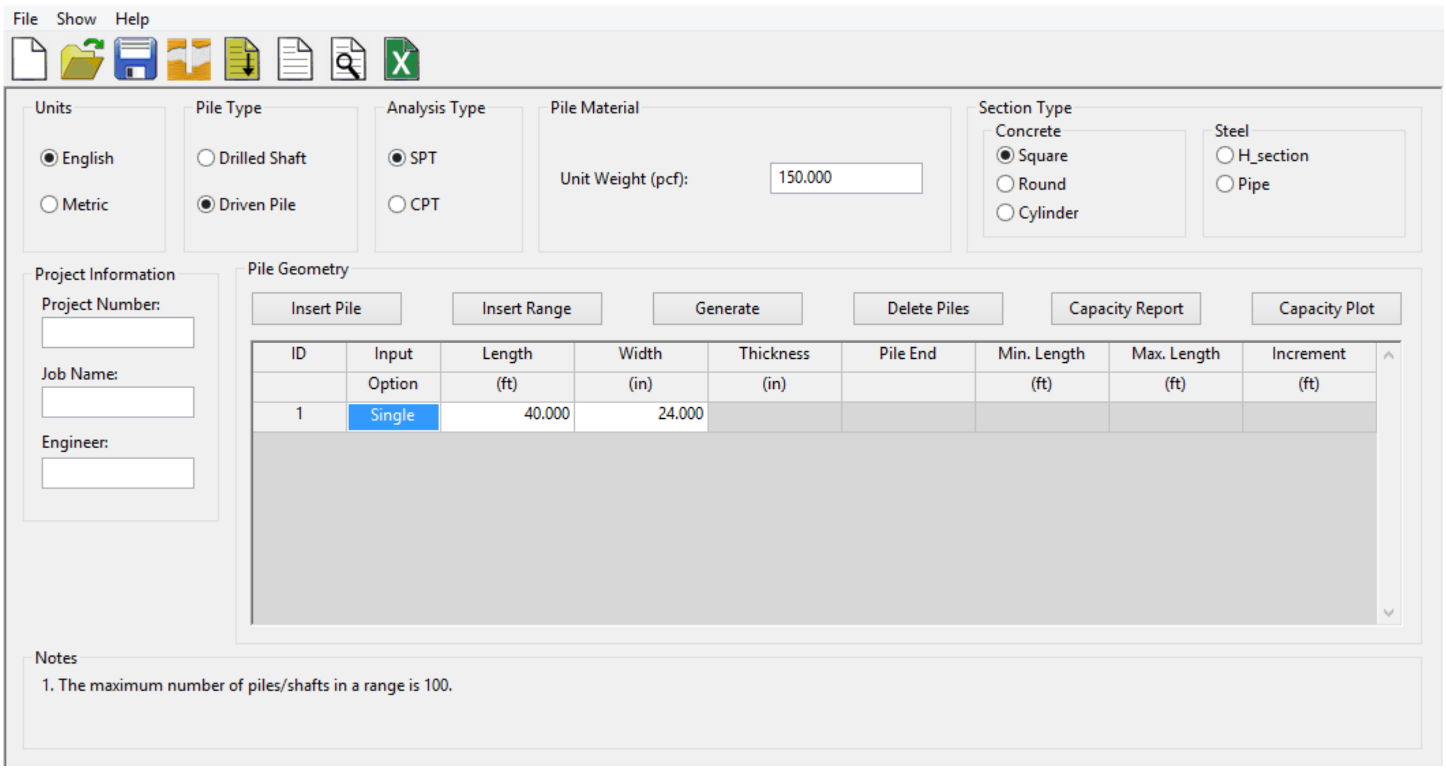


Figure: 4.2.a Staring Screen

1- In the Opening Screen, select:

Units= English Type= Driven Pile Analysis Type= SPT Section Type= Square.

2- In "Pile Material", introduce the unit weight in pcf of the pile material

Unit Weight= 150 pcf

3- In "Pile Geometry", click "Insert Pile" and introduce its parameters:

Input= Single Length= 40ft Width= 24in

4- Select form the “Show” menu, Boring log or click the Boring Log icon in the Toolbar to open the Boring Log window.

Figure: 4.2.b Boring Log

5- In the Boring Log Screen, select:

Ground Surface= 0.0ft

6- Click “Insert Layer” as many times as layers needed and introduce soil parameters:

Soil Type= 3 (Sand) N.Blows= 15 Blows/ft

Note: FB-Deep differs from the program SPT97 in that the last entry in FB-Deep does NOT equal 0

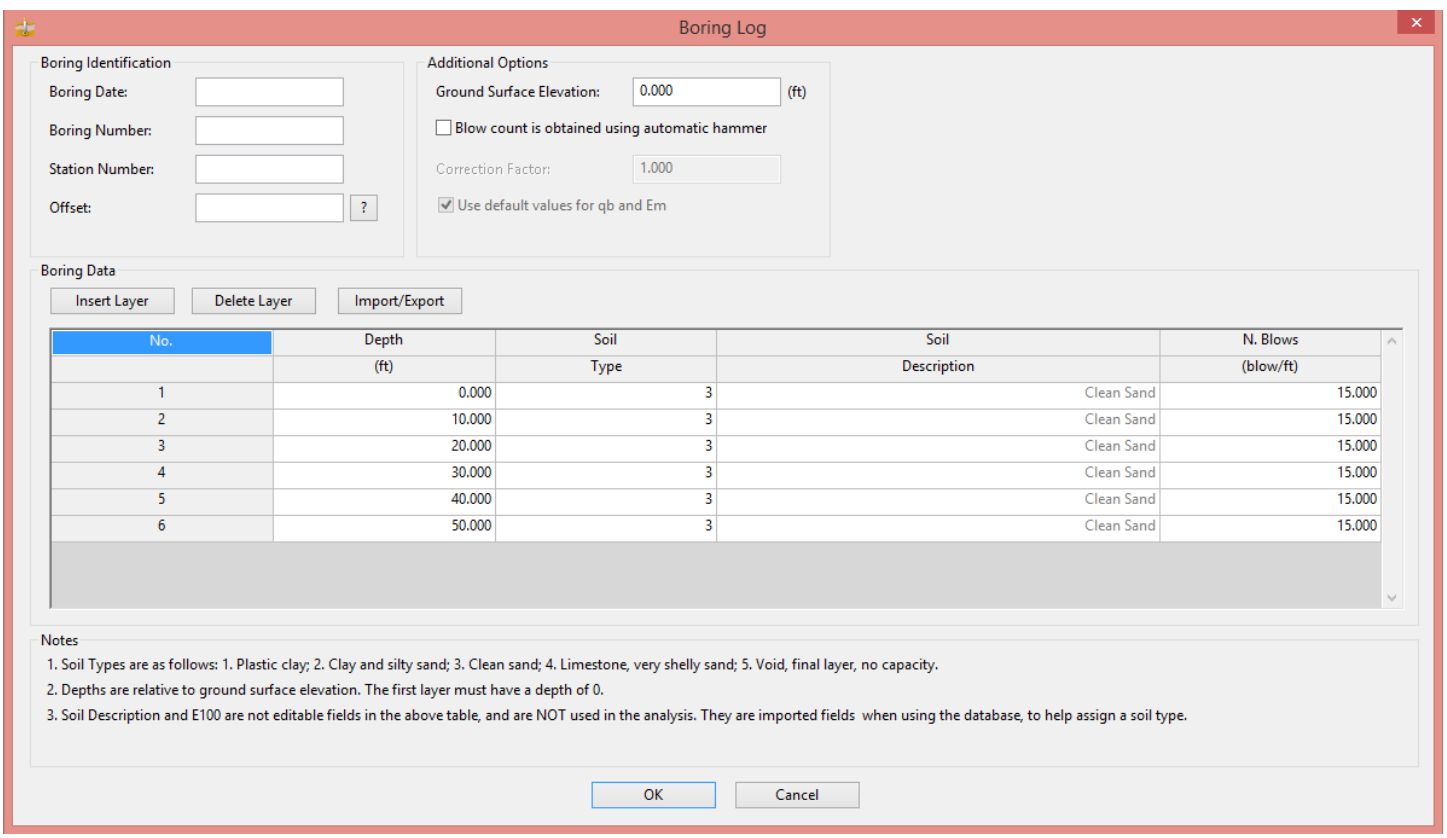


Figure: 4.2.c Boring Log Screen

7- Click “Ok” to go back to the Main screen.

8- In the Main Screen click “Capacity Report” or in the “Show” menu and select “Shaft Capacity Report (Brief Report)” or if preferable “Shaft Capacity Report (Detailed Report)”.

Test Pile Length (ft)	Pile Width (in)	Ultimate Side Friction (tons)	Mobilized End Bearing (tons)	Estimated Davisson Capacity (tons)	Allowable Pile Capacity (tons)	Ultimate Pile Capacity (tons)
40.00	24.0	91.20	64.00	155.20	77.60	283.20

NOTES

----

1. MOBILIZED END BEARING IS 1/3 OF THE ORIGINAL RB-121 VALUES.

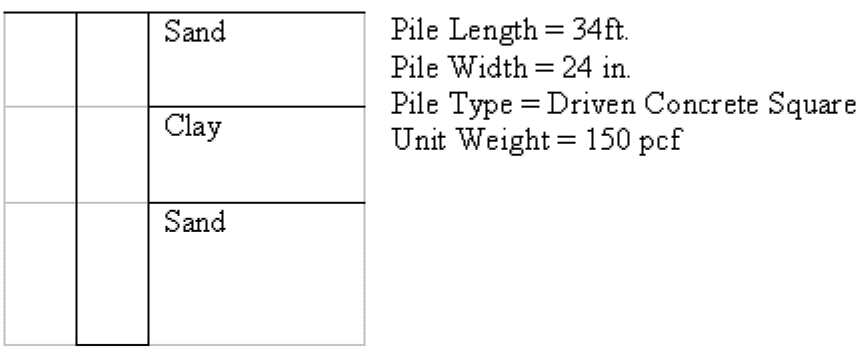
2. DAVISSON PILE CAPACITY IS AN ESTIMATE BASED ON FAILURE CRITERIA, AND EQUALS ULTIMATE SIDE FRICTION PLUS MOBILIZED END BEARING.

3. ALLOWABLE PILE CAPACITY IS 1/2 THE DAVISSON PILE CAPACITY.

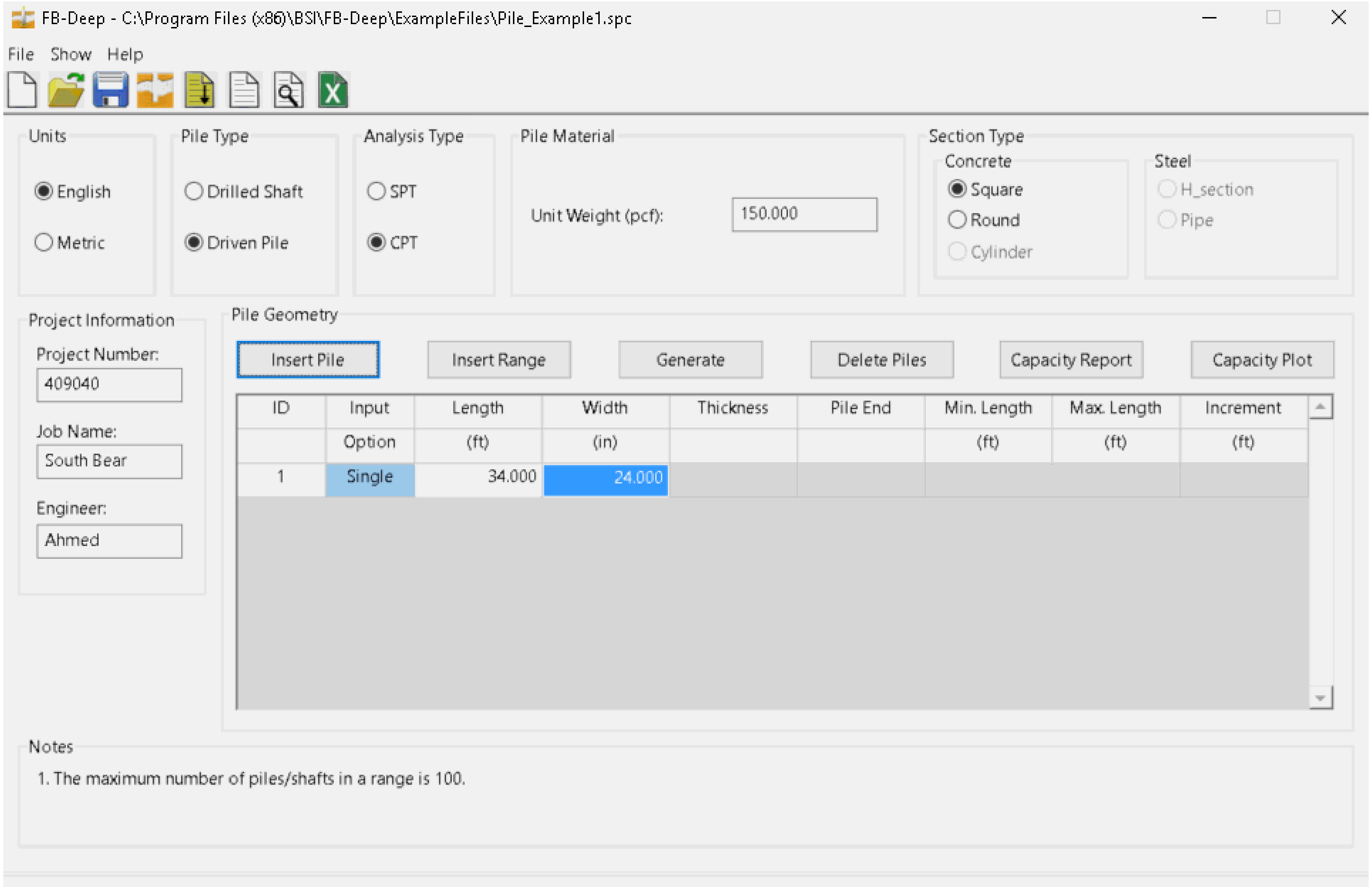
4. ULTIMATE PILE CAPACITY IS ULTIMATE SIDE FRICTION PLUS 3 x THE MOBILIZED END BEARING.
   EXCEPTION: FOR H-PILES TIPPED IN SAND OR LIMESTONE, THE ULTIMATE PILE CAPACITY IS ULTIMATE SIDE FRICTION PLUS 2 x THE MOBILIZED END BEARING.

4.2.2 CPT Method

4.2.2.1 Example 1: CPT Walkthrough Problem



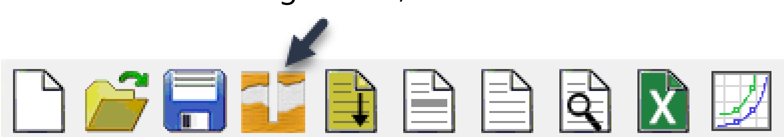
Opening Screen



Units = English, Pile Type = Driven

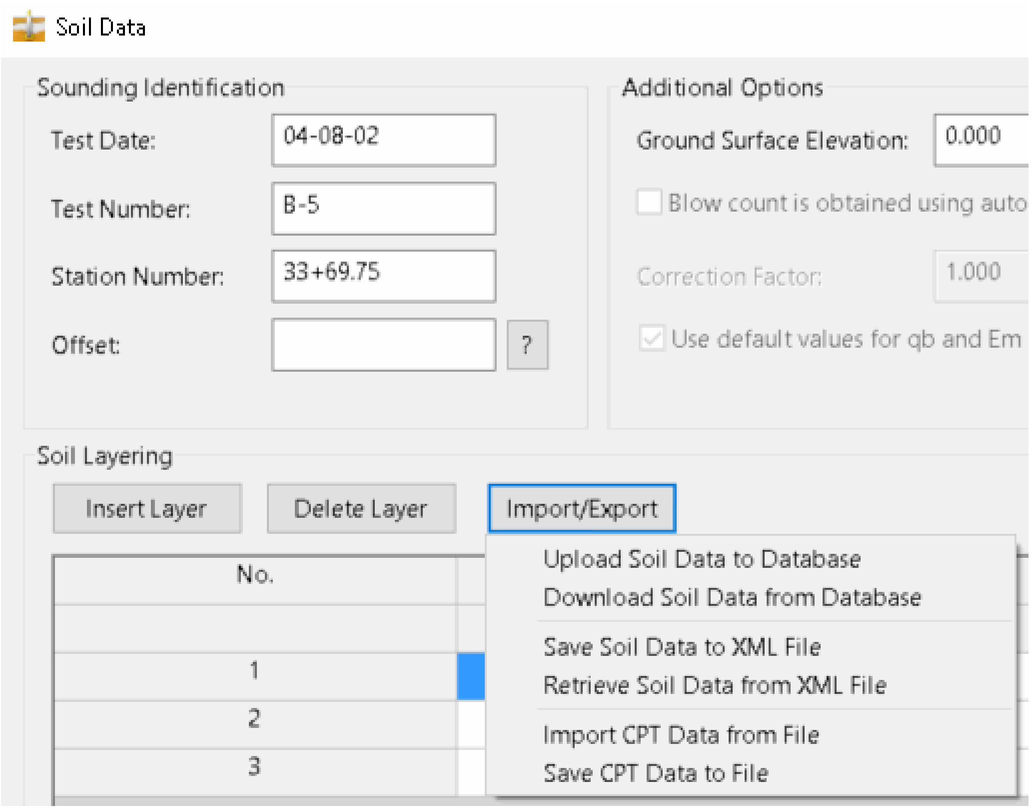
Unit Weight = 150, Section Type = Concrete Square

Click-Insert Pile: Length = 34, Width = 24



Click Soil (boring log) icon in Toolbar.

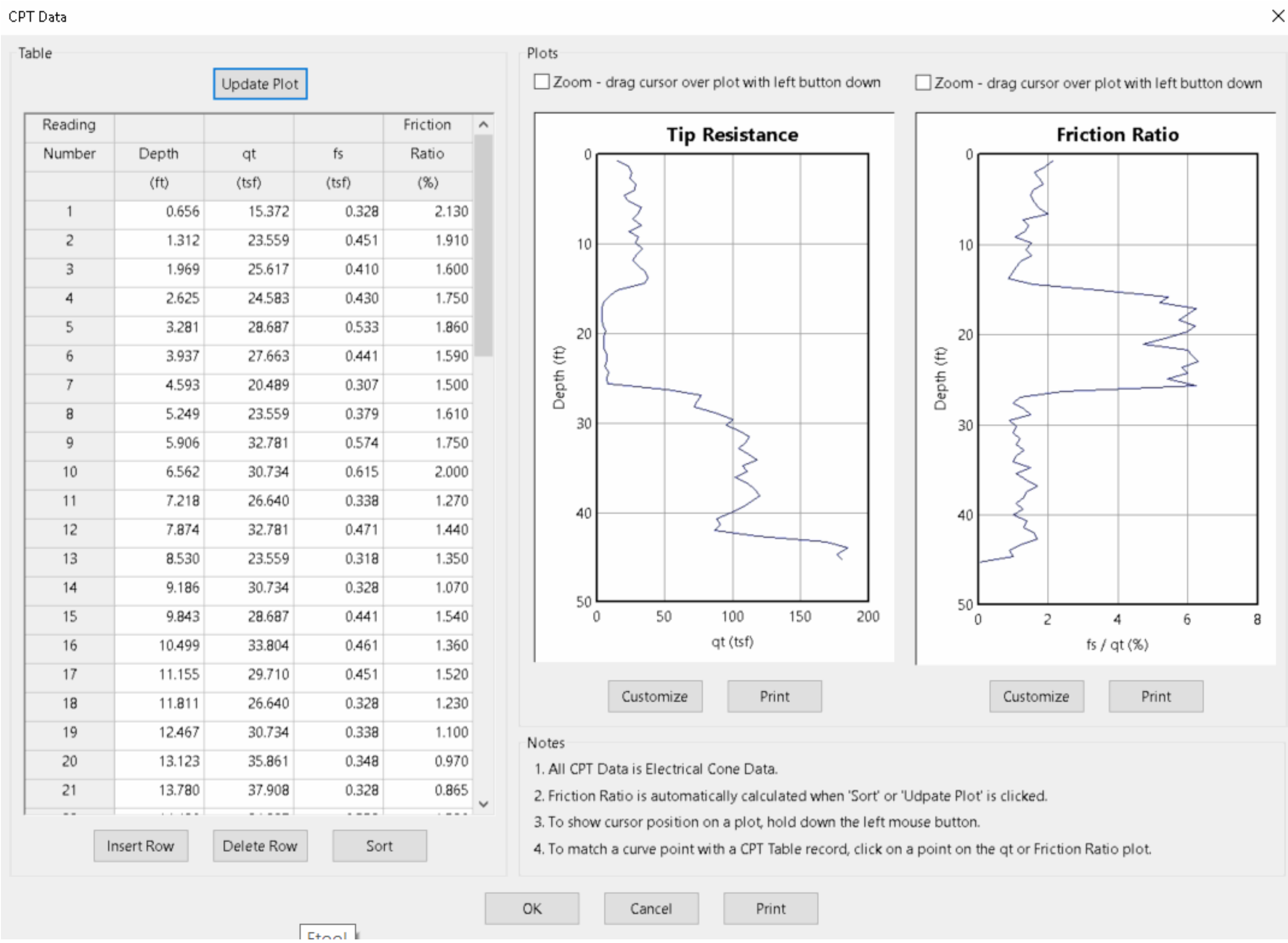
Soil Data Screen



Analysis Type = CPT-Schmertmann

Click "Import/Export" button, then select "Import CPT Data from File". CPT data can be imported as a .txt or .xls file. See "CPT Modeling" for correct units and file format. Then click the "CPT Data button" to launch the CPT Data Screen.

