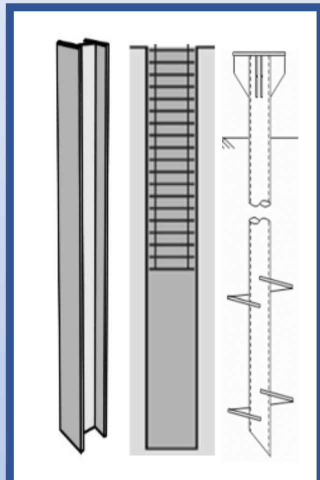
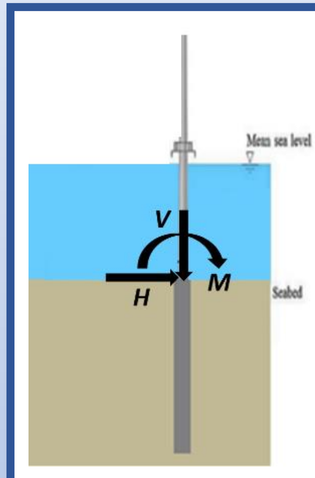
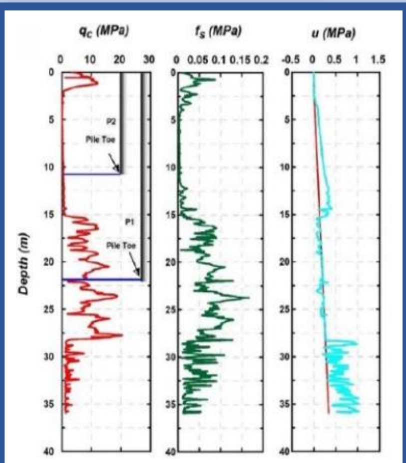
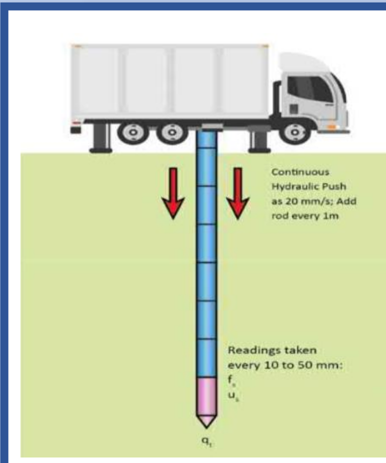
Short Course

Cone Penetration Tests (CPT & CPTu) Records for Deep Foundations Geotechnical Design









Abolfazl Eslami

Professor of Geotechnical Engineering,
Amirkabir University of Technology, AUT
Visiting Scholar in UCSD, 2022 - 2023

June 2023

Outline

1 **Deep Foundations: Geotechnical Design**

2 CPT & Pile Scale Effects

3 CPT- & CPTu-Based Methods

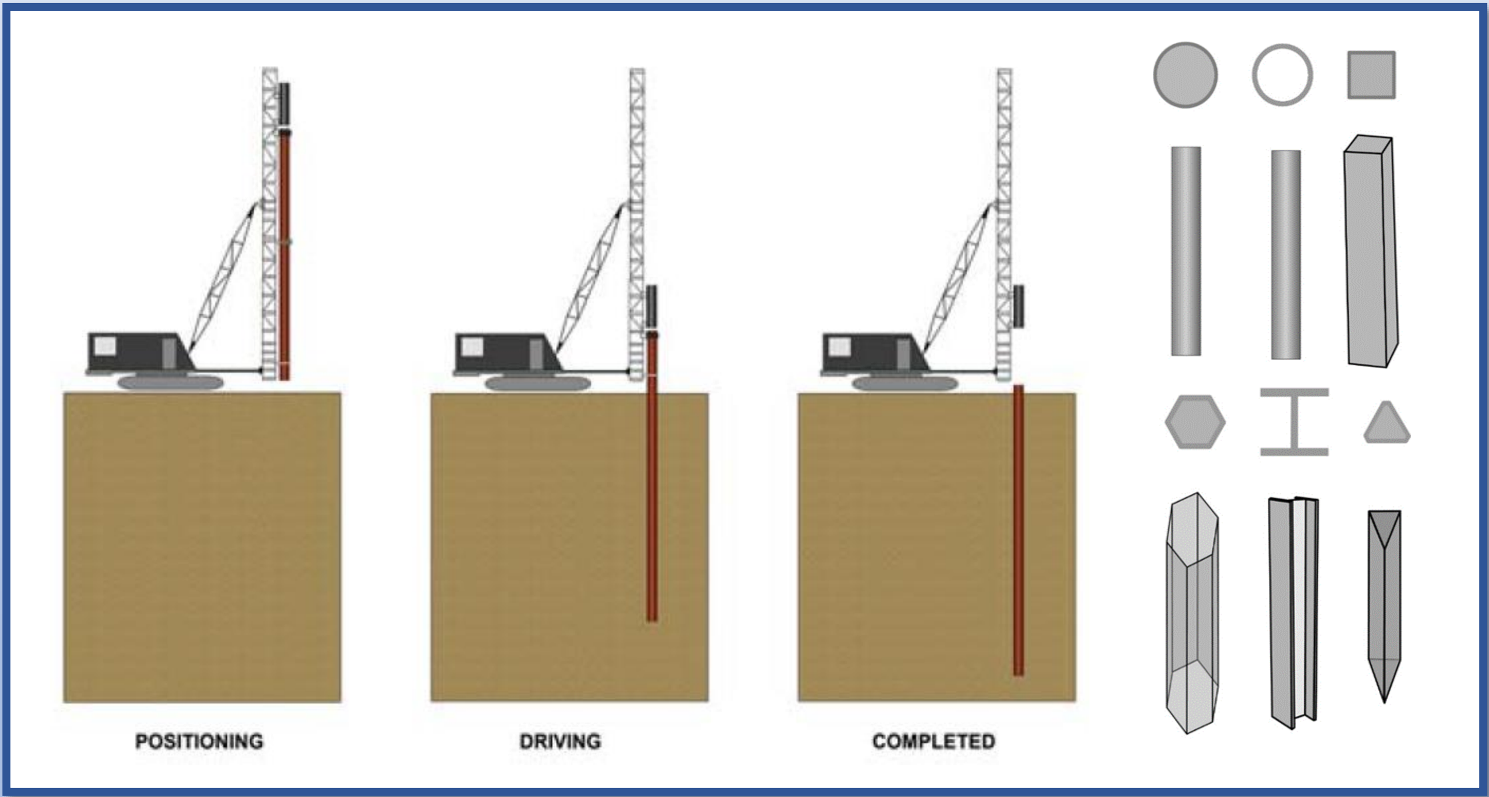
4 Worked Examples

5 Challenging Conditions

6 Case Histories

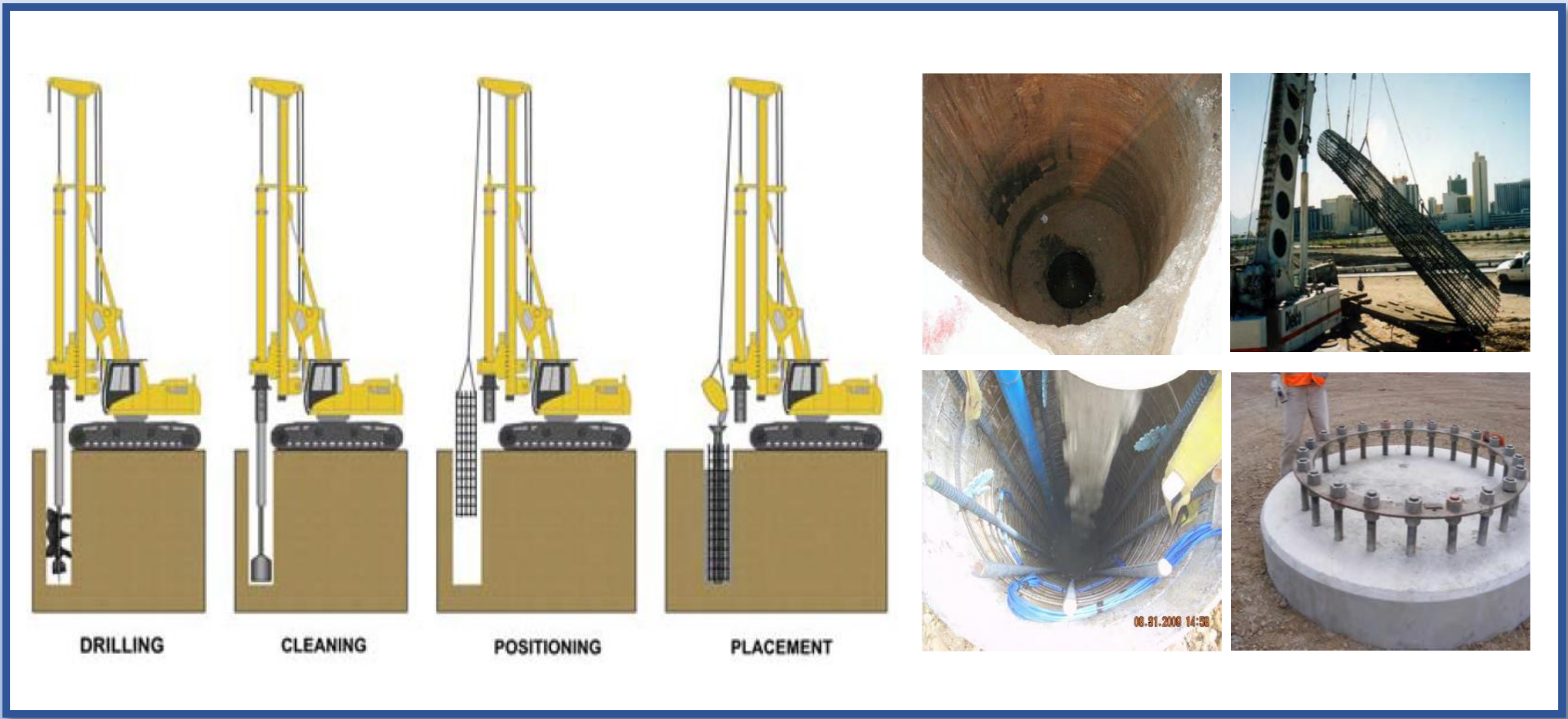
Different Types of Deep Foundations

- **Driven Piles**



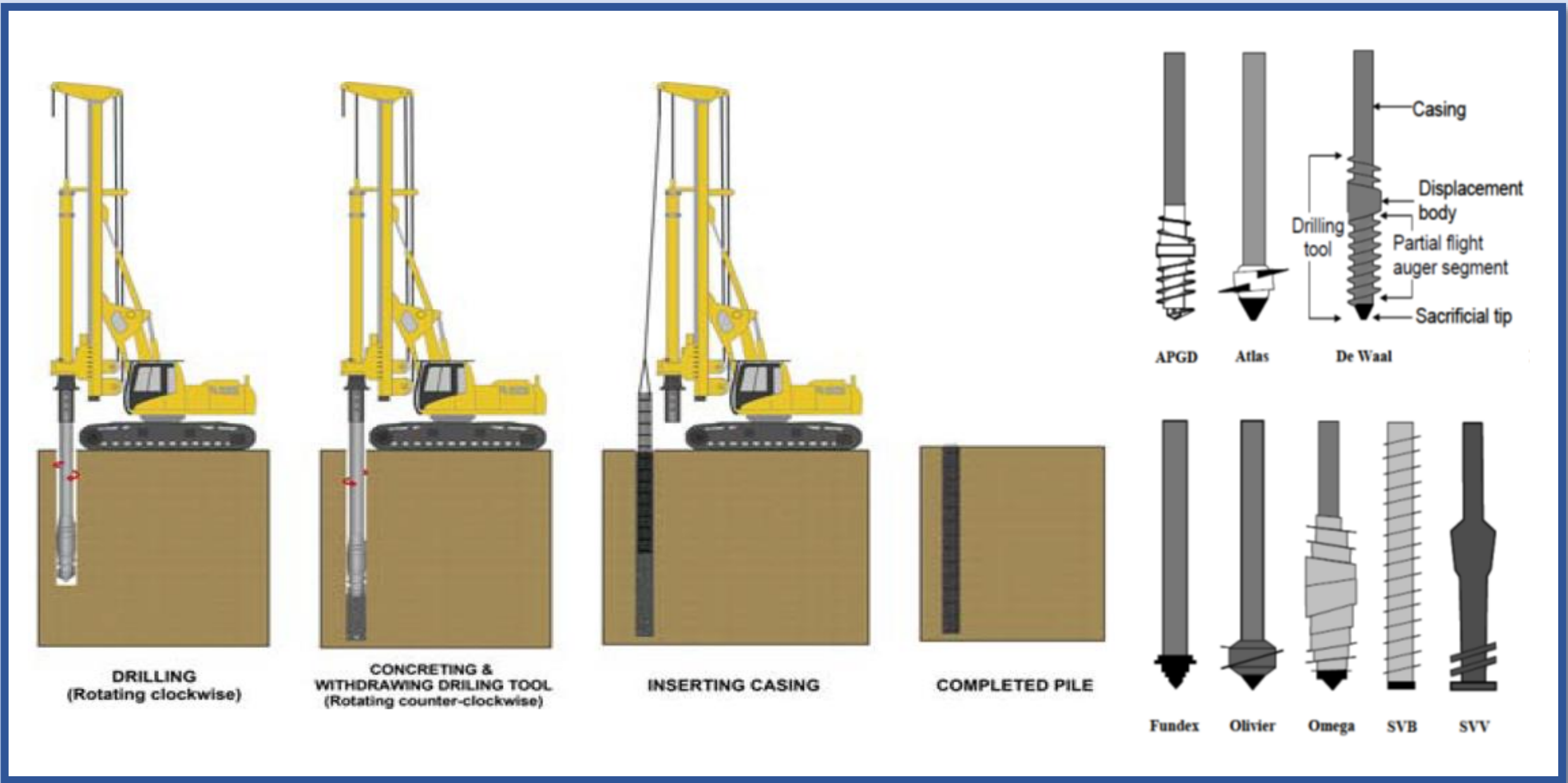
Different Types of Deep Foundations

- **Drilled Shafts**



Different Types of Deep Foundations

- **Drilled Displacement Piles (DDP)**



Necessity & Requirements

1. **<u>Upper soil strata have low resistance</u>, so are unable to bear the superstructure transferred load, and soil layers with more resistance are found at lower depths. In other words, even if mats are used, the bearing capacity is not provided by surface layers.**
2. **Despite resistant surface soil layers, there is a problem of "<u>scouring</u>," such as the scouring of structures adjacent to a beach.**
3. **<u>Large concentrated loads</u> should be transferred from the structure to the soil when the tolerance of these forces by shallow foundations, even mats, is impossible.**
4. **<u>The groundwater level is high</u>, or there is an artesian pressure in the soil layers, so it is impossible to construct shallow foundations.**
5. **It is necessary <u>to increase the hardness of soil under the machine foundations</u> to control the amplitude of foundation vibrations and control the system's normal frequency.**

