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A presentation of **UniPile** software for calculation of Capacity, Drag Force, Downdrag, and Settlement for Piles and Piled Foundations

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and

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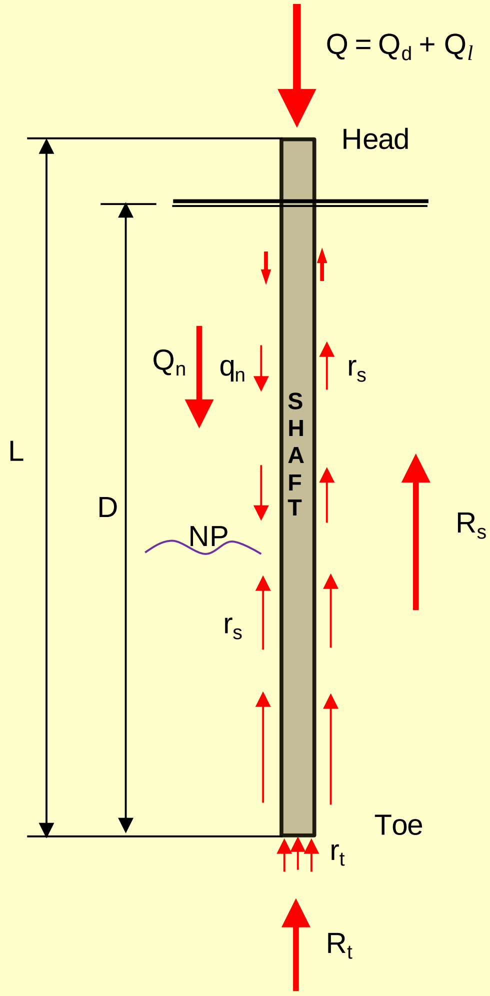
2° C.F.P.B. CONGRESO INTERNACIONAL DE FUNDACIONES PROFUNDAS DE BOLIVIA del 12 al 15 de mayo de 2015 SANTA CRUZ · BOLIVIA

Fellenius, B.H. and Goudreault P.A., , 2015. Background to UniPile.
Segundo Congreso Internacional de Fundaciones Profundas de Bolivia,
Santa Cruz May 12-15, Lecture, 58 p.

The Foundation Pile



Q = Load

Q_d = Dead load, Sustained load

Q_l = Live load, Transient load

r_s = Unit shaft resistance

R_s = Total shaft resistance

q_n = Unit negative skin friction

Q_n = Drag force

r_t = Unit toe resistance

R_t = Total toe resistance

L = Pile length

D = Embedment depth

NP = Neutral Plane

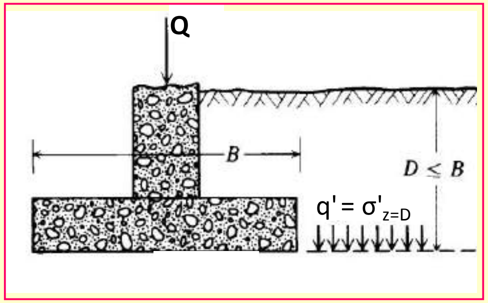
A_s = Circumferential area (m^2/m; ft^2/ft)

A_t = Pile toe area (m^2; ft^2)

A pile toe is really a footing with a long stem, so the bearing capacity formula applies, or does it?

The Bearing Capacity Formula

$$r_u = c' N_c + q' N_q + 0.5 b \gamma' N_\gamma$$



where r_u = ultimate unit resistance of the footing

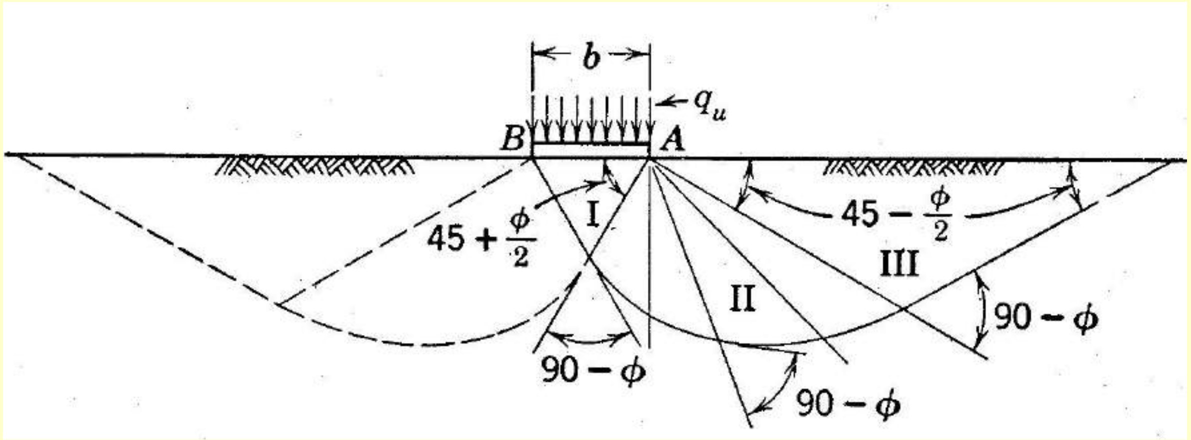
c' = effective cohesion intercept

B = footing width

q' = overburden effective stress at the foundation level

γ' = average effective unit weight of the soil below the foundation

N_c, N_q, N_γ = non-dimensional bearing capacity factors



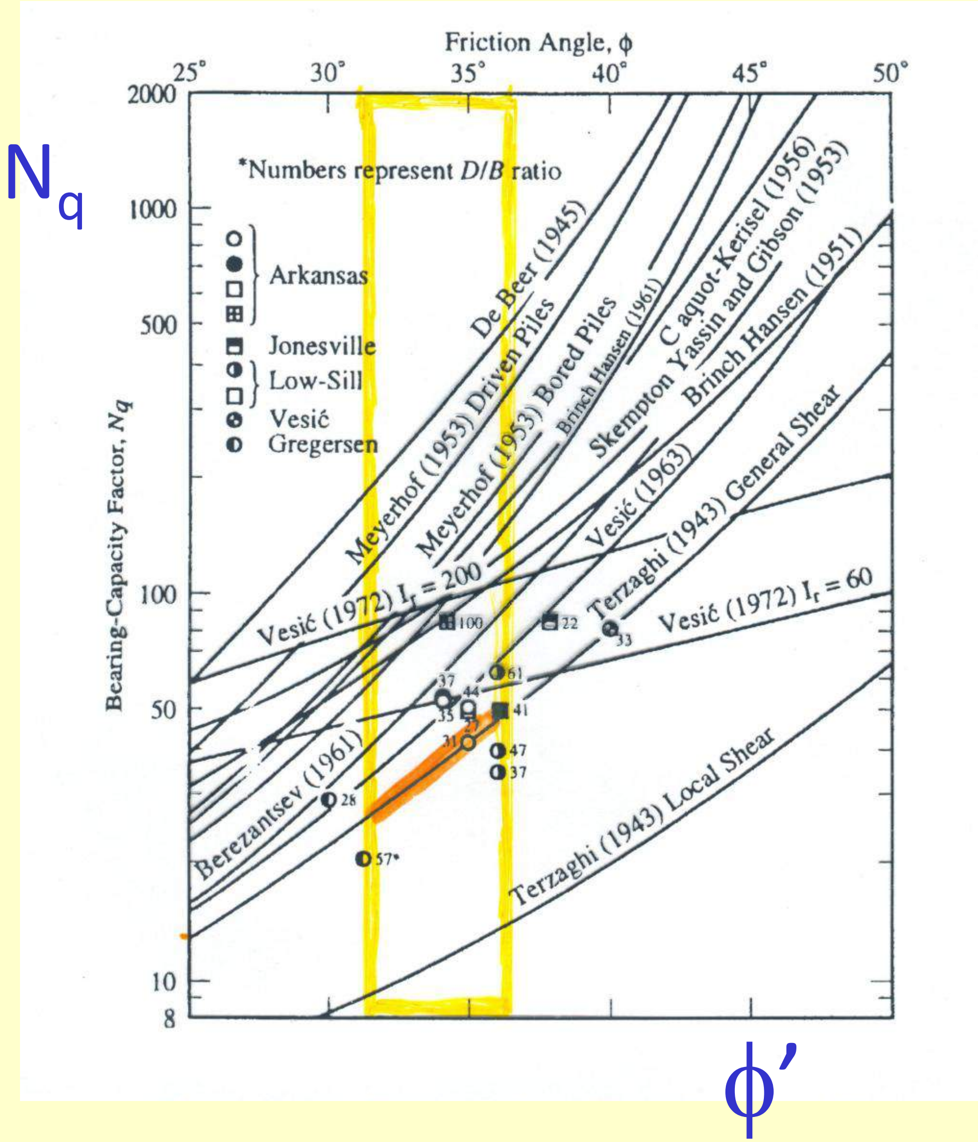
Factor of Safety, F_s

$$F_s = r_u/q$$

(q = Q/footing area)

$$r_t = q' N_q$$

N_q was determined in tests—model-scale tests



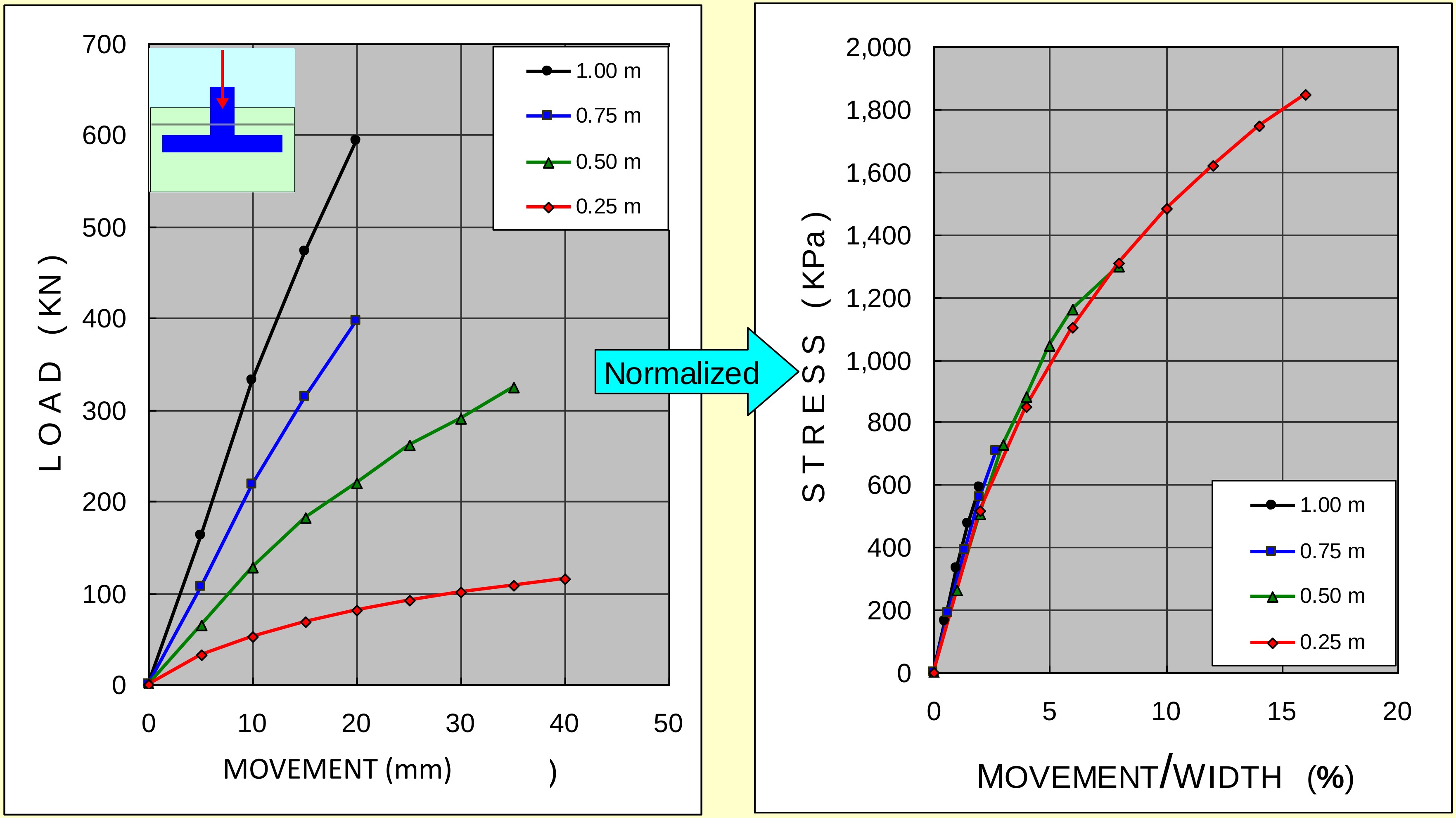
Min to max N_q ratio can be ≈200 for the same ϕ'!

The log-scale plot is necessary to show all curves with some degree of resolution.

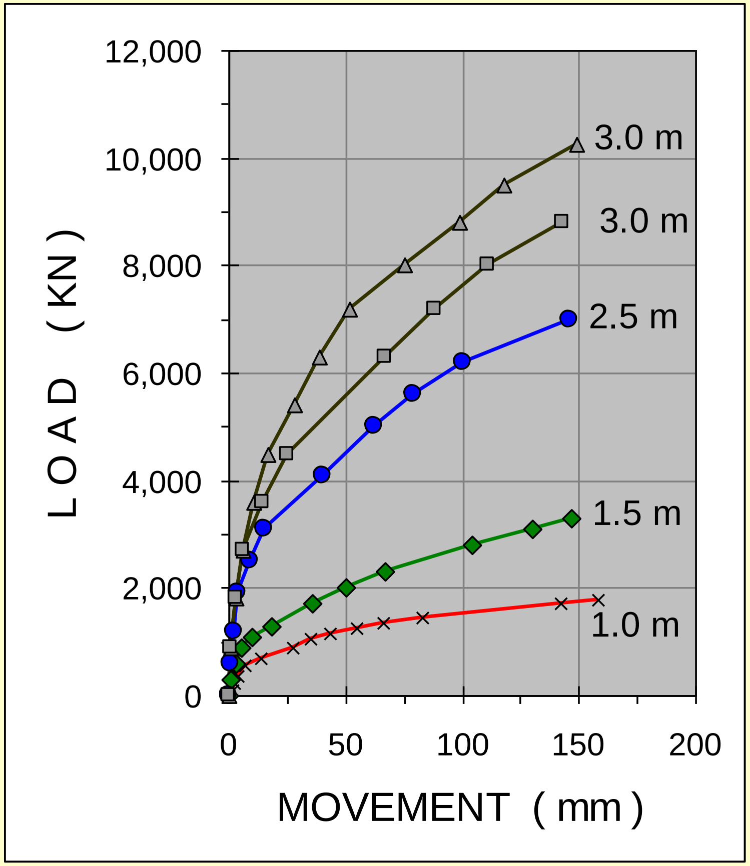
Why is it that nobody has realized that something must be wrong with the theory for the main factor, the N_q, to vary this much?

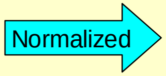
Let's compare to the reality?

Results of static loading tests on **0.25 m to 0.75 m** square **footings** in well graded **sand** (Data from Ismael, 1985)

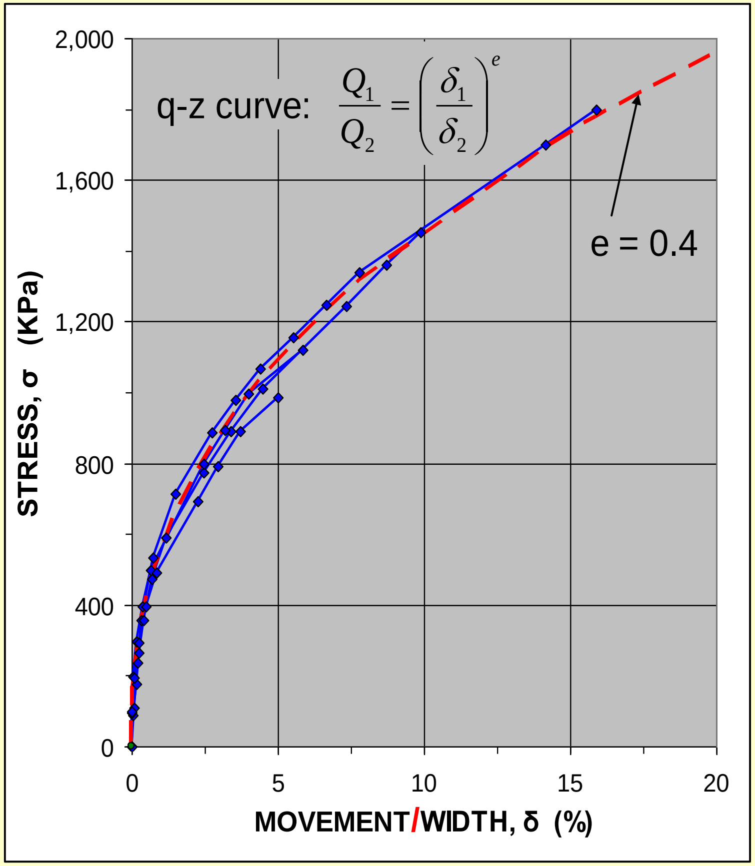


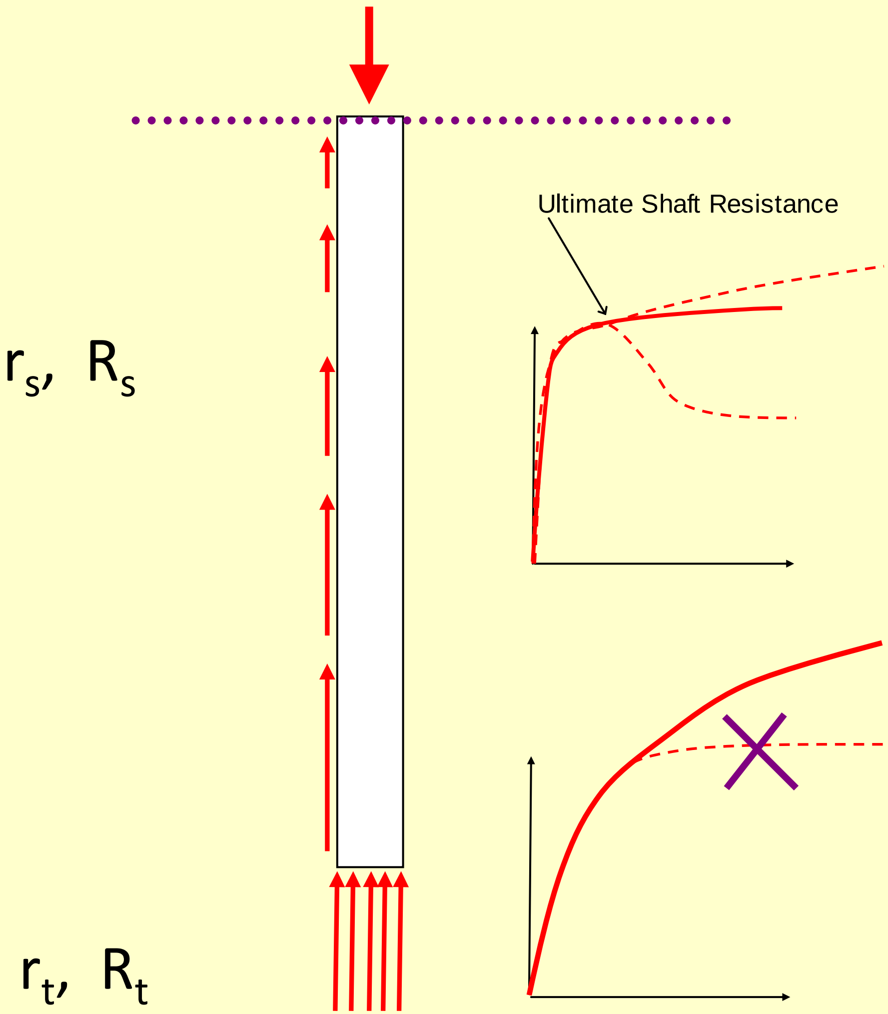
Load-Movement of Four Footings on Sand
Texas A&M University Experimental Site
J-L Briaud and R.M. Gibbens 1994,
ASCE GSP 41





The measured data normalized and with a fit of a q-z curve





Ultimate Toe Resistance does not exist other than as a definition of load at a certain movement

Ultimate Shaft Resistance can be a reality. An ultimate value can be determined. However, the required movement for a specific case can vary between a mm or two through 50 mm and beyond!