CPT Data Screen



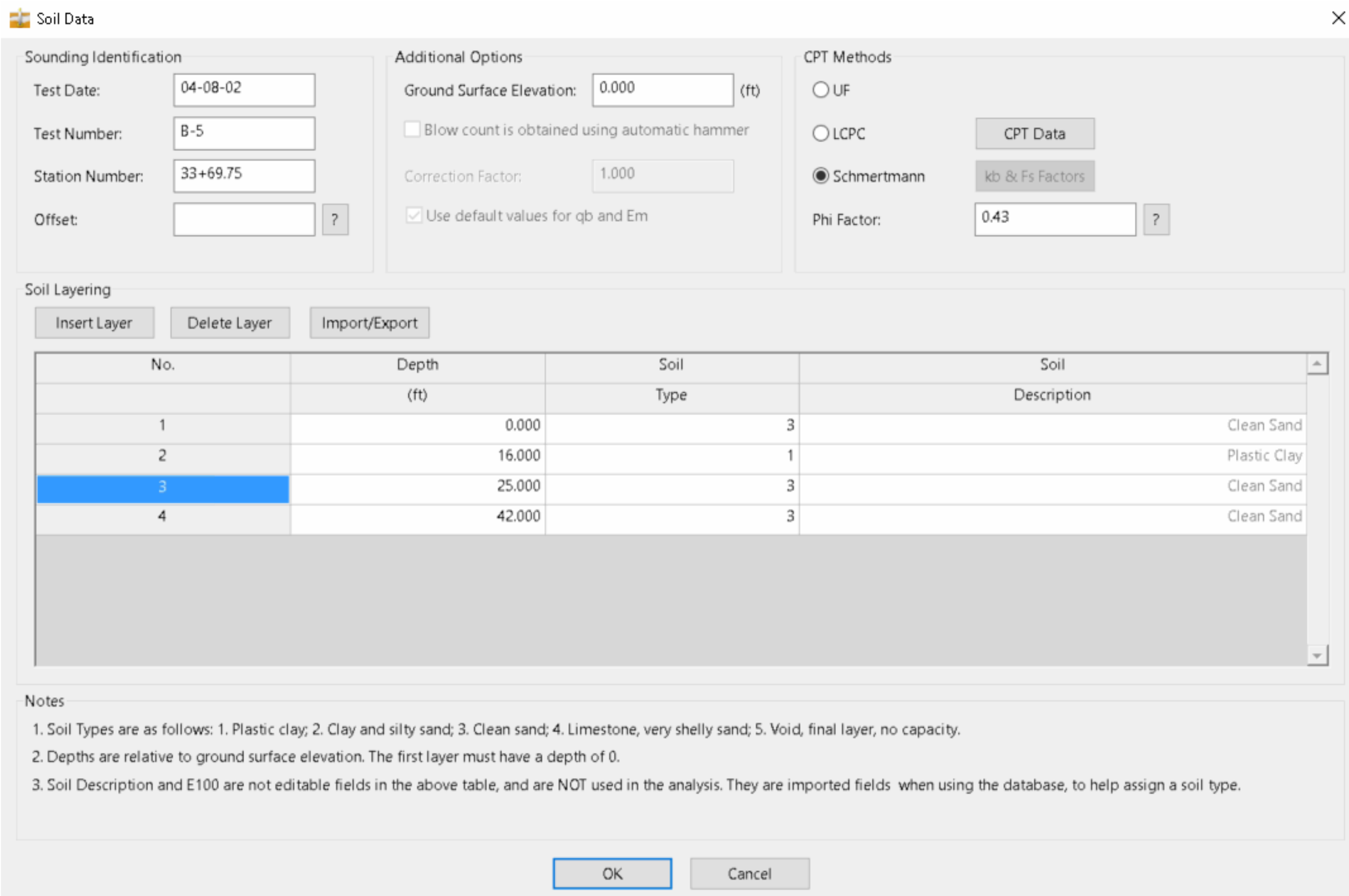
Reading Number	Depth (ft)	qt (tsf)	fs (tsf)	Friction Ratio (%)
1	0.656	15.372	0.328	2.130
2	1.312	23.559	0.451	1.910
3	1.969	25.617	0.410	1.600
4	2.625	24.583	0.430	1.750
5	3.281	28.687	0.533	1.860
6	3.937	27.663	0.441	1.590
7	4.593	20.489	0.307	1.500
8	5.249	23.559	0.379	1.610
9	5.906	32.781	0.574	1.750
10	6.562	30.734	0.615	2.000
11	7.218	26.640	0.338	1.270
12	7.874	32.781	0.471	1.440
13	8.530	23.559	0.318	1.350
14	9.186	30.734	0.328	1.070
15	9.843	28.687	0.441	1.540
16	10.499	33.804	0.461	1.360
17	11.155	29.710	0.451	1.520
18	11.811	26.640	0.328	1.230
19	12.467	30.734	0.338	1.100
20	13.123	35.861	0.348	0.970
21	13.780	37.908	0.328	0.865

Find layer changes by moving the cursor to the observed layer change on plot and hold down the left mouse button.

Comparing the cone bearing capacity graph to the friction ratio graph the user can see different soil layers as well as determine soil types. Cohesive soils have low cone bearing capacity values and high friction ratio values. This can be seen for the second layer with depths between depths of 16 and 25 ft. Cohesionless soils have high cone bearing capacity values and low friction ration values. This can be seen for first and third layers.

Click-OK when done

Soil Data Screen



Click – Insert Layer

Insert layer with depths found from CPT data Screen.

Note: Schmertmann method uses 2 soil descriptions, cohesive and cohesionless. For soil layer analysis the user should use soil types 1 and 3. For more information on the Schmertmann method see the FB-Deep Help Manual.

FB-Deep => Driven Piles => Method of Analysis => CPT =>Methodology => Schmertmann.

Click – OK when done

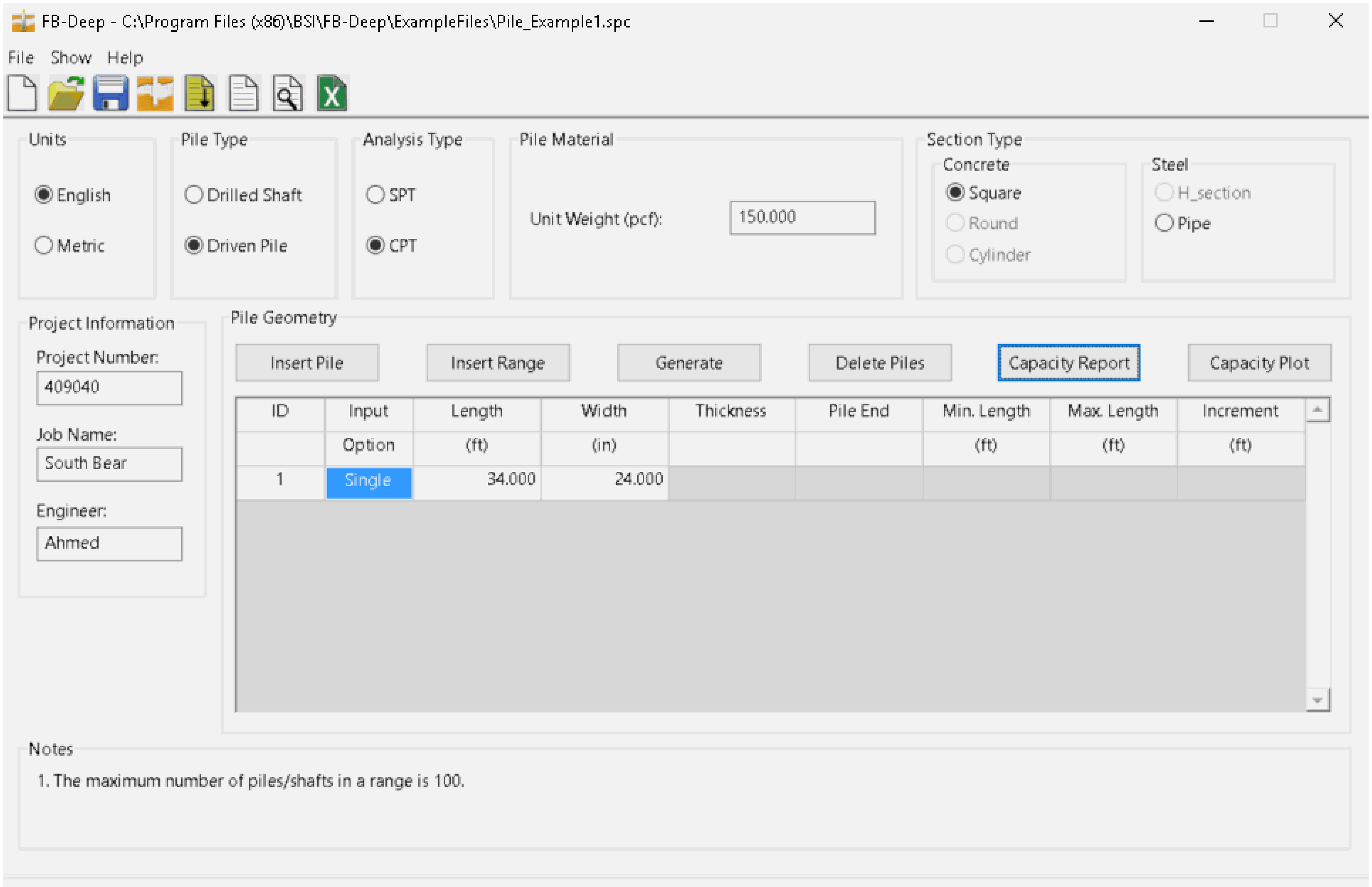
Methods of Determining Soil Type

1. Cohesive soils have low cone bearing capacity values and high friction ratio values. Cohesionless soils have high cone bearing capacity values and low friction ration values.

2. Soil types can be determined by using the UBC-1983 Soil Behavior Type.

3. Compare a corresponding SPT boring log.

Main Screen



Click – Cap. Report to view the Output

Driven Pile Capacity:

Test Pile Length (ft)	Pile Width (in)	Nominal Skin Friction Resistance (tons)	Nominal Tip Resistance (tons)	Nominal Resistance (tons)	Design Resistance (tons)	Phi Factor
34.0	24.0	112.29	278.61	390.90	168.09	0.430

4.2.2.1 Example 2: Schmertmann Method

1- In the Opening Screen, select:

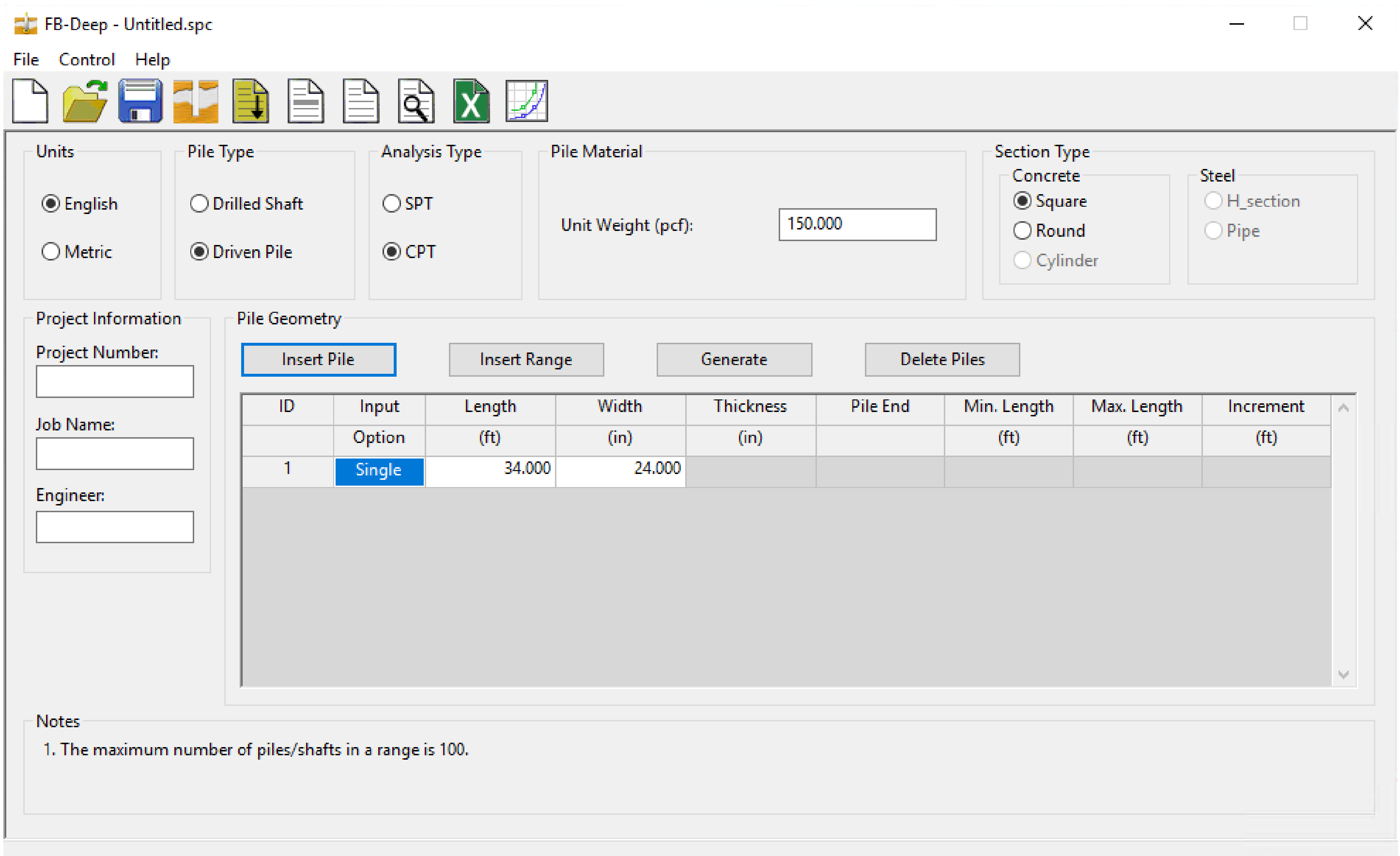
Units= English; Type= Driven Pile; Analysis Type= CPT; Section Type= Square

2- In “Pile Material”, introduce the unit weight in pcf of the pile material

Unit Weight=150 pcf

3- In “Pile Geometry”, click “Insert Pile” and introduce its parameters:

Input= Single ; Length= 34ft; Width= 24in



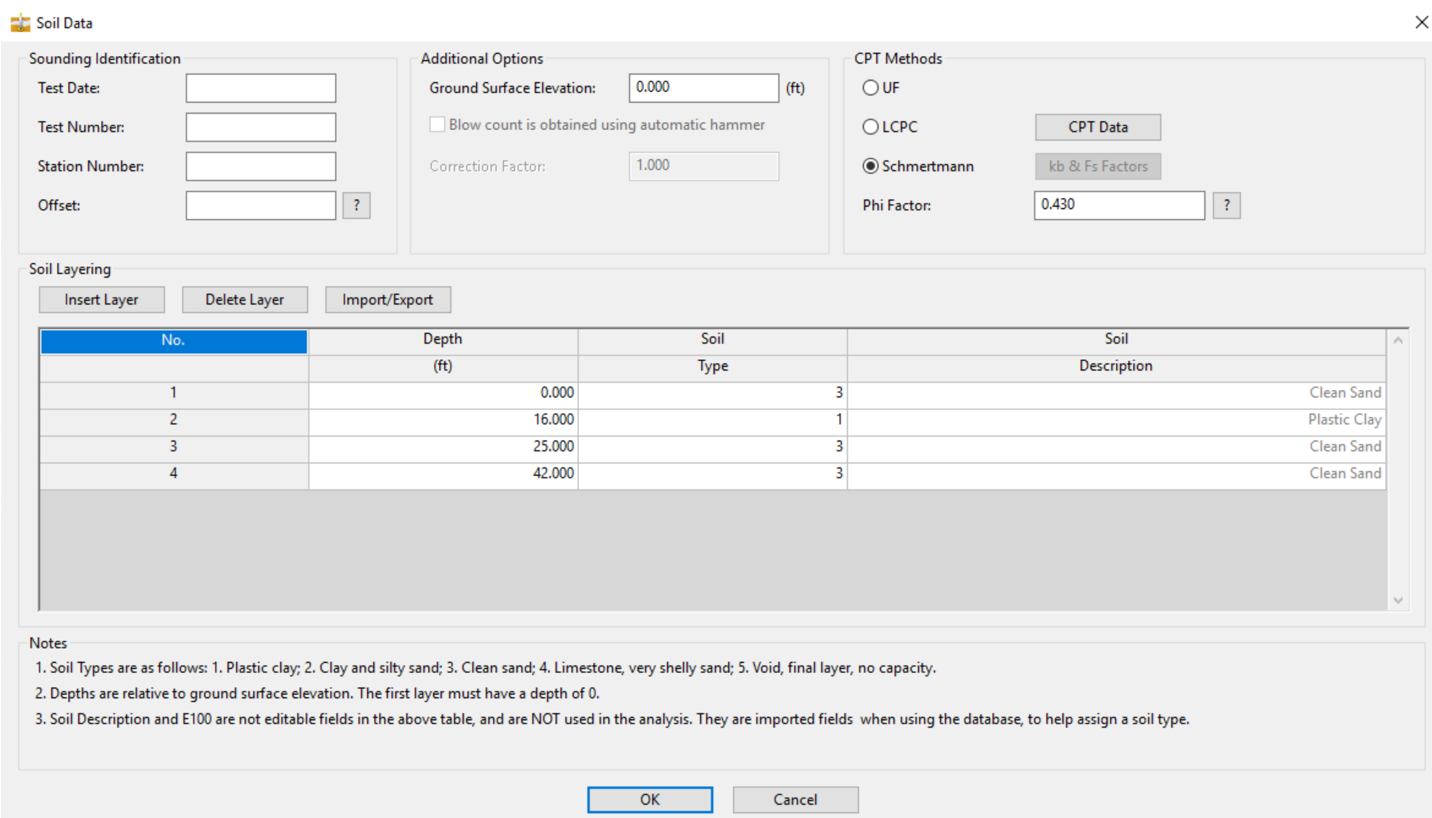
4- Select form the “Show” menu, Soil Data or click the Boring Log icon in the Toolbar to open the Boring Log window.

5- In the Soil Data Screen, select:

Ground Surface= 0.0ft; CPT Methods= Schmertmann; Phi Factor(default)= 0.430

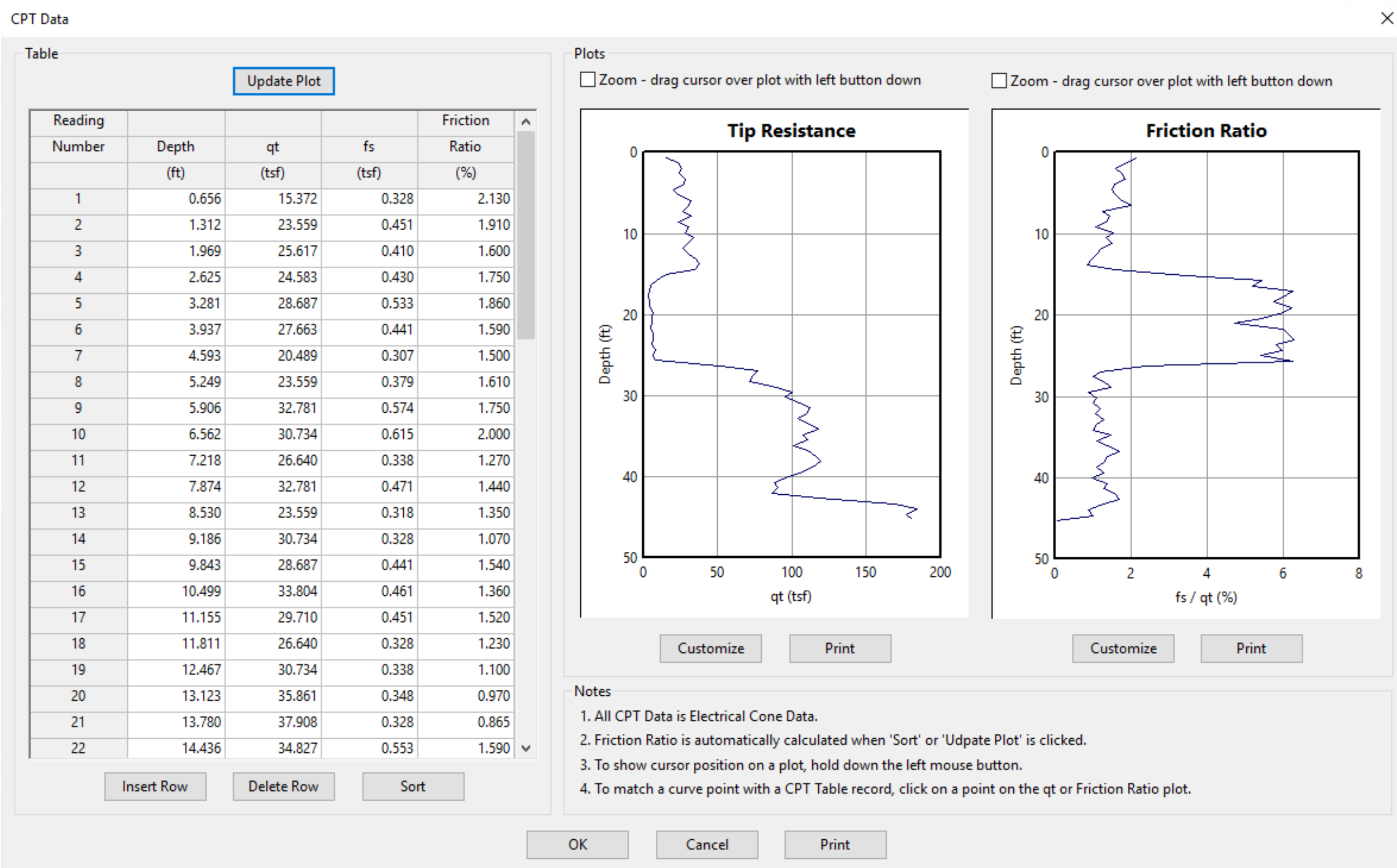
6- Click “Insert Layer” as many times as layers needed and introduce soil parameters:

Soil Type=3 (Sand), from (0.0ft-16.0ft) (25.0ft-42.0ft) Soil Type=1 (Clay), from (16.0ft-25.0ft)



Note: Schmertmann method uses 2 soil descriptions, cohesive and cohesionless. For soil layer analysis the user should use soil types 1 and 3. For more information on the Schmertmann method see "Schmertmann" .