Necessity & Requirements

6. **If there is <u>resistance to tensile or overturning forces</u> below the surface, or it is required to prevent the overturning of high structures.**
7. **It is necessary <u>to create restraint against lateral and earthquake forces</u>.**
8. **There is a need <u>to control landslides</u>, increase slope stability as well as support against ground motion.**
9. **In cases where it is essential to provide sufficient pullout capacity plus external stability in particular for <u>structures under combined loading (VMH).</u>**
10. **It is essential <u>to mitigate and control the seepage</u> through the implementation of some barriers.**
11. **There is a need <u>to enhance existing shallow foundations</u> capacity through intrusion or confinement using deep-seated elements.**

Getechnical Design Aspects

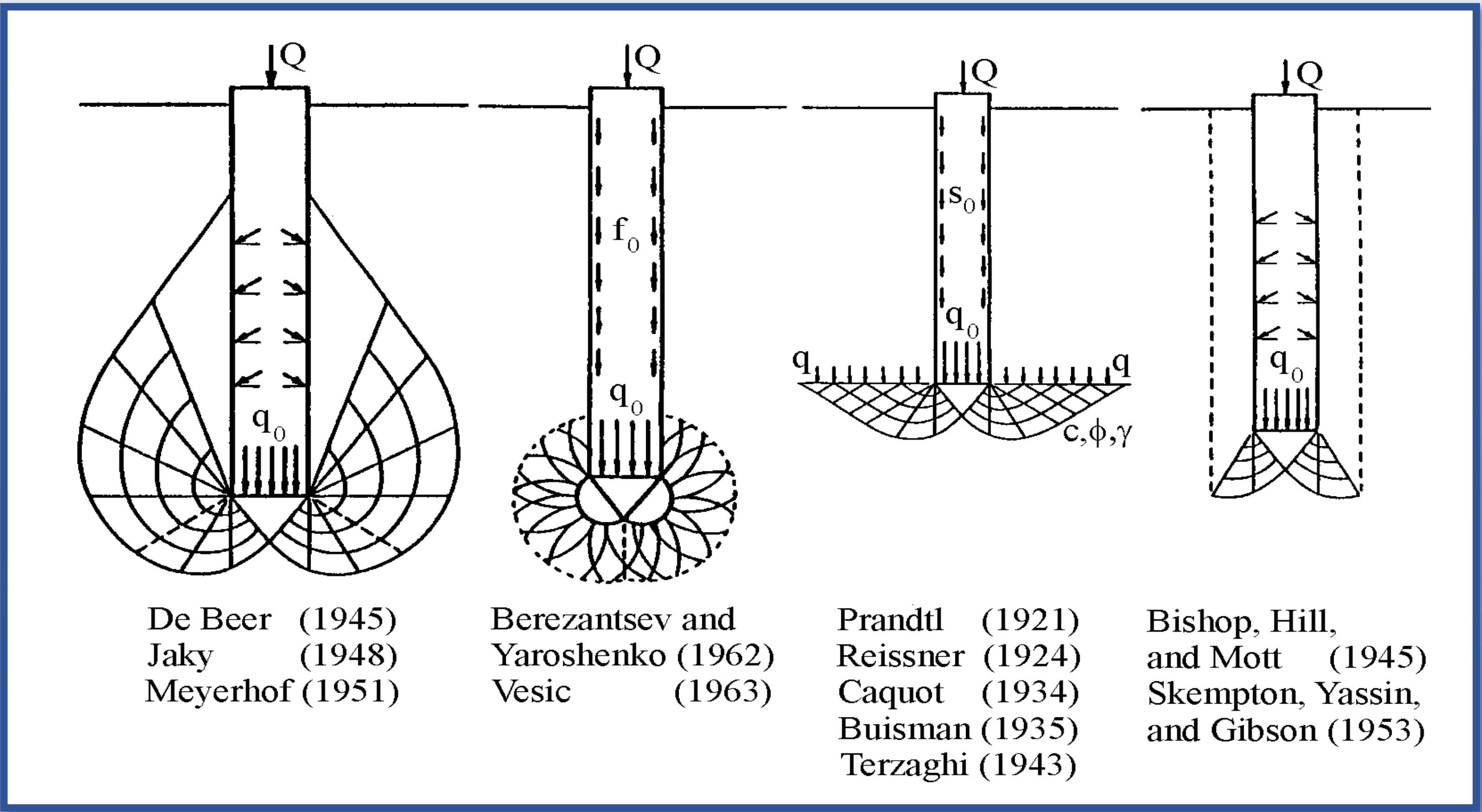
1. Bearing Capacity

2. Resistance Distribution

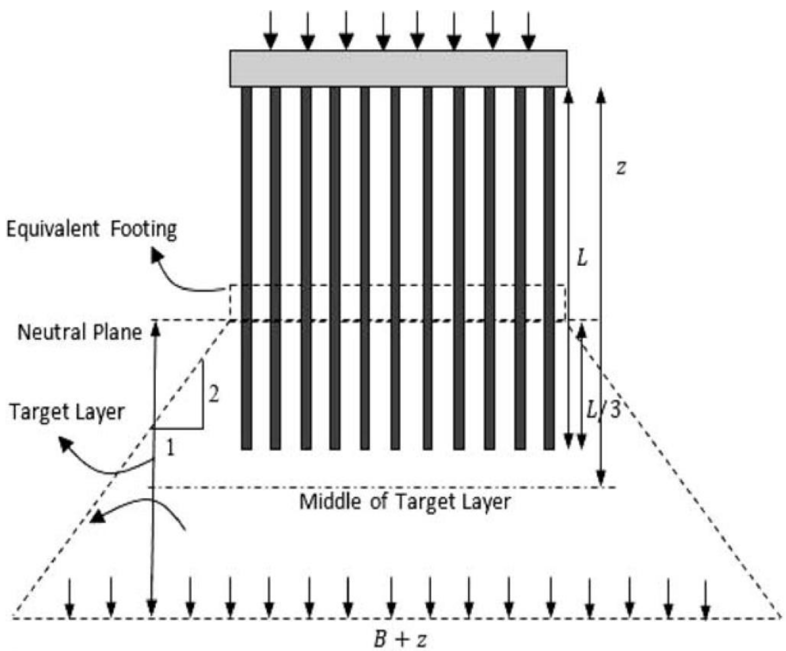
3. Settlement

4. Load - Displacement

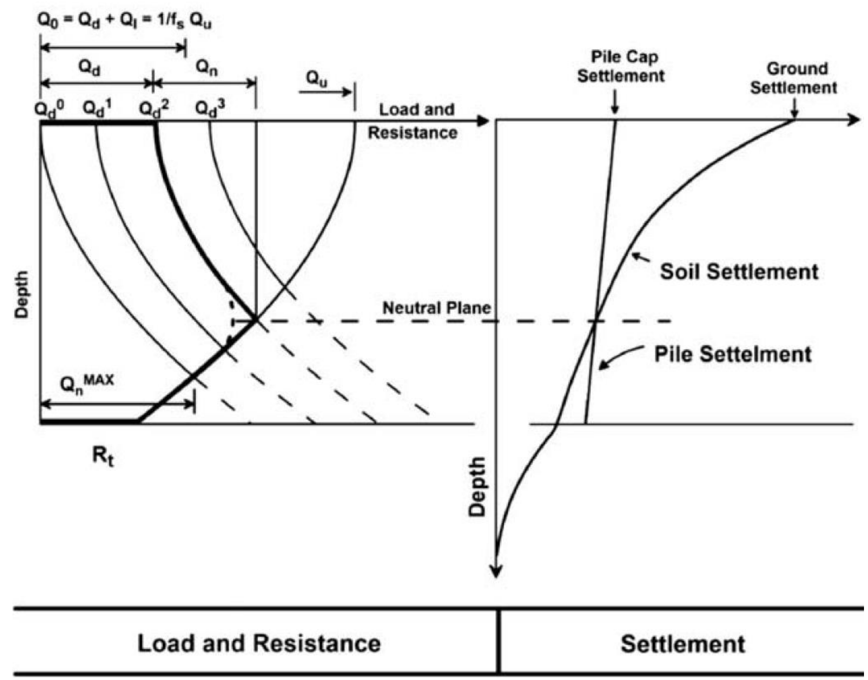
Failure Mechanisms for Bearing Capacity



Settlement & Resistance Distribution



Simple model to estimate pile group settlement proposed by Terzaghi and Peck (1948)



load, resistance, and settlement distribution along depth (Fellenius, 2015)

Direct Application for Settlement & Load-Displacement

- **Valikhah & Eslami (2019)**

$$\Delta H = \left(\frac{1}{mj}\left[\left(\frac{\sigma_0'+\Delta\sigma'}{\sigma_r'}\right)^j - \left(\frac{\sigma_0'}{\sigma_r'}\right)^j\right]\right)\times H$$

$$m = 0.25b \times \left(\frac{2B+1}{3B}\right)^3 \times q_c$$

b: penetration cone diameter

B: foundation width

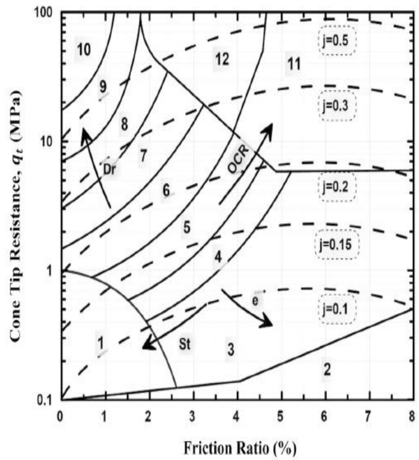
(b and B are in m and q_c is in kPa)

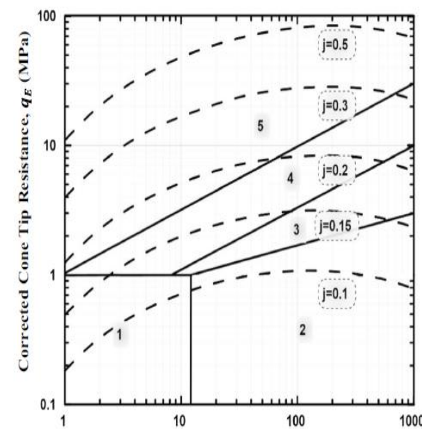
$$j = \frac{q_c}{x + y q_c}$$

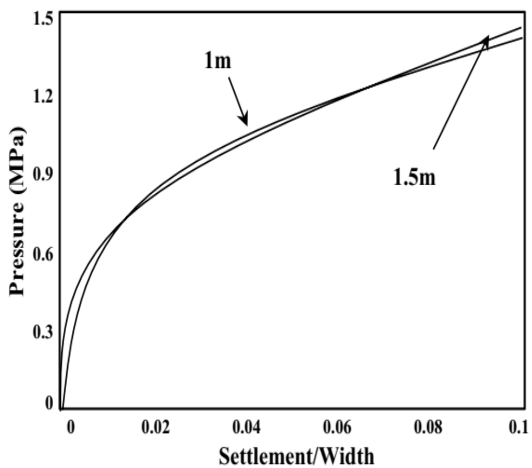
$$x = 0.02R_f + 0.5$$

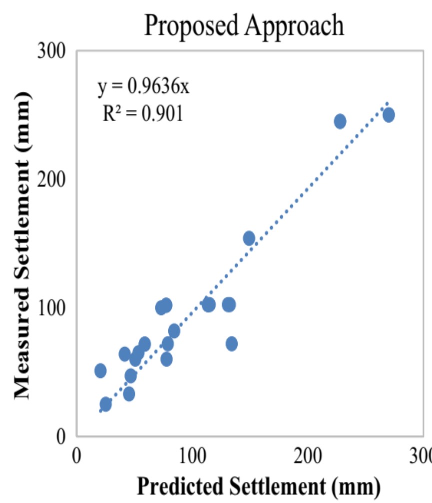
$$y = 7.53(\sigma_0')^{-0.25}$$

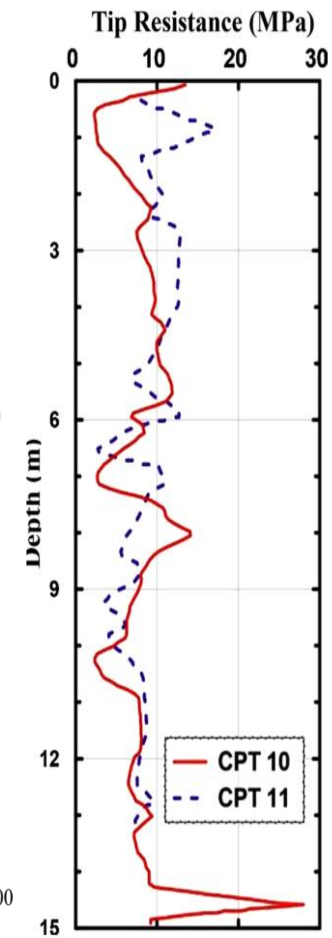
(q_c and σ_0' are in kPa)











How can we estimate the bearing capacity of piles?