… , but Ultimate Toe Resistance can never be. Toe capacity is a myth!

Analysis Methods for Determining
Shaft Resistance, r_s

The Total Stress Method

The SPT Method

The CPT and CPTU Methods

The **Beta Method**

Total Stress Method

"Alpha analysis"

$$r_s = \tau_u \qquad [= \alpha \tau_u]$$

where r_s = unit shaft resistance

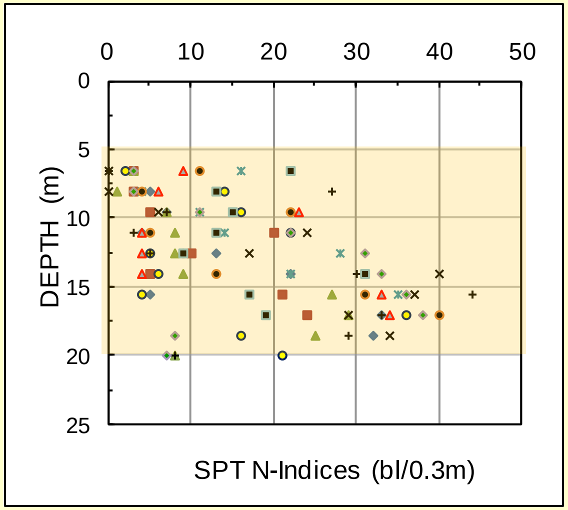
τ_u = undrained shear strength

α = reduction coefficient for τ_u > ≈100 KPa

The undrained shear strength can be obtained from unconfined compression tests, field vane shear tests, or, to be fancy, from consolidated, undrained triaxial tests. Or, better, back-calculated from the results of instrumented static loading tests. **However, if those tests indicate that the unit shaft resistance is constant with depth in a homogeneous soil, don't trust the records! Also, the analysis results would only fit a pile of the same embedment length as the test pile.**

The SPT Method

Meyerhof (1976)



Which value would you pick for use in calculating pile capacity?

$$r_s = n\,N\,D$$

where r_s = ultimate unit **shaft** resistance (N/m^3)

n = a coefficient

N = average N-index along the pile shaft (taken as a pure number)

D = embedment depth

n = $2{\cdot}10^3$ for driven piles and $1{\cdot}10^3$ for bored piles (N/m^3)

[English units: 0.02 for driven piles and 0.01 for bored piles (t/ft^3)]

For unit toe resistance, r_t, Meyerhof's method applies the N-index at the pile toe times a toe coefficient = $400{\cdot}10^3$ for driven piles and $120{\cdot}10^3$ for bored piles (N/m^3)

[English units: n = 4 for driven piles and n = 1 for bored piles (t/ft^3)]

The SPT Method

Decourt (1988; 1995)

$$r_s = \alpha(2.8N + 10)\,D$$

where r_s = ultimate unit **shaft** resistance (N/m³)

α = a coefficient

N = average N-index along the pile shaft (taken as a pure number)

D = embedment depth

Shaft Coefficient α

Soil Type Type	Displacement Piles	Non-Displacement Piles
Sand	$1 \cdot 10^3$	$0.6 \cdot 10^3$
Sandy Silt	$1 \cdot 10^3$	$0.5 \cdot 10^3$
Clayey Silt	$1 \cdot 10^3$	$1 \cdot 10^3$
Clay	$1 \cdot 10^3$	$1 \cdot 10^3$

For unit toe resistance in sand, Decourt's method applies the N-index at the pile toe times a toe coefficient = $325 \cdot 10^3$ for driven piles and $165 \cdot 10^3$ for bored piles (N/m³)

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CPT and CPTU Methods

for Calculating the Ultimate

Resistance (Capacity) of a Pile

Schmertmann and Nottingham (1975 and 1978)

deRuiter and Beringen (1979)

Meyerhof (1976)

LCPC, Bustamante and Gianeselli (1982)

ICP, Jardine, Chow, Overy, and Standing (2005)

Eslami and Fellenius (1997)

The CPT and CPTU Methods

Schmertmann and Nottingham (1975 and 1978)

$$r_t = C_{OCR}\, q_{ca}$$

$$r_s = K_f\, f_s$$ CLAY and SAND

$$r_s = K_c\, q_c$$ SAND (alternative)

where r_t = pile unit toe resistance (**<15 MPa**)

C_{OCR} = correlation coefficient governed by the overconsolidation ratio, OCR, of the soil

q_{ca} = arithmetic average of q_c in an influence zone*)

K_f = a coefficient depends on pile shape and material, cone type, and embedment ratio. In sand, the coefficient ranges from 0.8 through 2.0, and, in clay, it ranges from 0.2 through 1.25.

K_c = a dimensionless coefficient; a function of the pile type, ranging from 0.8 % through 1.8 %

q_c = cone resistance (total; uncorrected for pore pressure on cone shoulder)

*) The Influence zone is 8b above and 4b below pile toe

Eslami and Fellenius (1997)

$$r_t = C_t\ q_{Eg}$$

$$r_s = C_s\ q_E$$

$$C_t = \frac{1}{3b}$$ b in metre

$$C_t = \frac{12}{b}$$ b in inch

b = pile diameter

r_t = pile unit toe resistance

C_t = toe correlation coefficient (toe adjustment factor)—equal to unity in most cases

q_{Eg} = **geometric** average of the cone stress over the influence*) zone after correction for pore pressure on the shoulder and adjustment to "effective" stress

r_s = pile unit shaft resistance

C_s = shaft correlation coefficient, which is a function of soil type determined **from the CPT/CPTU soil profiling chart**

q_E = cone stress after correction for pore pressure on the cone shoulder and adjustment to "effective" stress

*) The Influence zone is 8b above and 4b below pile toe

Shaft Correlation Coefficient

Soil Type**)	**C_s**
Soft sensitive soils	8.0 %
Clay	5.0 %
Stiff clay and Clay and silt mixture	2.5 %
Sandy silt and silt	1.5 %
Fine sand and silty sand	1.0 %
Sand to sandy gravel	0.4 %

) **determined directly from the CPTU soil profiling

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Pile Capacity or, rather, **Load-Transfer** follows principles of effective stress and is best analyzed using the **Beta method**

Shaft Resistance in **Sand and in Clay** — Beta-method

Unit Shaft Resistance, r_s

$$r_s = \beta \sigma'_v$$

$$r_s = \tan\phi' K_s \sigma'_v$$

where
- r_s = unit shaft resistance
- β = Bjerrum-Burland coefficient
- σ'_v = effective overburden stress
- K_s = earth stress ratio = σ'_h / σ'_v

Approximate Range of Beta-coefficients

SOIL TYPE	Phi	Beta
Clay	25 - 30	0.20 - 0.35
Silt	28 - 34	0.25 - 0.50
Sand	32 - 40	0.30 - 0.90
Gravel	35 - 45	0.35 - 0.80

0.05 - 0.80+ !

These ranges are typical values found in some cases. In any given case, actual values may deviate considerably from those in the table.

Practice is to apply different values to driven as opposed to bored piles, but ....

Total Resistance ("Capacity"); Load Distribution

$$Q_{ult} = R_s + R_t$$

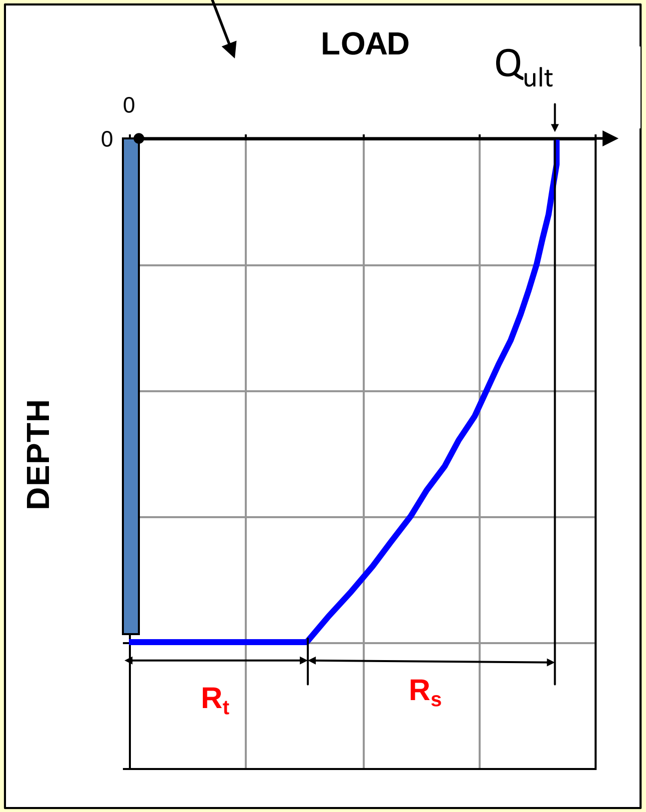
$Q_{ult} = Q_u$ = Ultimate resistance = Capacity

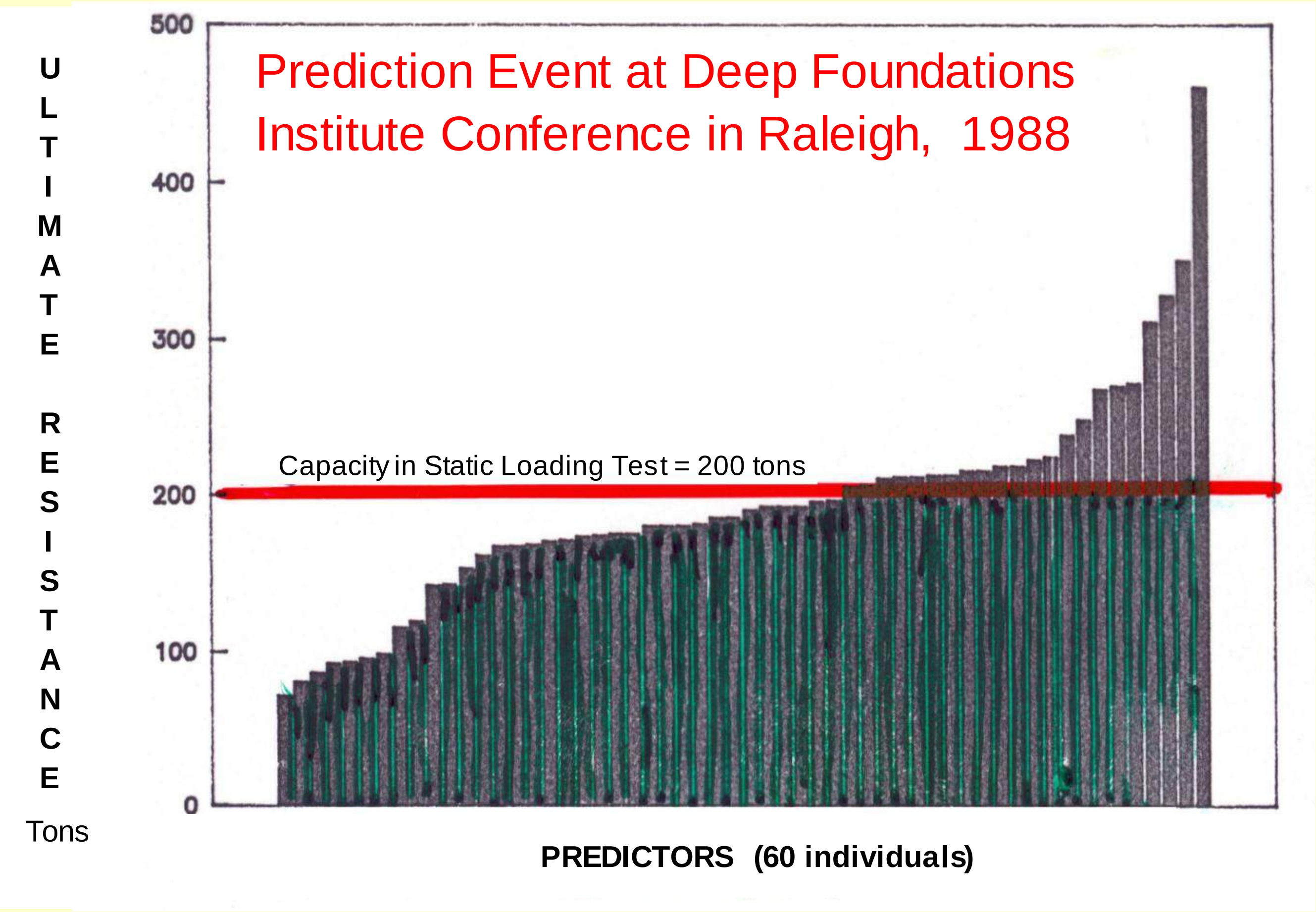
R_s = Shaft resistance

R_t = Toe resistance

$$Q_z = Q_u - \int A_s \beta \sigma_z' \, dz \; = \; Q_u \; - \; R_s$$

Effective stress–Beta-analysis–is the method closest to the real response of a pile to an imposed load



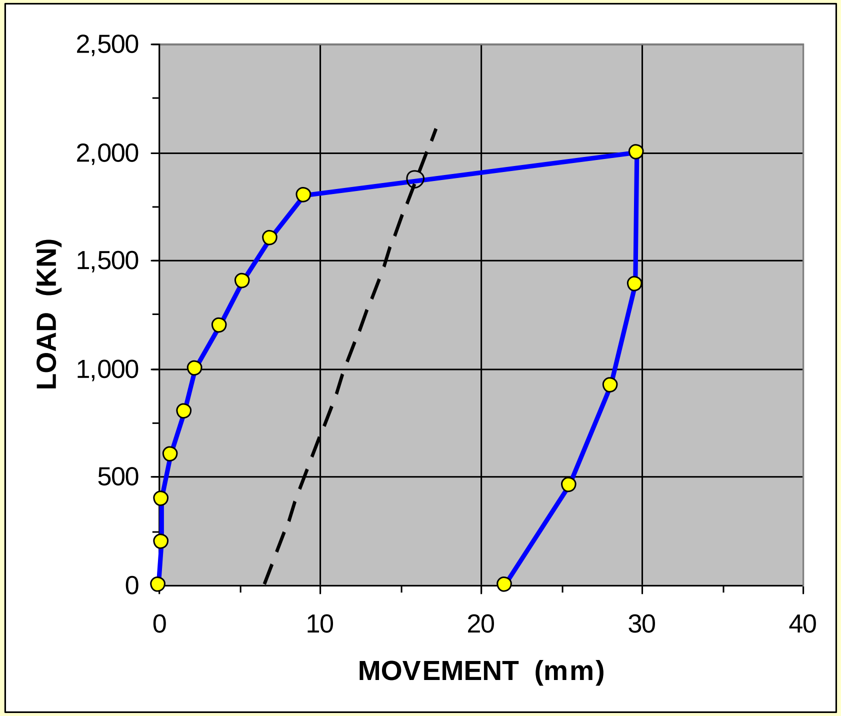


44 ft embedment, 12.5 inch square precast concrete driven through compact silt and into dense sand

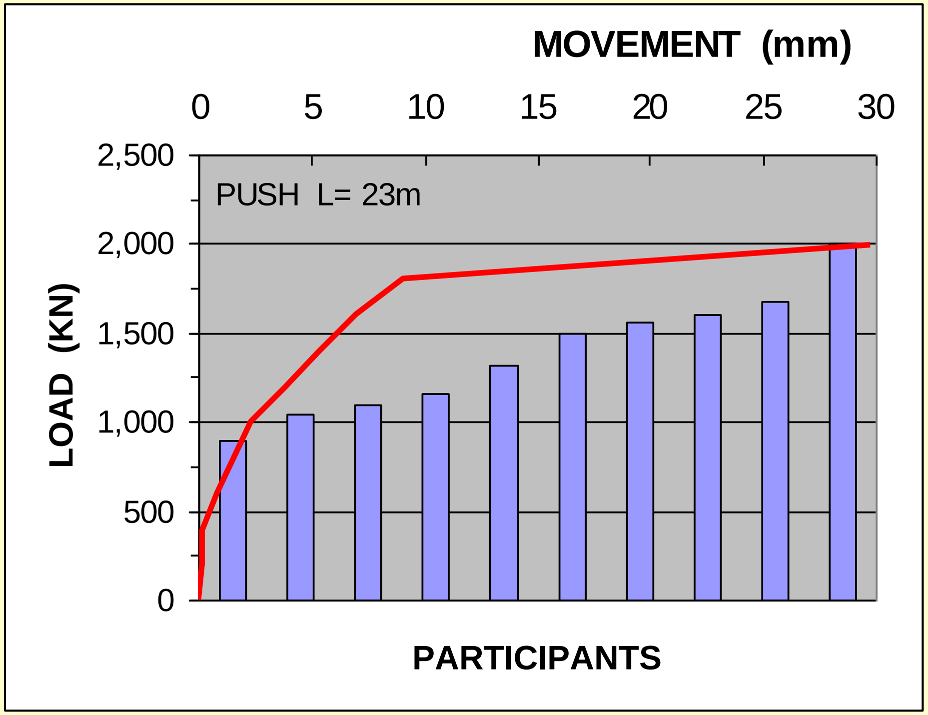
Brazil 2004: Bored pile (Omega screw pile) 23 m long, 310 mm diameter