7- Click “Import/Export” button, then select "Import CPT Data from File" to upload the results obtained for the CPT testing. CPT data can be imported as a .txt or .xls file. See "CPT Modeling" for correct units and file format. Then click the "CPT Data" button to launch the CPT Data Screen.



Reading Number	Depth (ft)	qt (tsf)	fs (tsf)	Friction Ratio (%)
1	0.656	15.372	0.328	2.130
2	1.312	23.559	0.451	1.910
3	1.969	25.617	0.410	1.600
4	2.625	24.583	0.430	1.750
5	3.281	28.687	0.533	1.860
6	3.937	27.663	0.441	1.590
7	4.593	20.489	0.307	1.500
8	5.249	23.559	0.379	1.610
9	5.906	32.781	0.574	1.750
10	6.562	30.734	0.615	2.000
11	7.218	26.640	0.338	1.270
12	7.874	32.781	0.471	1.440
13	8.530	23.559	0.318	1.350
14	9.186	30.734	0.328	1.070
15	9.843	28.687	0.441	1.540
16	10.499	33.804	0.461	1.360
17	11.155	29.710	0.451	1.520
18	11.811	26.640	0.328	1.230
19	12.467	30.734	0.338	1.100
20	13.123	35.861	0.348	0.970
21	13.780	37.908	0.328	0.865
22	14.436	34.827	0.553	1.590

8- Once the file is upload, it can be seen by selecting “CPT Data” located in the Soil Data screen.

Note: Find layer changes by moving the cursor to the observed layer change on plot and hold down the left mouse button.

Comparing the cone bearing capacity graph to the friction ratio graph the user can see different soil layers as well as determine soil types. Cohesive soils have low cone bearing capacity values and high friction ratio values. This can be seen for the second layer with depths between depths of 16 and 25 ft. Cohesionless soils have high cone bearing capacity values and low friction ration values. This can be seen for first and third layers.

9- Click “Ok” twice to go back to the Main screen.

10- In the Main Screen click “Show” menu and select “Shaft Capacity Report (Brief Report)”.

Section Type: Square
Pile Width: 24.00 (in)

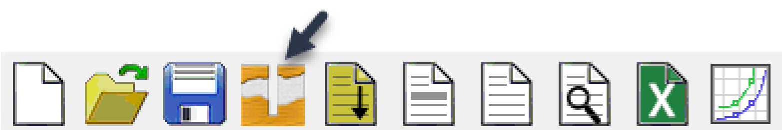
Test Pile Length (ft)	Pile Width (in)	Nominal Skin Friction Resistance (tons)	Nominal Tip Resistance (tons)	Nominal Resistance (tons)	Design Resistance (tons)	Phi Factor
34.00	24.0	112.29	278.61	390.90	168.09	0.430

4.2.2.1 Example 3: UF Method

Now that you are able to navigate the FB-Deep for CPT analysis, you are now prepared to run an analysis for a complex set of data. The following data is from the SR 417 extension in Orlando, Florida.

First you should begin with interpreting your CPT data.

Opening Screen

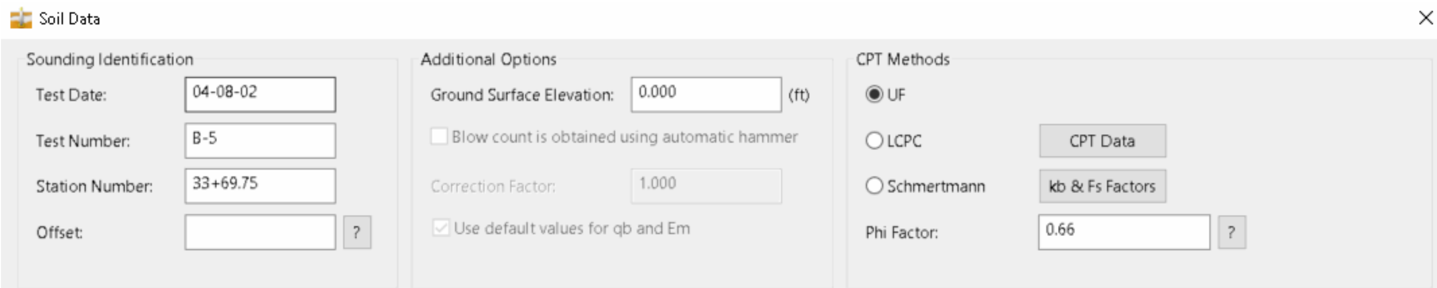


Units = English

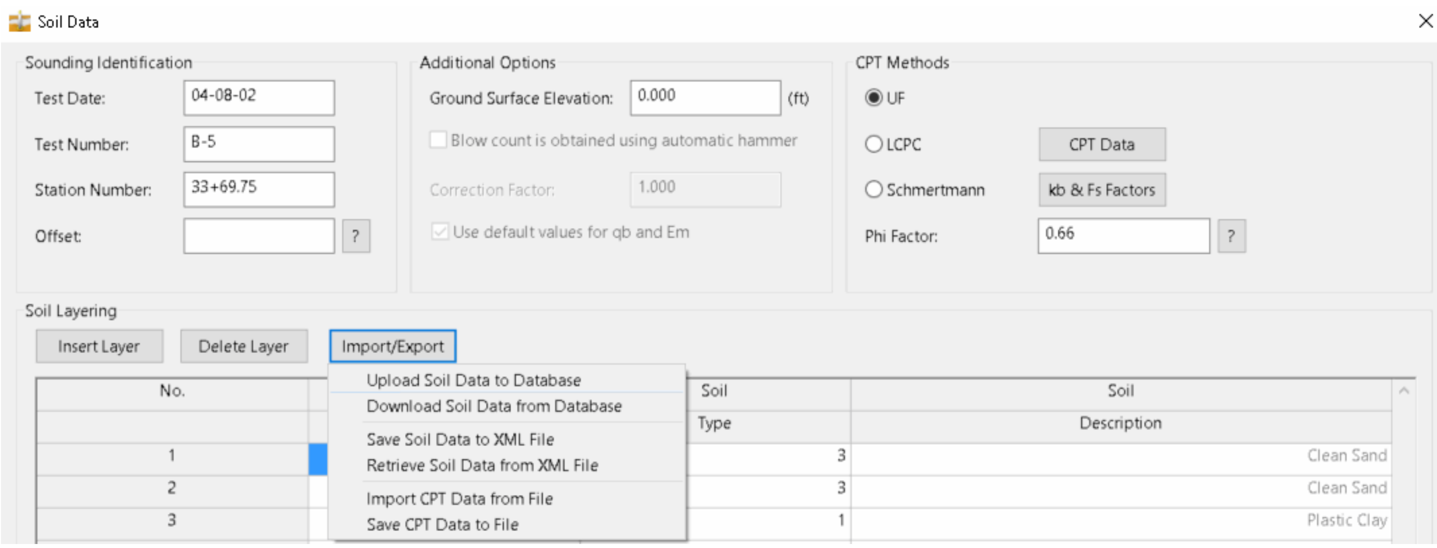
Pile Type = Driven Pile

Click boring log icon in Toolbar.

Soil Data Screen

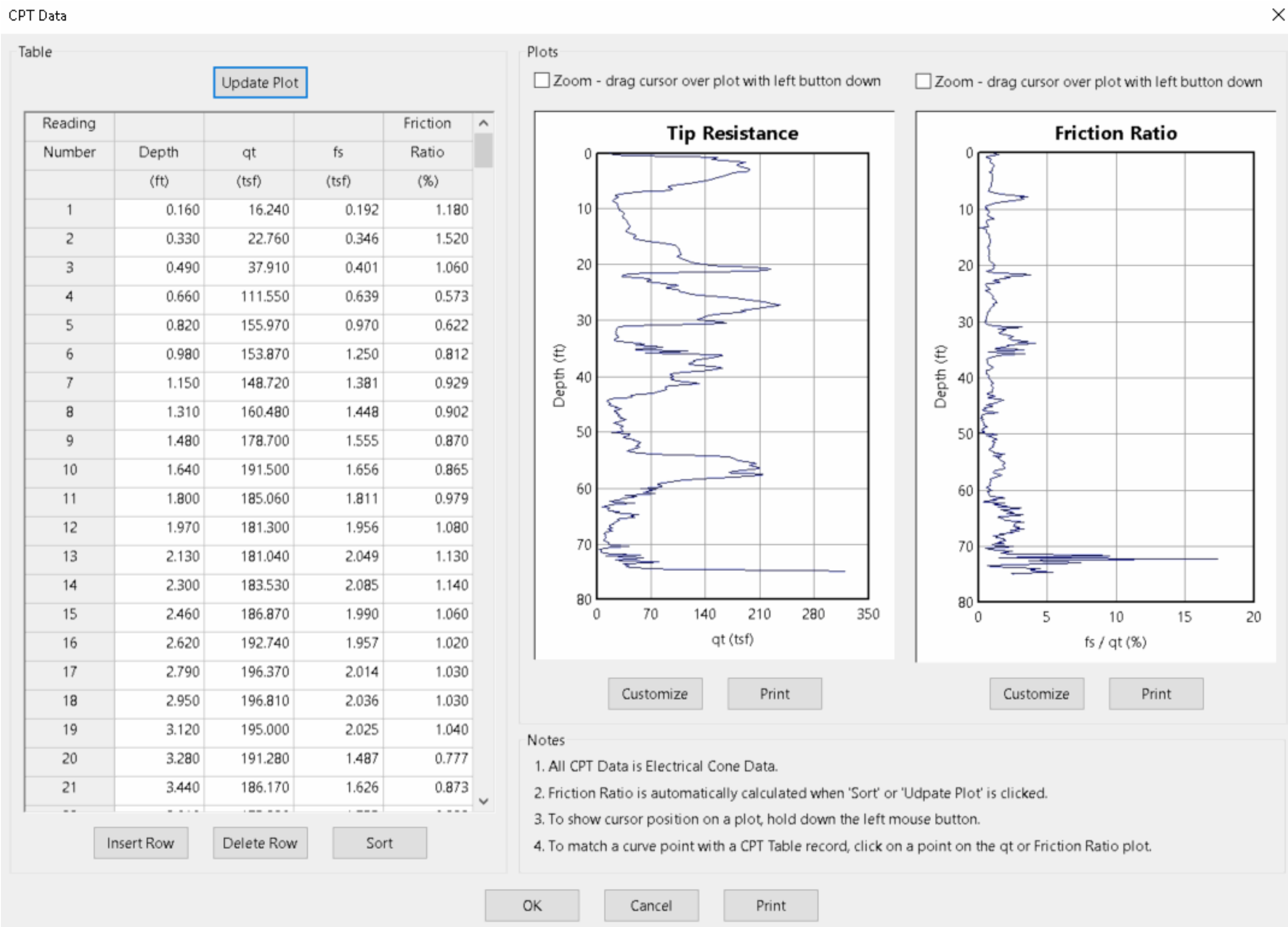


Analysis Type = CPT-UF



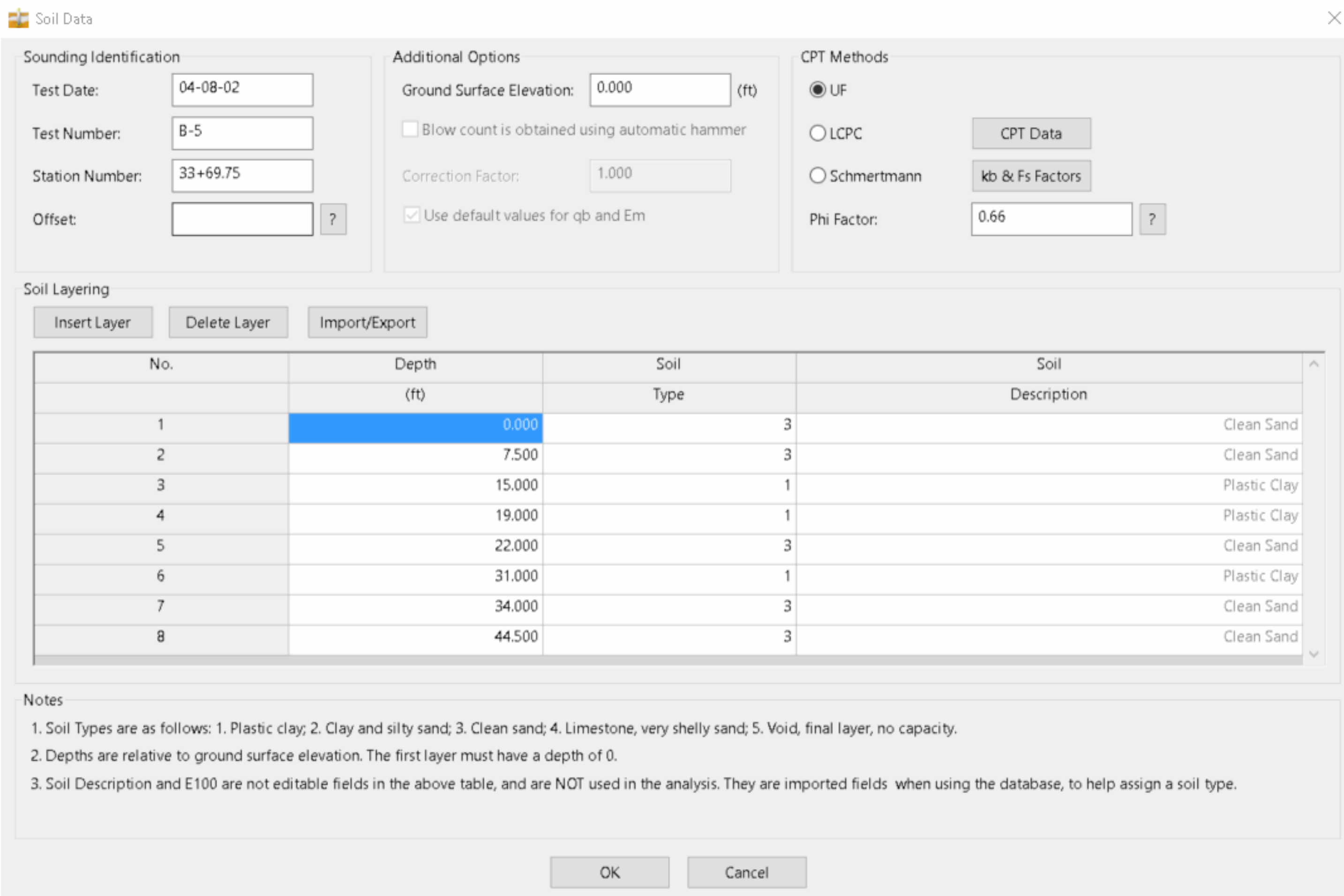
Click “Import/Export” button, then select "Import CPT Data from File" to upload the results obtained for the CPT testing. CPT data can be imported as a .txt or .xls file. See "CPT Modeling" for correct units and file format. Then click the "CPT Data" button to launch the CPT Data Screen.

CPT Data Screen



Find layer changes by moving the cursor to the observed layer change on plot and hold down the left mouse button.

Click-OK when done



Click – Insert Layer

Insert layer with depths found from CPT data Screen.

Insert Soil Types.

Note: The UF method uses all 5 soil type descriptions. For more information on the UF method see the FB-Deep Help Manual.

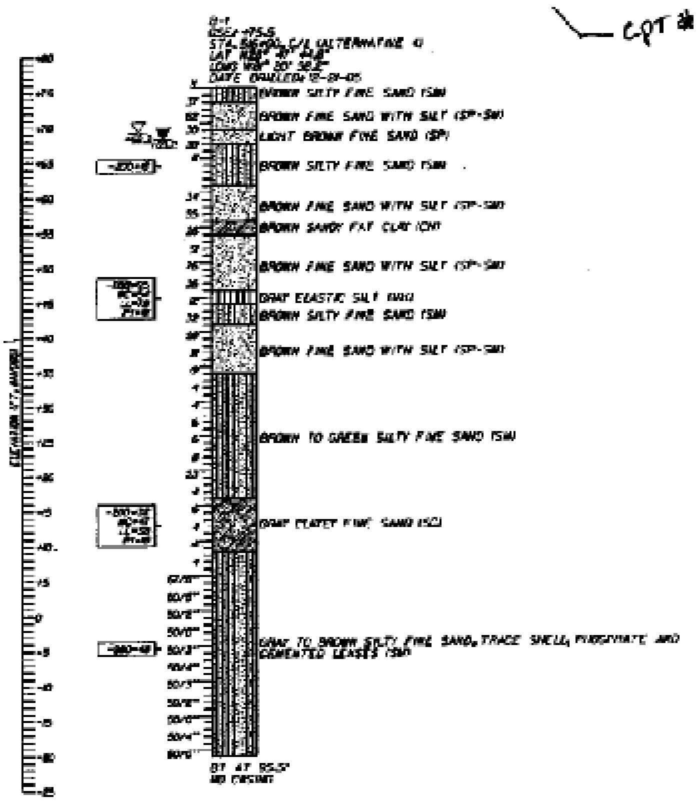
FB-Deep => Driven Piles => Method of Analysis => CPT =>Methodology => UF.

Methods of Determining Soil Type

1. Cohesive soils have low cone bearing capacity values (qc) and high friction ratio values. Cohesionless soils have high cone bearing capacity values and low friction ration values.
2. Soil types can be determined by using the UBC-1983 Soil Behavior Type.
3. Compare a corresponding SPT boring log.

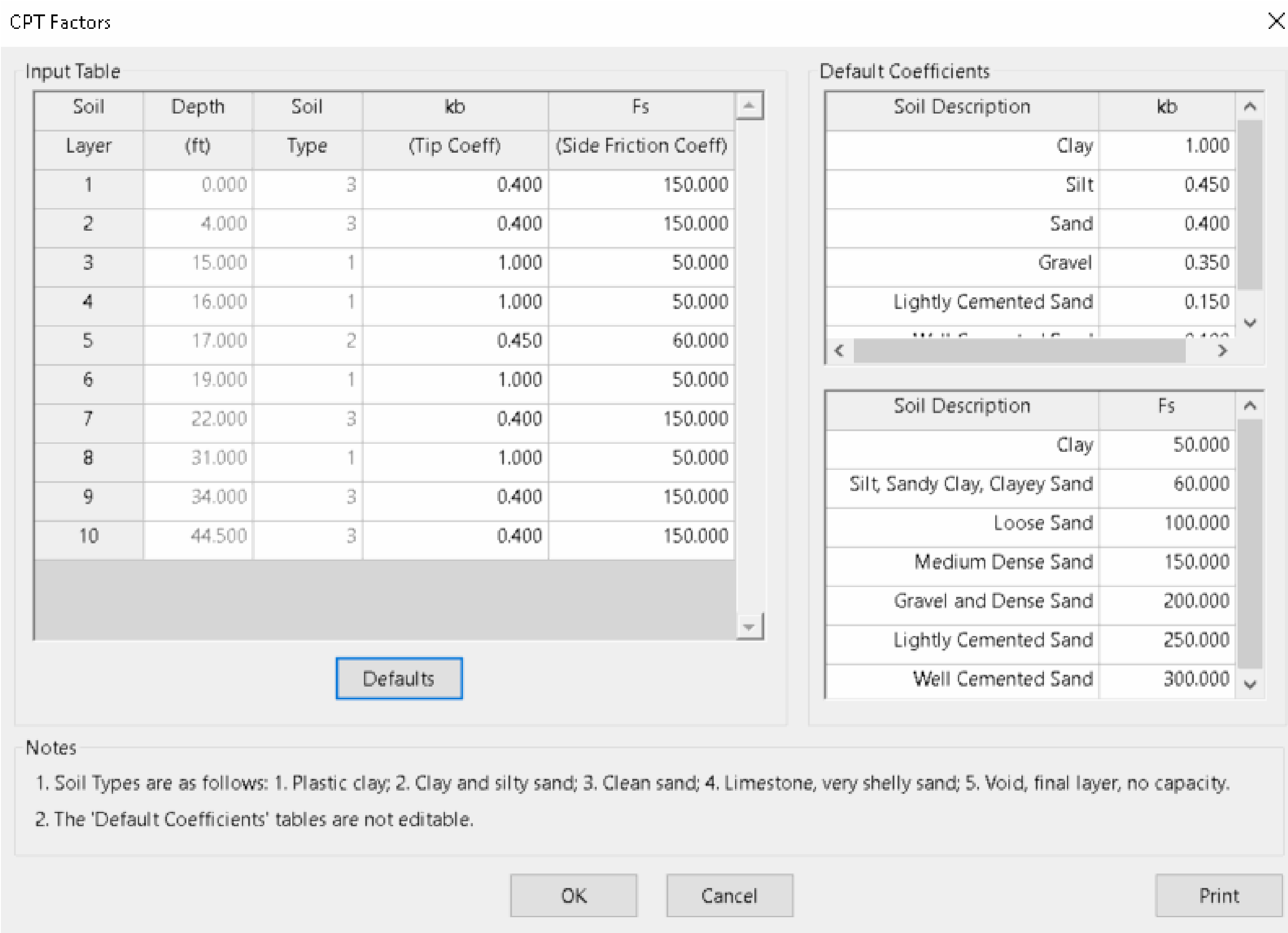
See SPT boring below for help with soil layering description.