1. Static Methods

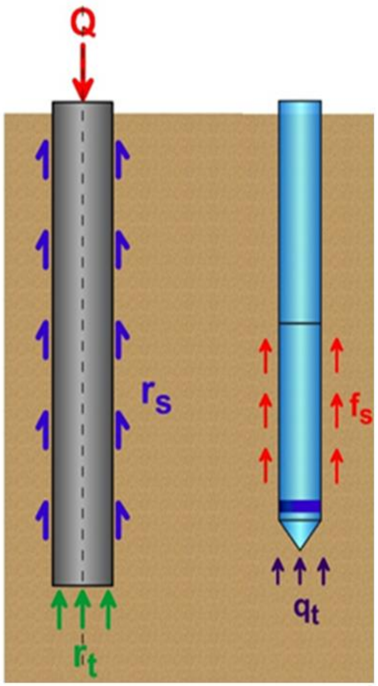
2. In-situ Tests

3. Static Loading Test

4. Dynamic Methods

5. Numerical Analysis

Role in Deep Foundations Axial Capacity

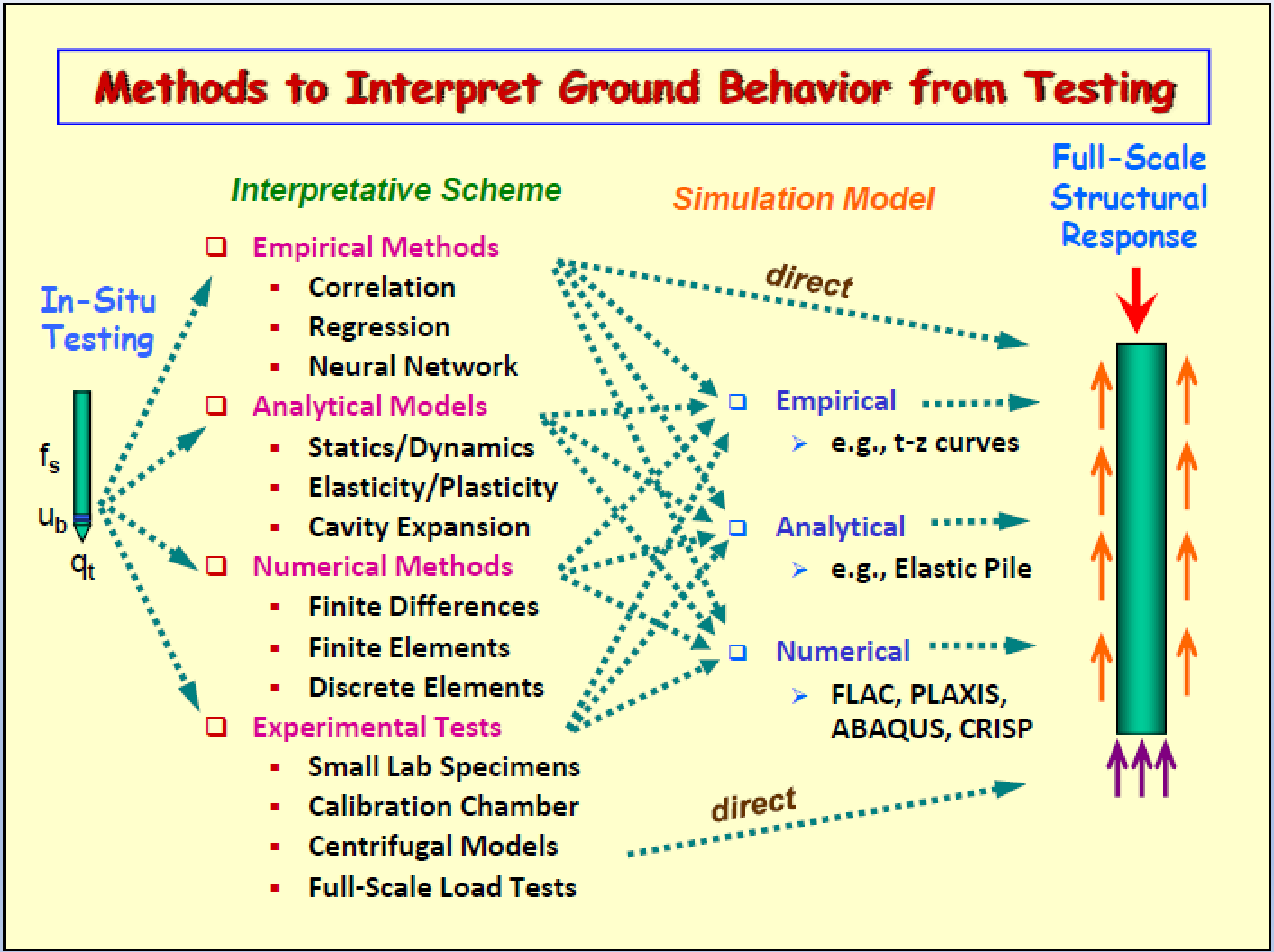


Indirect Approach: q_c & f_s → C,Ø → r_t, r_s

Direct Approach:
- q_c → r_t
- f_s → r_s

Penetrometers can be realized as a *model pile*

In-situ Tests Interpretation



Pile Bearing Capacity

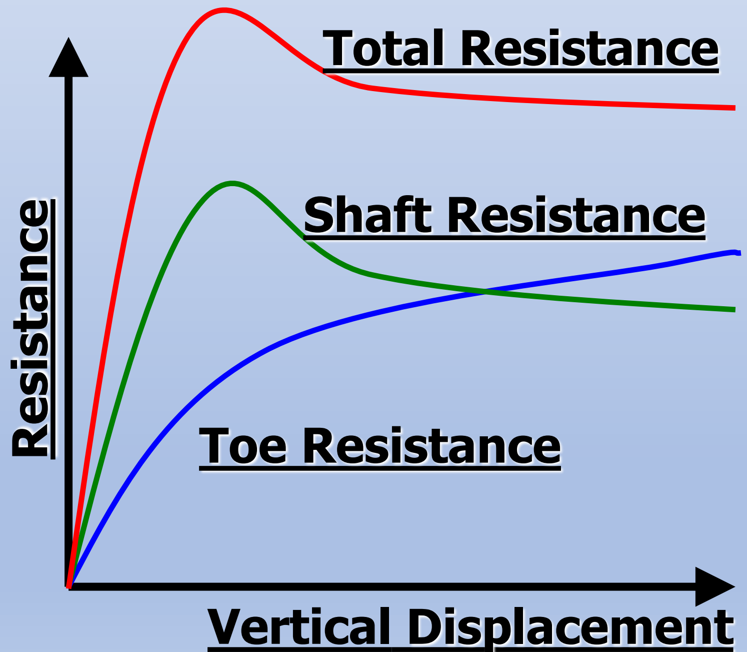
Bearing Capacity = **Toe Capacity** + **Shaft Capacity**

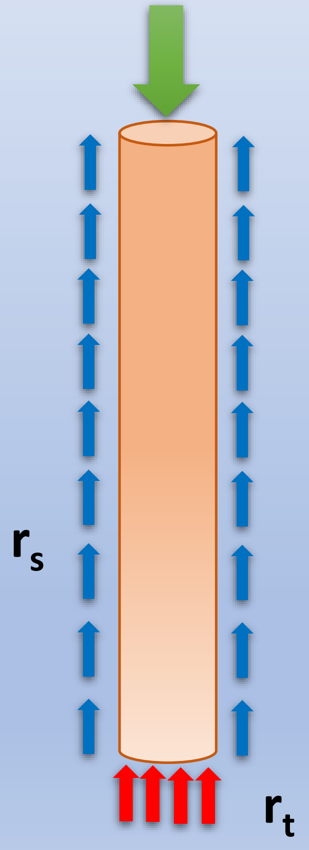
$$R_t = r_t . A_t$$

$$R_s = r_s . A_s . D_f$$

$$R_u = R_t + R_s$$

$$P_a = \frac{R_u}{FS}$$





Pile Bearing Capacity- Static Analysis – Indirect Approach

Toe Resistance

$$r_t = CN_C^* + \overline{q}N_q^* + 0.5\gamma BN_\gamma^*$$

Neglecting the third term $\rightarrow$ $r_t = CN_C^* + \gamma D_F .N_q^*$

For cohesive soils (undrained condition) $\rightarrow$ $r_t = CN_C^*$

For non-cohesive soils (drained condition) $\rightarrow$ $r_t = \overline{q}.N_q^*$

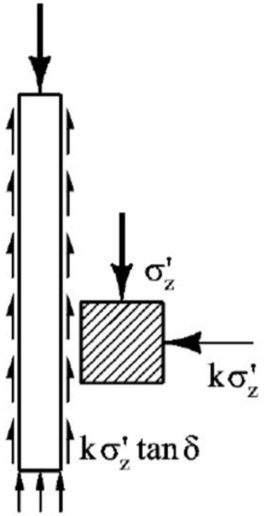
Pile Bearing Capacity- Static Analysis – Indirect Approach

Shaft Resistance

Effective stress analysis (ESA)

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

$$\beta = K \cdot \tan \delta$$



Pile type	K/K_o	Construction method (Bored piles)	K/K_o
Jetted piles	1/2 ~ 2/3	Dry construction with minimal sidewall disturbance and prompt concreting	1.0
Drilled shaft, cast-in-place	2/3 ~ 1	Slurry construction—good workmanship	1.0
Driven pile, small displacement	3/4 ~ 5/4	Slurry construction—poor workmanship	2/3
Driven pile, large displacement	1 ~ 2	Casing under water	5/6
References	(Kulhawy 1984)	(Reese and O'Neill 1989)	

Pile material	δ/ϕ'	Construction method (Bored piles)	δ/ϕ'
Rough concrete (cast-in-place)	1.0	Open hole or temporary casing	1.0
Smooth concrete (precast)	0.8~1.0	Slurry method—minimal slurry cake	1.0
Rough steel (corrugated)	0.7~0.9	Slurry method—heavy slurry cake	0.8
Smooth steel (coated)	0.5~0.7	Permanent casing	0.7
Timber (pressure-treated)	0.8~0.9		
References	(Kulhawy 1984)	(Reese and O'Neill 1989)	

Wei Dong Guo (2012)

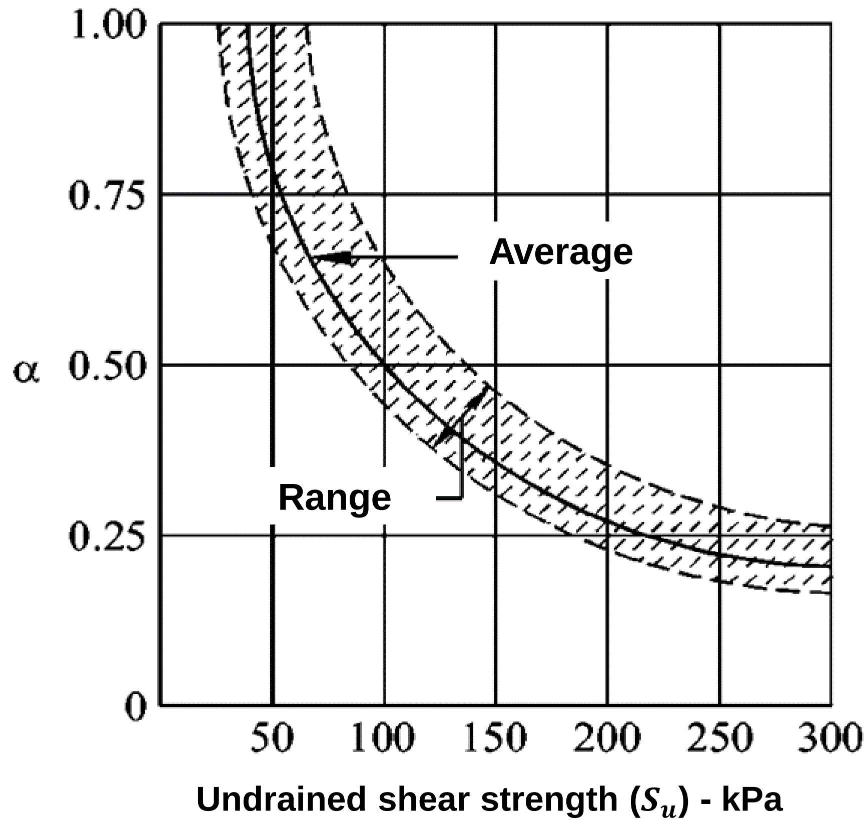
Pile Bearing Capacity- Static Analysis – Indirect Approach

Shaft Resistance

Total stress analysis (TSA)

$$\mathbf{r_s = \alpha S_u}$$

$$\alpha = 0.21 + 0.26\left(\frac{P_a}{C_u}\right) \leq 1$$



Pile Bearing Capacity- Static Analysis – Indirect Approach

Unified Pile Design (CFEM)

$$\mathbf{r_t = N_t \times \sigma'_{z=D_f}}$$

$$\mathbf{r_s = \beta \times \sigma'_{z-avg}}$$

$\sigma'_{Z=D_f}$ is the effective vertical stress at depth $Z = D_f$

The values of β and N_t are as given in Table

Soil type	Soil friction angle	Drilled Shafts		Driven Piles	
		β	N_t	β	N_t
Clay	25-30	0.25-0.32	3-10	0.25-0.32	3-10
Silt	28-34	0.2-0.3	10-30	0.3-0.5	20-40
Loose sand		0.2-0.4	20-30	0.3-0.8	30-80
Medium sand	32-42	0.3-0.5	30-60	0.6-1	50-120
Dense sand		0.4-0.6	50-100	0.8-1.2	100-120
Gravel	35-45	0.4-0.7	80-150	0.8-1.5	150-350

Pile Bearing Capacity- Static Analysis – Indirect Approach

API (2011)

For cohesive soils

$$r_t = 9S_u$$

$$r_s = \alpha S_u$$

For $\Psi \leq 1 \rightarrow \alpha = 0.5\Psi^{-0.5}$
For $\Psi > 1 \rightarrow \alpha = 0.5\Psi^{-0.25}$
with the constraint that $\alpha \leq 1$