Static Loading Test on a 23 m 310 mm bored pile

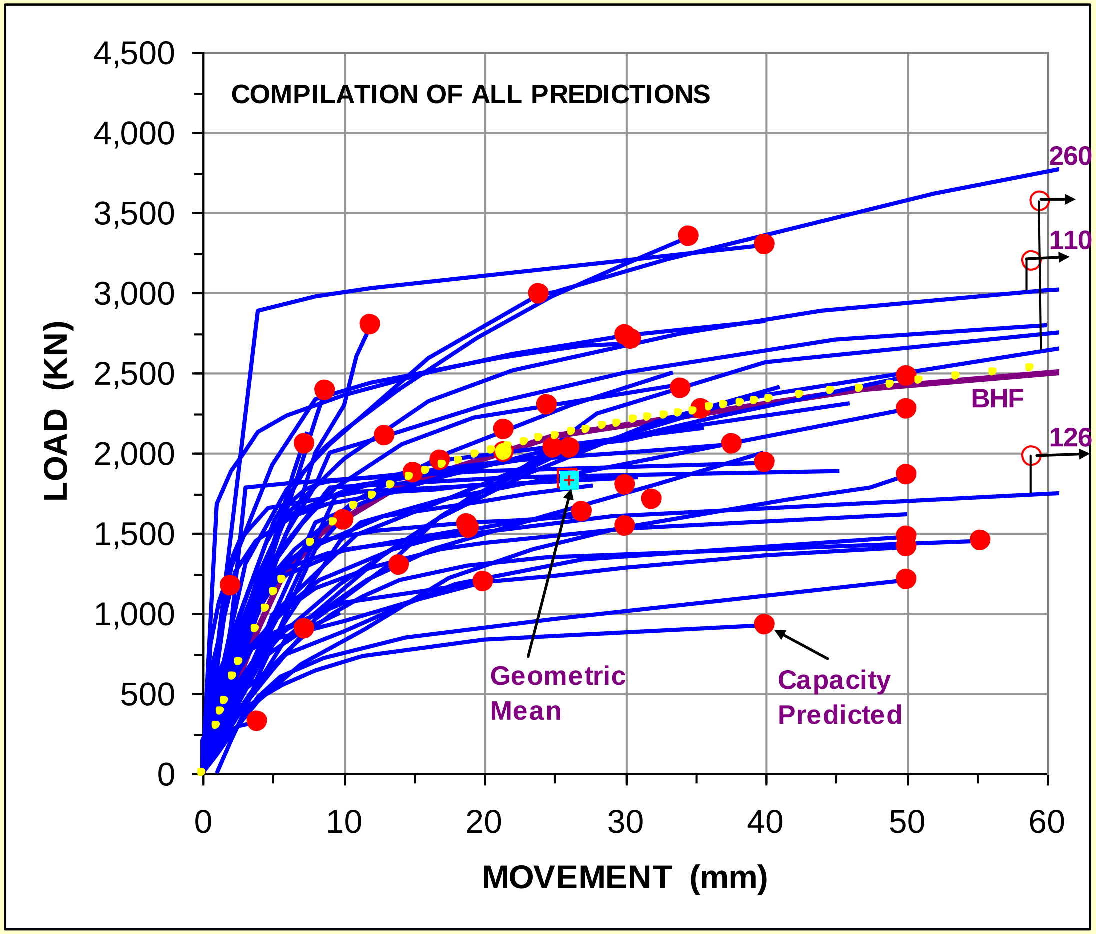
Load-Movement Response



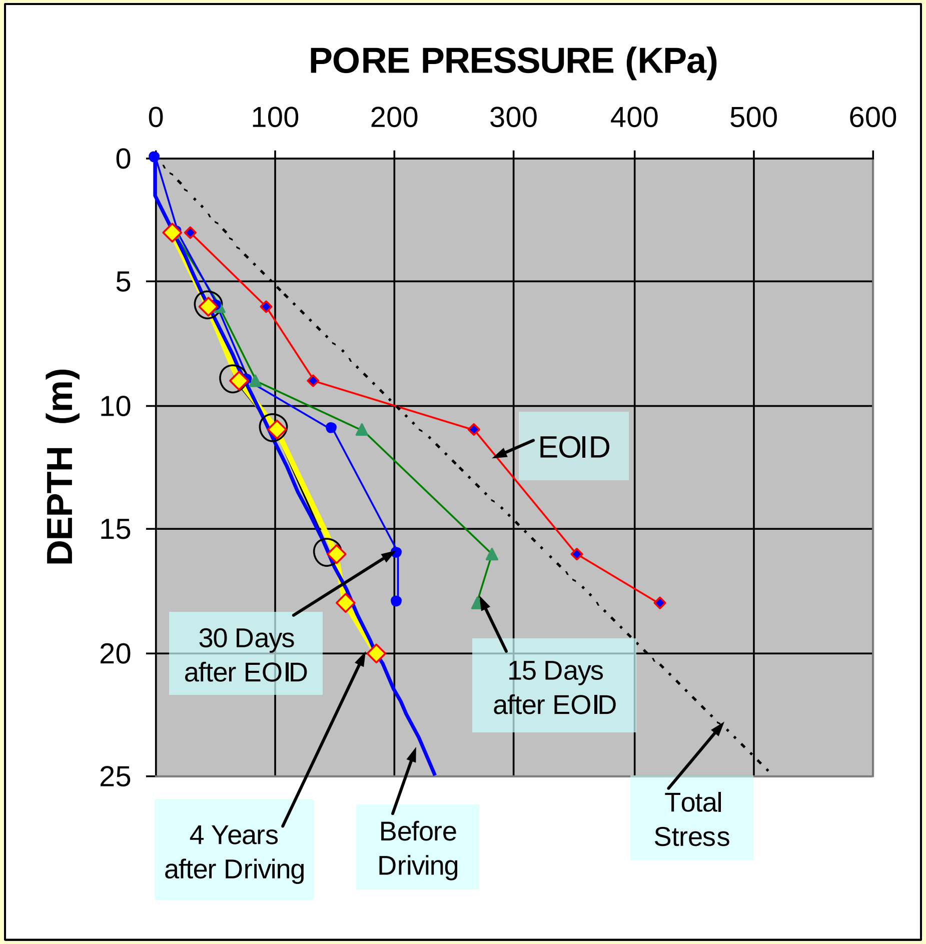
Prediction Compilation



Compilation of predicted load-movement curves and capacities Bolivia 2013

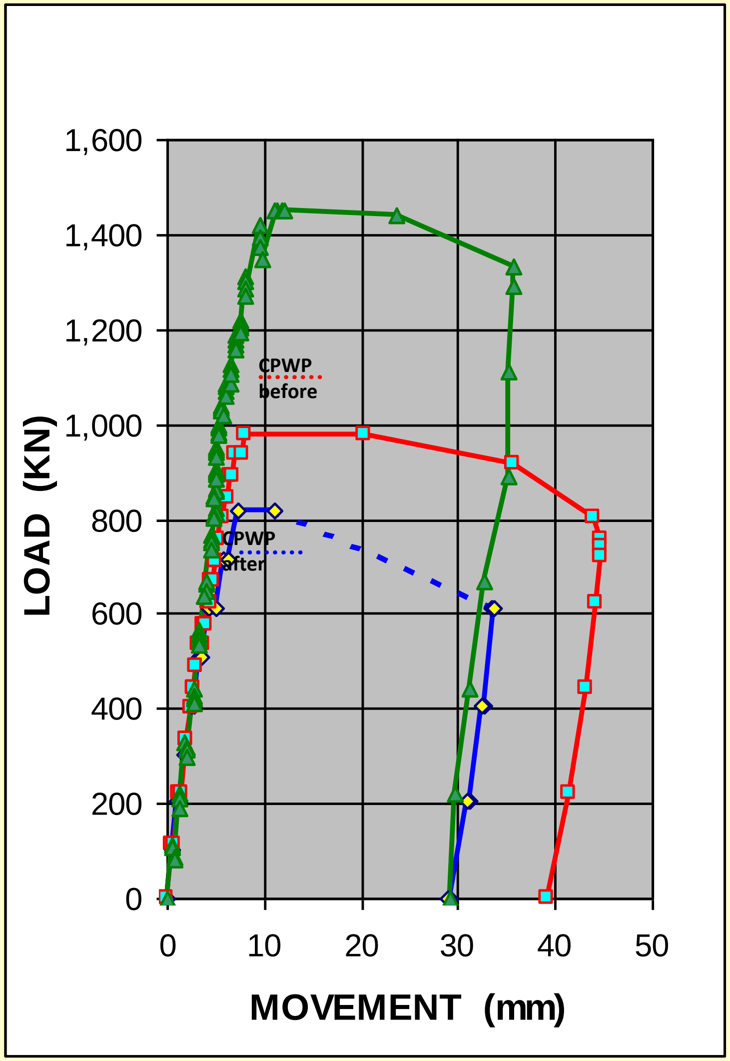


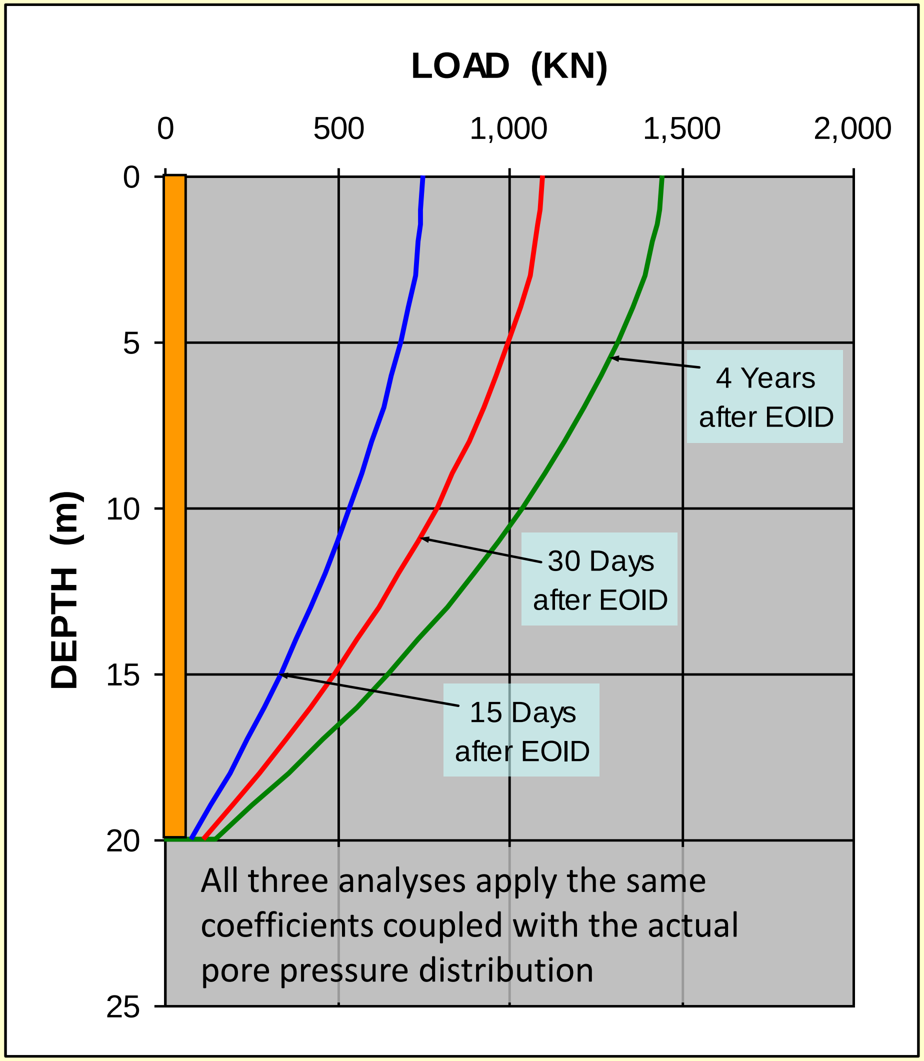
Pore Pressure Dissipation



Paddle River, Alberta,
Canada (Fellenius 2008)

Effective Stress Analysis





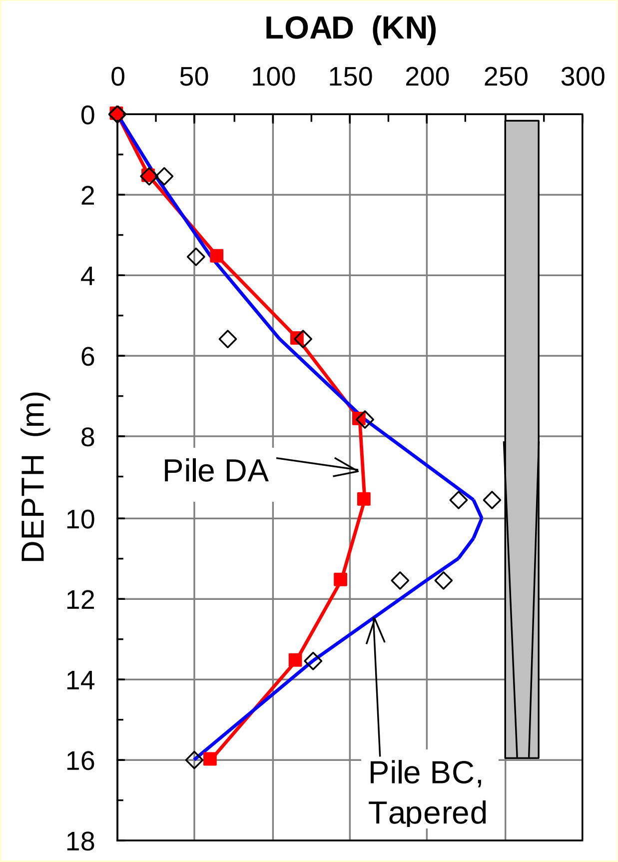
Load Distributions—Measured in the static loading tests and fitted to UniPile analysis

Paddle River, Alberta, Canada (Fellenius 2008)

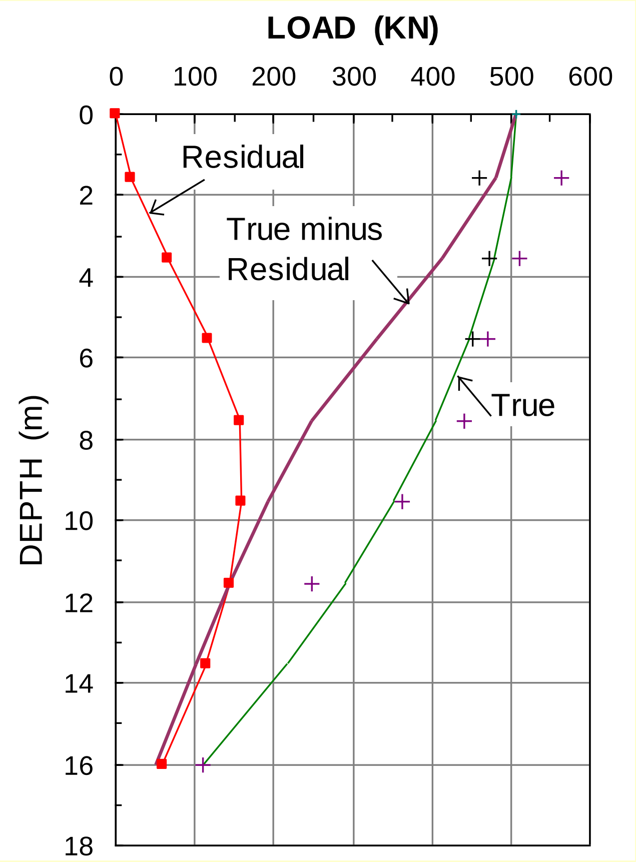
If we want to know the load distribution, we can measure it. But, what we measure is the increase of load in the pile due to the load applied to the pile head. What about the load in the pile that was there before we started the test?

That is, the Residual load.

Example from Gregersen et al., 1973

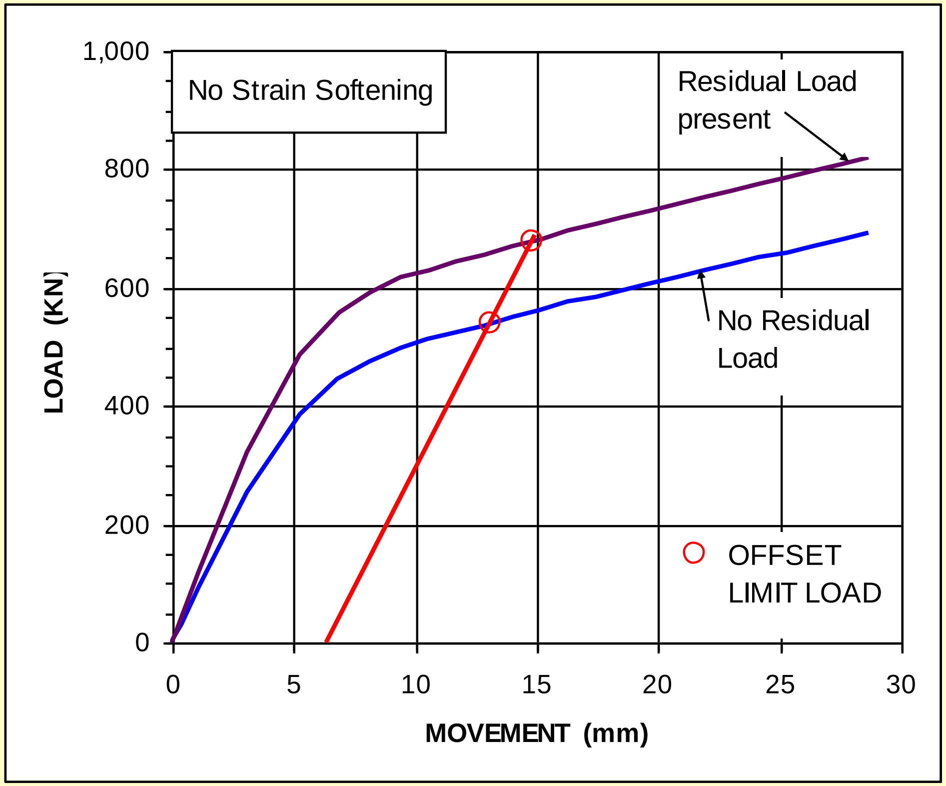


Distribution of residual load in Piles DA and BC
before start of the loading test



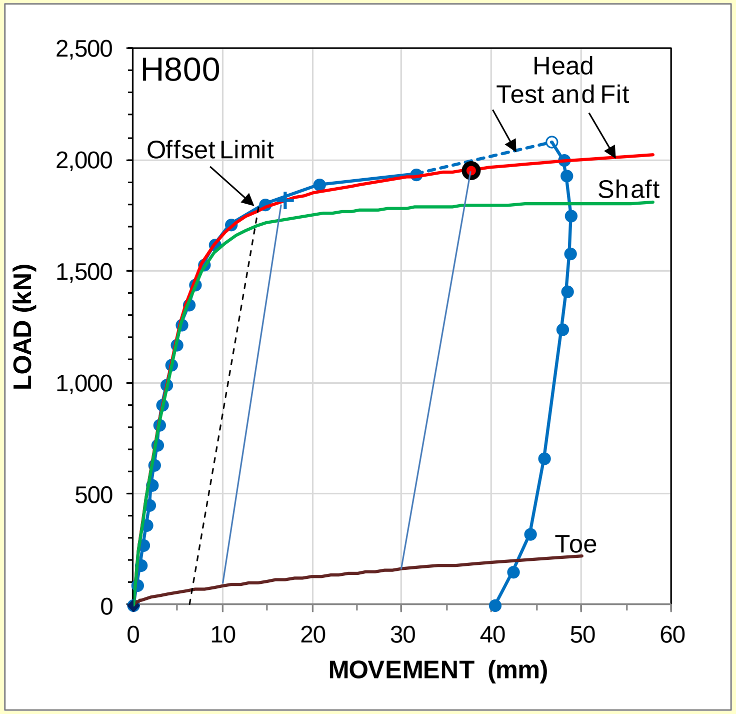
Load and resistance in Pile DA
for the maximum test load

Presence of residual load is not just of academic interest

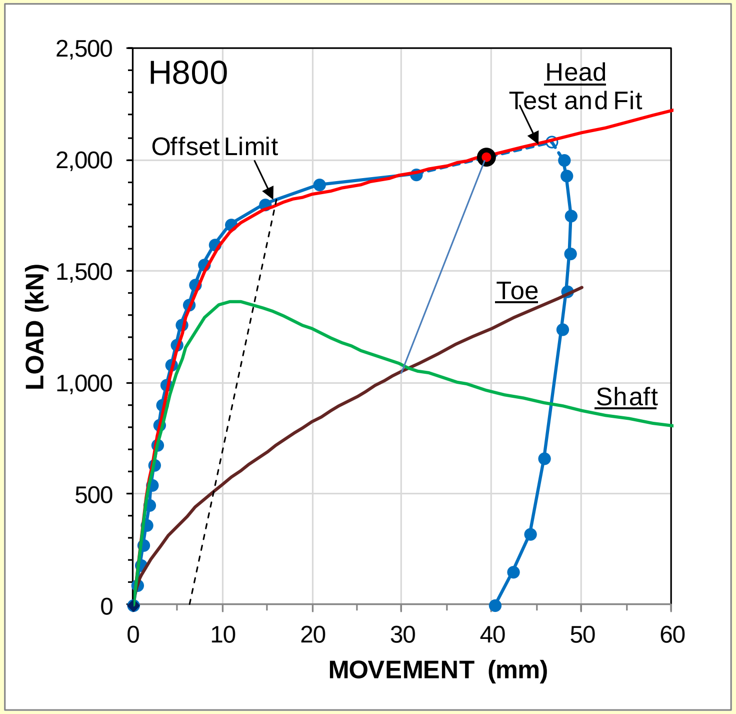


Separation of shaft and toe resistances

According to the Meyerhof et al.