Click – OK when done



Corresponding SPT Boring from SR 417 extension.

Default Factors



CPT Factors

Input Table

Soil Layer	Depth (ft)	Soil Type	kb (Tip Coeff)	Fs (Side Friction Coeff)
1	0.000	3	0.400	150.000
2	4.000	3	0.400	150.000
3	15.000	1	1.000	50.000
4	16.000	1	1.000	50.000
5	17.000	2	0.450	60.000
6	19.000	1	1.000	50.000
7	22.000	3	0.400	150.000
8	31.000	1	1.000	50.000
9	34.000	3	0.400	150.000
10	44.500	3	0.400	150.000

Defaults

Default Coefficients

Soil Description	kb
Clay	1.000
Silt	0.450
Sand	0.400
Gravel	0.350
Lightly Cemented Sand	0.150

Soil Description	Fs
Clay	50.000
Silt, Sandy Clay, Clayey Sand	60.000
Loose Sand	100.000
Medium Dense Sand	150.000
Gravel and Dense Sand	200.000
Lightly Cemented Sand	250.000
Well Cemented Sand	300.000

Notes

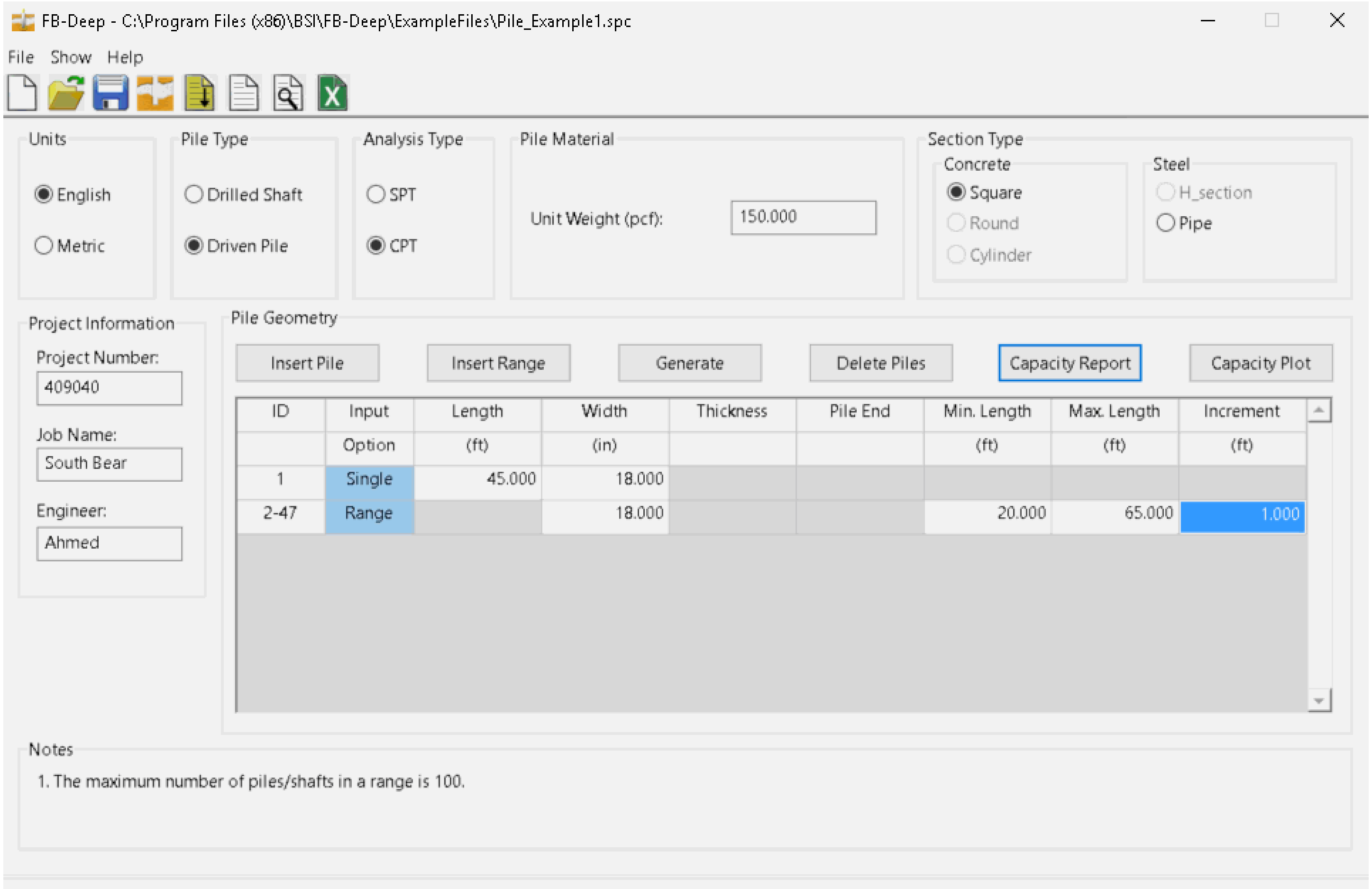
1. Soil Types are as follows: 1. Plastic clay; 2. Clay and silty sand; 3. Clean sand; 4. Limestone, very shelly sand; 5. Void, final layer, no capacity.
2. The 'Default Coefficients' tables are not editable.

OK Cancel Print

For this example since there are no cemented layers seen in the SPT boring, the default factors can be used.

Click – Yes

Main Screen



Pile Type = Driven

Unit Weight = 150pcf

Section Type = Concrete Square

Click-Insert Pile: Length = 45, Width = 18

Click – Cap. Report to view the Output

Driven Pile Capacity:

Test Pile Length (ft)	Pile Width (in)	Nominal Skin Friction Resistance (tons)	Nominal Tip Resistance (tons)	Nominal Resistance (tons)	Design Resistance (tons)	Phi Factor
45.0	18.0	242.03	25.28	267.31	176.42	0.660

4.2.2.1 Example 4: LCPC Method

Now that you are able to navigate the FB-Deep for CPT analysis, you are now prepared to run an analysis for a complex set of data. The following data is from the SR 417 extension in Orlando, Florida.

First you should begin with interpreting your CPT data.

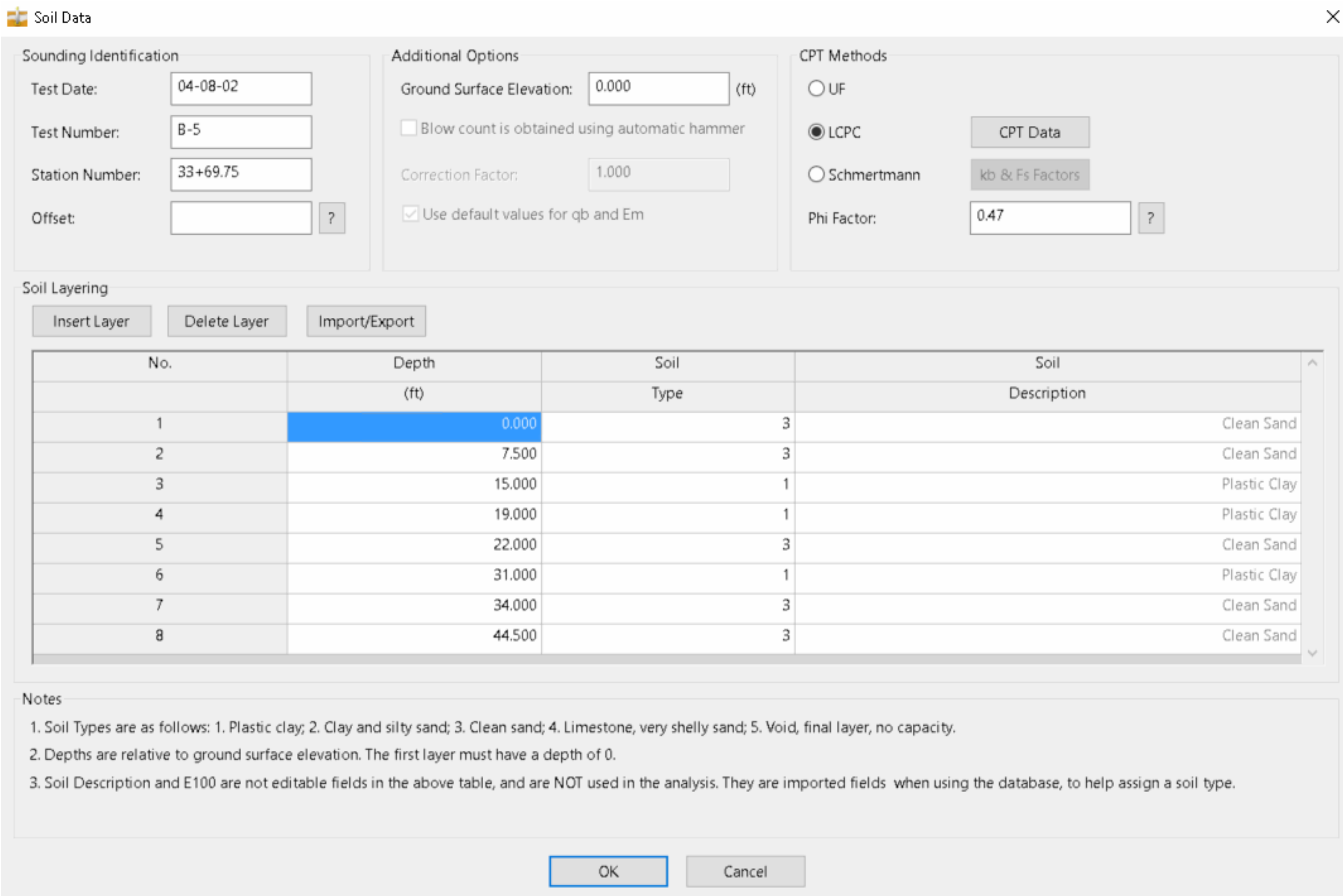
Opening Screen

Units = English

Pile Type = Driven Pile

Click boring log icon in Toolbar.

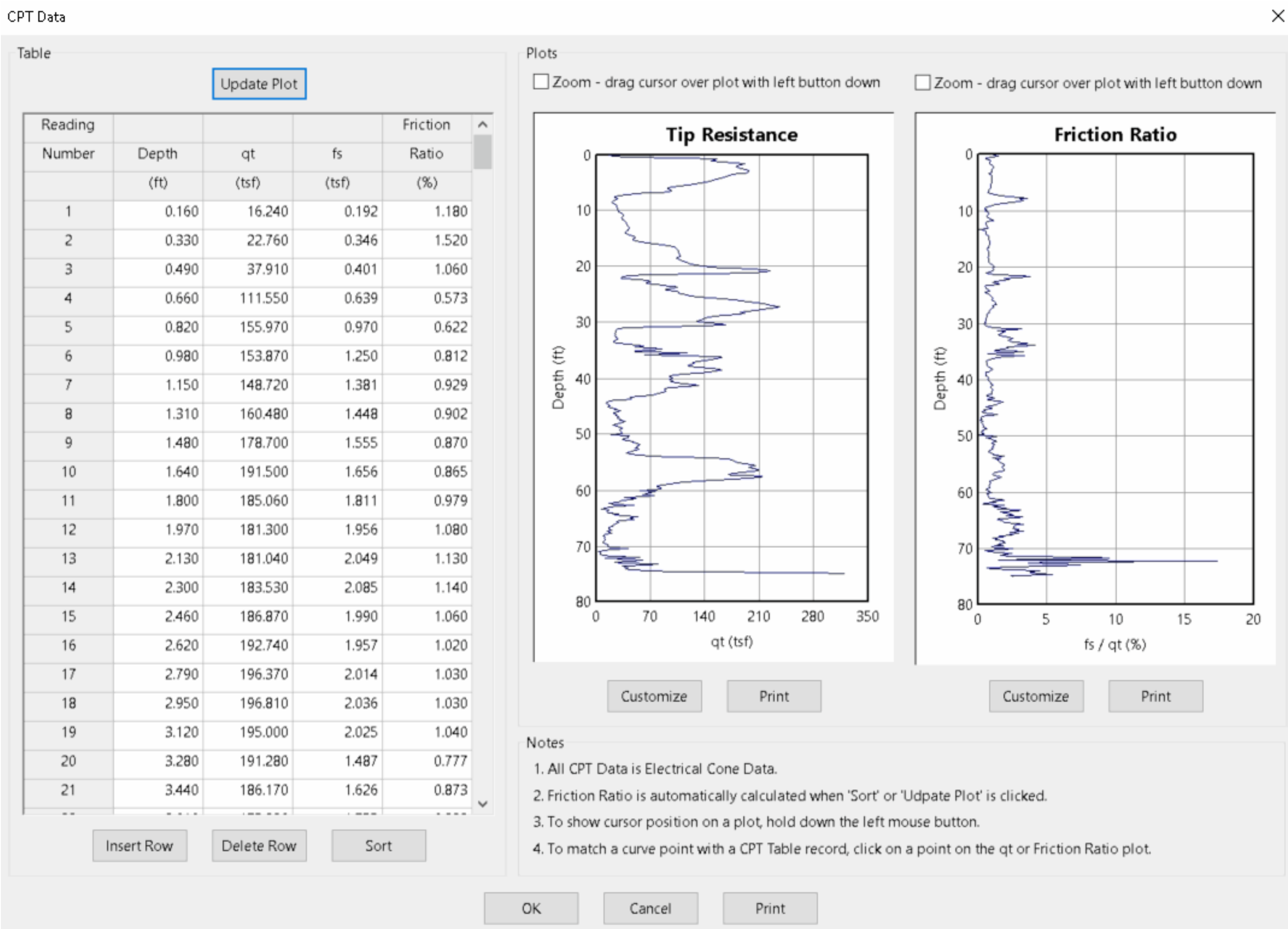
Soil Data Screen



Analysis Type = CPT-LCPC

Click Import/Export Soil Data button, then select Import CPT Data from File. Then select the file "CPT_LCPC_1.txt", which is located in C:\Program Files\BSI\FB-Deep. CPT data can be imported as a .txt or .xls file. See Modeling under Driven Piles -> CPT for correct file format.

Then click the CPT Data button to launch the CPT Data Screen

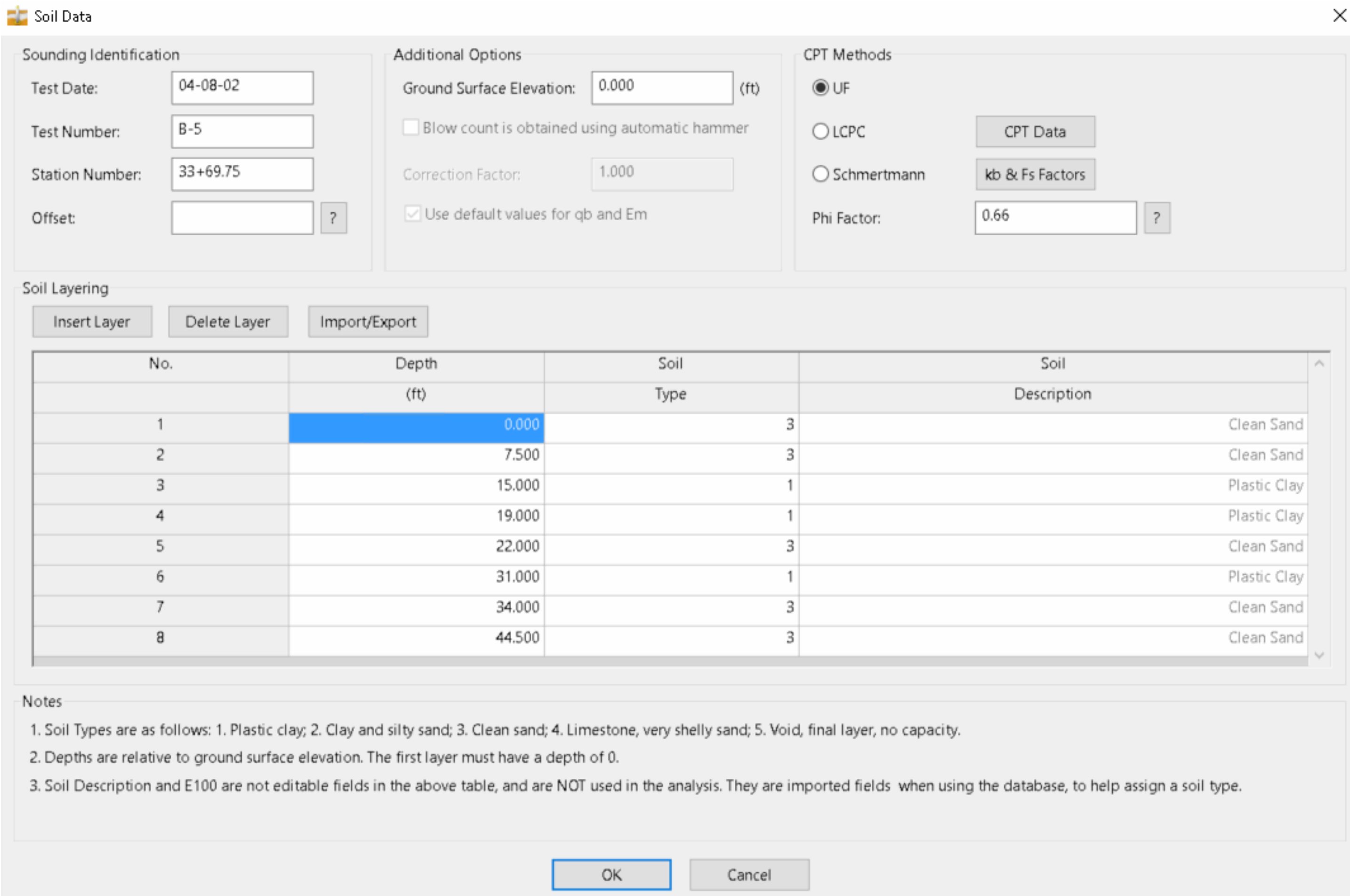


CPT Data Screen

Find layer changes by moving the cursor to the observed layer change on plot and hold down the left mouse button.

Click-OK when done

Soil Data Screen



Click – Insert Layer

Insert layer with depths found from CPT data Screen.

Insert Soil Types.

Note: The LCPC method uses all 5 soil type descriptions. For more information on the LCPC method see the FB-Deep Help Manual.

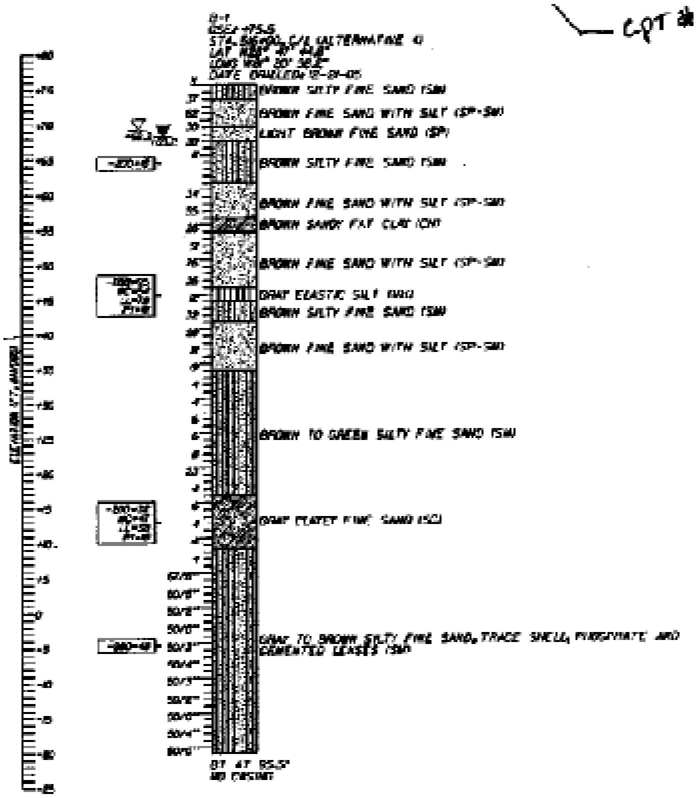
FB-Deep => Driven Piles => Method of Analysis => CPT =>Methodology => LCPC.