$\Psi = \frac{S_u}{p'_0\,(z)}$, $p'_0\,(z)$ = effective stress at depth z

For cohesionless soils

$$r_t = N_q \times \sigma'_{z=D_f}$$
$$r_s = \beta \times \sigma'_{z-avg}$$

Relative Density[a]	Soil Description	β	Limiting Shaft Friction Values (kPa)	N_q	Limiting End Bearing Values (MPa)
Very loose	Sand	Not applicable[d]	Not applicable[d]	Not applicable[d]	Not applicable[d]
Loose	Sand				
Loose	Sand-silt[b]				
Medium dense	Silt				
Dense	Silt				
Medium dense	Sand-silt[b]	0.29	67	12	3
Medium dense	Sand	0.37	81	20	5
Dense	Sand-silt[b]				
Dense	Sand	0.46	96	40	10
Very dense	Sand-silt[b]				
Very dense	Sand	0.56	115	50	12

Note: The listed parameters are intended as guidelines only. Other values may be justified in cases where detailed information such as CPT records, strength tests on high-quality samples, model tests, or pile driving performance, is available.

a: The definitions for the relative density percentage description are as follows:

Very loose, 0-15; Loose, 15-35; Medium dense, 35-65; Dense, 65-85; Very dense, 85-100.

b: Sand-silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

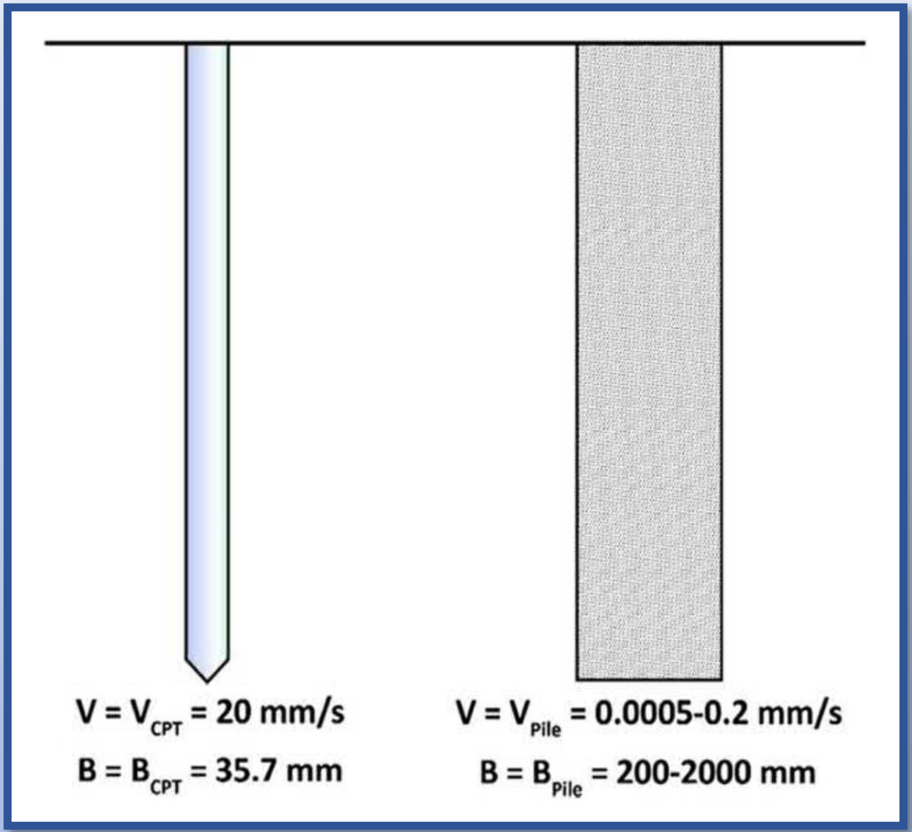
c; Design parameters given in previous editions for these soil/relative density combinations may be unconservative. Hence, it is recommended to use CPT-based methods.

1. Deep Foundations: Geotechnical Design
2. **CPT & Pile Scale Effects**
3. CPT- & CPTu-Based Methods
4. Worked Examples
5. Challenging Conditions
6. Case Histories

Scale Effect Correlations

- **Determinant Factors for Toe Capacity**

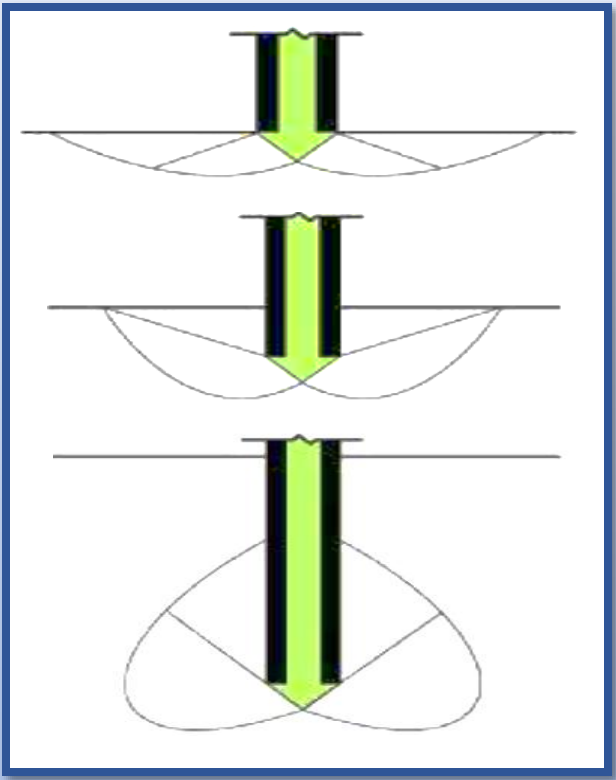
1. Embedment depth
2. Influence zone
3. Data production processing and averaging
4. Diameter
5. Nonhomogeneous condition
6. Penetration rate and failure mechanism
7. Ultimate capacity interpretation



Schematic view of pile and cone penetration test differences in material, penetration rate, and dimensions (Eslami et al., 2020)

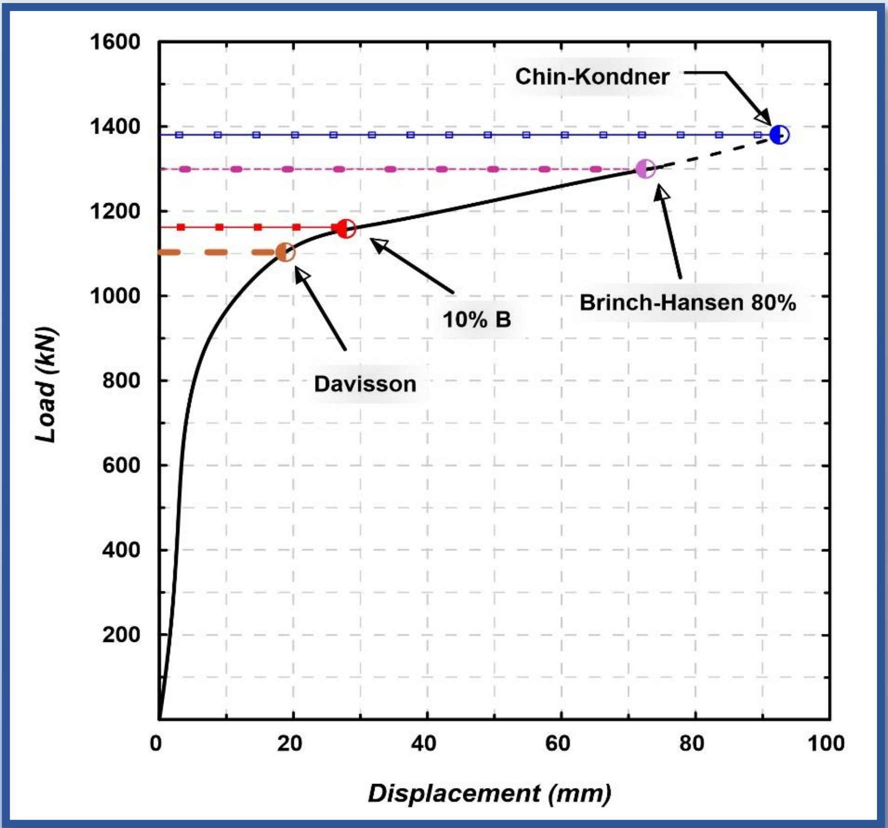
Scale Effect Correlations

Embedment Depth



Schematic view of transformation of shear failure from shallow to deep (Eslami et al., 2020)

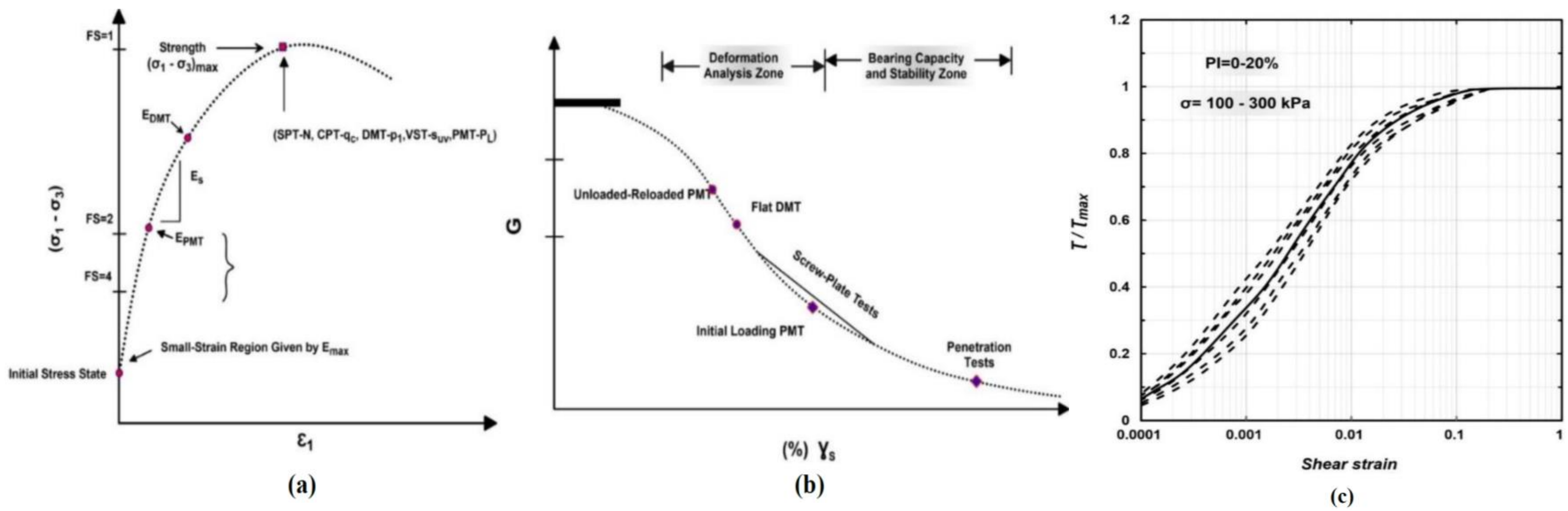
Ultimate Capacity Condition



Interpretation of load displacement diagram for Case 001-L&D31 (Moshfeghi & Eslami, 2016)

Scale Effect Correlations

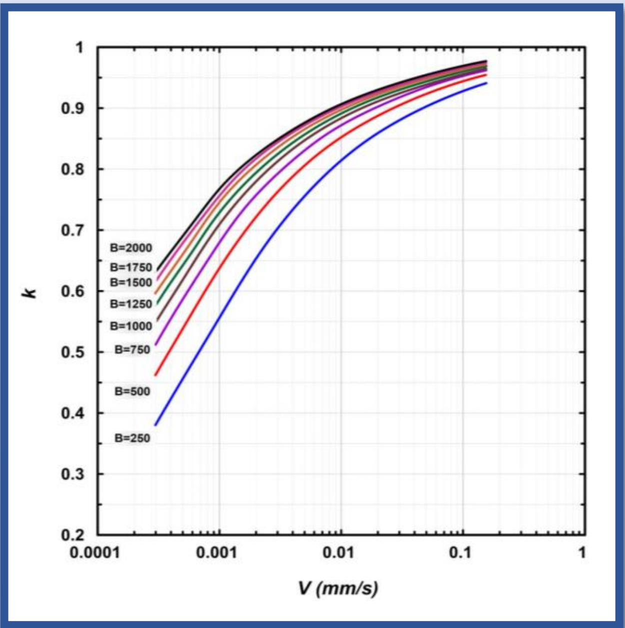
$$\frac{\gamma_{pile}}{\gamma_{CPT}} = (\frac{V_{pile}}{V_{CPT}})^{0.61} . (\frac{D_{pile}}{D_{CPT}})^{0.5} \qquad \gamma_{pile} = 0.0255\ (V_{pile})^{0.61} . (D_{pile})^{0.5}$$



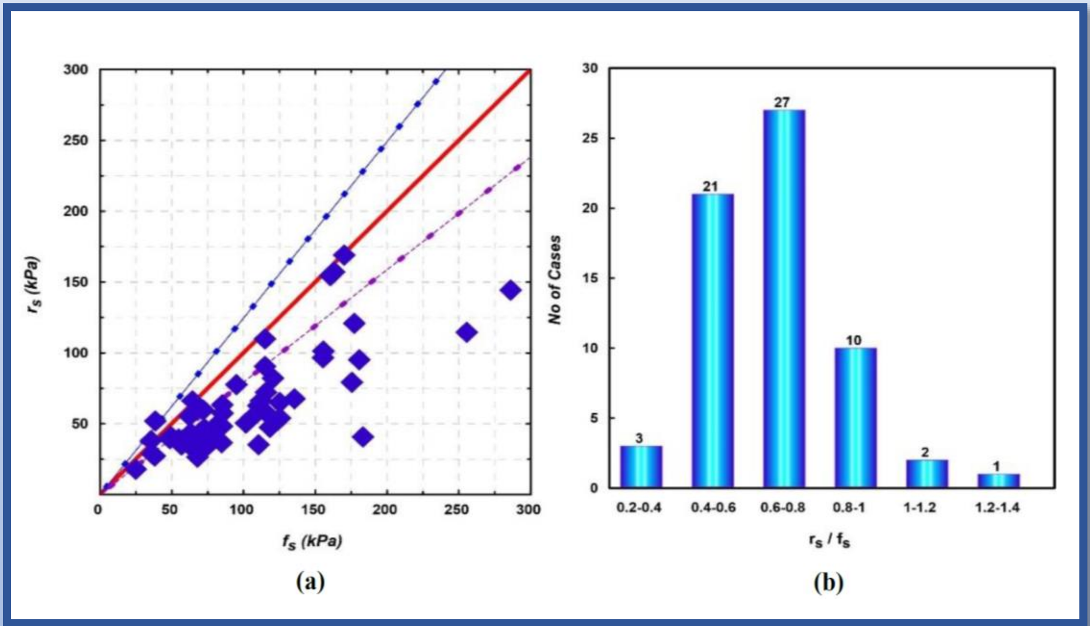
Stress strain Strength curves for different in situ tests; (a) strength measured by in situ tests at the peak of the stress strain curve, (b) variation of shear modulus with strain level (c) Variation of shear stress with shear strain (Sabatani et al., 2002)