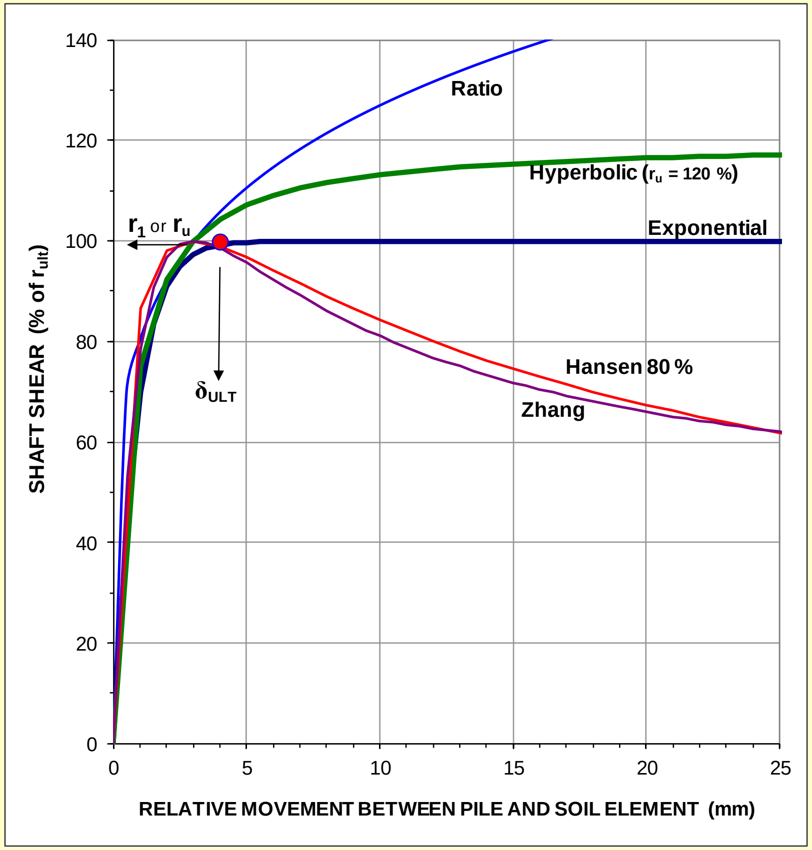


More likely



Meyerhof, G.G., Brown, J.D., and Mouland, G.D., 1981.
Predictions of friction capacity in a till.
Proceedings of the ICSMFE, Stockholm, June 15-19, Vol. 2, pp. 777-780

t-z and q-z functions



Strain-hardening

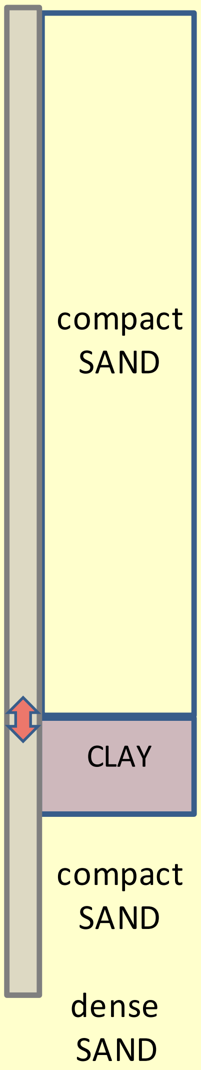
Elastic-plastic

Strain-softening

Note, the diagram assumes that all curves pass through the point for 100-% load and 5-mm movement. However, the movement can vary widely in a specific case .

Assigning applicable t-z and q-z functions is fundamental to the analysis and vital for determining pile response and achieving reliable design of piled foundations. Confidence in a design is obtained from back-analysis of results of static loading tests. Next is an example of such analysis

Analysis of the results of a bidirectional test on a 21 m long bored pile

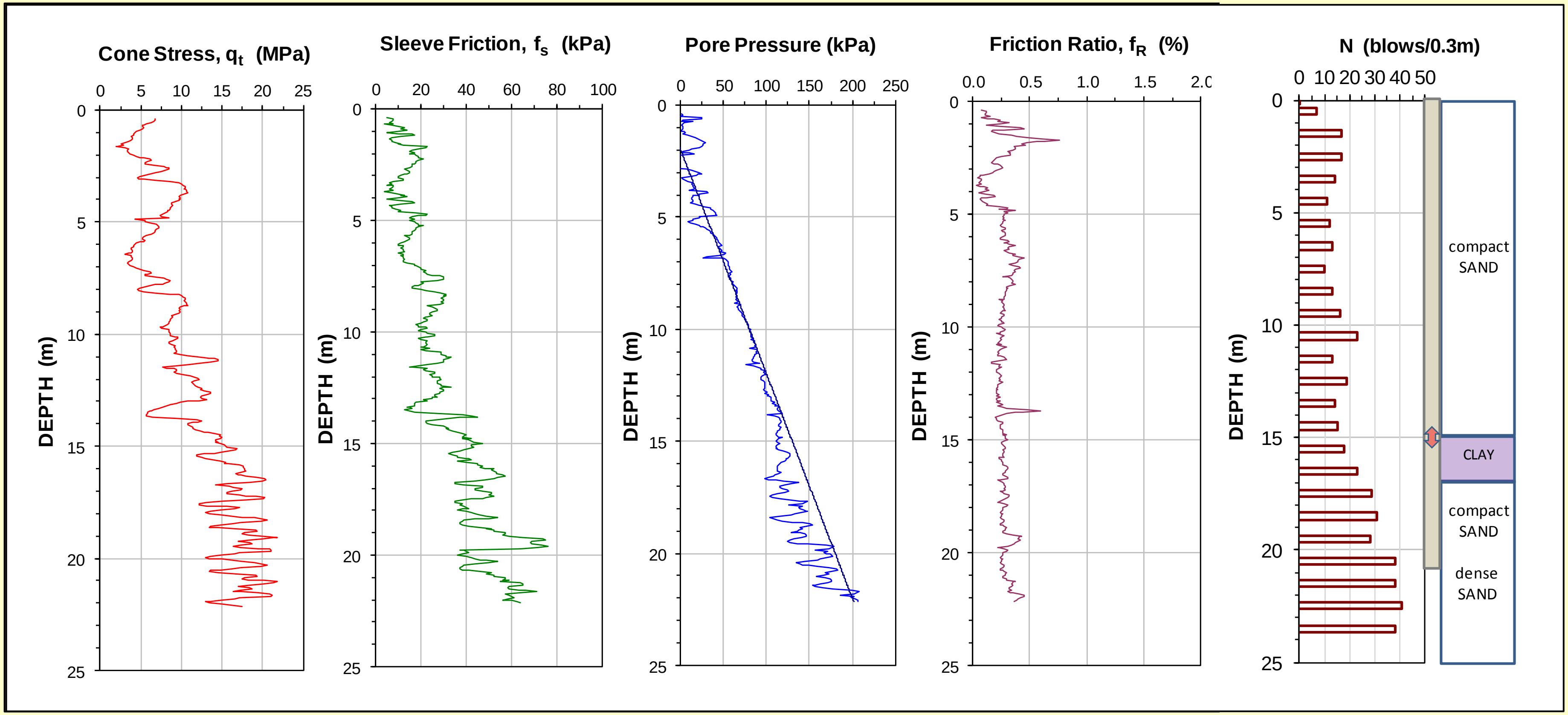


The sand becomes very dense at about 35 m depth

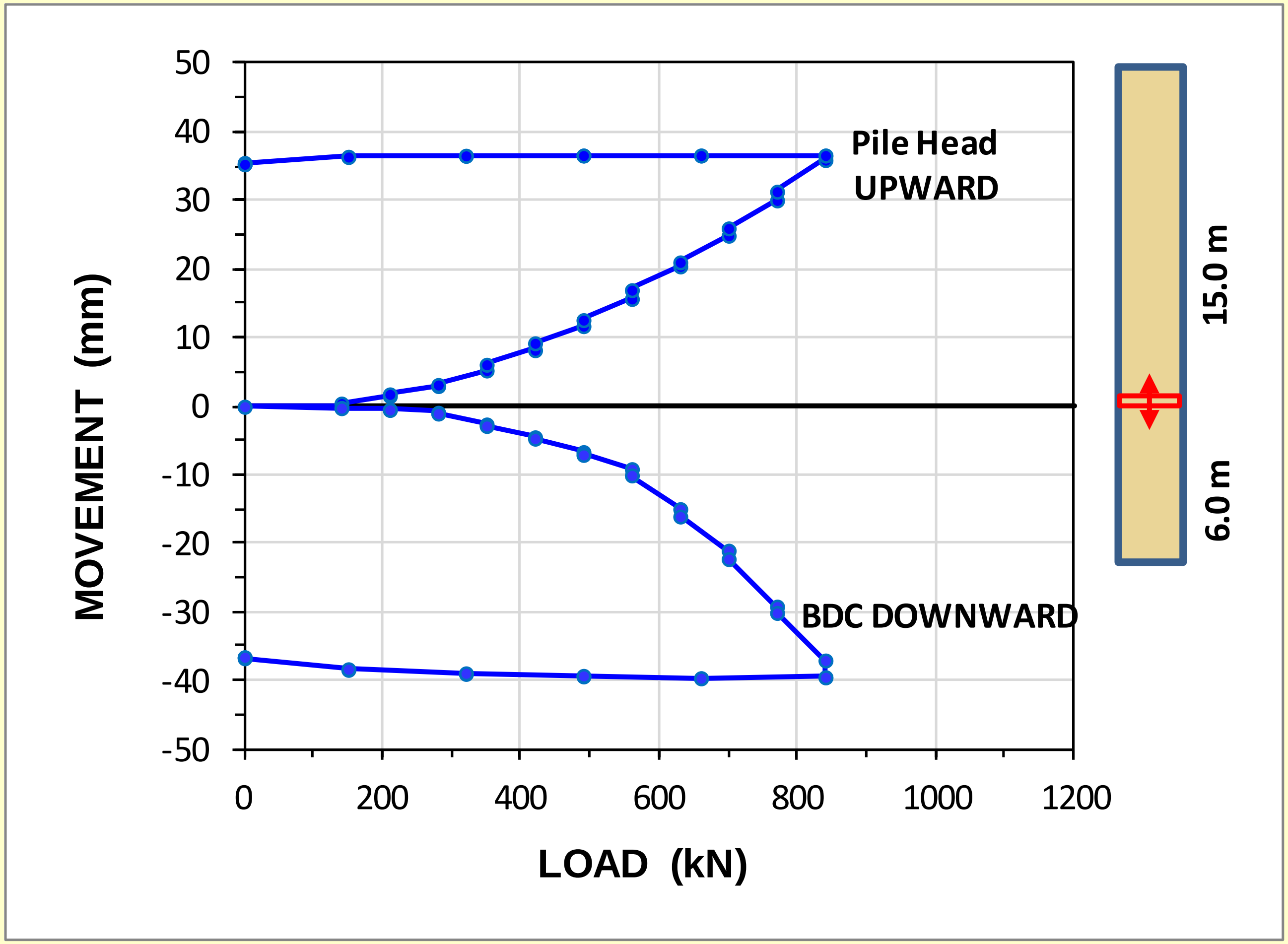
A bidirectional test was performed on a 500-mm diameter, 21 m long, bored pile constructed through compact to dense sand by driving a steel-pipe to full depth, cleaning out the pipe, while keeping the pipe filled with betonite slurry, withdrawing the pipe, and, finally, tremie-replacing the slurry with concrete. The bidirectional cell (BDC) was attached to the reinforcing cage inserted into the fresh concrete. The BDC was placed at 15 m depth below the ground surface.

The pile will be one a group of 16 piles (4 rows by 4 columns) installed at a 4-diameter center-to-center distance. Each pile is assigned a working load of 1,000 kN.

The soil profile determined by CPTU and SPT

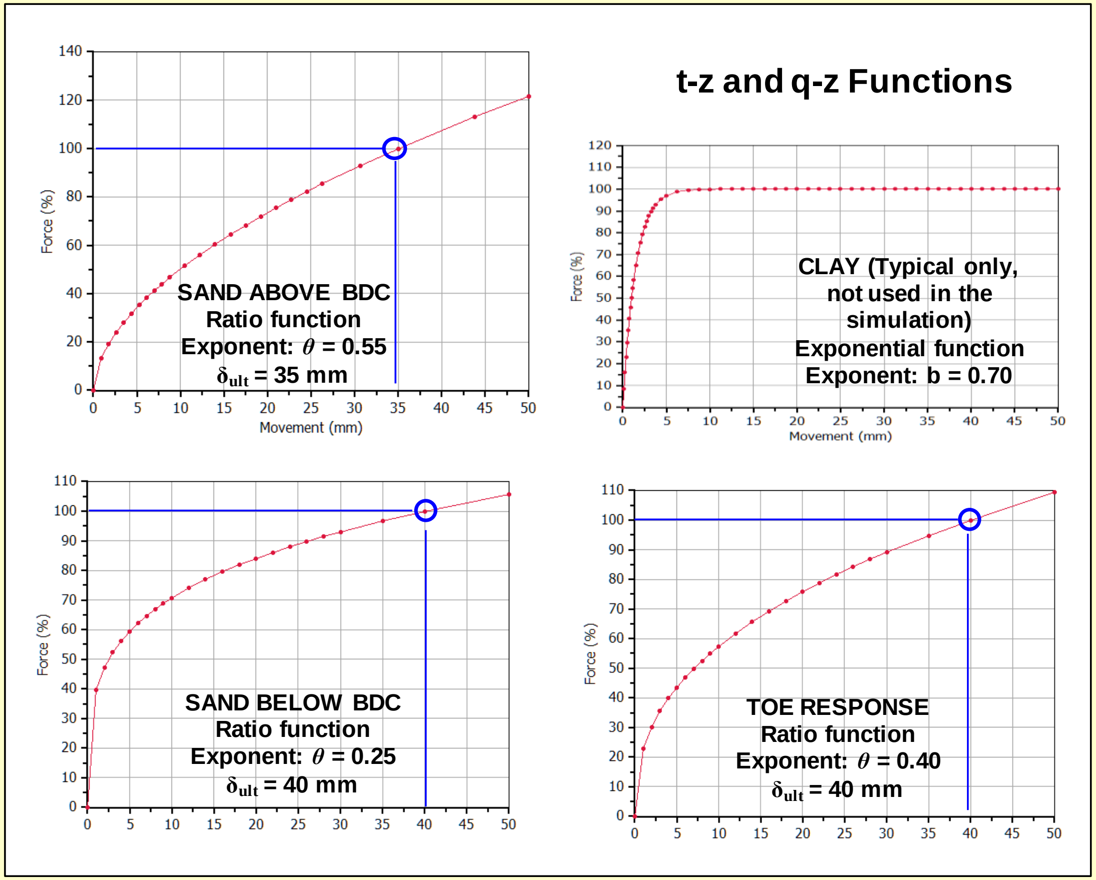


The results of the bidirectional test



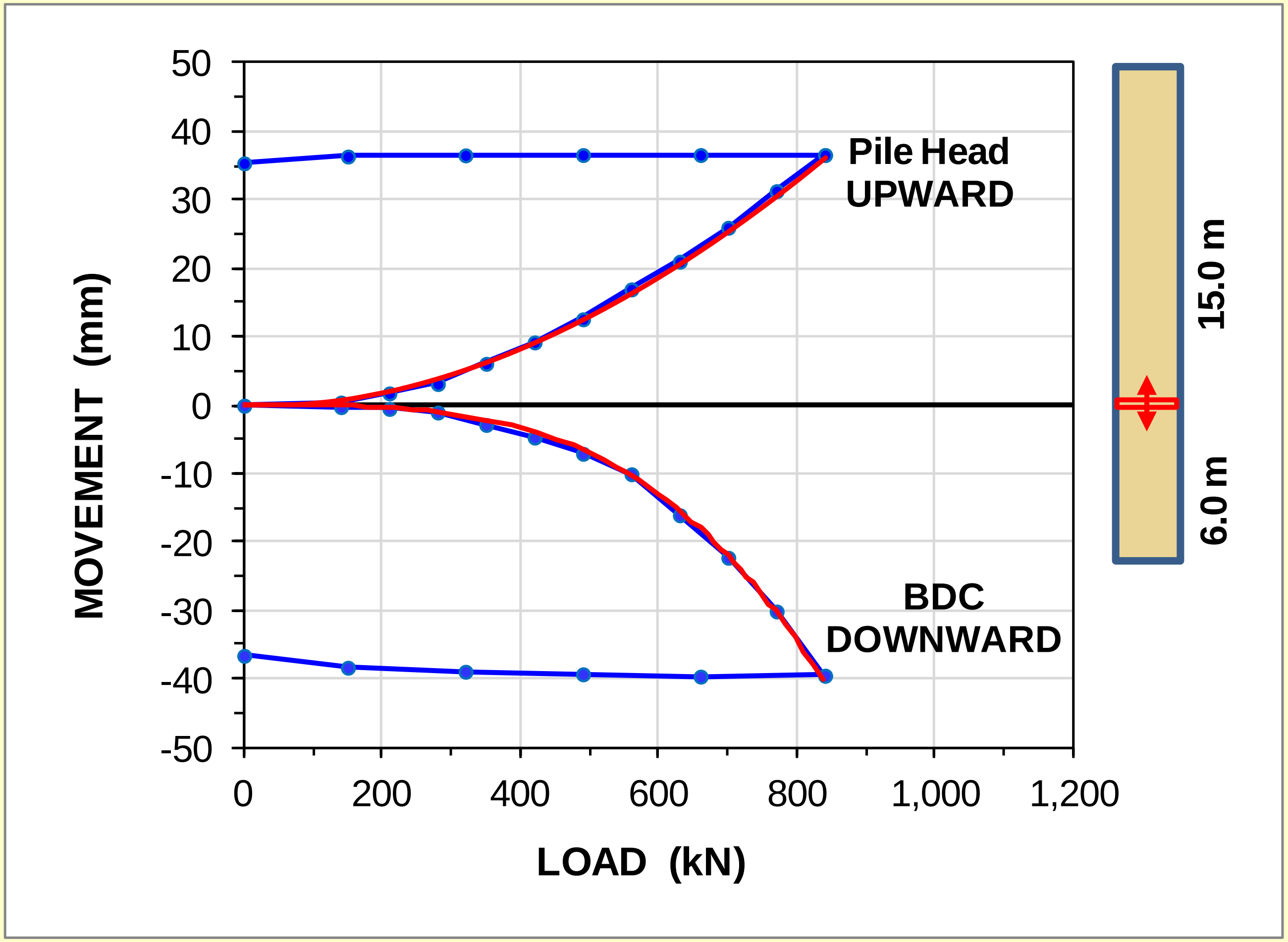
Acknowledgment: The bidirectional test data are courtesy of Arcos Egenharia de Solos Ltda., Belo Horizonte, Brazil.

To fit a simulation of the test to the results, first input is the effective stress parameter (ß) that returns the maximum measured upward load (840 kN), which was measured at the maximum upward movement (35 mm). Then, “promising” t-z curves are tried until one is obtained that, for a specific coefficient returns a fit to the measured upward curve. Then, for the downward fit, t-z and q-z curves have to be tried until a fit of the downward load (840 kN) and the downward movement (40 mm) is obtained.