Methods of Determining Soil Type

1. Cohesive soils have low cone bearing capacity values (qc) and high friction ratio values. Cohesionless soils have high cone bearing capacity values and low friction ration values.
2. Soil types can be determined by using the UBC-1983 Soil Behavior Type.
3. Compare a corresponding SPT boring log.

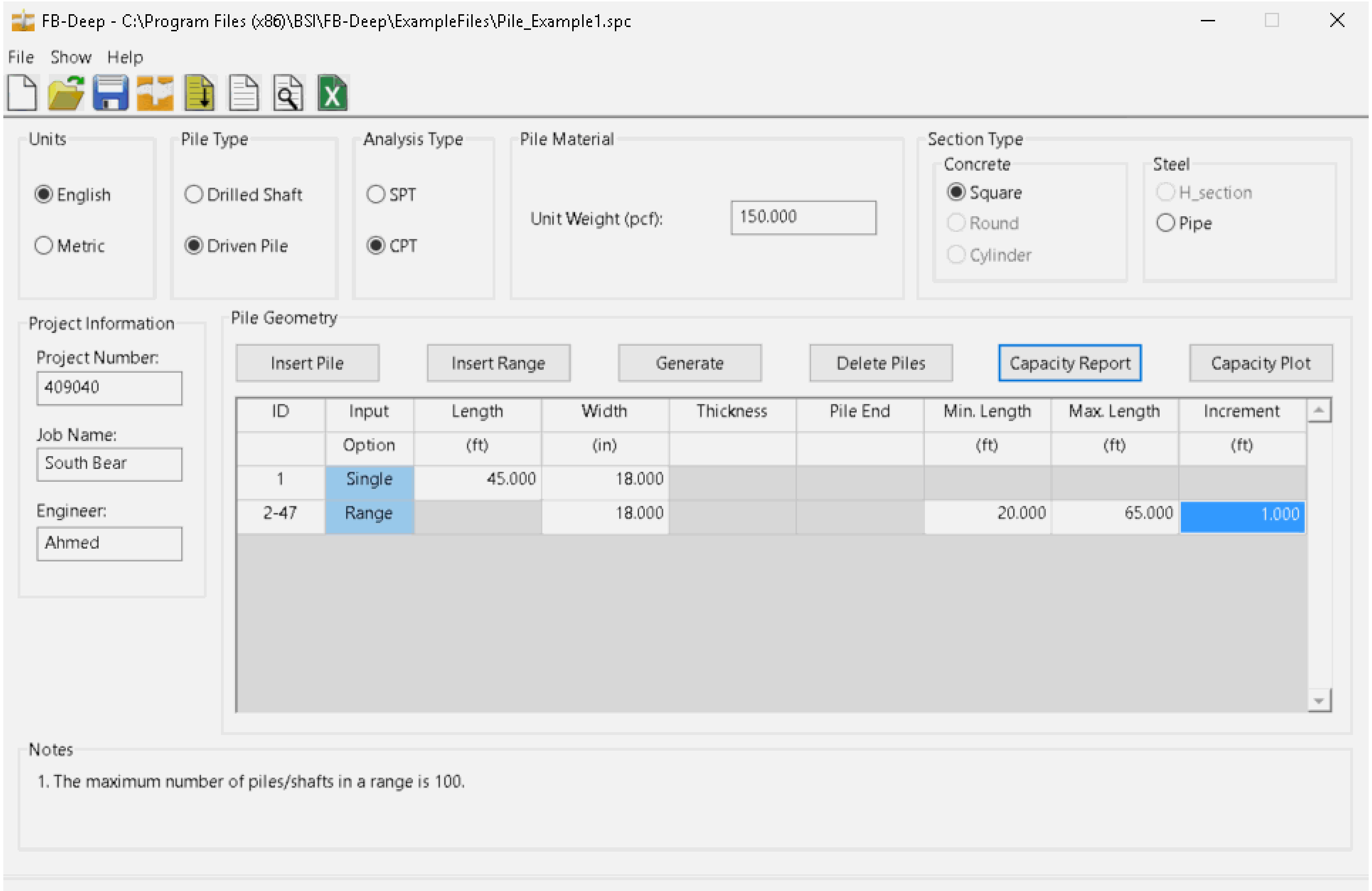
See SPT boring below for help with soil layering description.

Click – OK when done



Corresponding SPT Boring from SR 417 extension.

Main Screen



Pile Type = Driven

Unit Weight = 150pcf

Section Type = Concrete Square

Click-Insert Pile: Length = 45, Width = 18

Click – Cap. Report to view the Output

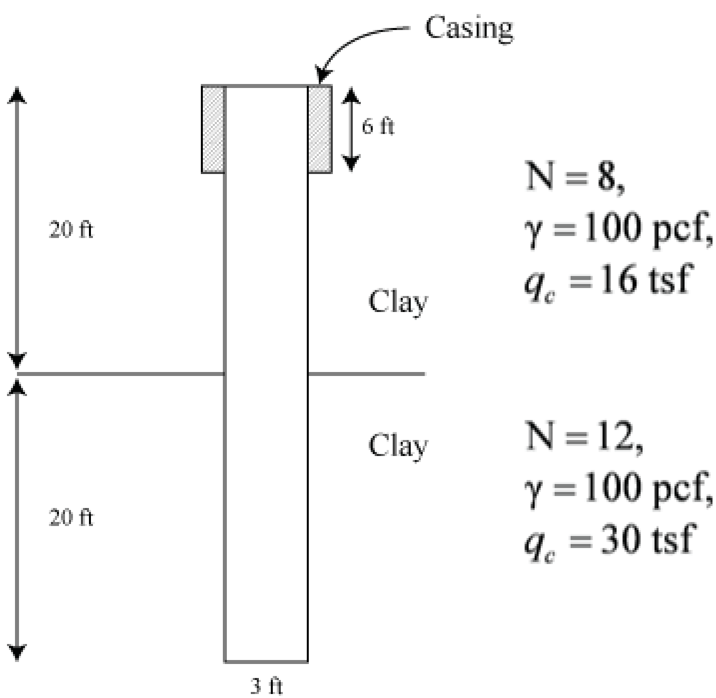
Driven Pile Capacity:

Test Pile Length (ft)	Pile Width (in)	Nominal Skin Friction Resistance (tons)	Nominal Tip Resistance (tons)	Nominal Resistance (tons)	Design Resistance (tons)	Phi Factor
45.0	18.0	216.65	23.12	239.77	112.69	0.470

4.3 Drilled Shaft Examples

1. Clay Layer
2. Sand Overlaying Rock Layer

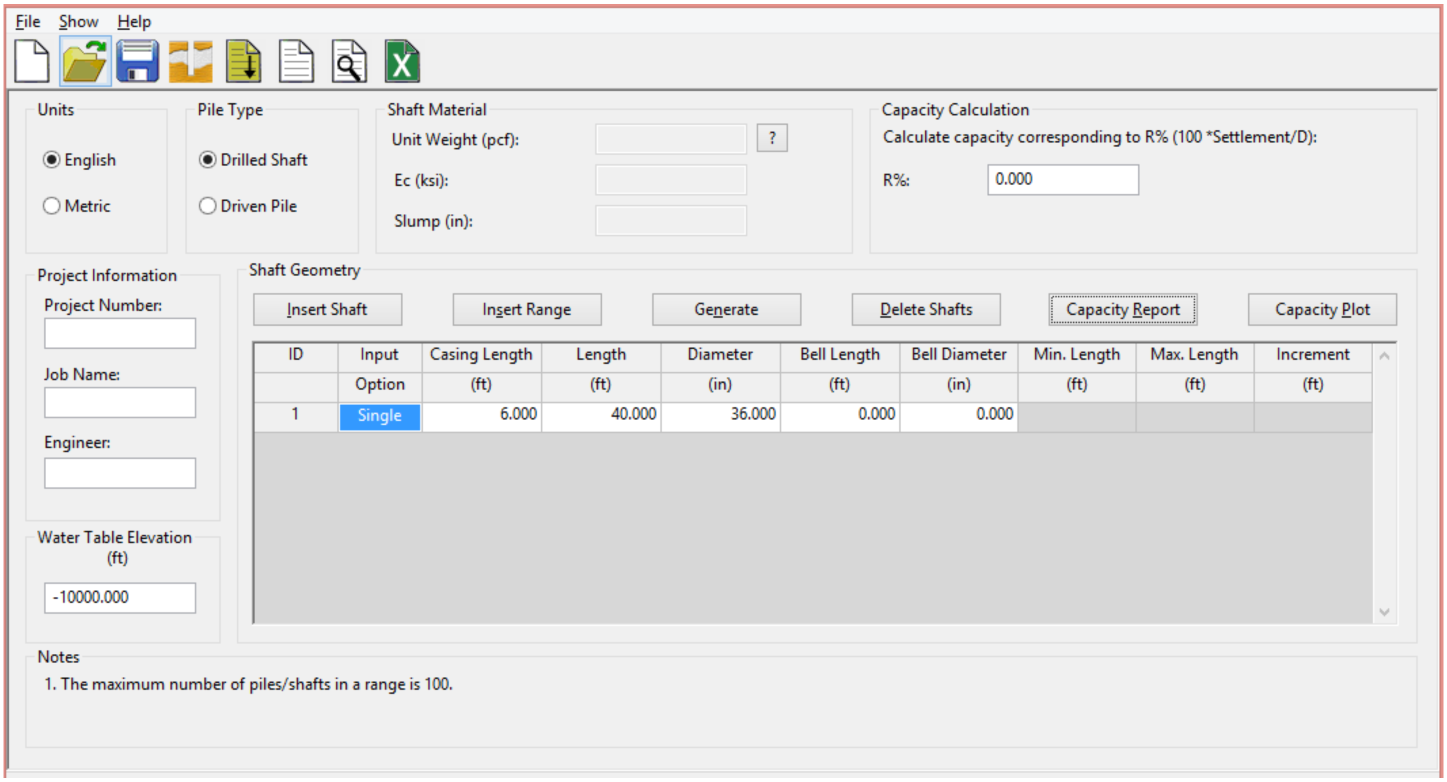
4.3.1 Shaft with Clay and Casing



Example 1 – Section1: Drilled Shaft

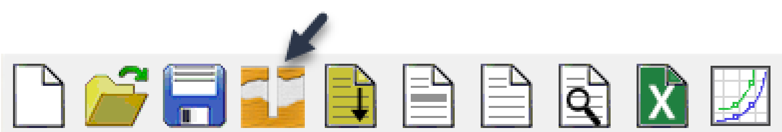
1- In the Opening Screen, Select:
Units= English; Type= Drilled Shaft

2- In “Shaft Geometry”, click “Insert Shaft” and introduce its parameters:
Input= Single; Casing Length= 6ft; Length= 40ft; Diameter= 36in



Note: Because the Shaft does not include bell, its dimensions stay zero.

3- Select form the "Show" menu, Boring log or click the Boring Log icon in the Toolbar to open the Boring Log window.



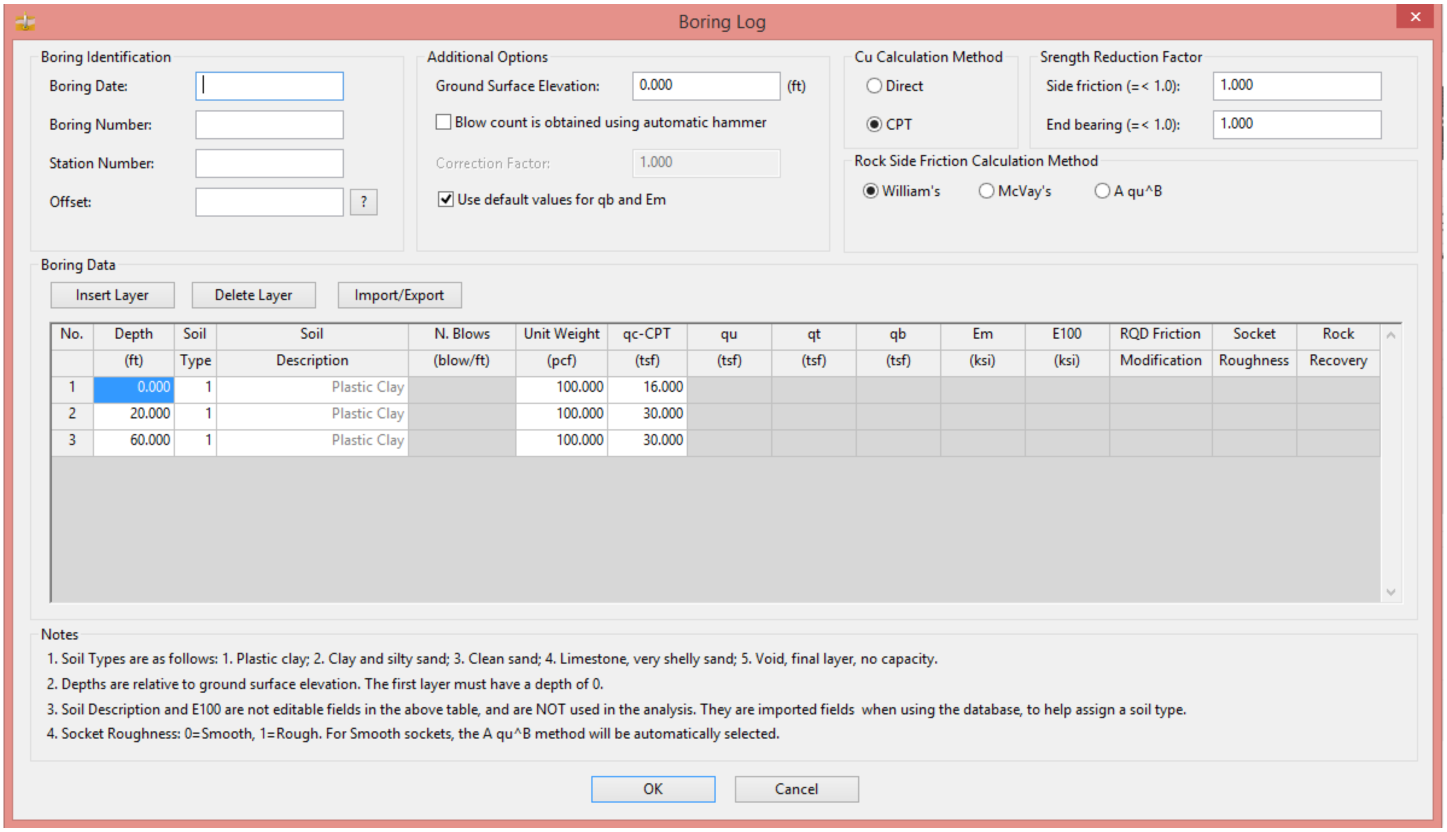
4- In the Boring Log Screen, select:

Ground Surface= 0.0ft; Cu Calculation Method= CPT

5- Click "Insert Layer" as many times as layers needed. Introduce soil parameters:

Soil Type=1 (Clay); Unit Weight=100 pcf; qc=16 tsf (for first layer); qc=30 tsf (for second and third layer)

Note: For this example, it is necessary to include three layers as the third layer marks the end of the second one described in the problem statement.



If instead of using CPT method to find the Undrain Shear Stress (Cu), it is used Direct Method, an average of the Cu for each layer is introduced in the Boring Log.

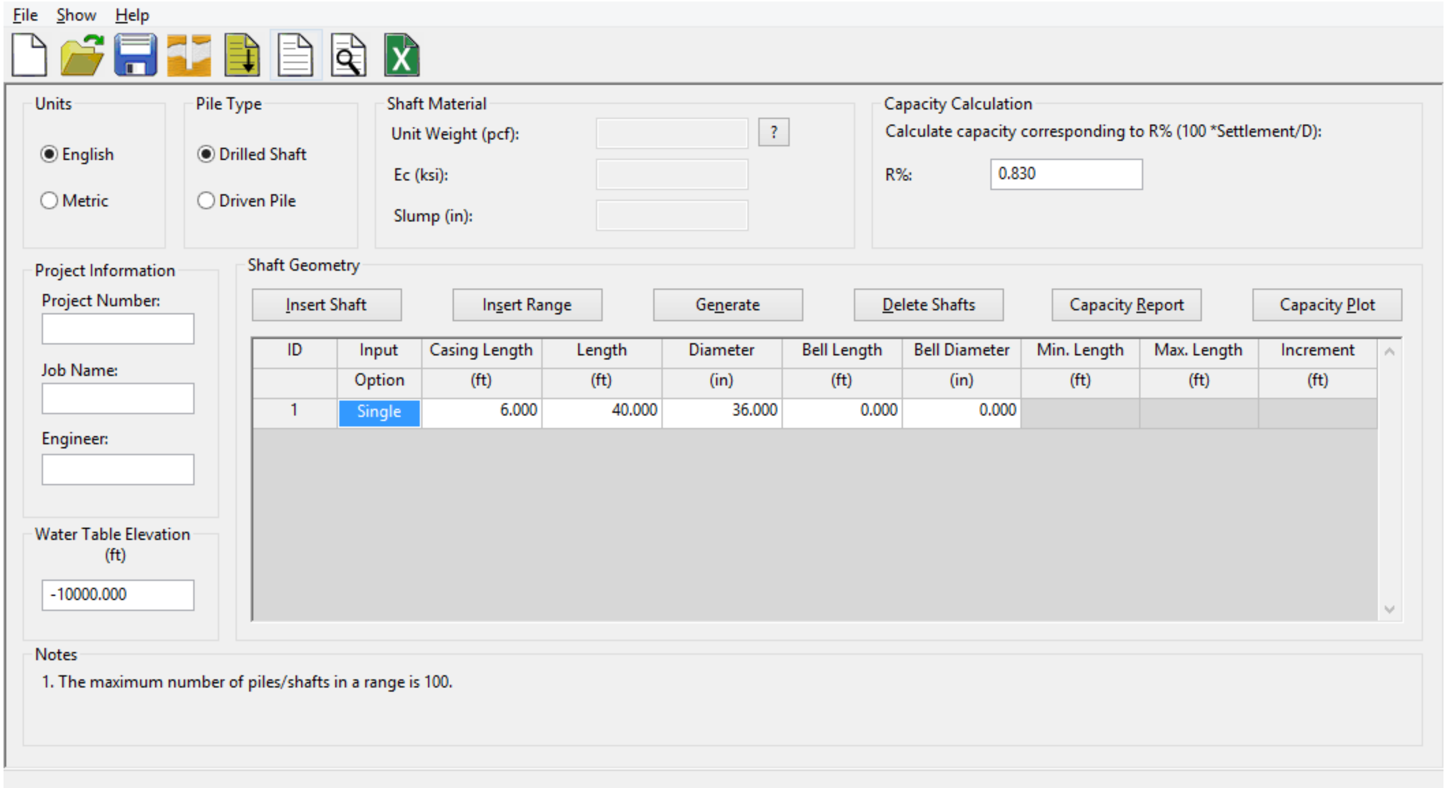
6- Click “Ok” to go back to the Main screen.

7- In the Main Screen click “Capacity Report” or in the “Show” menu and select “Shaft Capacity Report (Brief Report)".

```
Skin friction capacity
----------------------
  Strength reduction factor for skin-friction = 1.00

  Layer  Top           Ult Skin
   ID    Elev.  Thick. Friction           Soil Type
         (ft)   (ft)    (Tons)
  ----- -------- ------ -------- ------------------------------
     1     0.00  60.00          1- Plastic Clay                 <--- Bearing layer
  (* IN LAYERS ABOVE BEARING LAYER)

  Ultimate skin friction in layers above bearing layer =    0.00(tons)
  Ultimate skin friction in bearing layer              =  242.42(tons)
  Total Skin Friction                                  =  242.42(tons)

End bearing capacity
--------------------
  Soil type of end bearing layer:  1- Plastic Clay
  Strength reduction factor for End-bearing =  1.00

  ELEVATION  UNIT E. B.
    (ft)       (tsf)
  ---------- ----------
     -40.00      16.80  <-- Shaft tip elevation
     -49.00      16.53  <-- 3.0B below shaft tip

  Average unit end bearing above Shaft tip          =   16.87(tsf)
  Average unit end bearing below Shaft tip          =   16.67(tsf)
  Average unit end bearing in vicinity of Shaft tip =   16.77(tsf)

  Uncorrected mobilized end bearing capacity              =  118.51(tons)
  Corrected mobilized end bearing capacity for wide shaft  =  118.51(tons)

Shaft Capacity
-------------
  For Probability of Failure, Pf = 0.1%, factor of safety equals 2.4
  Ultimate Shaft capacity =  360.93(tons)
  Allowable Shaft Capacity (Factor of Safety = 2.4) =  150.39(tons)
```

a) For settlement of 0.3″ → R%= 0.83%

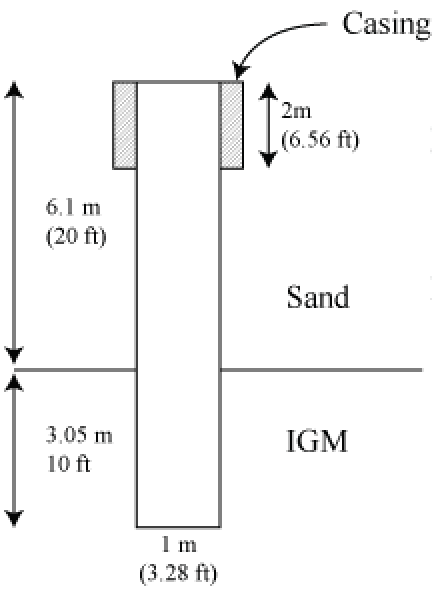
8- In the Main Screen, introduce the value for R% to calculate the capacity corresponding to the allowable settlement.