Scale Effect Correlations

$$r_s = k.f_s$$



Determining k regarding pile diameter and pile penetration rate (Eslami et al., 2020)



(a) Comparison of r_s and f_s, (b) distribution of r_s/f_s values for Eslami et al. (2013) database (Eslami et al., 2020)

1 Deep Foundations: Geotechnical Design

2 CPT & Pile Scale Effects

3 CPT- & CPTu-Based Methods

4 Worked Examples

5 Challenging Conditions

6 Case Histories

Direct Application for Deep Foundations Axial Capacity

List of common CPT- and CPTu-based methods for pile bearing capacity

No.	Method/ Reference	No.	Method/ Reference
1	Begemann (1963, 1965, 1969)	15	Fugro-05 (Kolk et al. 2005)
2	Meyerhof (1956, 1976, 1983)	16	UCD-05 (Gavin and Lehane 2005)
3	Aoki and Velloso (1975)	17	ICP-05 (Jardine et al. 2005)
4	Nottingham (1975), Schmertmann (1978)	18	UWA-05 (Lehane et al. 2005)
5	Penpile (Clisby et al.1978)	19	NGI-05 (Clausen et al. 2005)
6	Dutch (de Ruiter & Beringen 1979)	20	Cambridge-05 (White & Bolton 2005)
7	Philipponnat (1980)	21	Togiliani (2008)
8	LCPC (Bustamante & Gianeselli 1982)	22	German (Kempfert and Becker 2010)
9	Cone-m (Tumay & Fakhroo 1982)	23	UCD-11 (Igoe et al. 2010, 2011)
10	Price and Wardle (1982)	24	V–K (Van Dijk and Kolk 2011)
11	Gwizdala (1984)	25	SEU (Cai et al. 2011, 2012)
12	UniCone (Eslami & Fellenius 1997)	26	HKU (Yu and Yang 2012)
13	KTRI (Takesue et al. 1998)	27	UWA-13 (Lehane et al. 2013)
14	TCD-03 (Gavin and Lehane 2003)	28	Modified UniCone (Niazi and Mayne 2016)

Summary of Commonly Used CPT-Based Methods

Method/references	Pile unit side resistance (r_s)	Pile unit end bearing (r_t)
Meyerhof (1976)	$r_s = kf_s$ k =1 $r_s = cq_c$ c =0.5%	$r_t = q_{c.a}c_1c_2$ $c_1 = \left(\frac{B+0.5}{2B}\right)^n, c_2 = \frac{D_b}{10B}$ D_b *bearing embedment depth* $n = 1\ (loose), 2\ (medium\ dense), 3\ (dense)$
LCPC (Bustamante and Gianeselli, 1982)	$r_s = \frac{1}{k_s} q_c$ $k_s = 30 - 150$	$r_t = \mathrm{k_b q_{eq}}$ $k_b = 0.4 \sim 0.55$
Dutch method (de Ruiter and Beringen 1979)	Compression: $r_s = \min[f_s, \frac{q_c}{300}, 120\ kPa]$ $Tension$: $r_s = \min[f_s, \frac{q_c}{400}, 120\ kPa]$	Similar to Nottingham (1975) and Schmertmann (1978)
Nottingham (1975) Schmertmann (1978)	$r_s = C_s q_c$ $r_s = Kf_s$ $C_s = 0.8{\sim}1.8\%$, $K = 0.8{\sim}2(sand)$	$r_t = q_{ca}$
Unicone (Eslami and Fellenius, 1997)	$r_s = c_{se} \times q_E$ $q_E = q_t - u_2$ $c_{se} = 0.3{\sim}8\%$	$r_t = c_{te} \times q_{Eg}$ $q_{cg} = (q_{c1} \times q_{c2} \times q_{c3} \times \cdots \times q_{cn})^{\frac{1}{n}}$ $c_{te} = 1$

Summary of Commonly Used CPT-Based Methods

UWA-05 method (Lehane et al., 2005)	unit side resistance (rs)	$r_s = \frac{f_t}{f_c}\left[0.03 q_c A_{rs,eff}{}^{0.3}\left[\max\left(\frac{h}{B},2\right)\right]^{-0.5} + \Delta\sigma'_{rd}\right]\tan\delta_f$ $A_{rs,eff} = 1 - IFR\left(\frac{B_i}{B}\right)^2, \frac{f_t}{f_c} = 1\ in\ compression, 0.75\ in\ tension$ $IFR_{mean} \approx min\left[1, (\frac{B_i(m)}{1.5(m)})^{0.2}\right]$
	unit end bearing (r_t)	$\frac{r_{t0.1}}{q_{c,avg}} = 0.15 + 0.45 A_{rb,eff}$ $A_{rb,eff} = 1 - FFR\left(\frac{B_i{}^2}{B^2}\right), FFR \approx min\left[1, (\frac{B_i(m)}{1.5(m)})^{0.2}\right]$
Fugro-05 method (Kolk et al., 2005)	unit side resistance (r_s)	Compression Loading: $h/R^* \geq 4 : r_s = 0.08 q_c \left(\frac{\sigma'_{v0}}{p_{ref}}\right)^{0.05}\left(\frac{h}{R^*}\right)^{-0.90}$ $h/R^* \leq 4: r_s = 0.08 q_c \left(\frac{\sigma'_{v0}}{p_{ref}}\right)^{0.05}(4)^{-0.90}\left(\frac{h}{4R^*}\right)$ Tension Loading: $r_s = 0.045 q_c \left(\frac{\sigma'_{v0}}{p_{ref}}\right)^{0.15}\left(\max(\frac{h}{R^*},4)\right)^{-0.85}$
	unit end bearing (r_t)	$r_{t0.1} = 8.5 q_{c,avg}(\frac{p_{ref}}{q_{c,avg}})^{0.5} A_r{}^{0.25}$ $A_r = 1 - \left(\frac{B_i{}^2}{B^2}\right)$

Summary of Commonly Used CPT-Based Methods

ICP-05 method (Jardine et al., 2005)	unit side resistance (r_s)	$r_s = a\left[0.029bq_c(\frac{\sigma'_{v0}}{p_{ref}})^{0.13}\left[\max(\frac{h}{R^*},8)\right]^{-0.38} + \Delta\sigma'_{rd}\right]\tan\delta_f$ a = 0.9 (OE piles in tension), 1.0 (all other cases), b = 0.8 (tension), 1.0 (compression), δ_f measured or estimated as fctn(d_{50})
	unit end bearing (r_t)	$\frac{r_{t0.1}}{q_{c,avg}} = max\left[1-0.5\log\left(\frac{B}{B_{CPT}}\right),0.3\right]$ The pile is fully plugged if: $B_i < 0.02(D_r-30)$ or $B_i < 0.083\left(\frac{q_{c,avg}}{p_{ref}}\right)B_{CPT}$ Fully plugged: $\frac{r_{t0.1}}{q_{c,avg}} = max\left[0.5-0.25\log\left(\frac{B}{B_{CPT}}\right),0.15,A_r\right]$ Coring: $\frac{q_{b0.1}}{q_{c,avg}} = A_r$
NGI-05 method (Clausen et al., 2005)	unit side resistance (r_s)	$r_s = \left(\frac{z}{Dp_{ref}F_{D_r}F_{sig}F_{tip}F_{load}F_{mat}}\right) \geq 0.1\sigma'_{v0}$ $F_{D_r} = 2.1(D_r-0.1)^{1.7}$, $F_{sig} = (\frac{\sigma'_{v0}}{p_{pa}})^{0.25}$, $F_{tip} = 1.0\ (driven\ OE), 1.6\ (driven\ CE)$ $F_{load} = 1.0\ (tension), 1.3\ (compression)$, $F_{mat} = 1.0\ (steel), 1.2\ (concrete)$
	unit end bearing (r_t)	Closed ended pile: $\frac{r_{t0.1}}{q_{c,tip}} = \frac{0.8}{1+D_r{}^2}$ Open ended pile: $Plugged$: $\frac{r_{t0.1}}{q_{c,tip}} = \frac{0.7}{1+3D_r{}^2}$ $Unplugged$: $r_{t0.1} = r_{t,ann}A_r + r_{t,plug}(1-A_r)$ $r_{t,ann} = q_{c,tip}$, $r_{t,plug} = \frac{12r_{s,avg}L}{\pi D_i}$, $r_{t0.1} = min(r_{t0.1,plugged}, r_{t0.1,unplugged})$

Meyerhof (1956, 1976, 1983)

Toe resistance: $r_t = q_{c.a} c_1 c_2$

$q_{c.a}$ = arithmetic average of q_c values in a zone ranging from "1b" below through "4b" above pile toe

$c_1 = \left(\frac{B+0.5}{2B}\right)^n$; modification factor for scale effect when $b > 0.5$, otherwise C_1=1

$c_2 = \frac{D_b}{10B}$; modification factor for penetration into dense strata when $D_b < 10b$, otherwise C_2=1

B = pile diameter (m)

n = an index; 1 for loose sand, 2 for medium dense sand, and 3 for dense sand

D_b = embedment of pile (m) in dense sand strata

Shaft resistance: $r_s = Kf_s$, $(K = 1)$; $r_s = cq_c$, $(c = 0.5\%)$

Eslami & Fellenius (1997)

Pile capacity by direct CPT and CPTu methods applied to 102 case histories

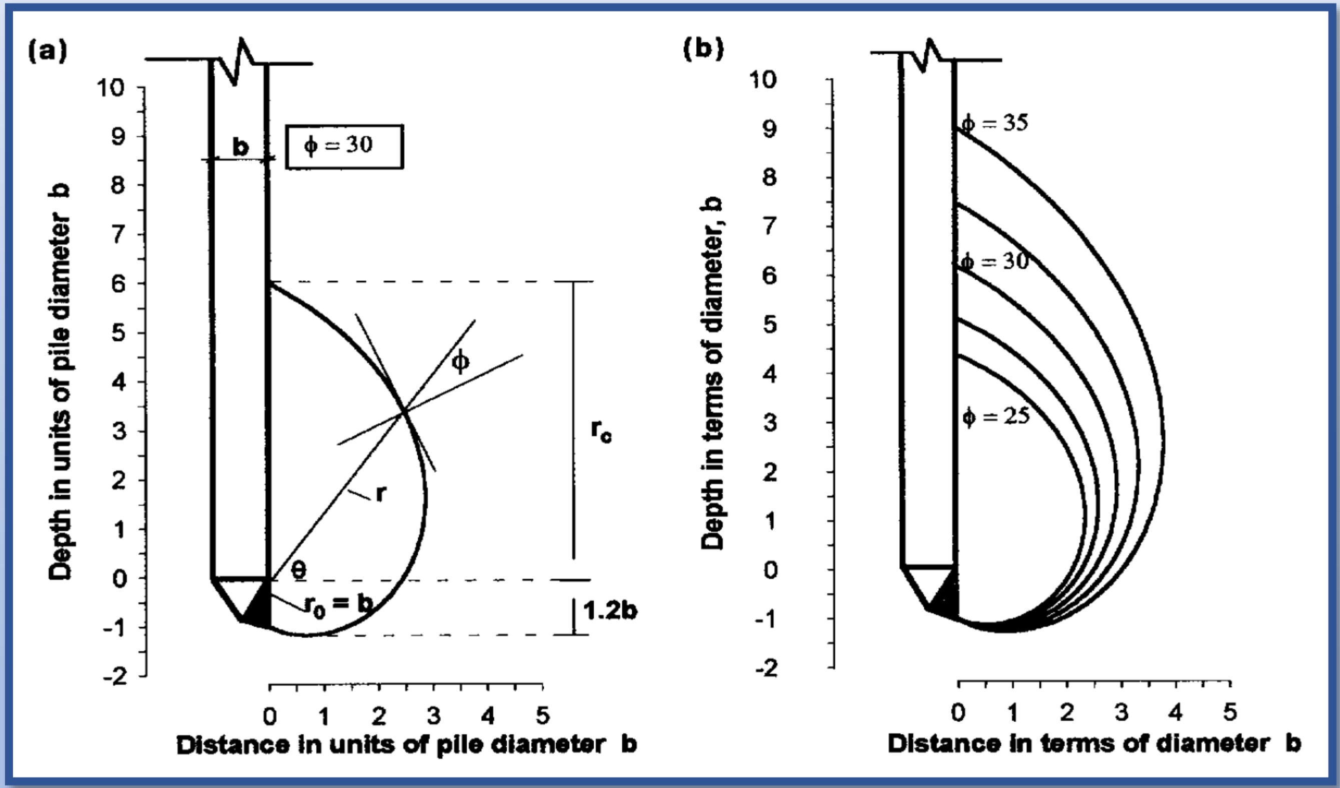
Abolfazl Eslami and Bengt H. Fellenius

Abstract: Six methods to determine axial pile capacity directly from cone penetration test (CPT) data are presented, discussed, and compared. Five of the methods are CPT methods that apply total stress and a filtered arithmetic average of cone resistance. One is a recently developed method, CPTu, that considers pore-water pressure and applies an unfiltered geometric average of cone resistance. To determine unit shaft resistance, the new method uses a new soil profiling chart based on CPTu data. The six methods are applied to 102 case histories combining CPTu data and capacities obtained in static loading tests in compression and tension. The pile capacities range from 80 to 8000 kN. The soil profiles range from soft to stiff clay, medium to dense sand, and mixtures of clay, silt, and sand. The pile embedment lengths range from 5 to 67 m and the pile diameters range from 200 to 900 mm. The new CPTu method for determining pile capacity demonstrates better agreement with the capacity determined in a static loading test and less scatter than by CPT methods.