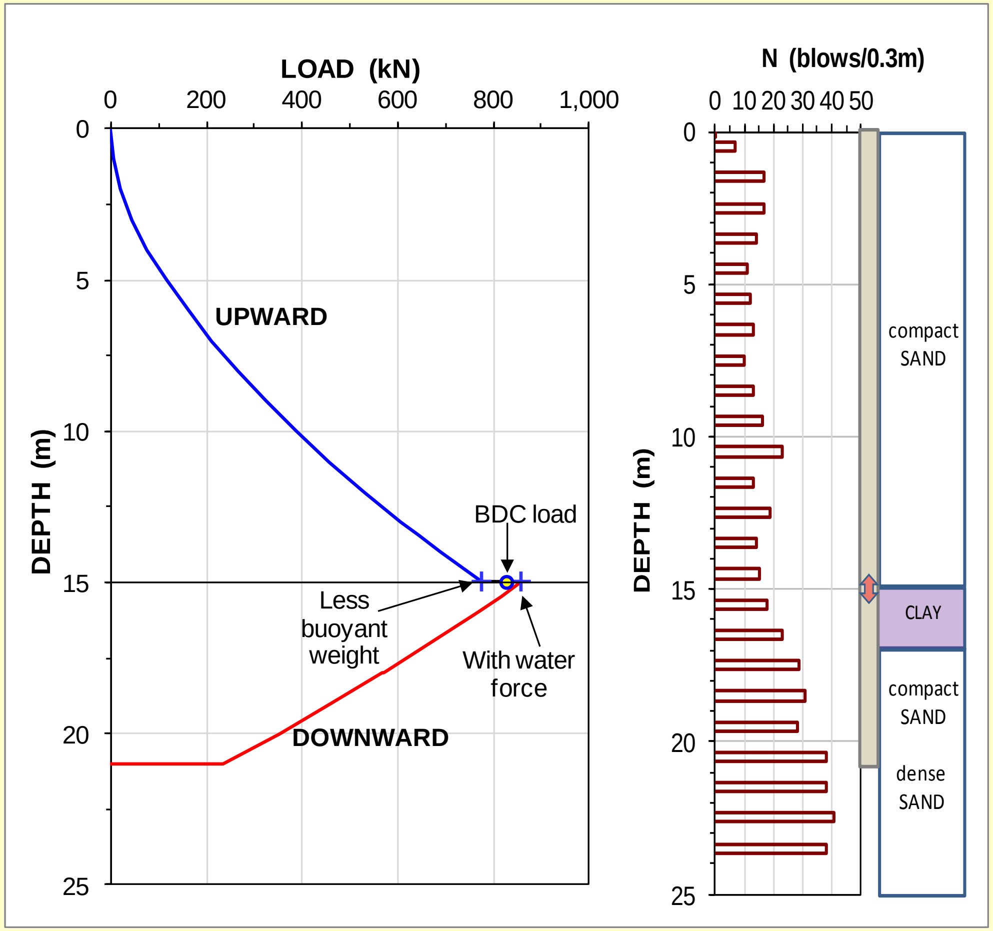


Usually for large movements, as in the example case, the t-z functions show a elastic-plastic response. However, for the example case , no such assumption fitted the results. In fact, the best fit was obtained with the Ratio Function for the entire length of the pile shaft.

The final fit of simulated curves to the measured

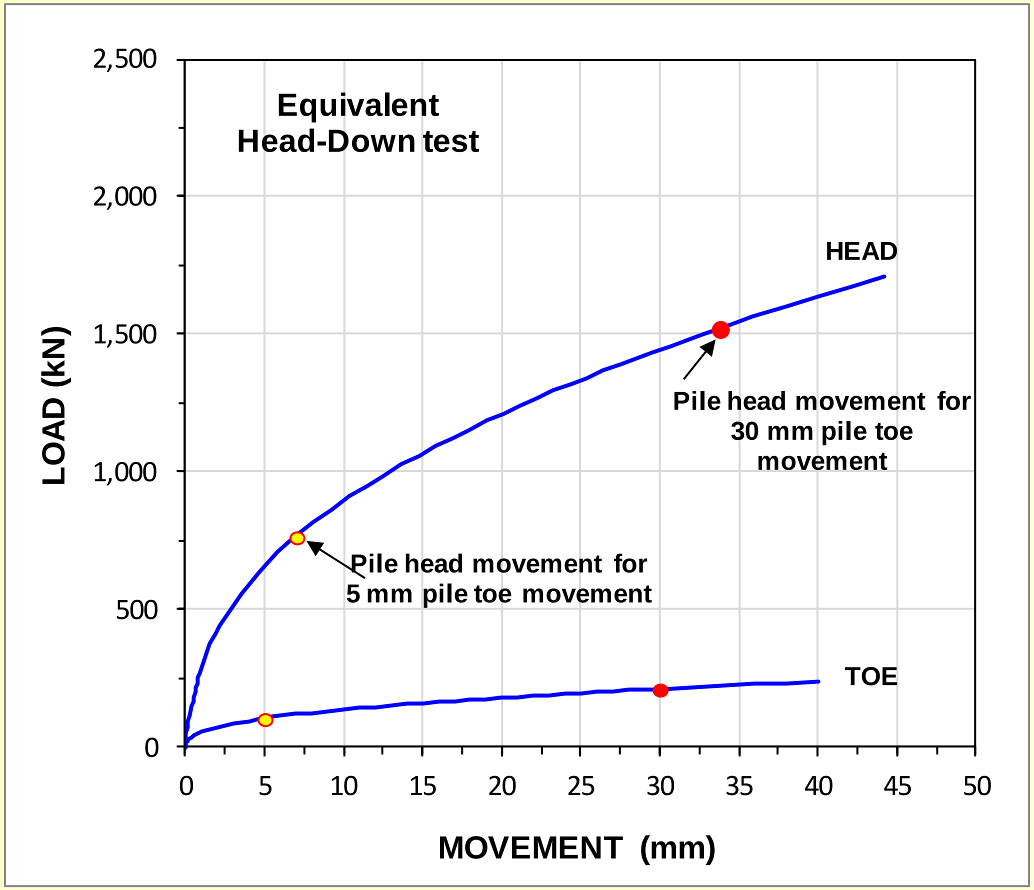


The test pile was not instrumented. Had it been, the load distribution of the bidirectional test as determined from the gage records, would have served to further detail the evaluation results. Note the below adjustment of the BDC load for the buoyant weight (upward) of the pile and the added water force (downward).



The analysis results appear to suggest that the pile is affected by a filter cake along the shaft and probably also a reduced toe resistance due to debris having collected at the pile toe between final cleaning and the placing of the concrete.

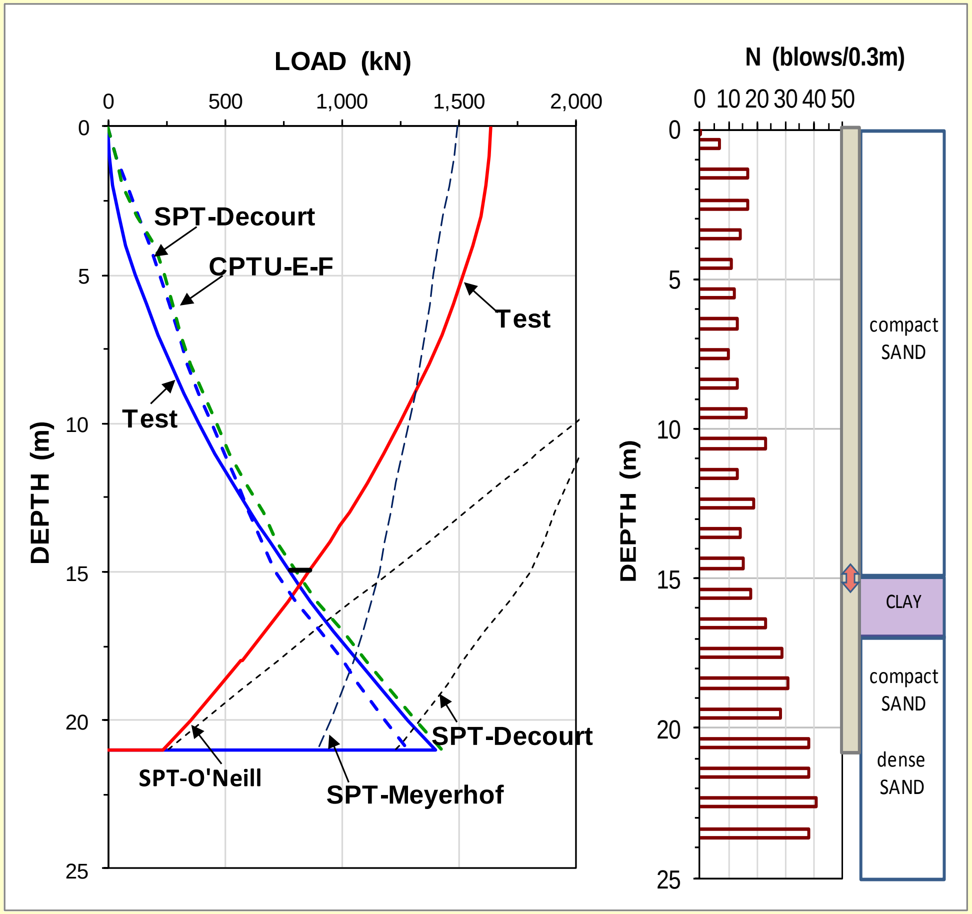
The final fit establishes the soil response and allows the equivalent head-down loading- test to be calculated



When there is no obvious point on the pile-head load-movement curve, the "capacity" of the pile has to be determined by one definition or other—there are dozens of such around. The first author prefers to define it as the pile-head load that resulted in a 30-mm pile toe movement. As to what safe working load to assign to a test, it often fits quite well to the pile head load that resulted in a 5-mm toe movement.

The most important aspect for a safe design is not the "capacity" found from the test data, but what the settlement of the structure supported by the pile(s) might be. How to calculate the settlement of a piled foundation is addressed a few slides down.

The final fit establishes also the equivalent head-down distributions of shaft resistance and equivalent head-down load distribution for the maximum load (and of any load in-between, for that matter). Load distributions have also been calculated from the SPT-indices using the Decourt, Meyerhof, and O'Neil-Reese methods, as well that from the Eslami-Fellenius CPTU-method.

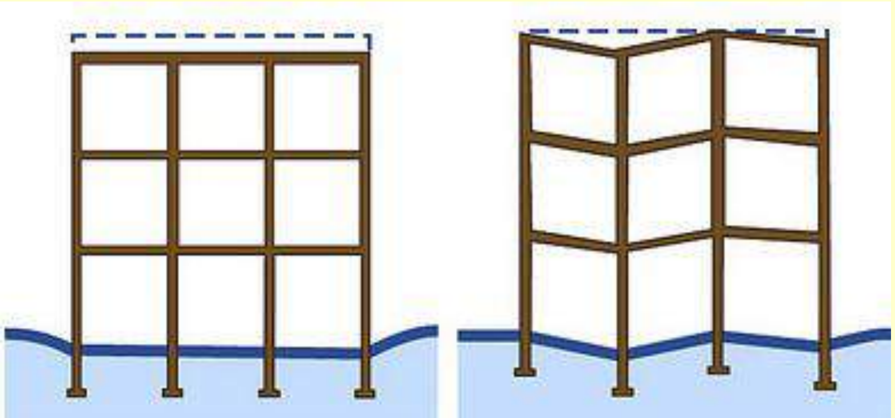


By fitting a UniPile simulation to the measured curves, we can determine all pertinent soil parameters, the applicable t-z and q-z functions, and the distribution of the equivalent head-down load-distribution. The results also enable making a comparison of the measured pile response to that calculated from the in-situ test methods.

However, capacity of the single pile is just one aspect of a piled foundation design. As mentioned, the key aspect is the foundation settlement.

Note, the analysis results suggest that the pile was more than usually affected by presence of a filter cake along the pile shaft and by some debris being present at the bottom of the shaft when the concrete was placed in the hole. An additional benefit of a UniPile analysis.

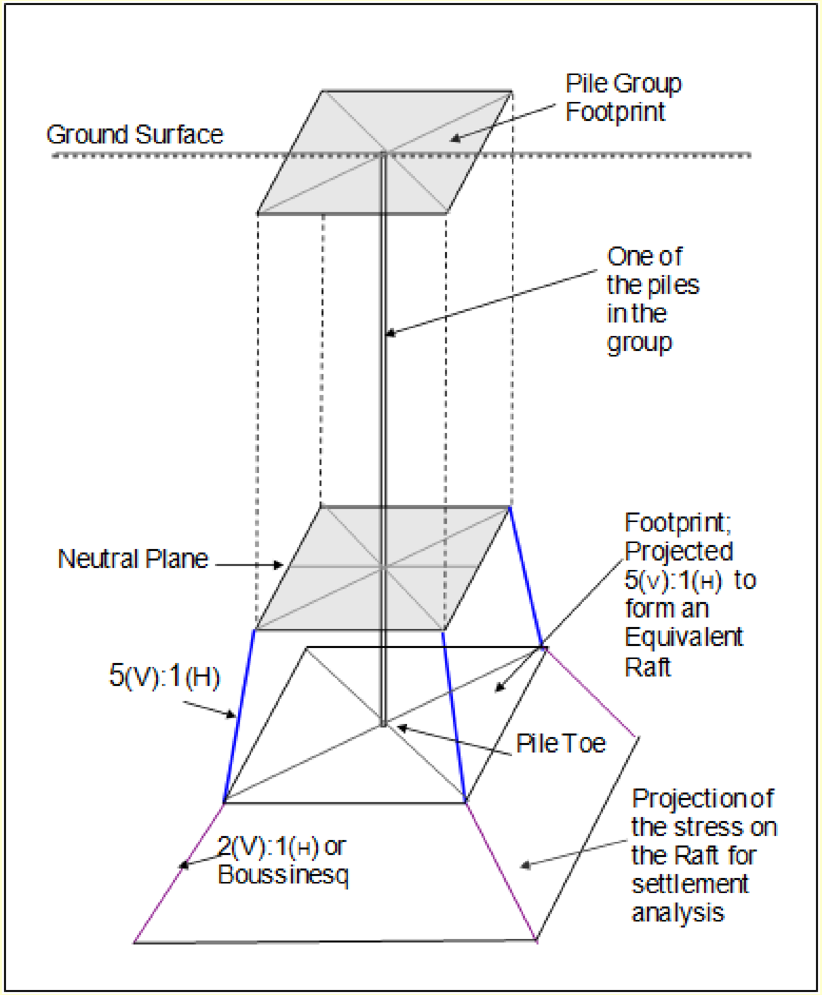
SETTLEMENT



Load placed on a pile causes downward movements of the pile head due to:

1. **'Elastic' compression** of the pile.
2. **Load transfer movement** -- the movement response of the soil.
3. **Settlement below the pile toe** due to the increase of stress in the soil. This is not important for single piles or small pile groups, but can be decisive for large pile groups, and where thick soil layers exist below the piles that receive increase of stress from sources other than the piles.

Settlement of a piled foundation



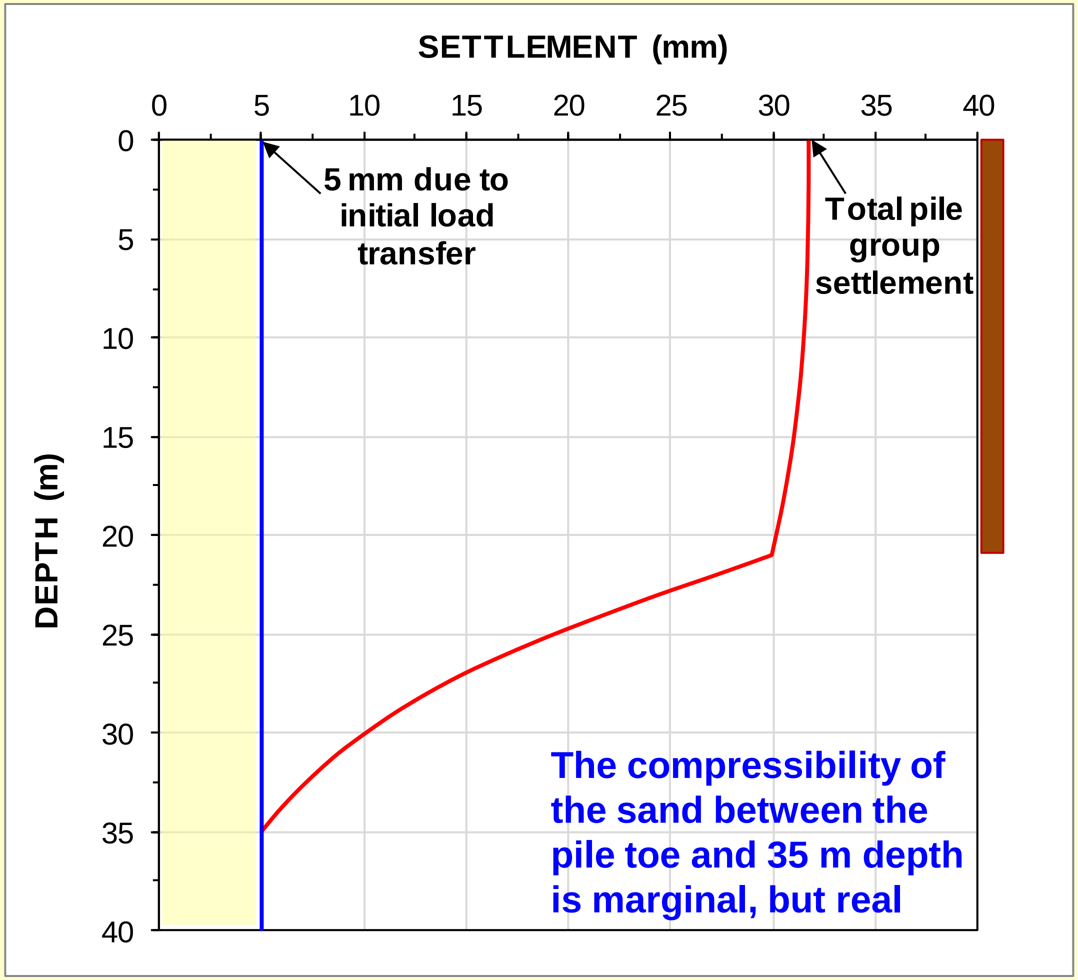
Distribution of stress for calculation of settlement

The depth to the Neutral Plane is 15.5 m. That depth is where the dead load applied to the pile starts to be distributed out into the soil.

The Unified Design Method developed by the first author considers this effect by widening the pile group foot-print area by a 5(V):1(H) from the N.P to the pile toe into an "Equivalent Raft" and applying the dead load to the raft.

Many other, very similar "Equivalent-Raft" approaches to calculating settlement of piled foundation are common in the industry. UniPile can also perform any such analysis as per the User preference and input.

The pile group (piled foundation) settlement as calculated by UniPile



For settlement calculations that include aspects of time, i.e., consolidation and secondary compression, the analysis is best performed in UniSettle, UniPile's "companion".