9- Click in “Capacity Report” or in the “Show” tab and select “Shaft Capacity Report (Brief Report).”

```
User-Defined Settlement = 0.83%
Shaft capacity at user-defined settlement =  293.12(tons)
```

4.3.2 Sand overlying Rock (IGM)



$\gamma = 100\,\text{pcf}\ (15.708\,\text{kN/m}^3)$

$\text{N} = 10$

$$\text{Limestone}: q_u = 10\,\text{tsf}\ (957.6\,\text{kPa},\ 0.96\,\text{MPa})$$

$$q_t = 1\,\text{tsf}\ (95.76\,\text{kPa},\ 0.096\,\text{MPa})$$

$$\gamma = 135\,\text{pcf}\ (21.2\,\text{kN/m}^3),\ \gamma_c = 20.4\,\text{kN/m}^3$$

$$E_c = 57{,}000\sqrt{f'_y} = 57{,}000\sqrt{5000\,\text{psi}}$$

$$= 4.03E6\ \text{psi}\ \ (2.77E6\,\text{kPa})$$

1- In the Opening Screen, Select:

Units= Metric; Type= Drilled Shaft

2- In “Shaft Geometry”, click “Insert Shaft” and introduce its parameters:

Input= Single; Casing Length= 2.0 m; Length=9.15 m; Diameter=1000mm

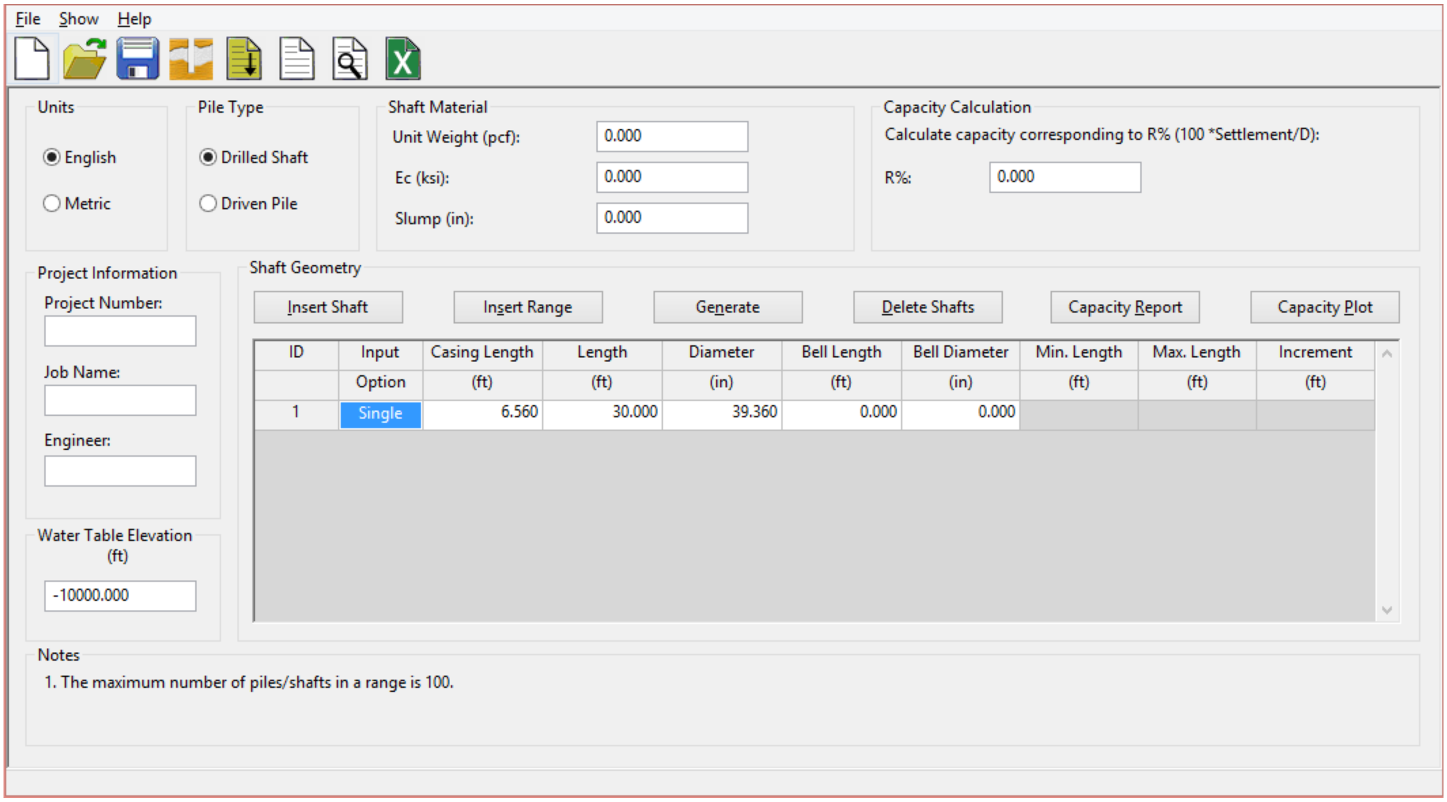


Figure: 4.3.a Opening Screen

Note: Because the Shaft does not include bell, its dimensions stay zero.

Figure: 4.3.b Boring Log Toolbar

3- Select form the "Show" tab, Boring log or click the Boring Log icon in the Toolbar to open the Boring Log window.

4- In the Boring Log Screen, click "Insert Layer" as many times as layers needed. Introduce soil parameters:

-For the First Layer: 0.0m-6.1m

Soil Type=3 (Sand) N.Blows=10 Unit Weight=15.708 kN/m3

-For the Second Layer:20ft-60ft

Soil Type=4 (Limestone) Unit Weight=21.2 kN/m3 qu=1957.6 kPa qt=95.76 kPa

Socket Roughness= 1 (Rough)

5- Also, in the Boring Log select:

Ground Surface=0.0ft Rock Side Friction Calculation Method= McVay's

Note: Because clay is not part of the soil profile, the method for calculating Cu is not relevant.

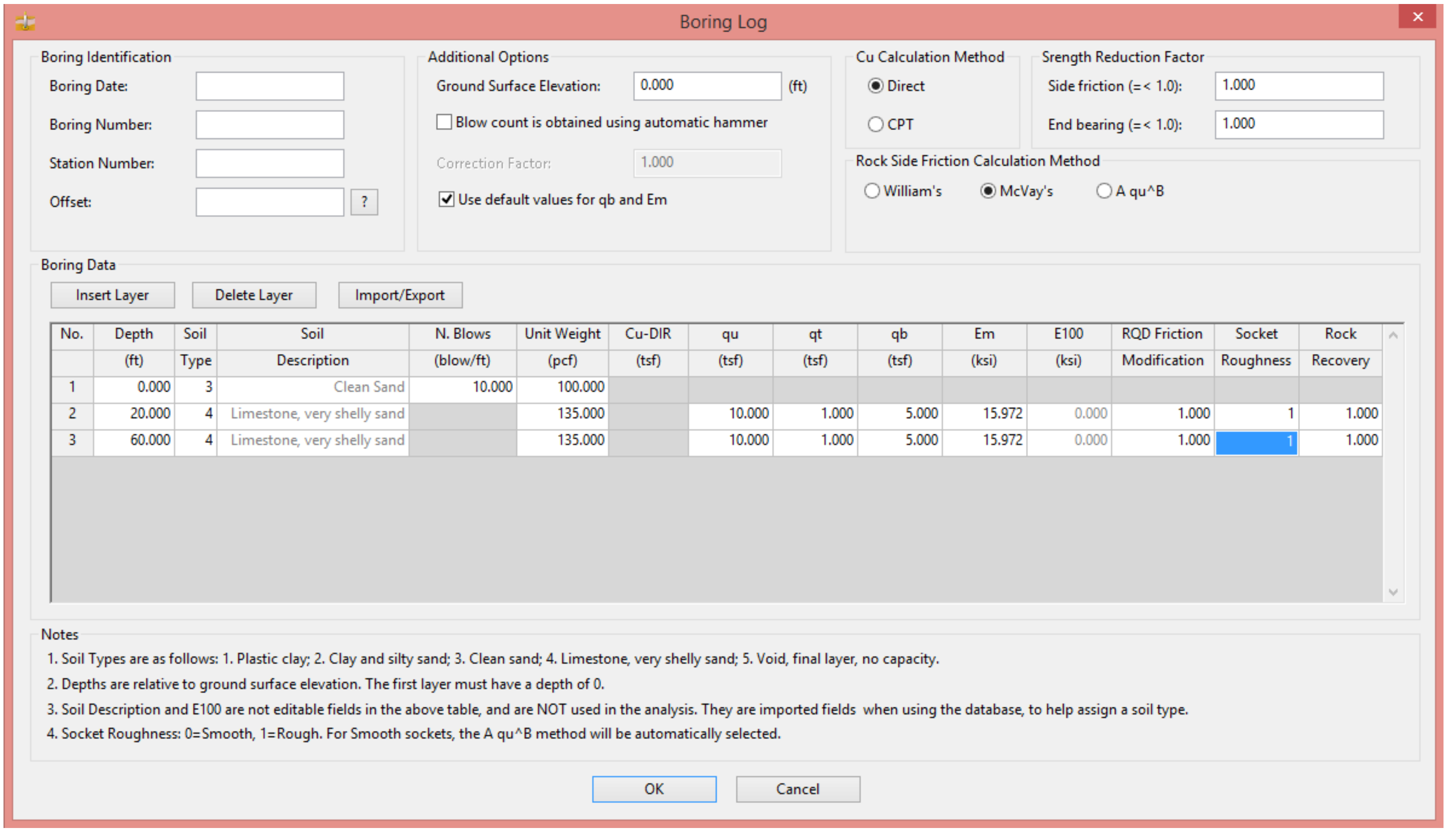


Figure: 4.3.c Boring Log Window

6- If the default values are to be used, click the button labeled "Default qb, Em." This will autofill the qb and Em space with the next value:

qb= qu/2= 478.790 kPa; Em= 115(qu)= 110.122 MPa

7- Click "Ok" to go back to the Main screen.

8- In the Main Screen, now that the program recognizes that preset of a Rock layer, it is necessary to enter the "Shaft Material" parameters:

Unit Weight=20.4 kN/m3; Ec=27785.1 MPa; Slump=152.4 mm

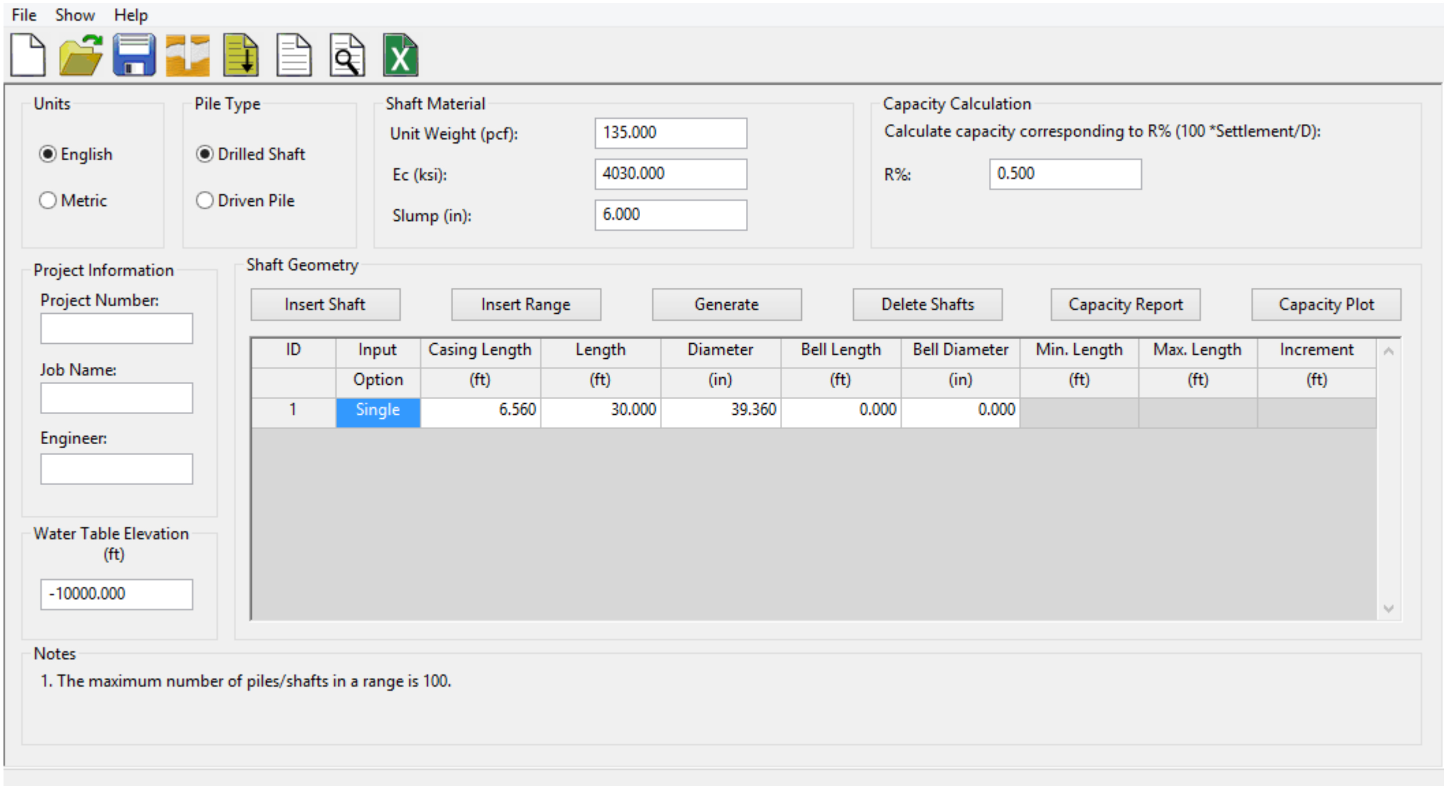


Figure: 4.3.d Shaft Material

9- Click "Capacity Report" or in the "Show" tab and select "Shaft Capacity Report (Brief Report) or (Detailed Report)".

```
Drilled Shaft Capacity (sorted by shaft diameter):
==================================================
  Strength reduction factors:  Skin-friction = 1.00, End-bearing =  1.00

   ID     Diameter     Length     Skin Fric.  End Bearing   Capacity
             (in)       (ft)        (tons)      (tons)       (tons)
  ----- ---------- ---------- ------------ ------------ ------------
     1      39.36      30.00      223.790       42.248      266.038
```

10- In the Main Screen, introduce the value for R% to calculate the capacity corresponding to the allowable settlement.

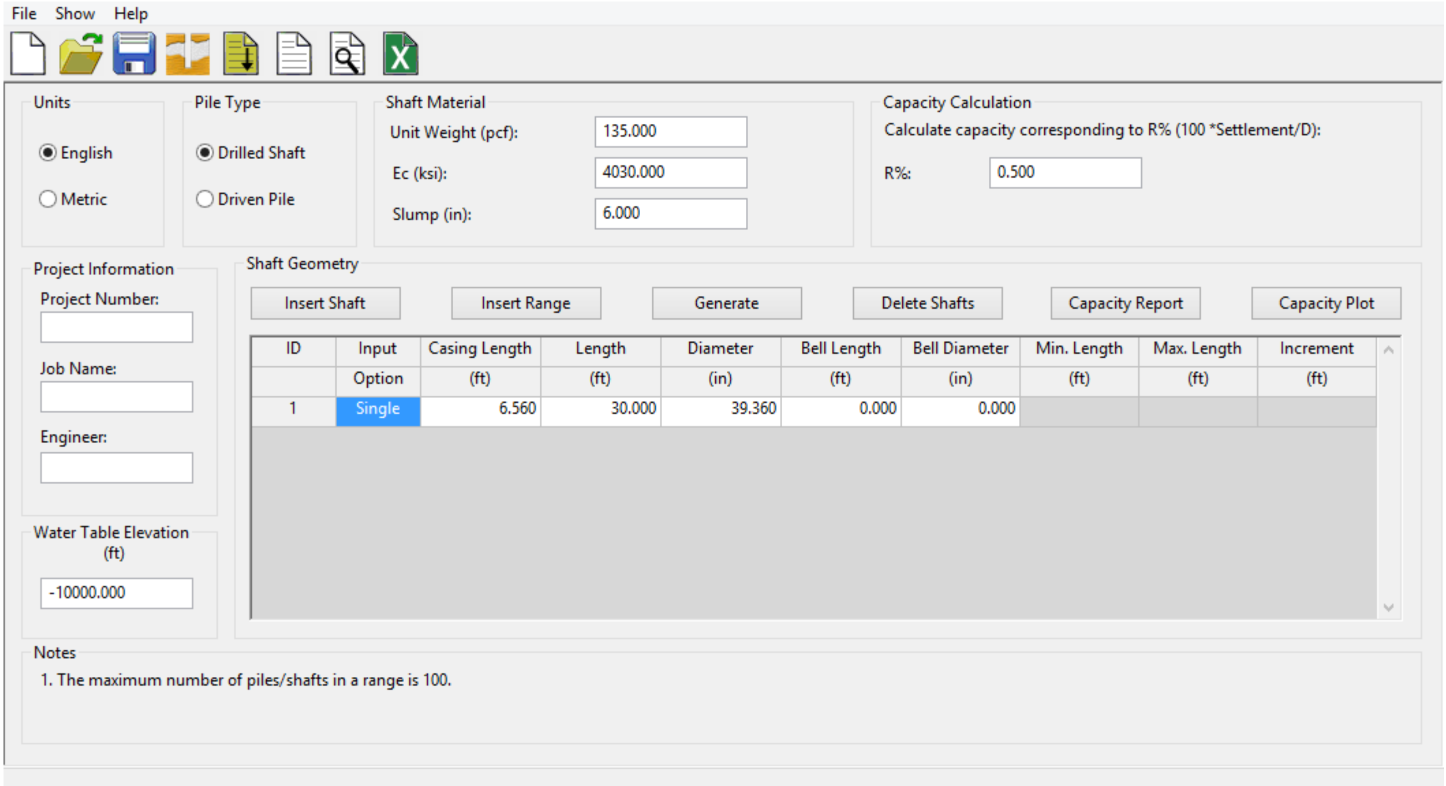


Figure: 4.3.e Settlement Capacity Calculation

11- Click in "Capacity Report" or in the "Show" tab and select "Shaft Capacity Report (Brief Report) or (Detailed Report)".

b) For settlement of 5.0mm → R%= 0.5%

```
User-Defined Settlement = 0.50%

  ID     Diameter     Length    Skin Fric.   End Bearing    Capacity
           (in)        (ft)       (tons)       (tons)        (tons)
 ----- ---------- ---------- ------------ ------------ ------------
     1      39.36      30.00      148.023       37.437      185.460
```

5 License Installation

5.1 Licensing

FB-Deep offers the following four licensing types:

1. SPT Analysis
2. SPT Analysis and CPT Analysis
3. SPT Analysis with Database Capability
4. SPT Analysis and CPT Analysis, with Database Capability

Depending on the license type purchased, certain features will be enabled/disabled. Multiple license types can be purchased on the same network license. In this instance, when FB-Deep is run from a workstation, the user is prompted to choose the license type for the current program run. For example, suppose a network license contains two seats, one of which is an SPT Analysis (type 1), and the other is SPT Analysis with Database Capability (type 3). When the type 1 license is being used, only the type 3 license would be simultaneously available.

5.2 License File

FB-Deep operates using a license file to determine its status. All shipped versions run in Demo mode as the default. The program can be "unlocked" into various modes including full version, networked or stand-alone. This unlocking can be done by hand, through phone contact with the Bridge Software Institute (http://bsi-web.ce.ufl.edu) or automatically through an internet connection to the BSI web server.

The program requires a license file to be installed. This license file is linked to the computer on which it is installed.

NOTE: You must have administrator rights on Windows NT or Windows 2000 to install FB-Deep or the license file on a server.

The following describes the modes and processes required:

Stand-Alone

A stand-alone or fixed license version is locked to run on a single machine and only that machine. The license file is installed on the individual machine.

Network Version

A network version is a floating license version that allows a fixed number of machines to run the program at any one time. For example, a three-seat installation allows three computers to run the program at the same time. The program is actually installed on any number of machines. For example, you can install the program on 20 computers in your network. However, only three of the 20 can use the program at the same time.

This installation requires a network server that shares a directory with all the computers wishing to run FB-Deep. The shared directory is where the license file is installed. All client machines must have read and write permissions for the shared directory in order for the program to run.

There is a separate install program for installing the license file on the server.

If your network installation has multiple servers, you will need to purchase multiple server versions.

Updating the license

Any installed version can have its permissions changed by entering encrypted numbers into the license file. This is done by choosing the Help->Update Software License option from the main menu. The update can be done by hand or automatically through the Internet.