Key words: cone penetration test, pile capacity, toe resistance, shaft resistance, soil classification.

Eslami & Fellenius (1997)

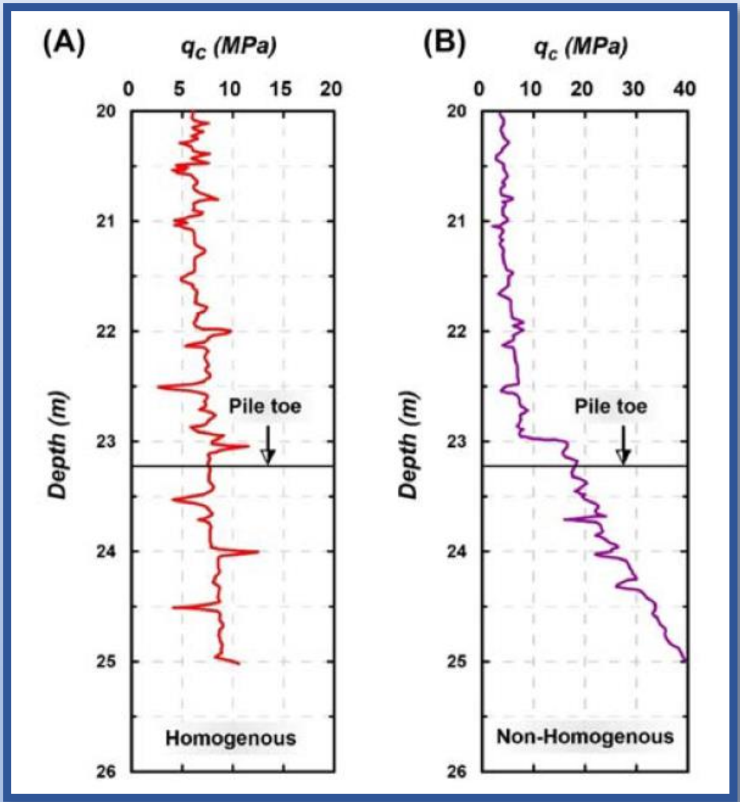
Toe Failure Zone



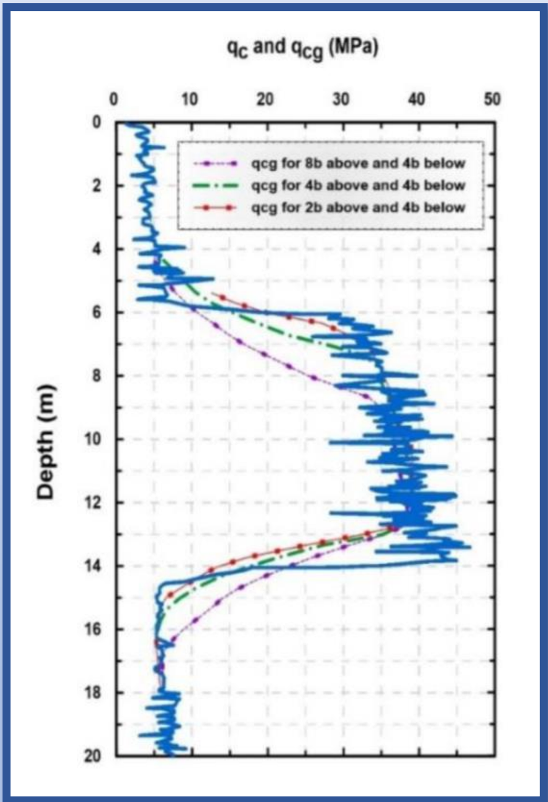
a) Principle of a logarithmic spiral rupture, b) rupture surfaces around pile toe for different soils (Eslami & Fellenius, 1997)

Eslami & Fellenius (1997)

Homogeneous and Nonhomogeneous Deposits



Comparison of pile unit toe resistance for different zones: (A) Homogeneous and (B) Nonhomogeneous



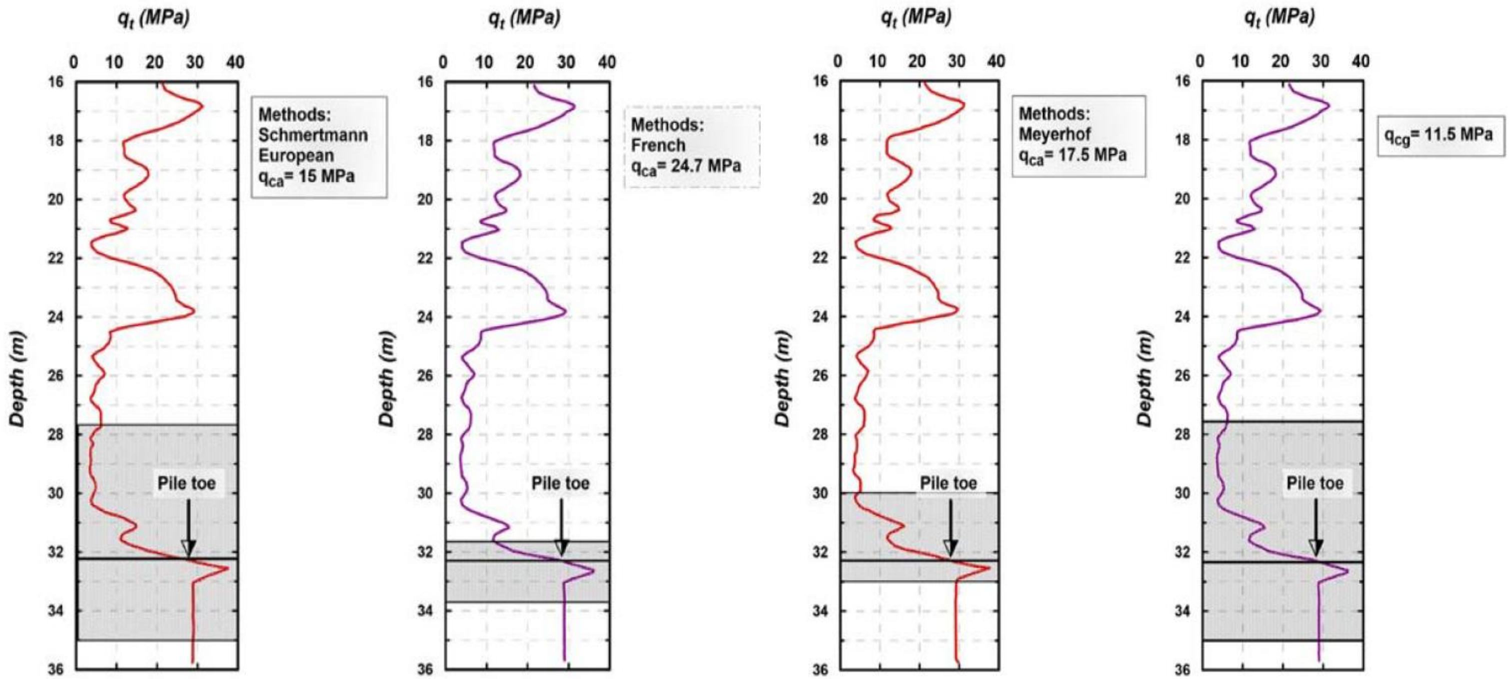
Comparison of cone resistance and calculated geometric average for a dense soil layer laid between loose layers

Eslami & Fellenius (1997)

Averaging

$$q_{ca} = \frac{q_{c1} + q_{c2} + \cdots + q_{cn}}{n}$$

$$q_{cg} = \sqrt{q_{c1} \times q_{c2} \times \cdots \times q_{cn}}$$



Example of comparison of average cone resistance for different CPT methods (Eslami & Fellenius, 1997)

Eslami & Fellenius (1997)

- **Toe Capacity**

$$r_t = c_t \times q_{Eg}$$

$$q_E = q_t - u$$

$$q_t = q_c + (1 - a)u_2$$

- **Shaft Capacity**

$$r_s = c_s \times q_{Eg}$$

$$q_{Eg} = \sqrt[n]{q_{E1} \times q_{E2} \times \cdots \times q_{En}}$$

Shaft coefficient correlation

Soil type	Cs
Soft sensitive soils	8.0%
Clay	5.0%
Stiff clay and mixture of clay and silt	2.5%
Mixture of silt and sand	1.0%
Sand	0.4%

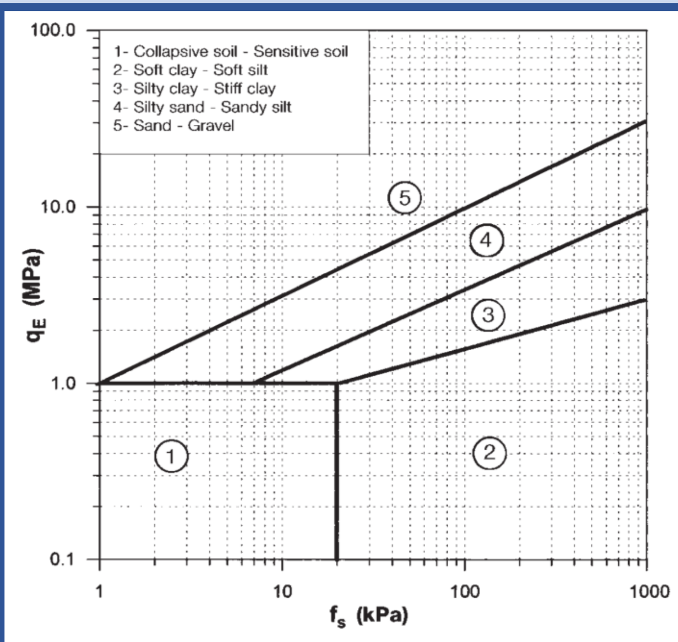
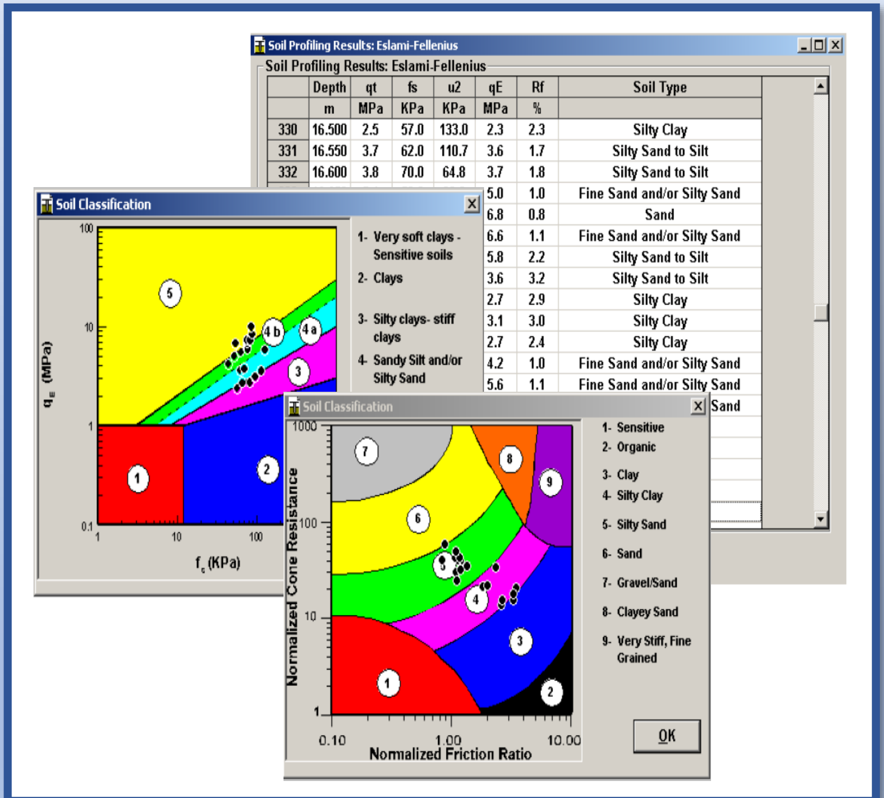


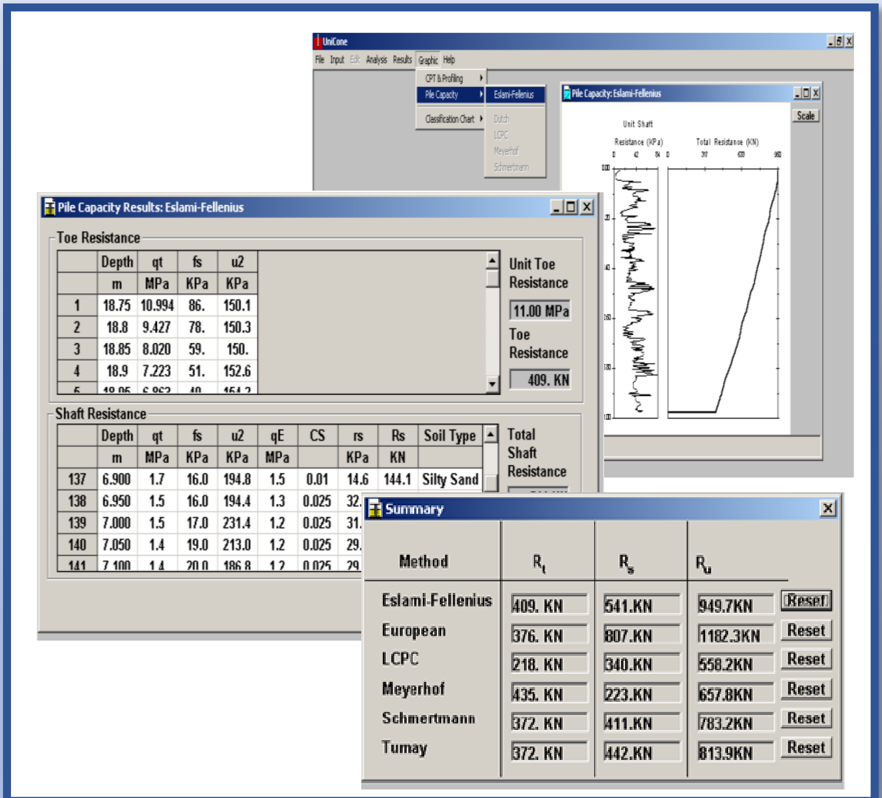
Chart for soil classification (Eslami & Fellenius, 1997)