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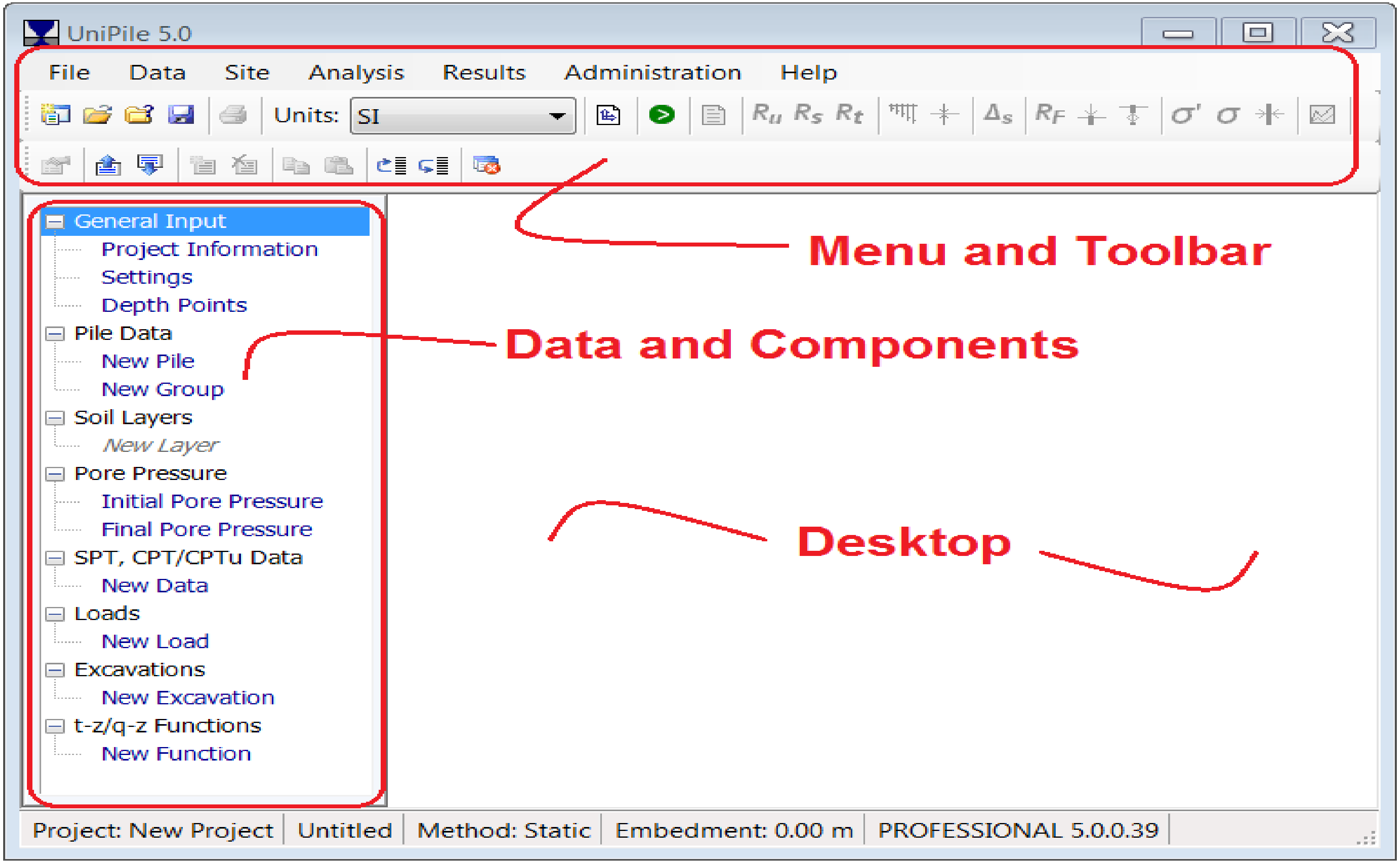
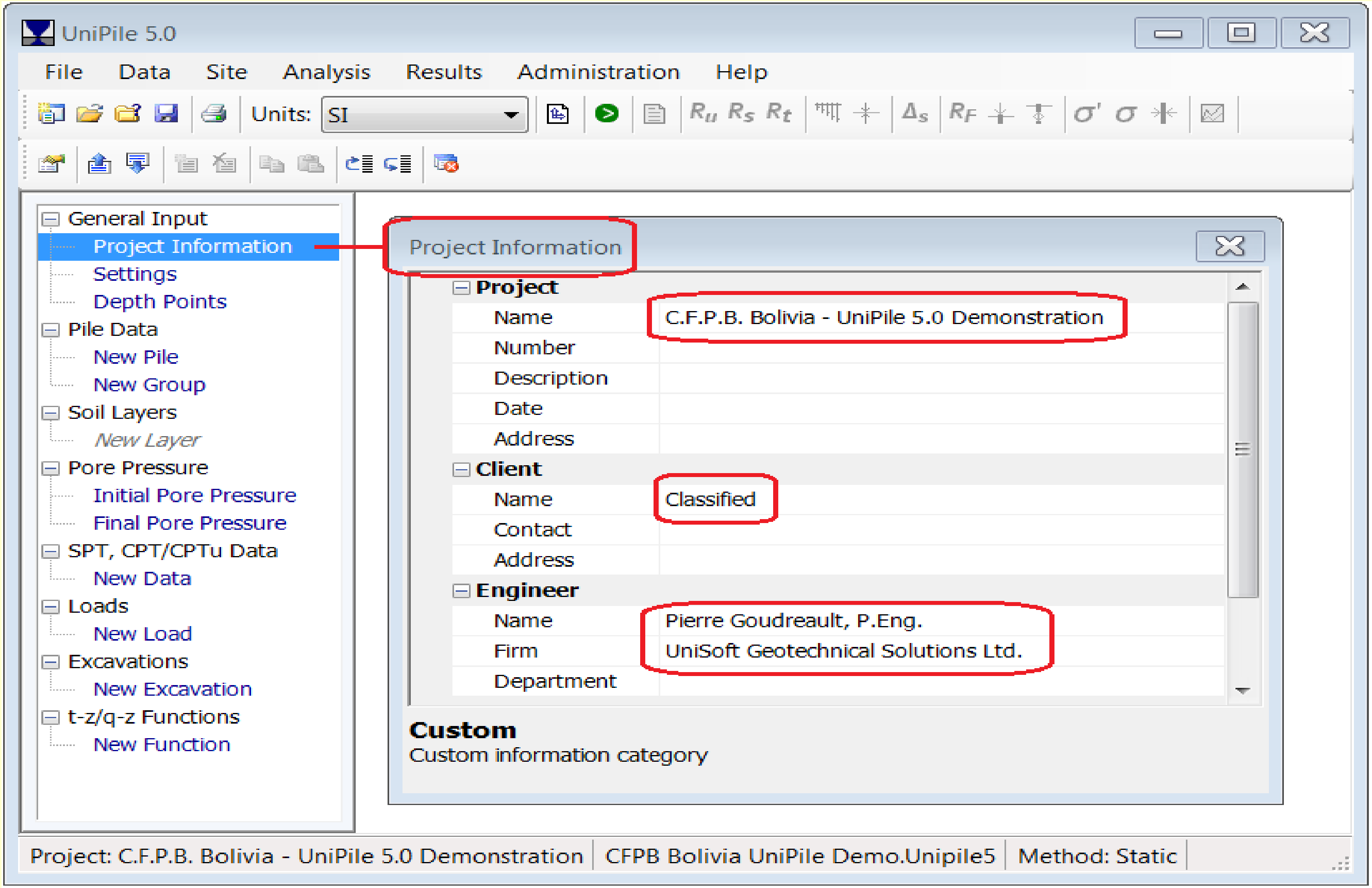
Using UniPile 5.0