E-mail/Fax/Phone License Update

This option is for installations that do not have an Internet connection. To do this installation, call the BSI support number (check the web for the phone number) and you will be stepped through the process. Numbers from your computer need to be given to the BSI representative and we can Fax or E-mail the encoded numbers you will need to type into the program.

Internet License Update

This option requires the computer on which you are installing the license file be connected to the Internet. Then, all numbers are communicated through the Internet and the license updated automatically. The computer can either be a stand-alone system or the network server for a multiple seat license.

Transfer License

There is a built in function that allows you to transfer you license to another machine. This allows you to move the license file from your current server or workstation to a new machine.

Troubleshooting

The license file (both for servers and individual workstations) is locked to a machine based on hardware components contained in the machine. If you change or modify your hardware (drives, motherboards etc) your installation may not function. To do this, you should first transfer the license, then modify your hardware, and then re-install the license on the machine.

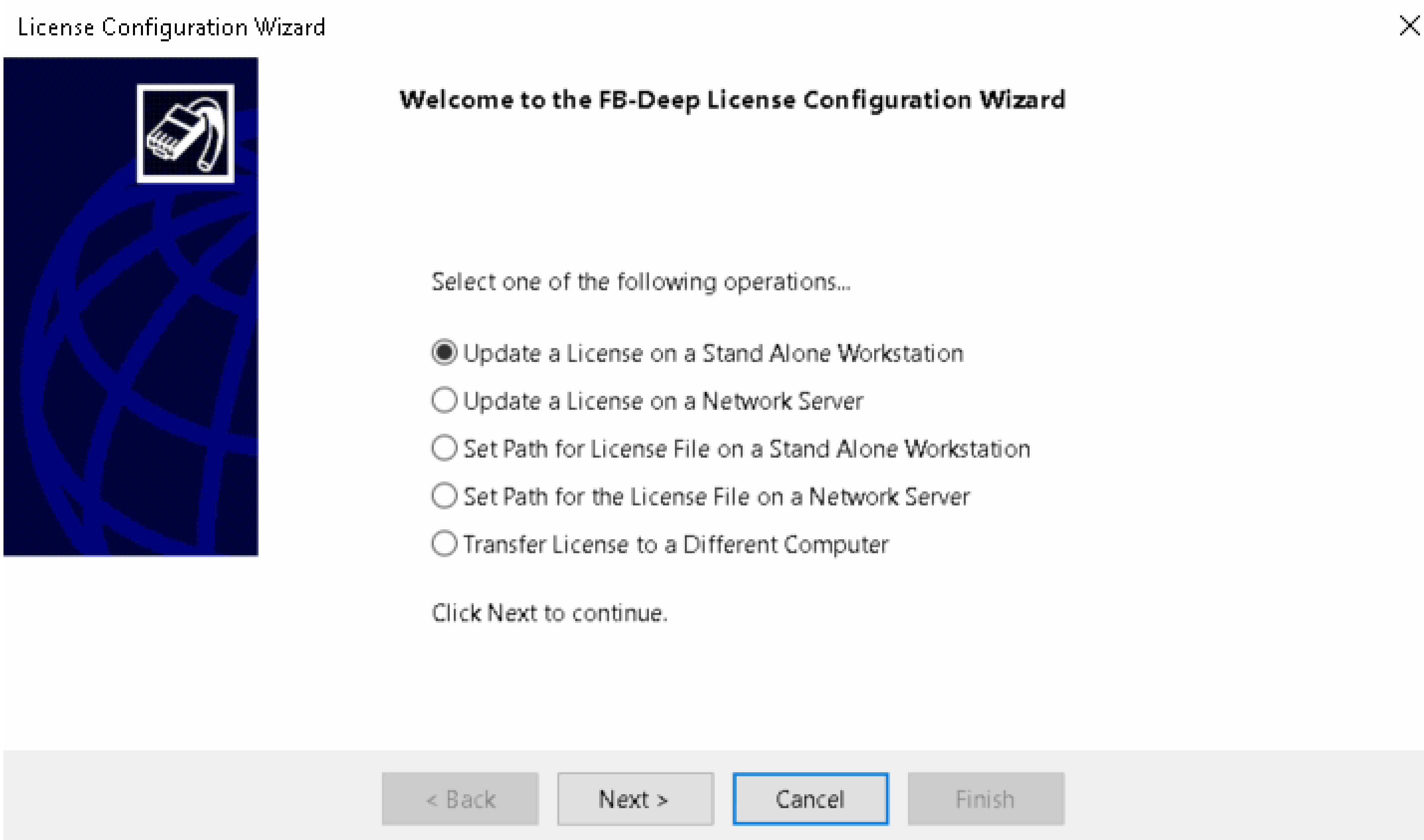
Novell systems: Be sure that the directory where the license file is saved is accessible to any user. The user must have read, write, modify, erase and create rights for that directory.

5.3 FB-Deep License Installation Help

Before updating the program license for the first time, the FB-Deep program will run in demo mode. While running in demo mode, the save option is disabled and the program execution is limited to 30 days. The reporting options are also limited to the first record, regardless of the number of records. After purchasing the program, these limitations can be removed by using the License Configuration Wizard.

To update the software license at any time, select Update Software License from the Help menu. Doing so brings up the License Configuration Wizard.

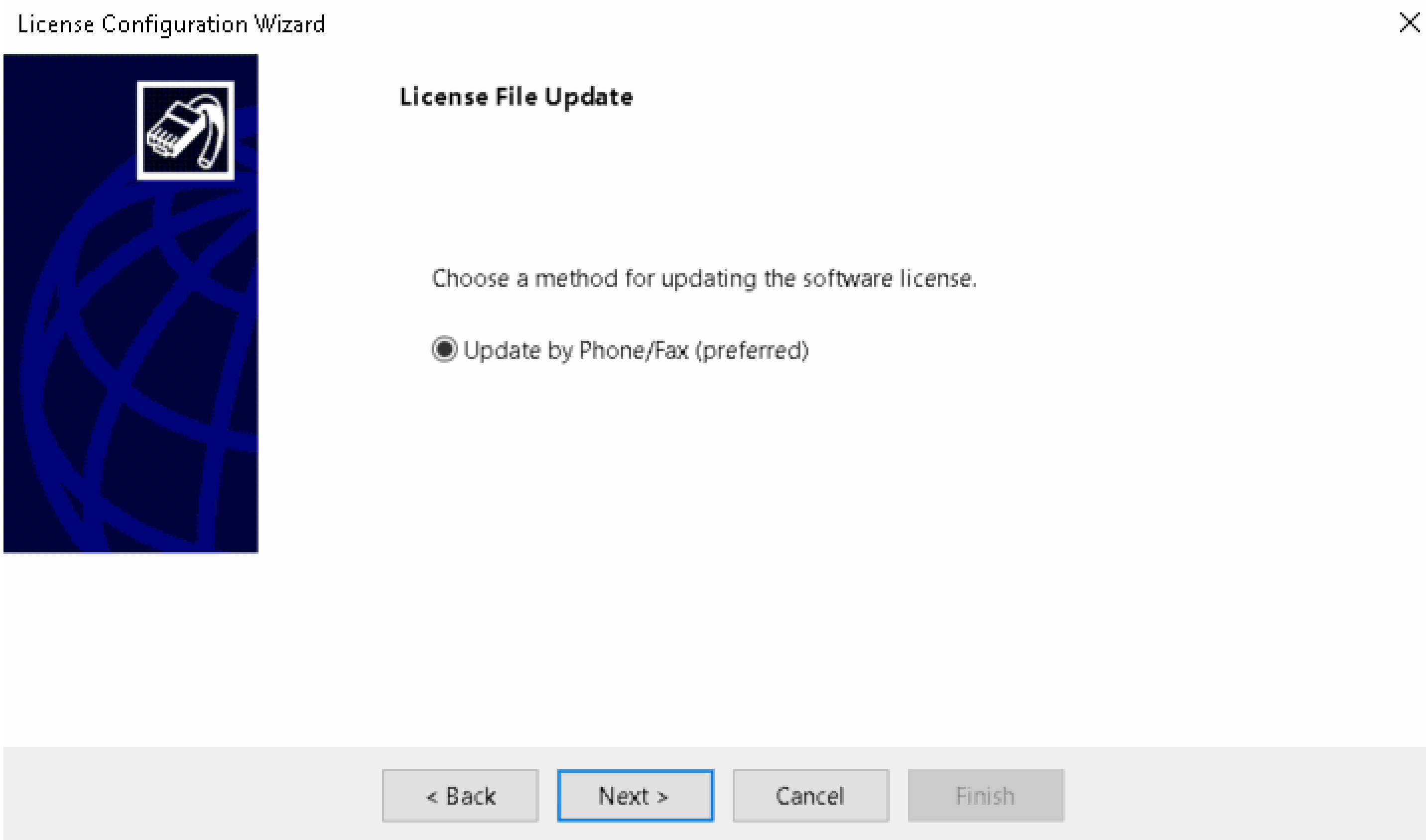
The initial License Configuration Wizard screen shows four options for updating the software license. The options are shown below:



5.4 Update a License on a Standalone Workstation

This option is used for a single installation of the software that does not rely on network to run the program. A license of this type is individually purchased per machine.

Click the Next button to continue. The next screen presents two methods for updating the software license. The first method allows the user to update the license by phone/fax. The second method is the preferred method, which allows the user to update the software license via an Internet connection. This method is preferred since it is completely automated, assuming that a user account has been established in advance and that the user can connect to the Bridge Software Institute (BSI) web server. The user account will be created when downloading the FB-Deep program.

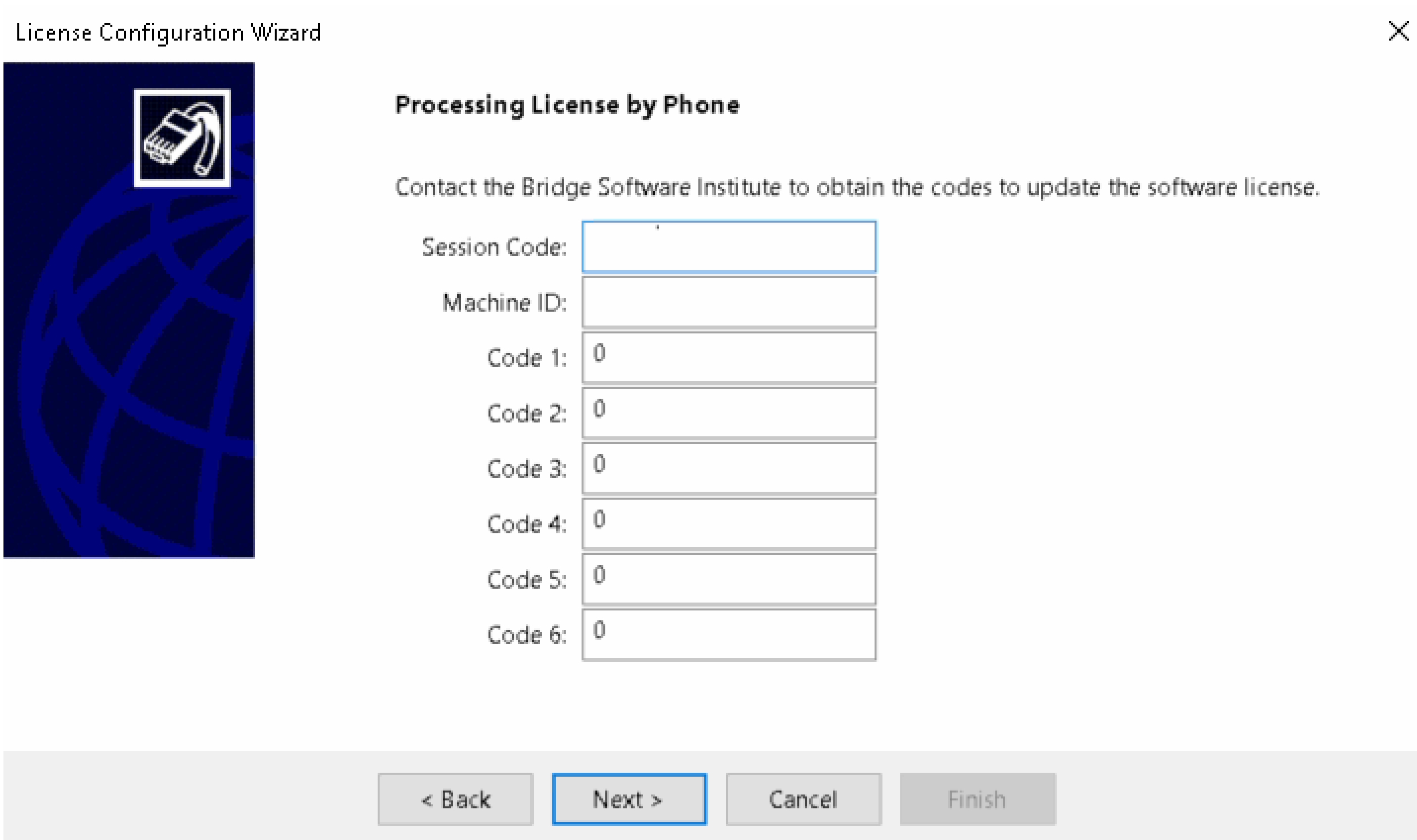


FB-Deep utilizes a license file to determine the program configuration. This license file must be updated by one of the two methods. If neither option is feasible, please contact the BSI for assistance.

License File Update by Phone/Fax

This option requires a phone call to the BSI. To update a license by phone/fax, select Update by Phone/Fax and click the next button to continue. The next screen shows a series of edit boxes for entering license data. The Session Code and Machine ID need to be given to the BSI representative. After validating the user's account information and status, the BSI representative will then supply the user with a series of numerical codes that will modify the configuration of the license file. If the numerical codes are entered correctly, the program will be unlocked and will run without any limitations. If any of the numerical codes are entered incorrectly, the wizard will prevent the user from advancing to the next screen.

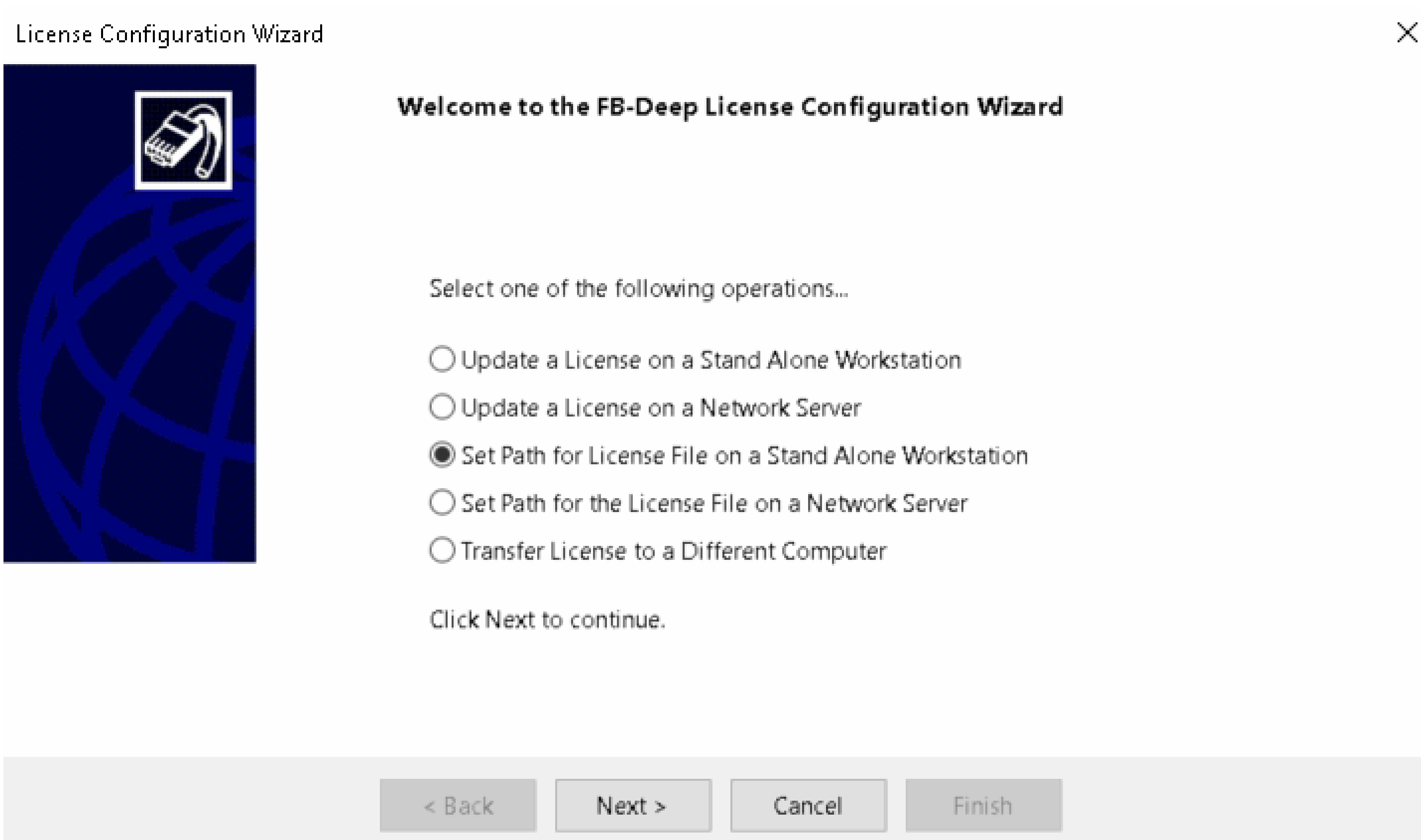
Click Next after entering the numerical codes.



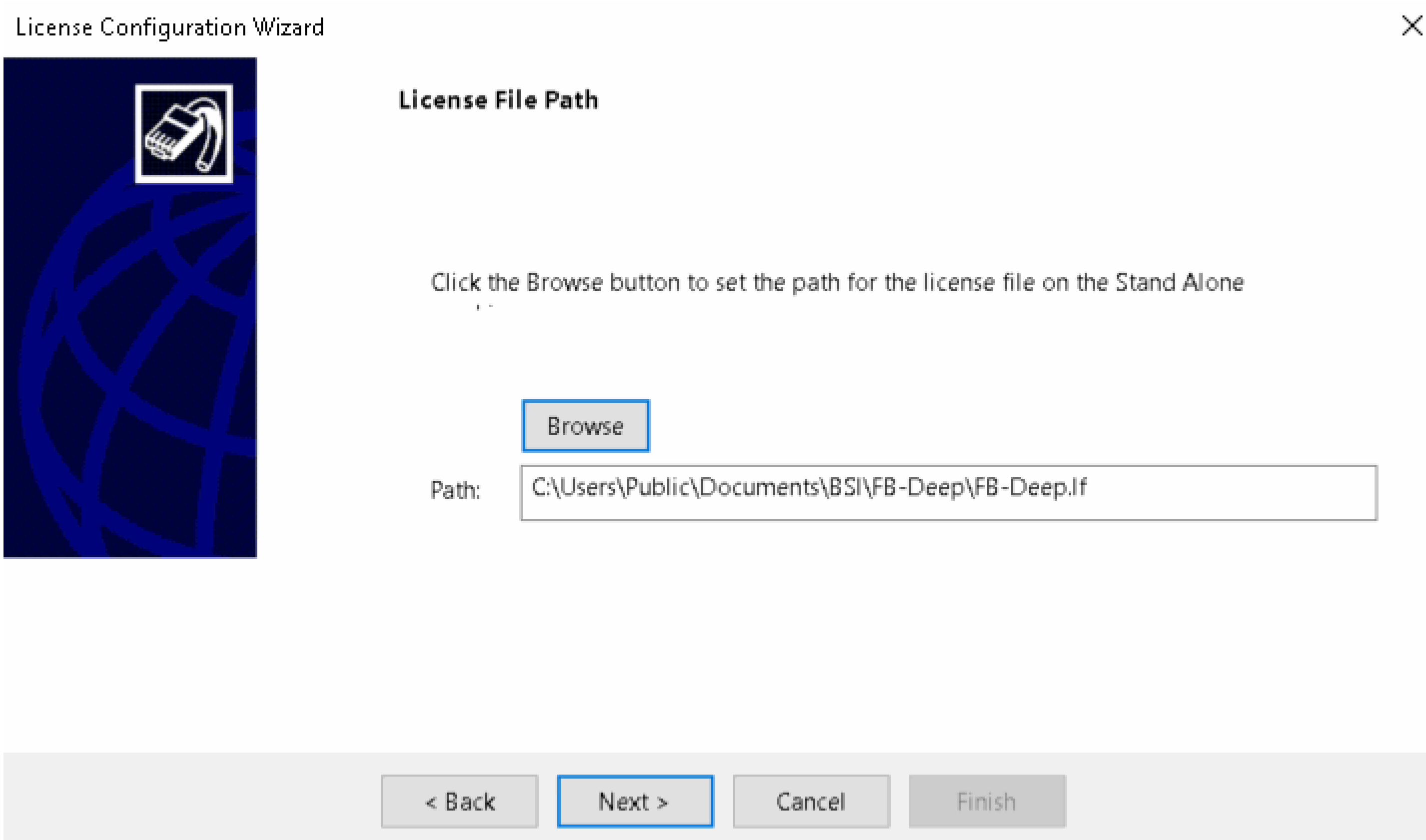
The Update Complete screen will then be shown after successfully entering the numerical codes. In order to apply the changes to the program configuration, the FB-Deep program needs to be restarted. Clicking the Finish button will update and automatically close the program. The program will now run in an unlocked state.