UniCone Software (Fellenius, Infante & Eslami, 2002)

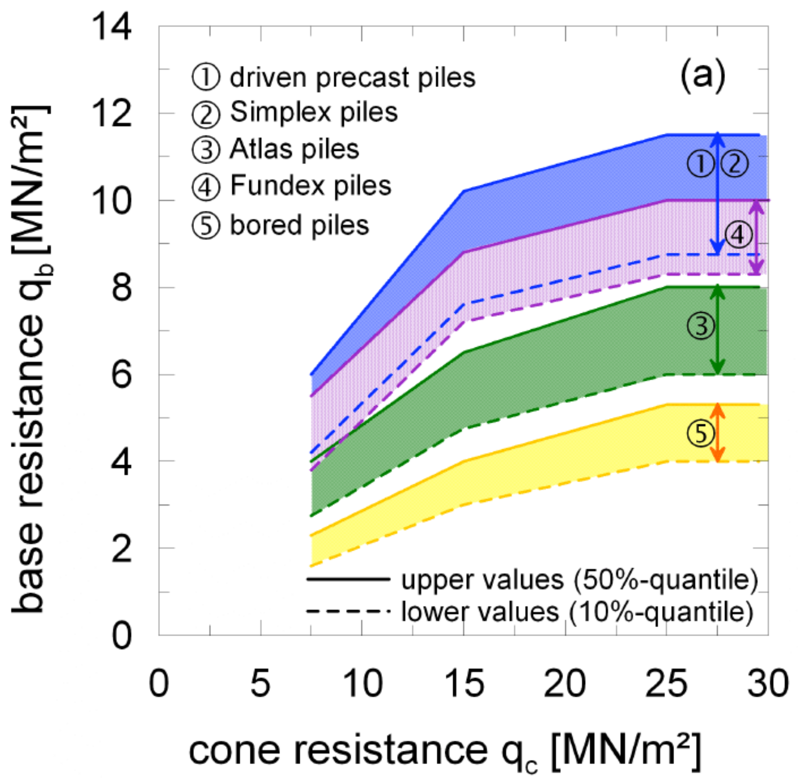
Pile Capacity Calculation

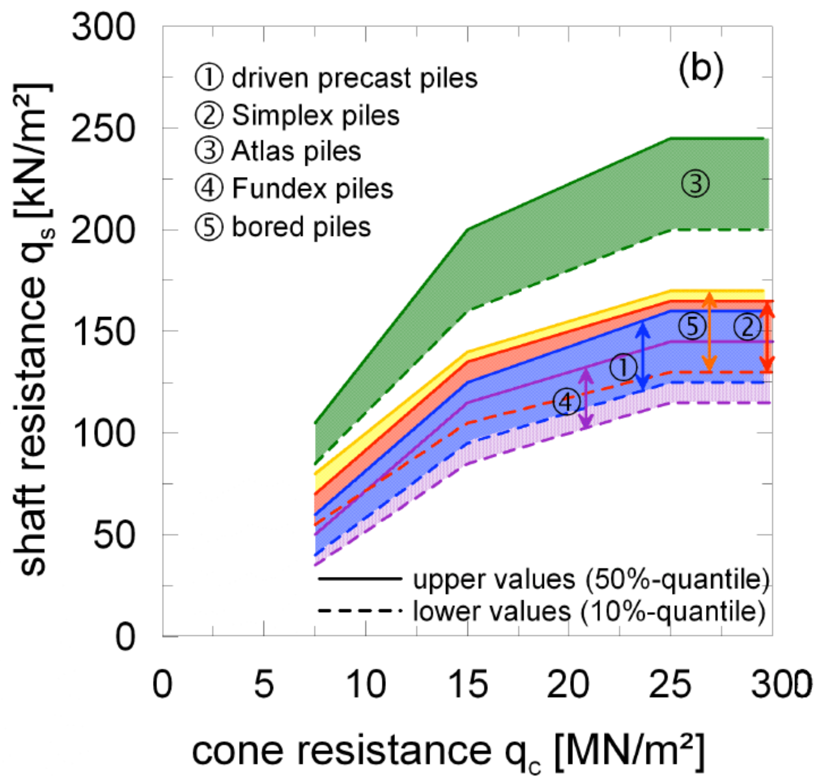


Soil Profiling



German Method (Kempfert & Becker, 2010)





1000 case records

Comments on the Methods

- The methods developed in 70s and 80s do not consider the more accurate measurements achievable by CPTu, since, it was before the piezocone was generally available.
- While the recommendations are specified to soil type (clay and sand) for a few methods, none of them, except for Eslami and Fellenius (1997) and enhanced UniCone (Niazi and Mayne, 2016), include a means for identifying the soil type from CPT data. Instead, the soil profile governing the coefficients relies on information from conventional boring and sampling, and laboratory testing, which may not be fully relevant to the CPT data.
- All of the CPT-based methods include random smoothing and filtering of the CPT data, that is, elimination of peaks and troughs that exposes the results to considerable subjective operator influence.
- The cone resistance (total resistance) has not been corrected for the pore pressure on the cone shoulder and, therefore, the data behind the methods include errors—smaller in sand, larger in clay. This matter, i.e. penetration pore pressure, u2, is realized by Eslami and Fellenius (1997).
- Most of the older methods employ total stress values, whereas in long term, effective stress governs pile capacity.

Comments on the Methods

- **Some of the methods are locally developed, that is, they are based on limited types of piles and soils, such as Schmertmann (1978) and Tummay and Fakhroo (1982).**
- **The upper limit resistance imposed on the unit toe resistance in the Schmertmann is not reasonable in very dense sands where values of pile unit toe resistance, rt, higher than 15 MPa frequently occur.**
- **Most of the direct methods involve a judgment in selecting the coefficient to apply to the average cone resistance to arrive at the unit toe resistance.**
- **Some methods such as Eslami and Fellenius (1997), NGI (2005), ICP (2005), UWA (2005), specify a certain criterion for evaluating the pile capacity from static loading test results that can be used as reference to the pile capacity estimated from CPT data. While, other methods have not introduced any criteria for pile ultimate capacity. Yet, the capacity of a pile is determined from the results of static loading tests, varies considerably with the method used to evaluate the test (Fellenius, 1975).**
- **The NGI (2005), ICP (2005), Fugro (2005), and UWA (2005) methods are included in the commentary of the new 22nd edition of the API RP 2A Recommendations (2006) and are applicable for displacement piles in sand. They are more or less following a similar format. For instance, they all consider the effects of friction fatigue and toe condition in open end piles. Also, except for the Fugro method, the dilation effects during pile loading are accounted.**

1 Deep Foundations: Geotechnical Design

2 CPT & Pile Scale Effects

3 CPT- & CPTu-Based Methods

4 **Worked Examples**

5 Challenging Conditions

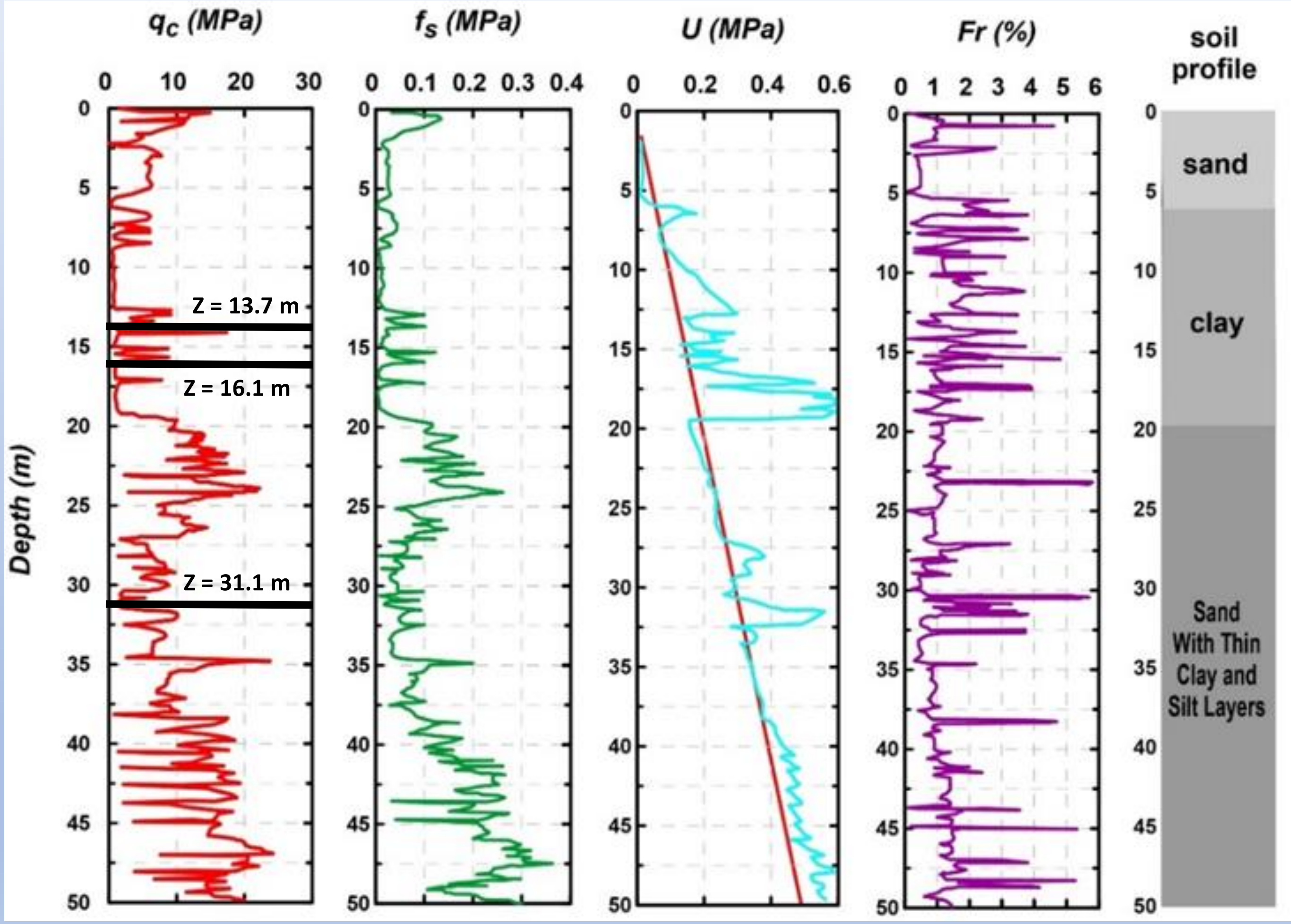
6 Case Histories

Example 2.1

The CPTu results of Fraser Delta, B.C., Canada are as shown below. The digitized CPTu in 0.5 m intervals is also presented. The bearing capacity of a pile with given specifications, using Eslami and Fellenius, Meyerhof, LCPC, and Schmertmann, would be as follows:

The pile is driven with a diameter of 324 mm and an embedment length of 13.7, 16.1, and 31.1. The results derived from the static pile load test are also presented for comparison.

Example 2.1



q_c (MPa)
f_s (MPa)
U (MPa)
Fr (%)
soil profile
Depth (m)
Z = 13.7 m
Z = 16.1 m
Z = 31.1 m
sand
clay
Sand With Thin Clay and Silt Layers

Example 2.1

Unit shaft resistance for the pile with a length of 31.1 m.

The geometric mean of cone resistance from surface to pile toe:

According to the CPTu profile, two layers are considered: one from the depth of 0-17m and below.

$R_s = C_s . q_E$

In soft clay (0-17.0 m): C_s**=0.08,** $\overline{q}_E = 0.1895$ MPa

In sand (17.0-31.0 m): C_s **=0.003,** $\overline{q}_E = 9.017$ MPa

$r_{1s} = 0.185 \times 0.08 = 0.0152$ MPa

$r_{2s} = 9.017 \times 0.003 = 0.027$ MPa

$R_s = r_s \times A_s$

$R_{1s} = 0.0152 \times \pi \times 0.324 \times 17 = 0.263$ MN $= 263$ kN

$R_{2s} = 0.027 \times \pi \times 0.324 \times (31.1 - 17) = 0.387$ MN $= 387$ kN

$R_s = R_{1s} + R_{2s} = 263 + 387 = 650$ kN

Example 2.1

Unit toe resistance:

-the averaging zone is 8B above and 4B below the pile toe. Considering the pile embedment depth, the averaging zone depth is from 29.5 m to 33.5 m.