UniPile 5.0

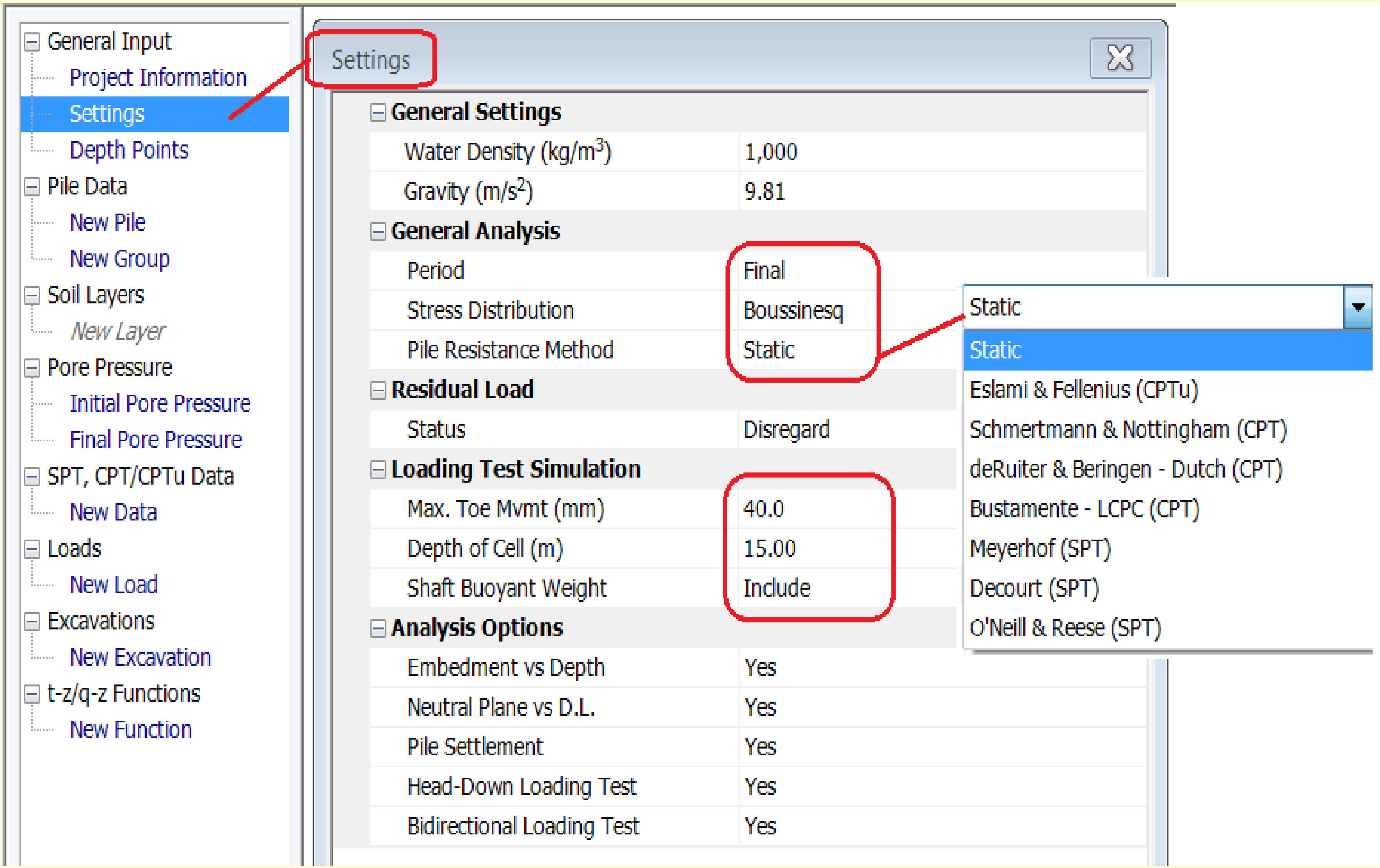
UniPile 5.0 Interface



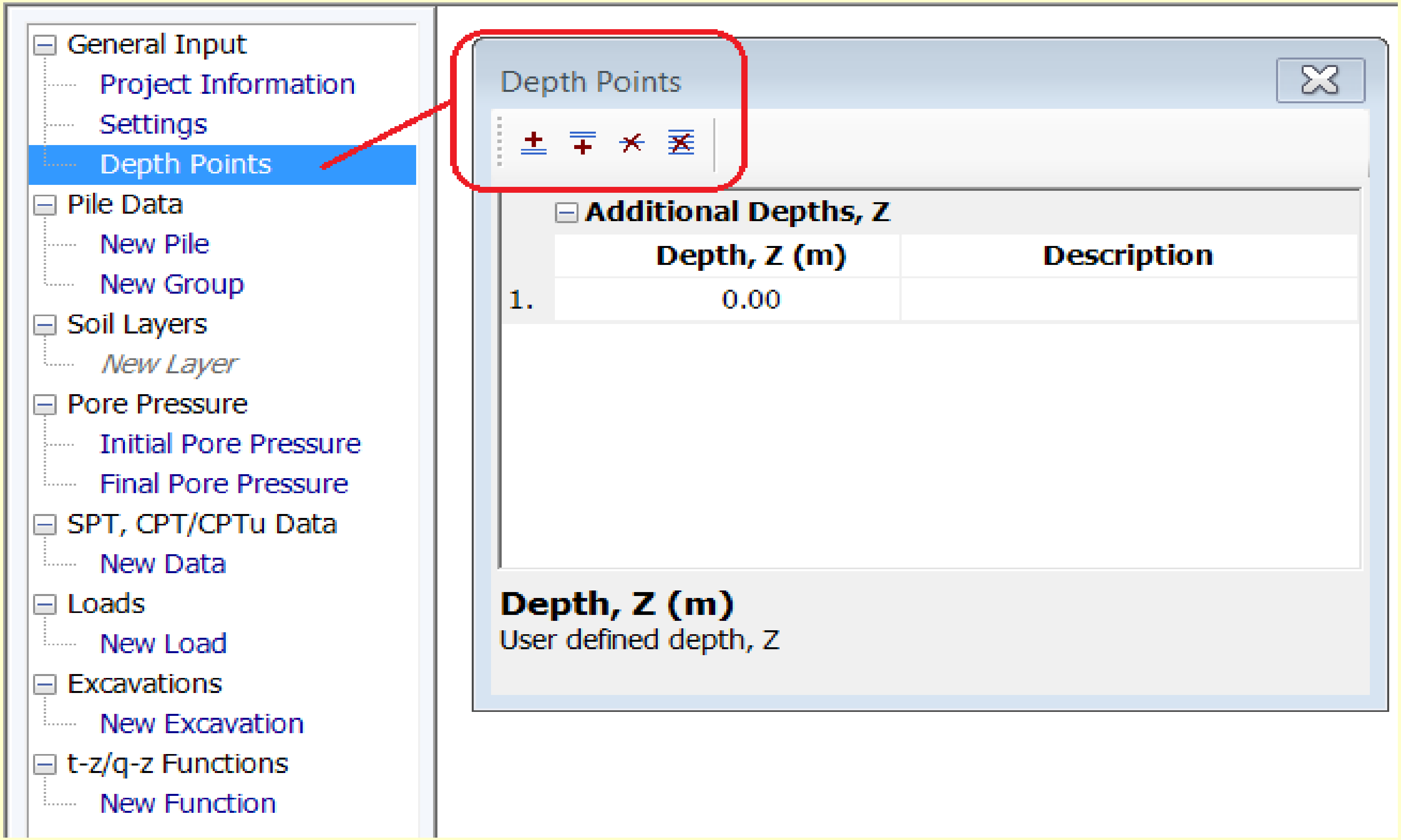
Project General Information



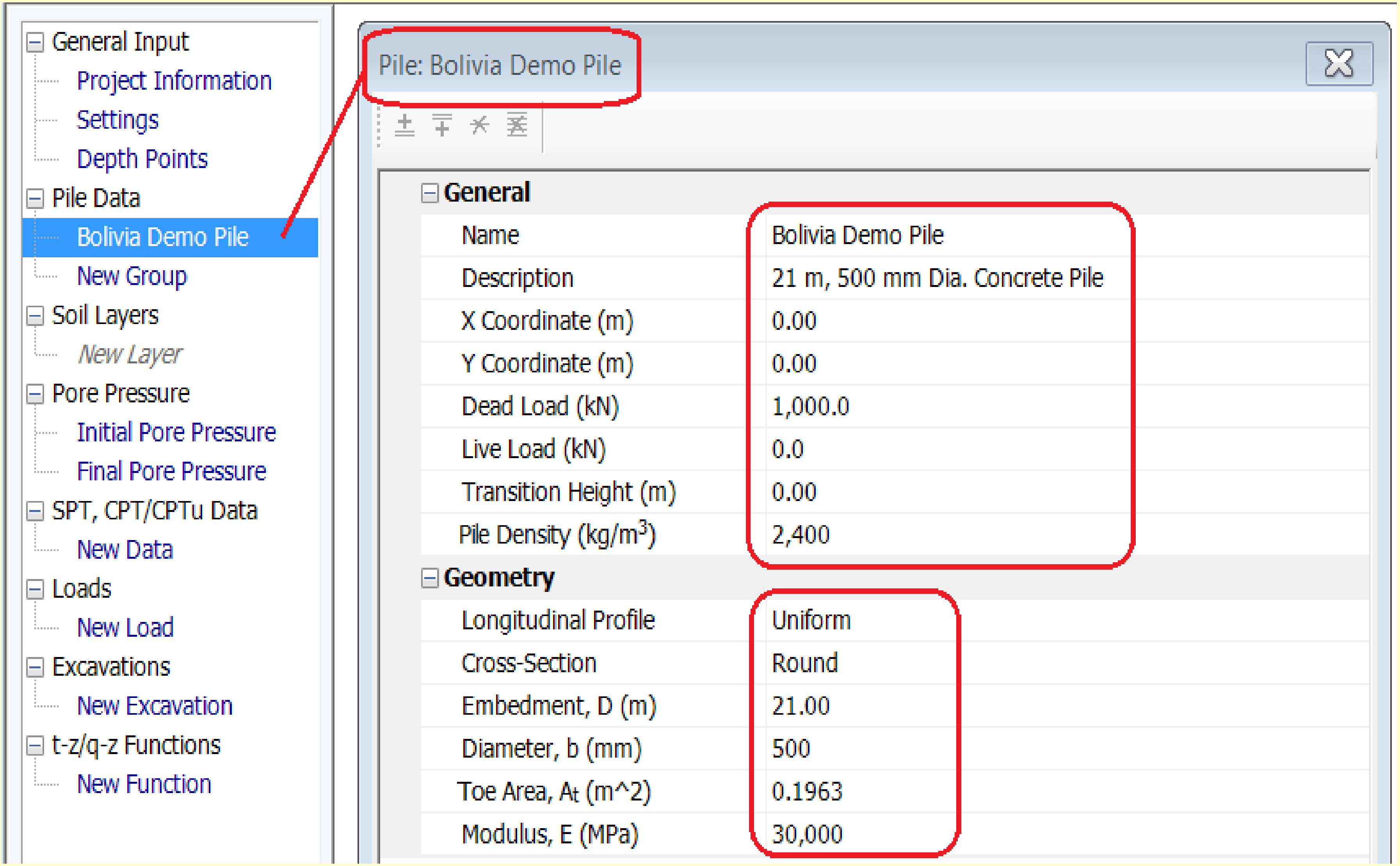
Settings and Defaults



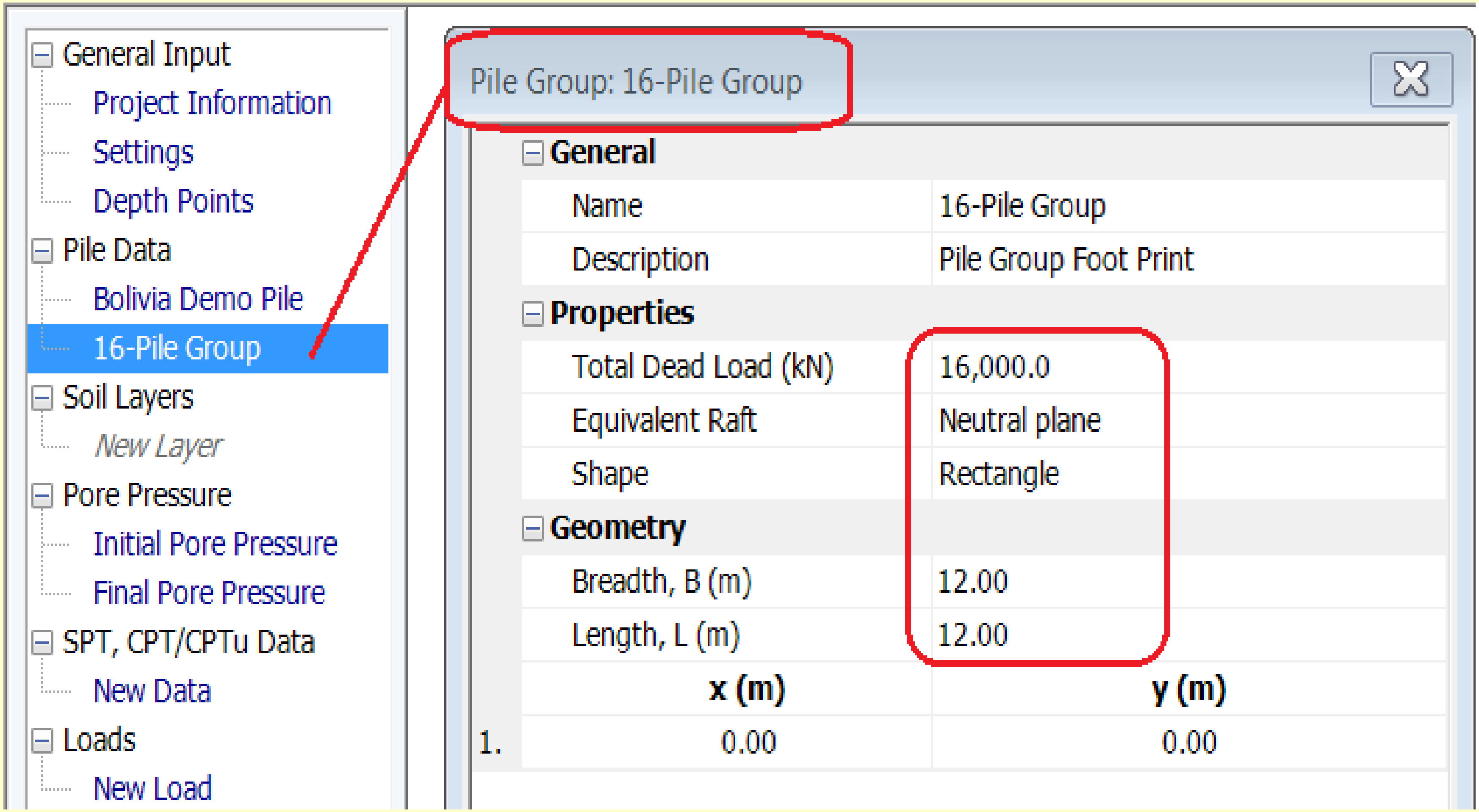
Additional Depth Points



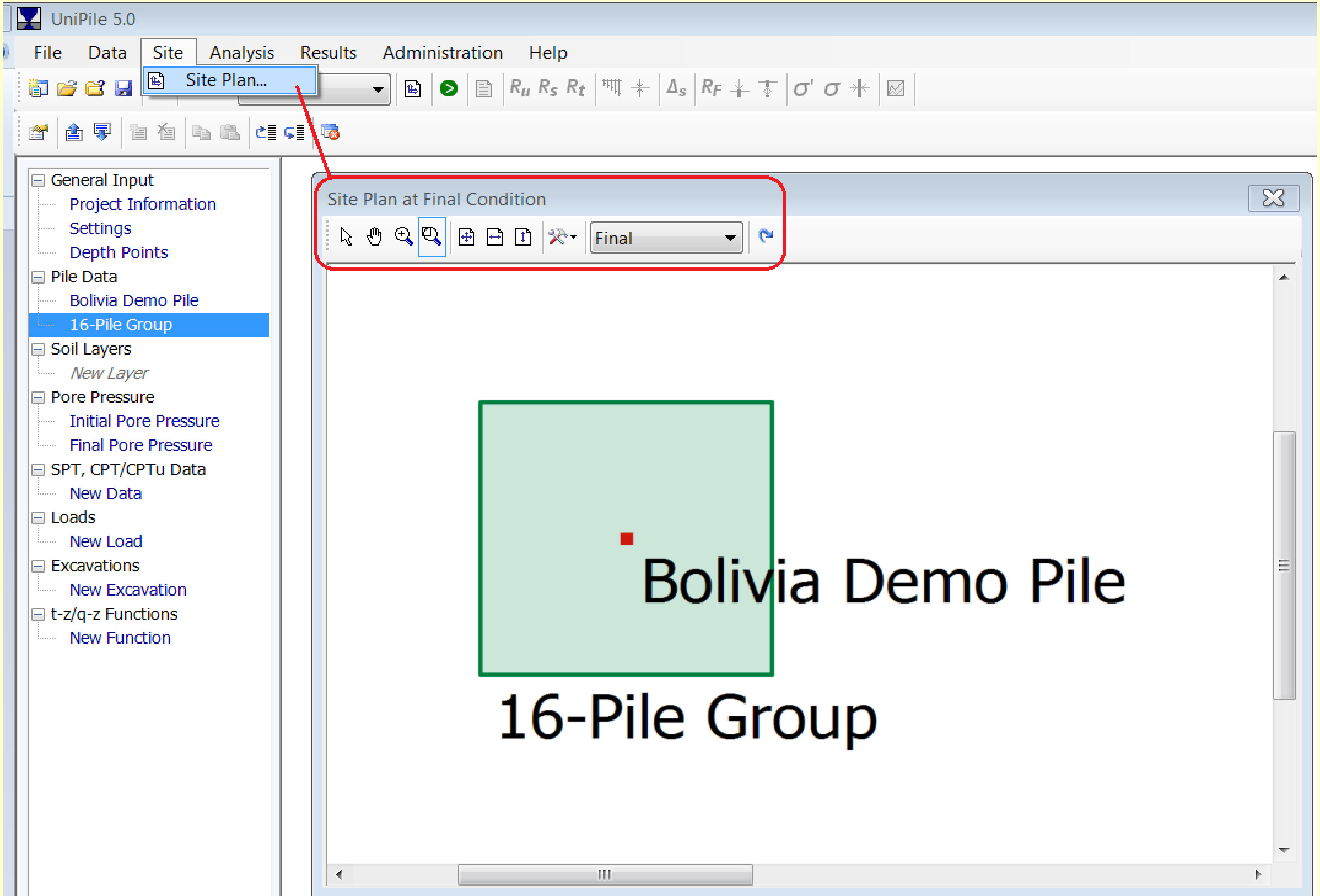
Pile Properties and Geometry



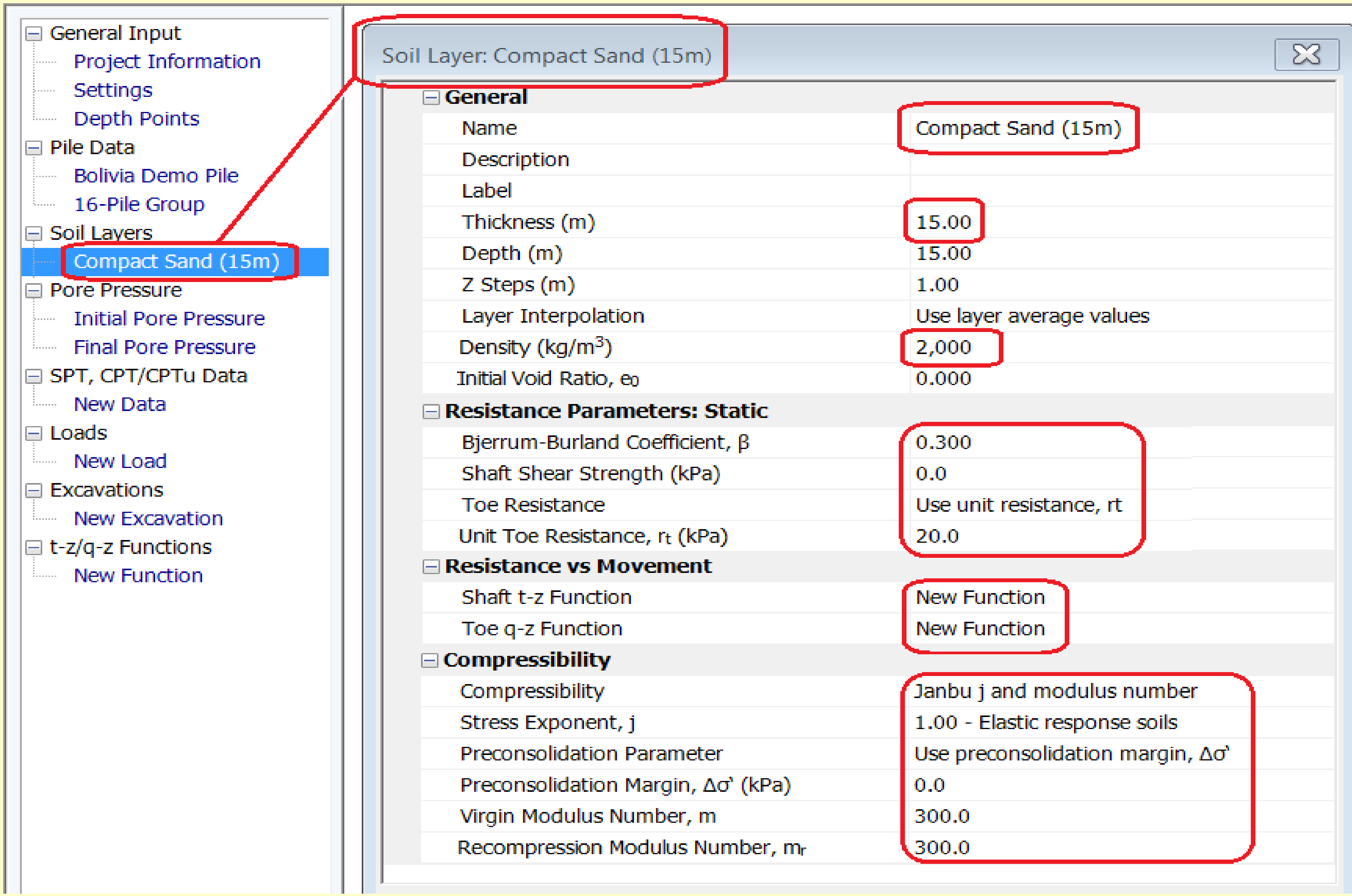
Pile Group Properties and Geometry (For Pile Group Settlement Analysis)