5.5 Set Client Path for a License File on a Network Server

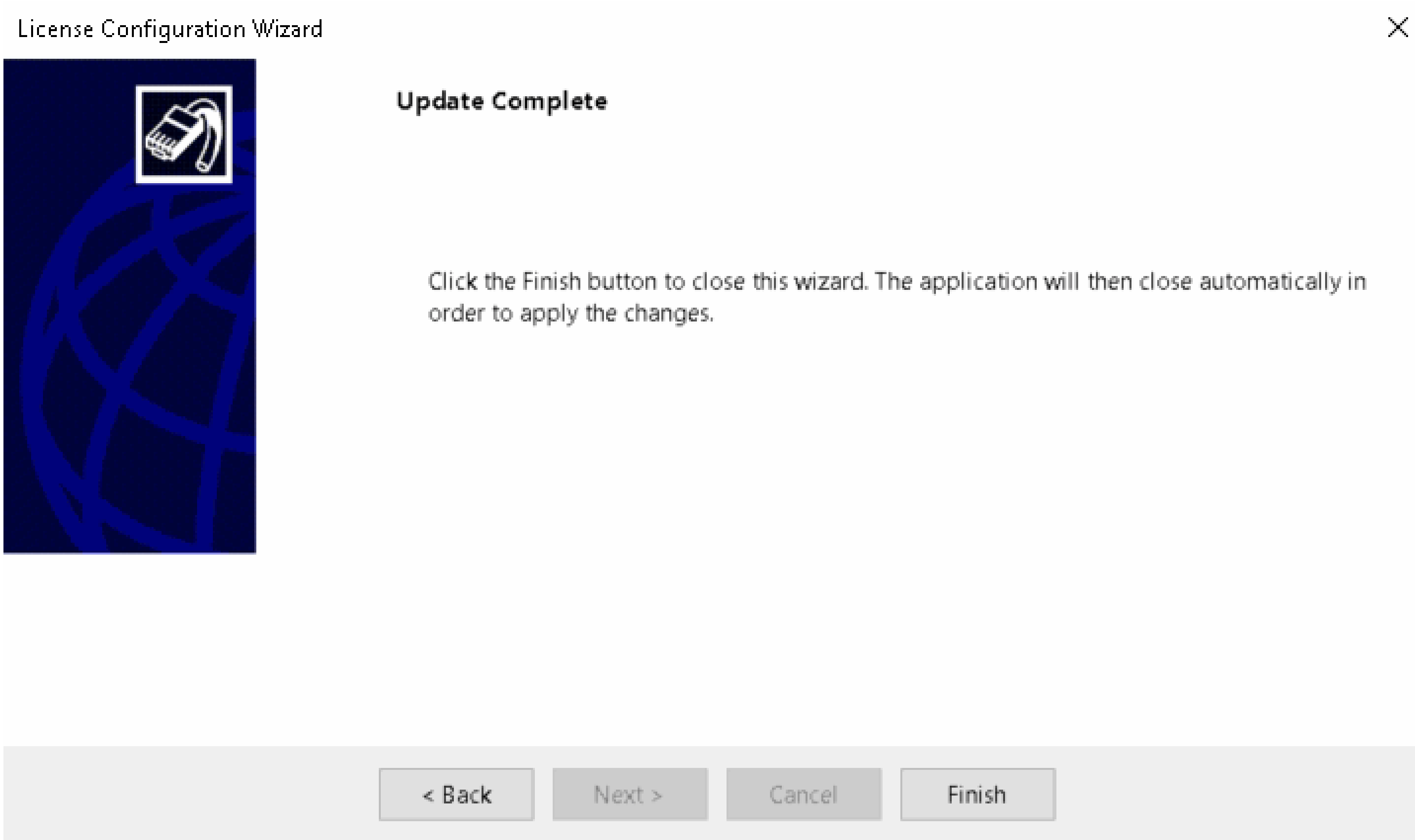
This option is used by the network client computer after a server license file has been configured and successfully installed on the network server (see LicServe Wizard). When a floating network license is purchased, the limiting factor is the number of network seats. The FB-Deep program can be installed on any number of client machines, however, the number of clients that can run the program at one time is limited by the number of network seats purchased. In order for the client machine to run the program using this scenario the client must locate the license file that has already been installed on the network server. Once this path has been established it will be saved so that the client machine will automatically find the license file each time the program is run.



Click the Next button to continue. The next screen asks the user to browse to the license file path on the network server. The user can either type the path or preferably click the Browse button to locate the file. The license file is named "ShaftSpt97.lf". Click the Browse button, locate the license file on the network server, and click Open to continue. You must browse through the network to locate the license file. You can not use a mapped drive letter.



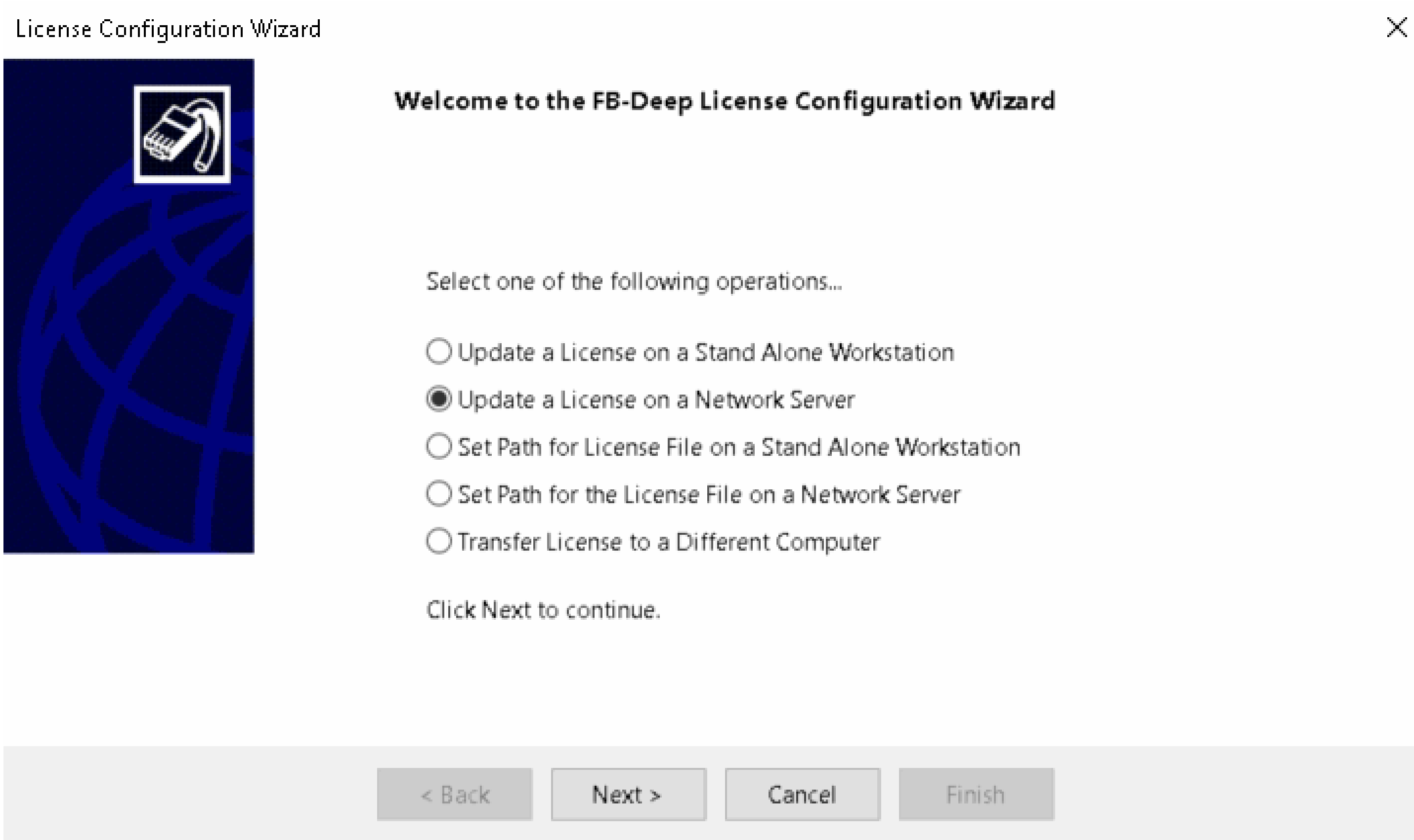
Click Next after locating the license file on the network server. The Update Complete page is now shown. In order to apply the changes to the program configuration, the FB-Deep program needs to be restarted. Clicking the Finish button will update and automatically close the program. The program will now run in an unlocked state.



5.6 Update/Install a License on a Network Server

This option is used for a single installation of the software on a network server. This license update is identical to stand alone workstation update, except that the license is configured on the network server. This option would be used to run the program directly on the server to take advantage of the server hardware configuration (i.e. more memory, hard disk space, etc.). A license of this type is individually purchases per machine.

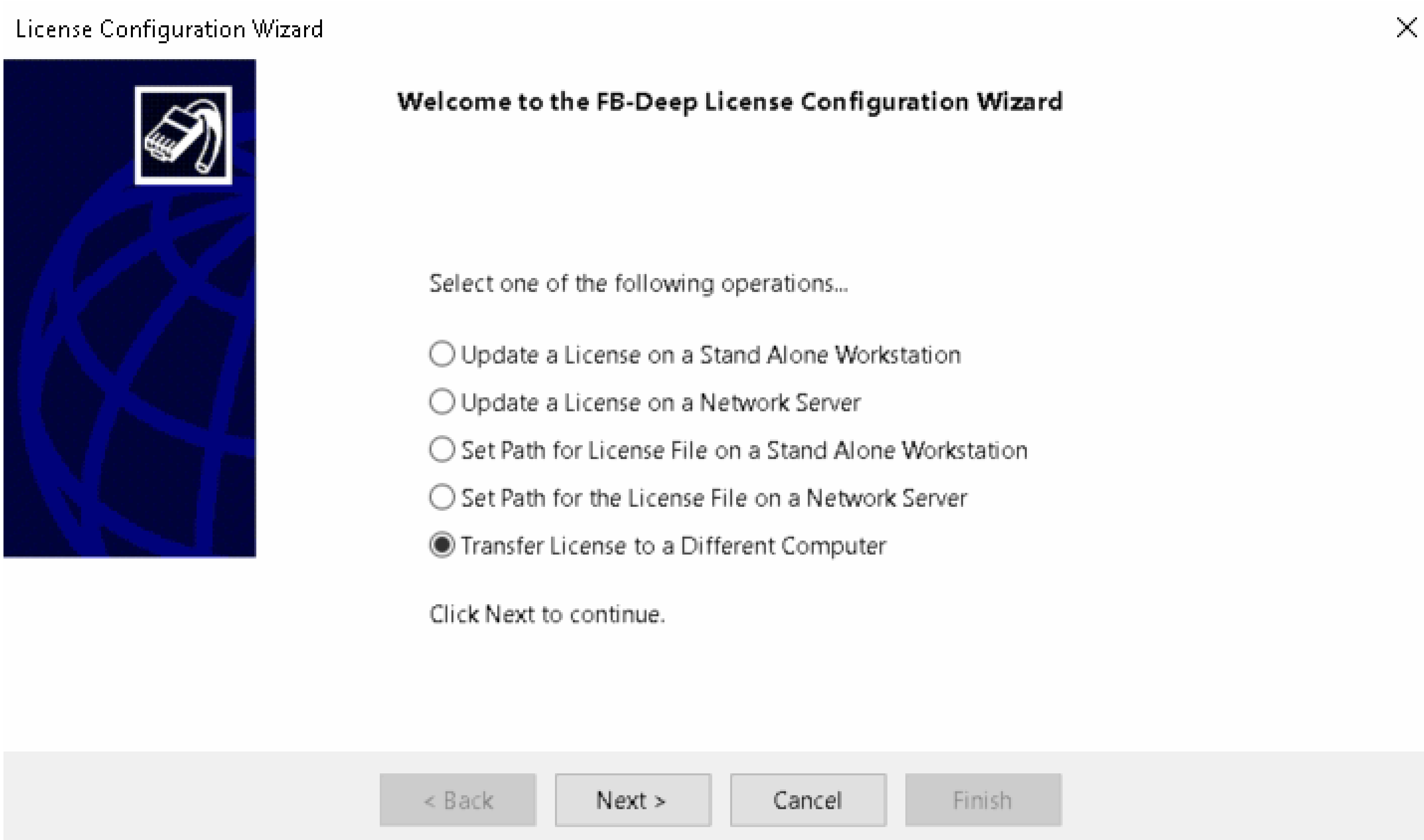
Select Update a License on a Network Server from the initial screen and follow the steps outline for Updating a License on a Stand Alone Workstation.



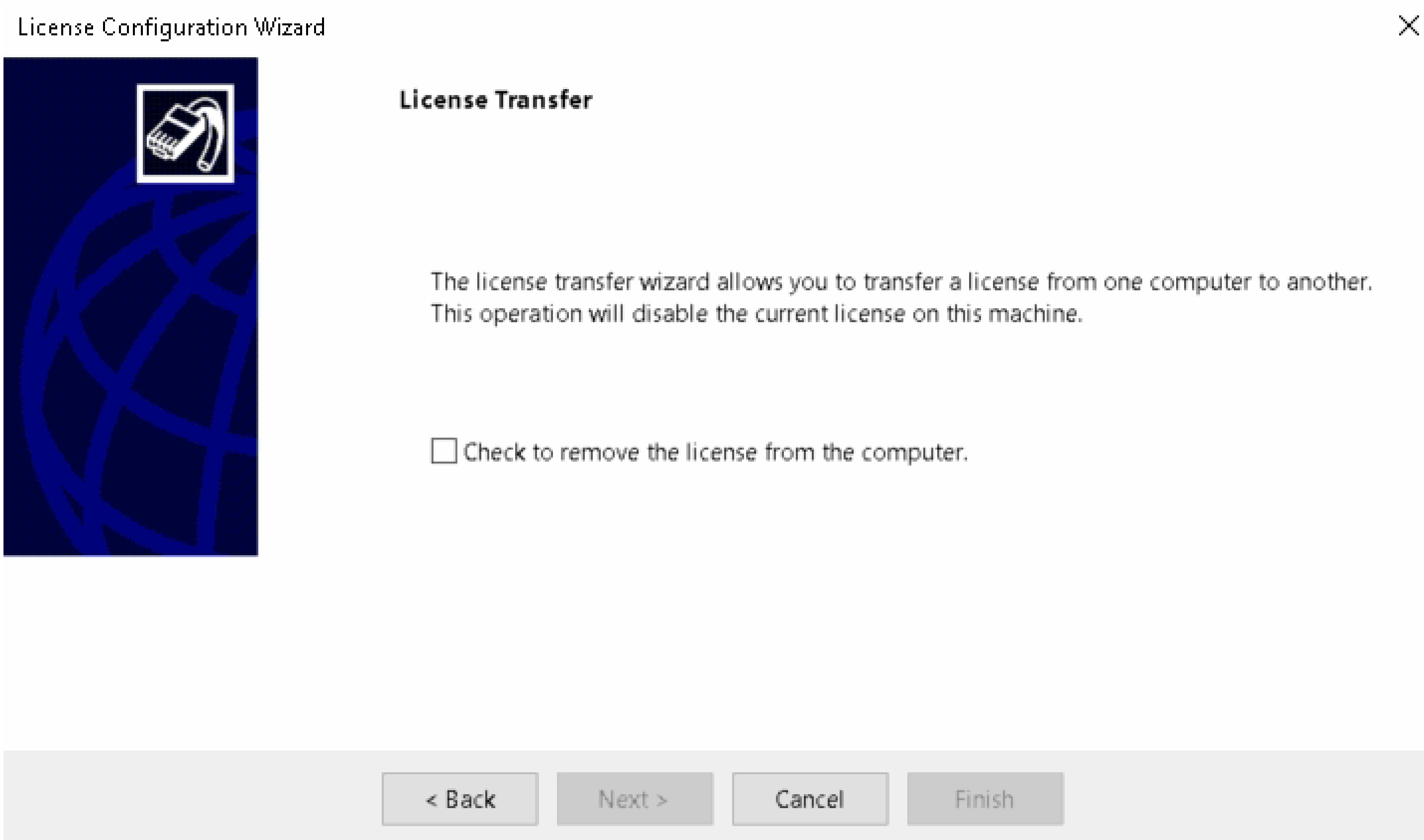
5.7 Transfer License to a Different Computer

This option is used to transfer a valid software license to another computer if the user no longer wishes to have the license on the current computer. Please note that selecting this option will invalidate the license file on the current machine. Also, this option is only valid for a stand along workstation installation of FB-Deep. Floating network installations are not applicable since the license is stored on the network server.

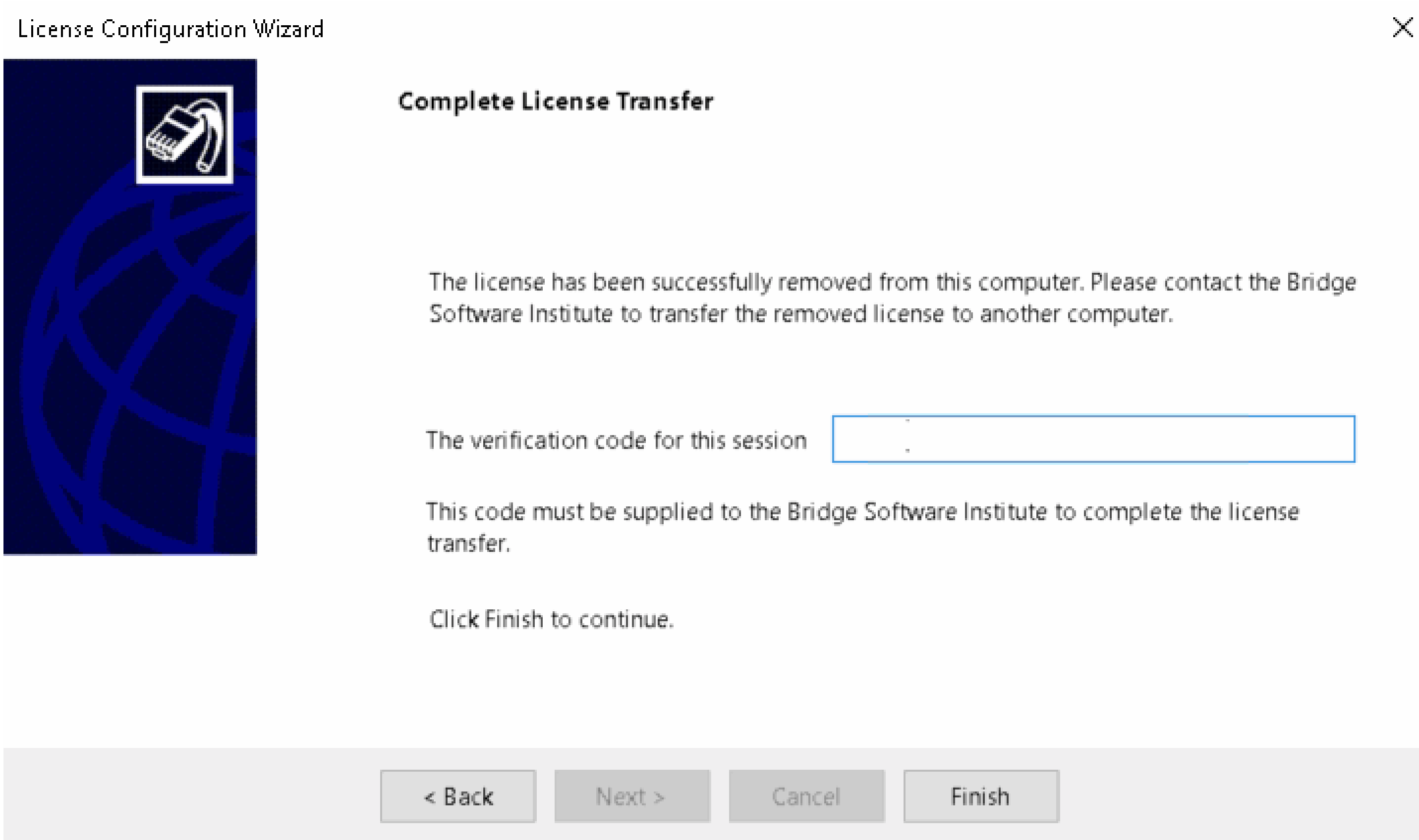
To proceed, select Transfer License to a Different Computer and click the Next button.



Because this process can not be reversed, the user must check the box to confirm the remove the license from the current computer before proceeding. Doing so will enable the Next button. Click the Next button to remove the license.



The next screen informs the user that the license has been successfully removed. A verification code is displayed on the screen (and written to the file "LicRemoval.txt" in the application directory). This code must be given to a BSI representative in order to complete the license transfer process and activate the license on another computer.



Click the Next button to continue. The Update Complete page is now shown. In order to apply the changes to the program configuration, the FB-Deep program needs to be restarted. Clicking the Finish button will update and automatically close the program. The program will now run in Demo mode.

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