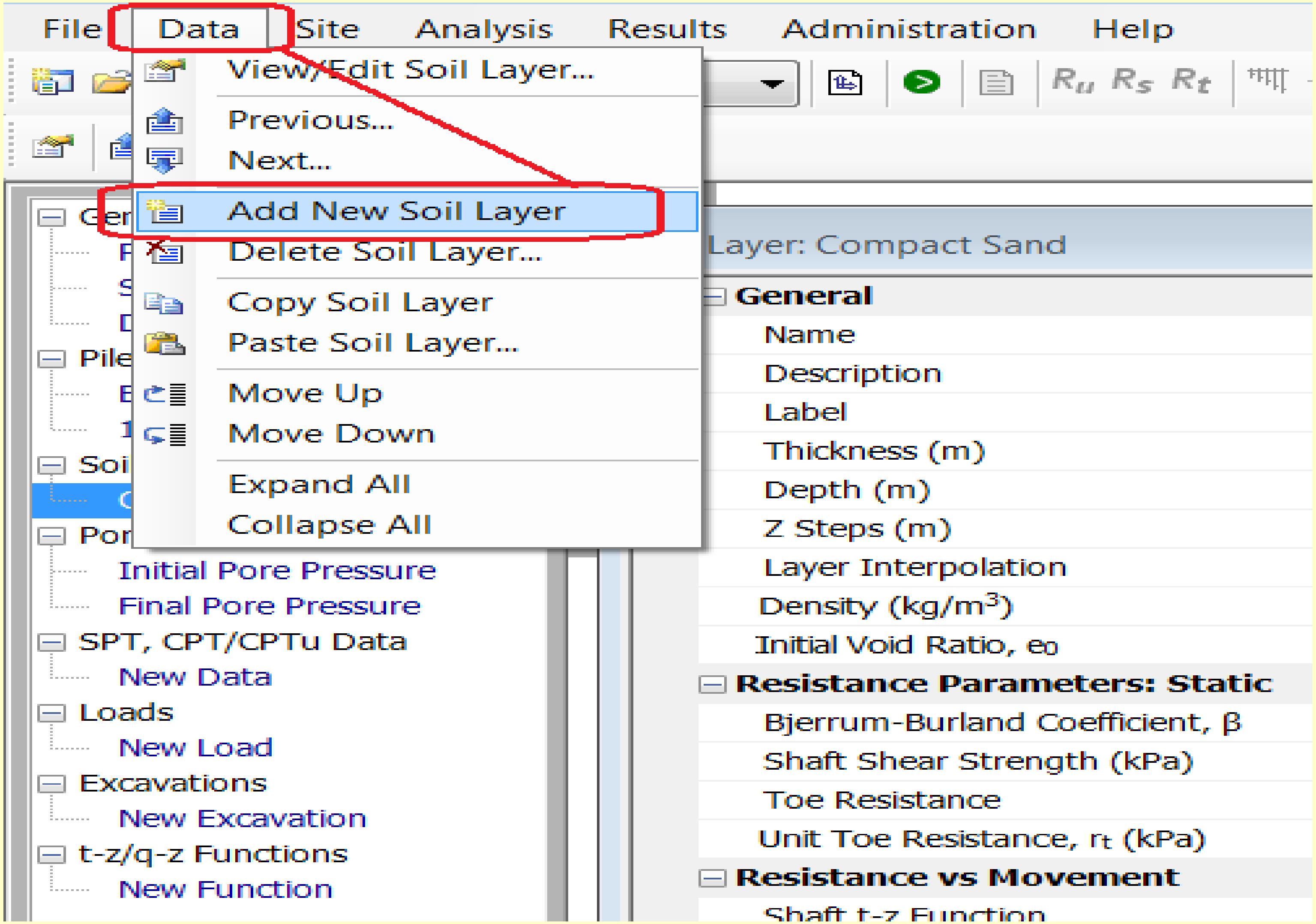
Project Site Plan View



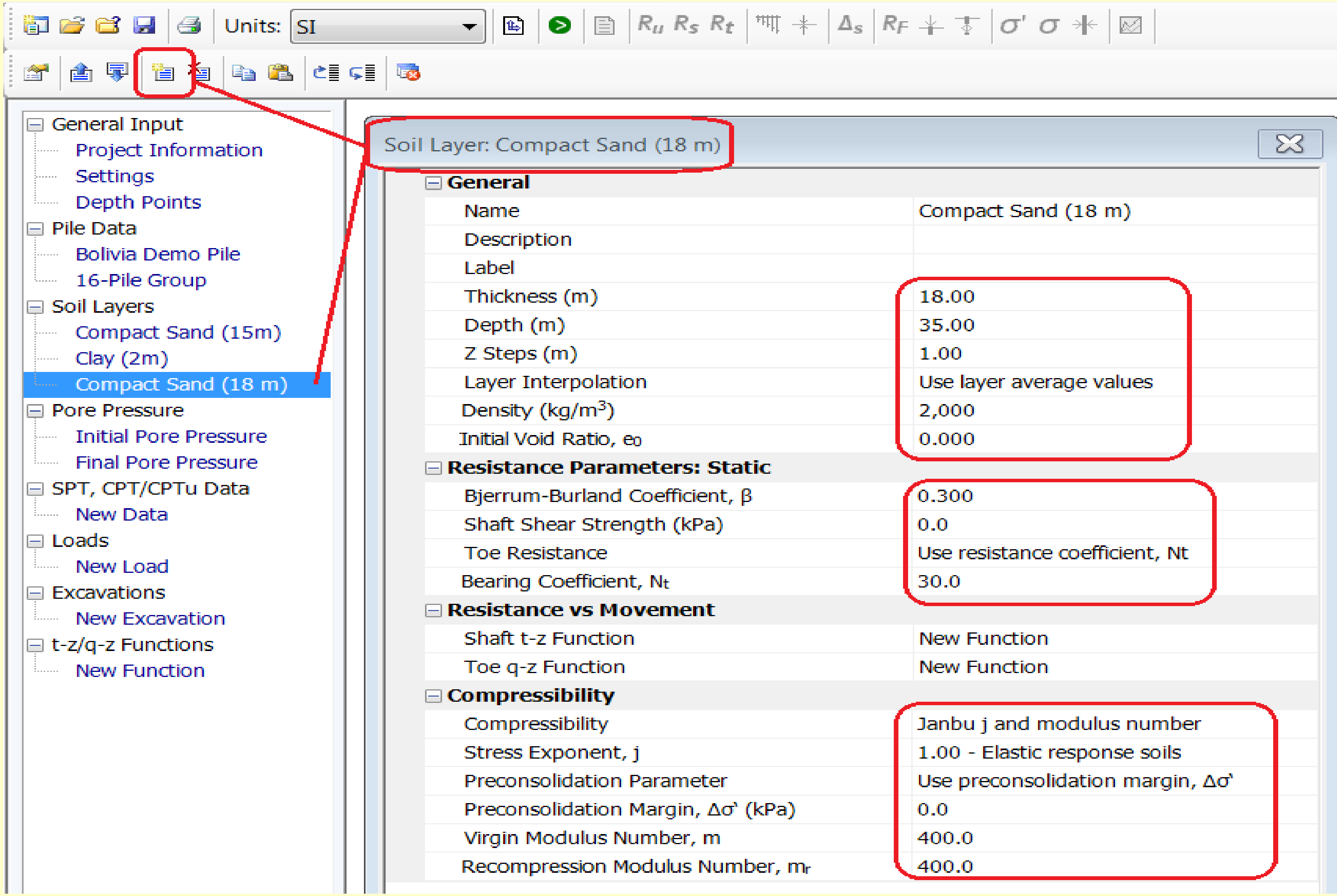
Soil Layer(s) Input



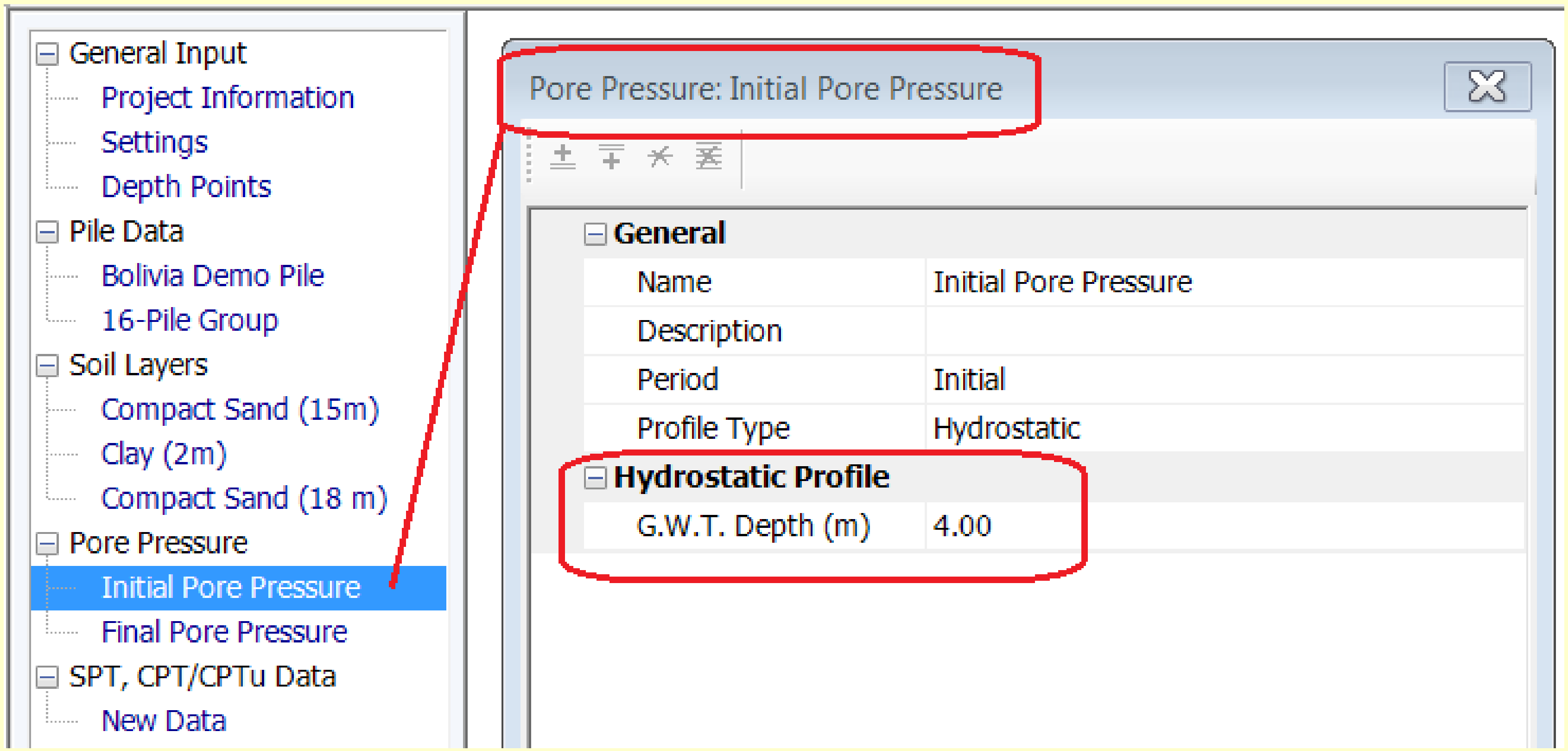
Add New Soil Layer



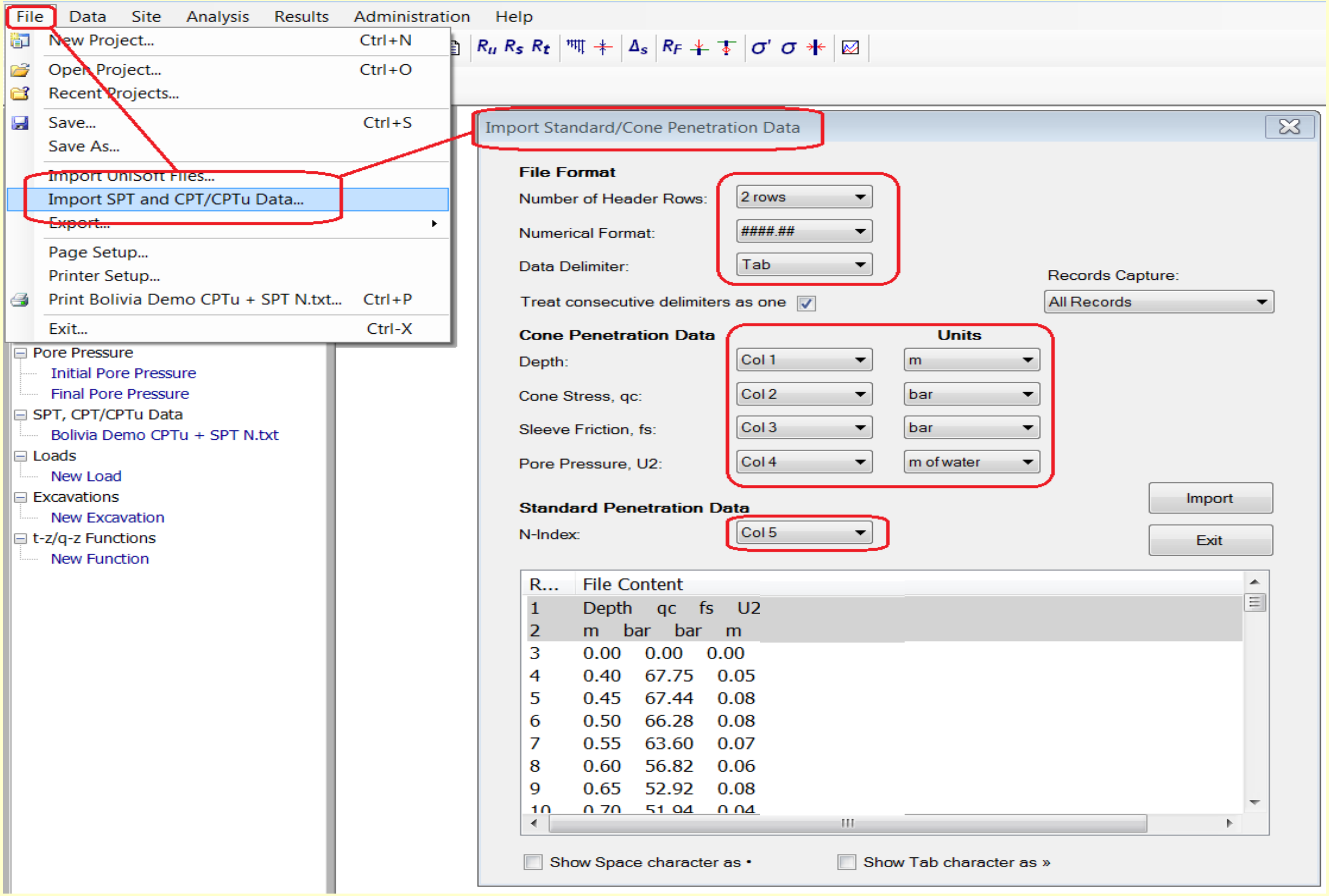
New Soil Layer Input



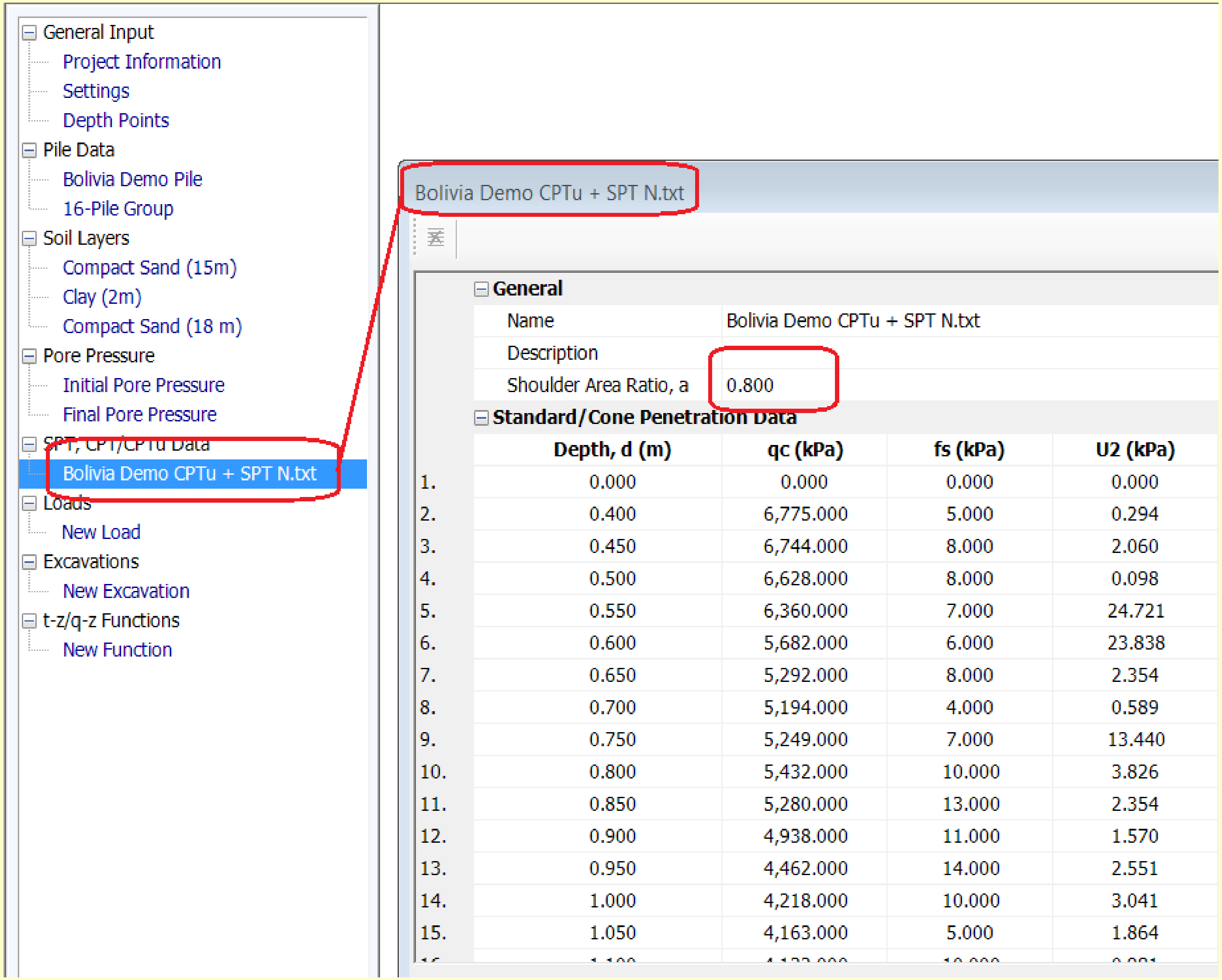
Enter Pore Pressures



Import CPT, CPTu, SPT Data

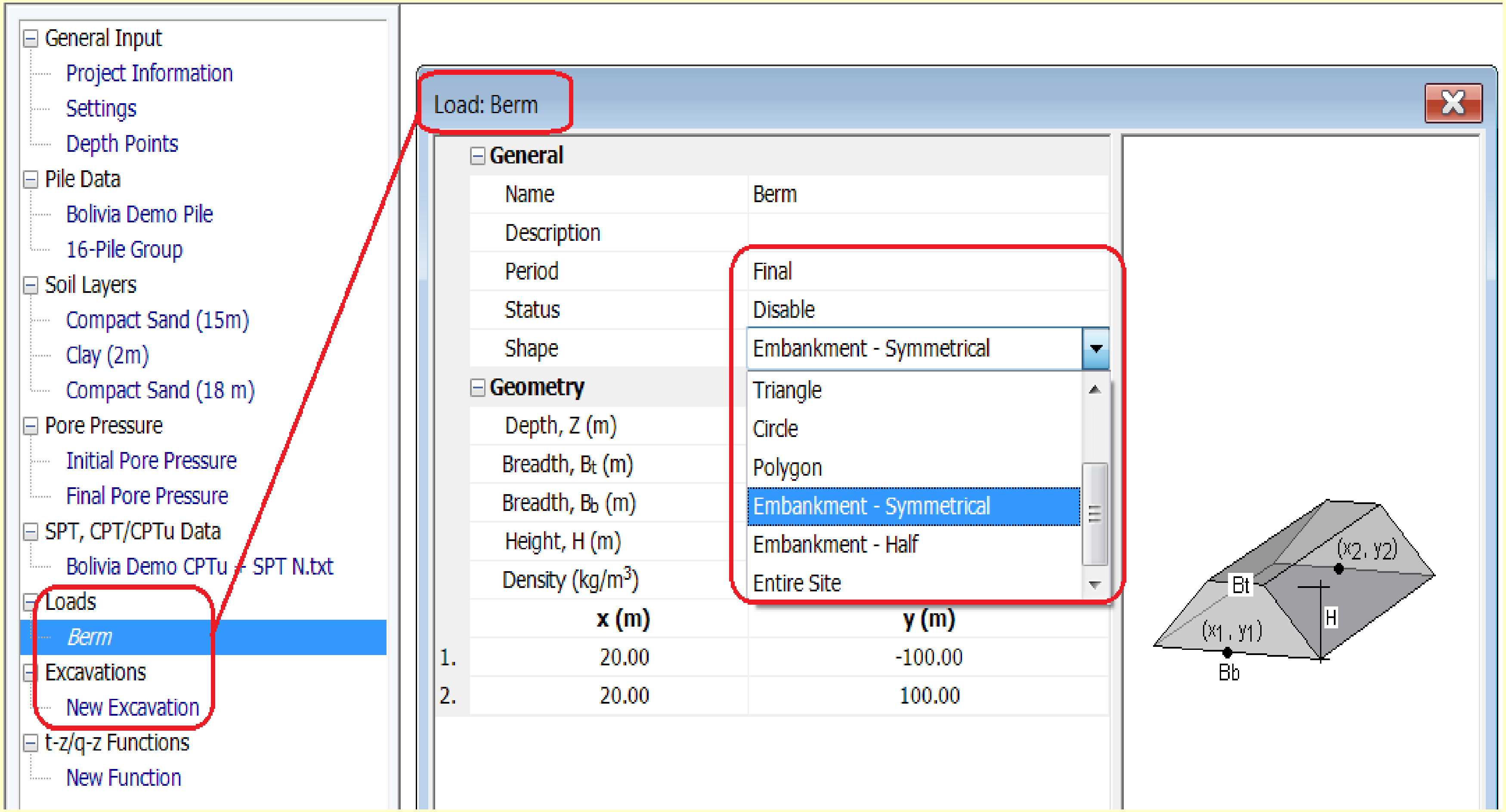


CPT, CPTu, SPT Data

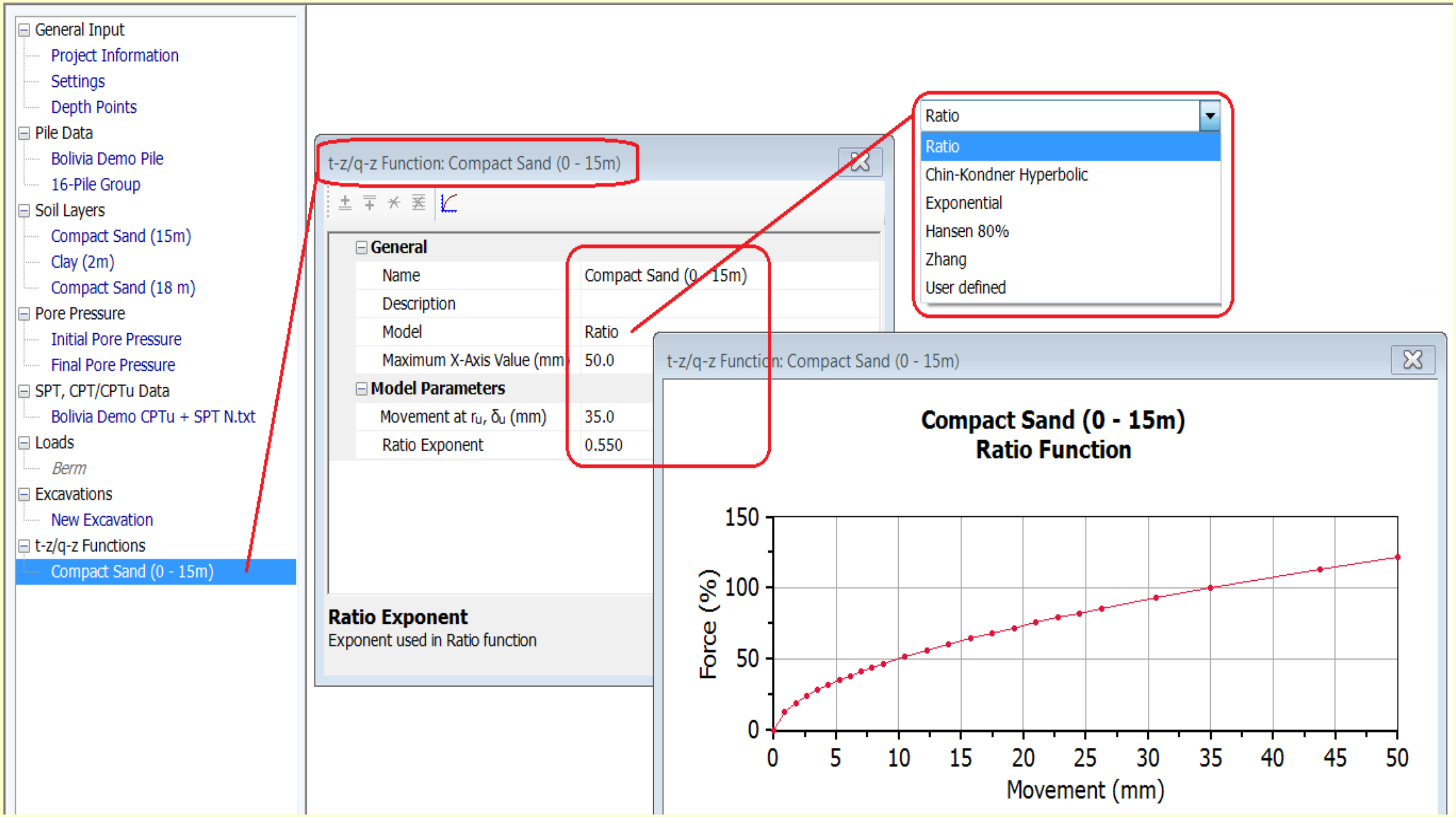


	Depth, d (m)	qc (kPa)	fs (kPa)	U2 (kPa)
1.	0.000	0.000	0.000	0.000
2.	0.400	6,775.000	5.000	0.294
3.	0.450	6,744.000	8.000	2.060
4.	0.500	6,628.000	8.000	0.098
5.	0.550	6,360.000	7.000	24.721
6.	0.600	5,682.000	6.000	23.838
7.	0.650	5,292.000	8.000	2.354
8.	0.700	5,194.000	4.000	0.589
9.	0.750	5,249.000	7.000	13.440
10.	0.800	5,432.000	10.000	3.826
11.	0.850	5,280.000	13.000	2.354
12.	0.900	4,938.000	11.000	1.570
13.	0.950	4,462.000	14.000	2.551
14.	1.000	4,218.000	10.000	3.041
15.	1.050	4,163.000	5.000	1.864

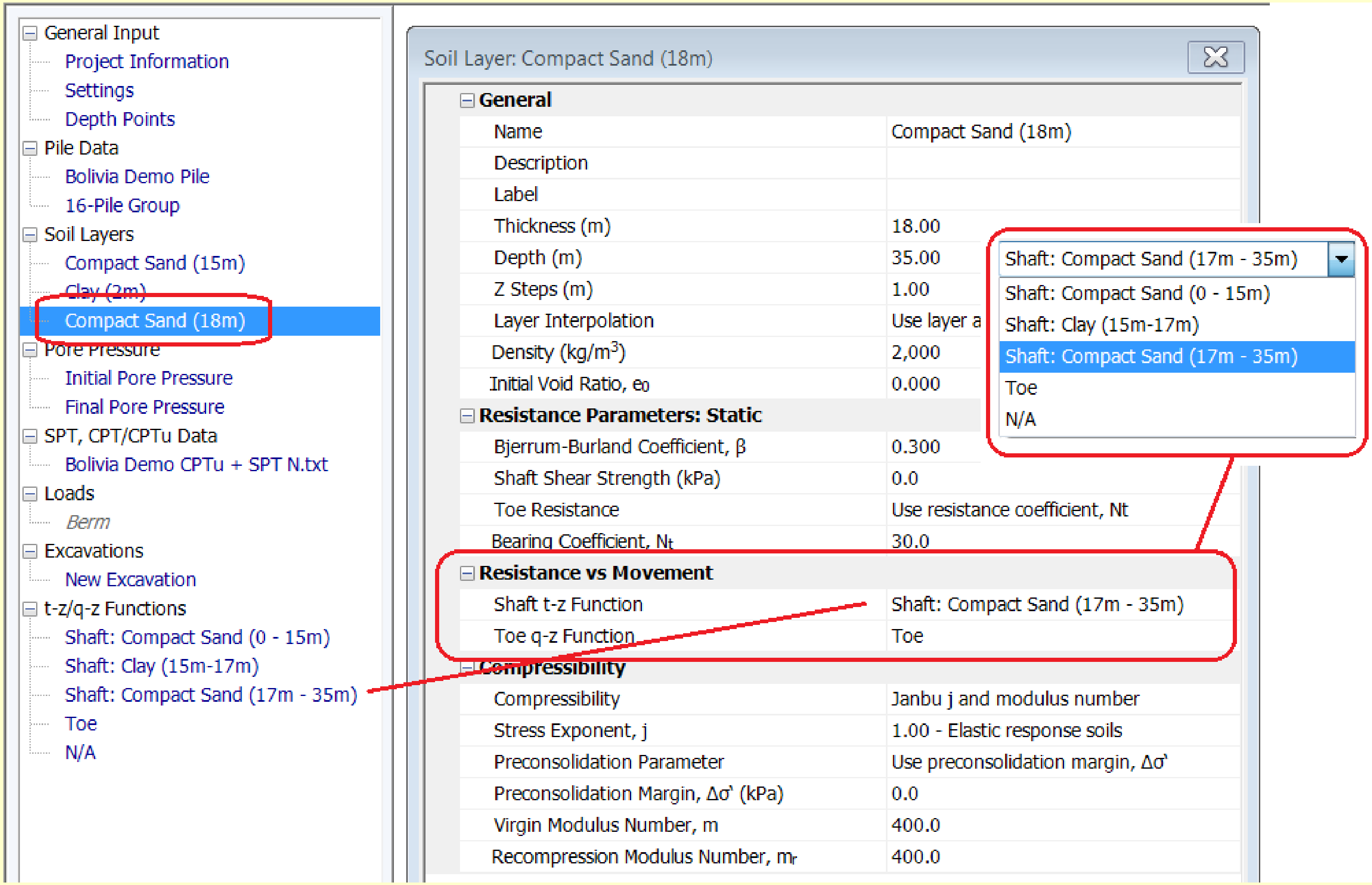
Enter Loads and Excavations



Define t-z and q-z Functions



Apply t-z and q-z to Soil Layer(s)



YES! But Does It Work?

- Static vs CPT, CPTu, SPT Analysis
- Embedment Analysis
- Add Transition Zone
- Pile group Settlement Analysis
- Head-Down Loading Test Simulation
- Bidirectional Loading Test Simulation

But What If?

- Non-Hydrostatic Pore Pressure
- Loads and Excavations are included
- Expanded-Base Pile
- Export results to Excel

Thank you for your attention

We'd welcome your questions