$$r_t = C_t \times \overline{q}_{\mathrm{E(geo)}}$$

$$\overline{q}_{\mathrm{E(geo)}} = 1.587\mathrm{MPa}$$

$$r_t = 1 \times 1.587 = 1.587$$

$$R_t = A_t \times r_t = \frac{\pi \times 0.324^2}{4} \times 1.587 = 0.131\mathrm{MN} = 131\ \mathrm{kN}$$

$$\mathrm{R} = R_t + R_s = 1005 + 131 = 1136\ \mathrm{kN}$$

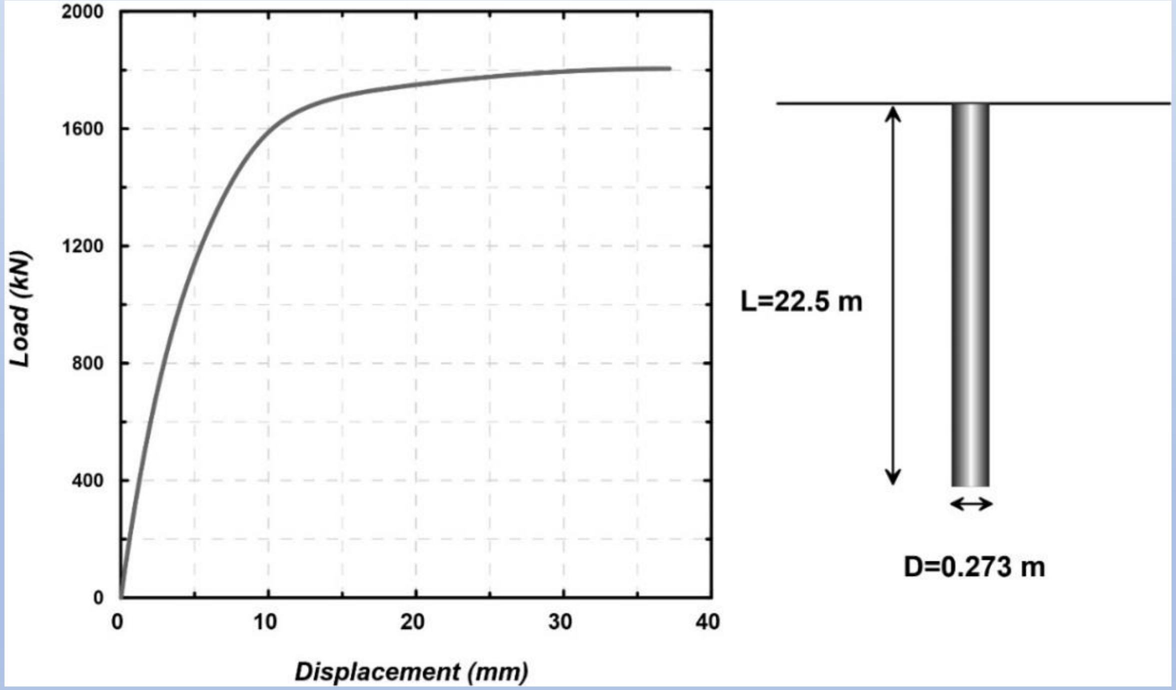
Example 2.1

The summary of the results for other methods and pile embedment depths is as follows:

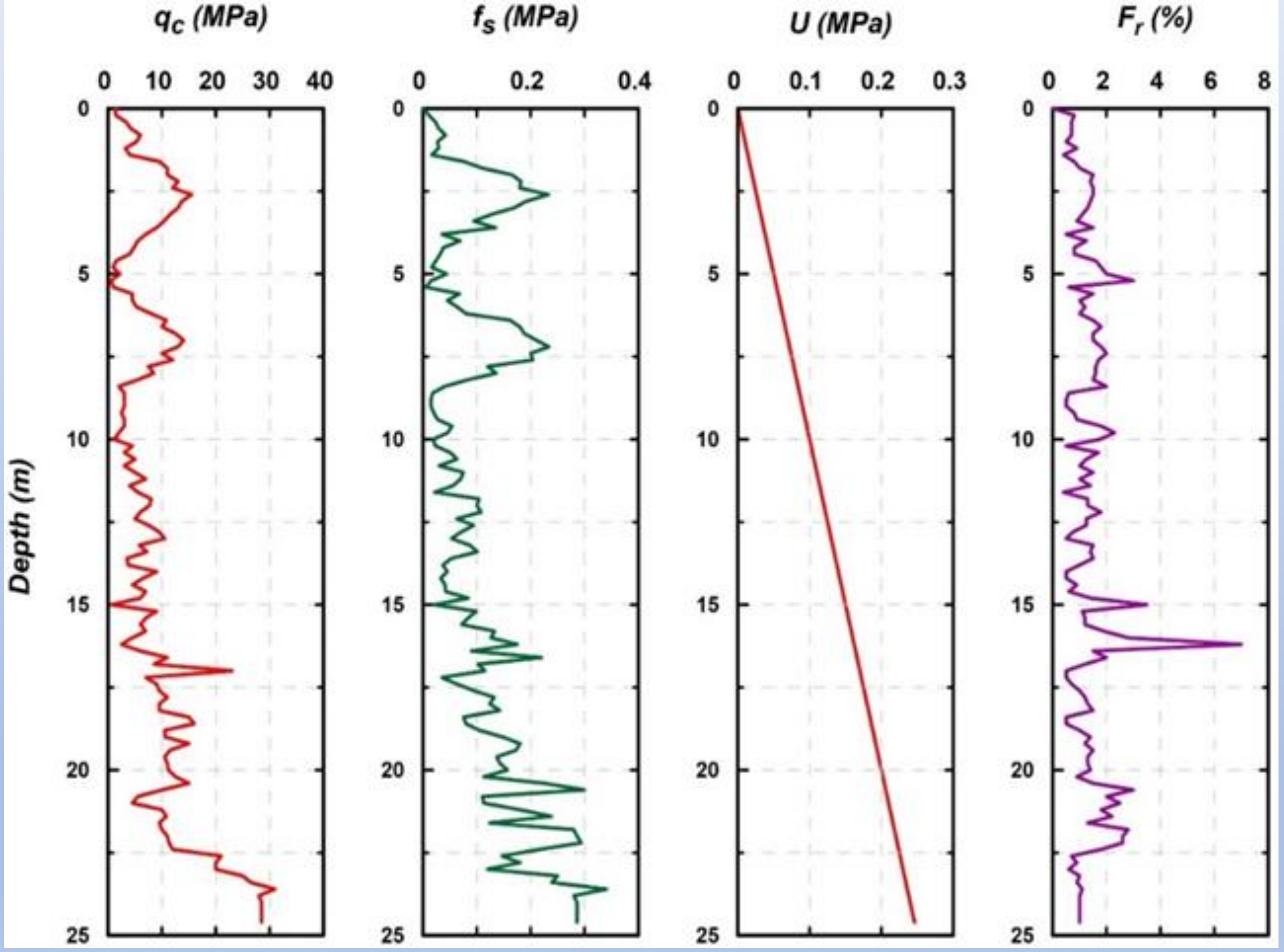
Pile No.	Bearing Capacity (kN)			
	Schmertmann	LCPC	UniCone	Measured Capacity
1	141	217	244	290
2	431	633	630	630
3	825	1053	1136	1100

Example 2.2

The bearing capacity of a closed-end driven steel pipe pile with the length and diameter of 22.5 m and 273 mm by ten of the presented methods.



Example 2.2



q_c (MPa)
0 10 20 30 40
f_s (MPa)
0 0.2 0.4
U (MPa)
0 0.1 0.2 0.3
F_r (%)
0 2 4 6 8
Depth (m)
0 5 10 15 20 25

Example 2.2

Results of Pile capacity estimations using CPT-based direct methods are encapsulated below.

Method	Bearing Capacity (kN)
Brinch Hansen 80% criterion	1620
Eslami and Fellenius (1997)	1713
LCPC	1161
Meyerhof	1451
Schmertmann	2078
UWA	1379
NGI	1410
Fugro	1162
ICP	1438
German-Upper Bound	1836
German-Lower Bound	1291
German-Average	1564

Example 2.3

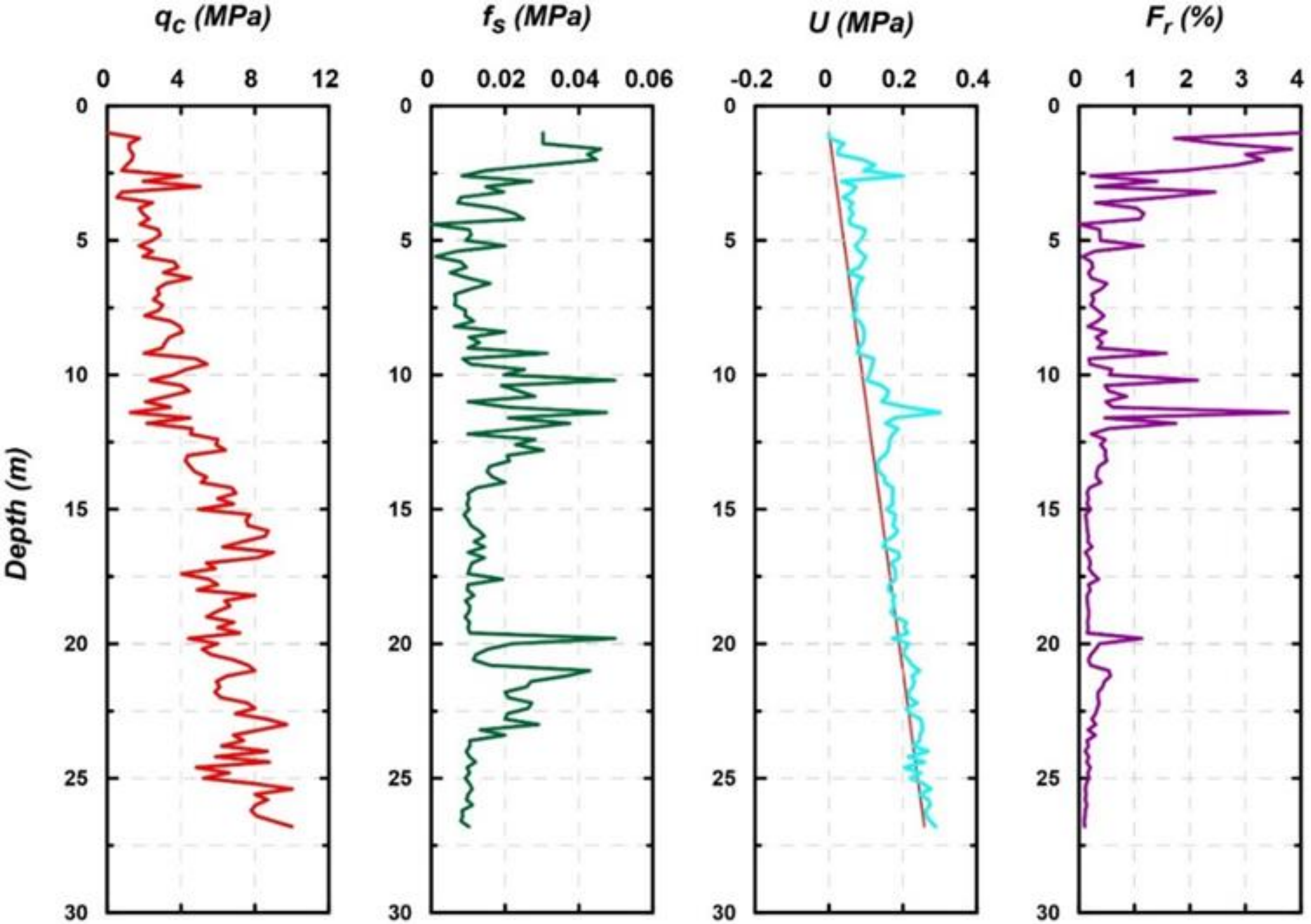
The bearing capacity of a square concrete pile with the characteristics presented below is as follows:

Case ID	134-1-FITTJA C
Reference	Axelsson (1998)
Location	Sweden
Shape	Square
Material	Concrete
Installation	Driven
Embedment Length, D (m)	19.0
Diameter, B (mm)	235
Cross Sectional Area, At (m2)	0.055
Perimeter (m)	0.940
GWL (m)	1.0

Example 2.3

Bearing Capacity from Dynamic Load Test (CAPWAP)		
1 day after installation	Toe Capacity, R_t (kN)	349
	Shaft Capacity, R_s (kN)	387
	Total Capacity, R_u (kN)	736
72 days after installation	Toe Capacity, R_t (kN)	319
	Shaft Capacity, R_s (kN)	1122
	Total Capacity, R_u (kN)	1441

Example 2.3



q_c (MPa)
0
4
8
12
f_s (MPa)
0
0.02
0.04
0.06
U (MPa)
-0.2
0
0.2
0.4
F_r (%)
0
1
2
3
4
Depth (m)
0
5
10
15
20
25
30

Example 2.3

Toe capacity:

calculating the average cone resistance using the minimum path rule.

$$q_{c,\,avg\ (0.7B\ below\ pile\ \mathrm{tip})} = 5.38\ \mathrm{MPa}$$

$$q_{c,\,avg\ (4B\ below\ pile\ \mathrm{tip})} = 5.96\ \mathrm{MPa}$$

$$\rightarrow q_{c,\,avg\ (0.7\mathrm{B\ bpt})} < q_{c,avg\ (4B\ \mathrm{bpt})}$$

$$q_{c,\,min\ (0.7B\ below\ pile\ \mathrm{tip})} = 5.38\ \mathrm{MPa}$$

$$q_{c,\,avg\ (8B\ above\ pile\ \mathrm{tip})} = 4.96\ \mathrm{MPa}$$

$$\rightarrow q_{c,\,avg} = \frac{\frac{5.38+5.38}{2} + 4.96}{2} = 5.17\ \mathrm{MPa}$$

The pile is closed-end $\rightarrow \frac{q_{b0.1}}{q_{c,avg}} = 0.6 \rightarrow q_{b0.1} = 0.6 \times 5.17 = 3.102\ \mathrm{MPa} = 3102\ \mathrm{kPa}$

$$Q_b = 3102 \times 0.055 = 171\ \mathrm{kN}$$

Example 2.3

$A_{rs,eff} = 1, B_{eff} = B = 0.265\ m, \Delta r = 0.02\ mm, tan\,\delta_f = 0.52$

Calculation of the pile capacity using the UWA method is as follows:

Z (m)	h/B	q_{c1N}	G (MPa)	$\Delta\sigma'_{rd}$ (kPa)	r_s (kPa)	ΔZ(m)	P(m)	ΔR_S (kN)	R_S (kN)
1.00	76.596	12364.7	0.01	0.00	0.07	0	0.940	0.00	0.00
1.40	74.894	63.37	12.19	4.15	4.33	0.40	0.940	1.63	1.63
1.80	73.191	52.46	16.28	5.54	5.46	0.40	0.940	2.05	3.68
2.20	71.489	32.03	17.20	5.86	4.99	0.40	0.940	1.88	5.56
2.60	69.787	105.87	28.43	9.68	12.55	0.40	0.940	4.72	10.27
3.00	68.085	118.73	32.90	11.20	15.36	0.40	0.940	5.78	16.05
3.40	66.383	11.36	17.82	6.07	4.17	0.40	0.940	1.57	17.62
3.80	64.681	34.34	26.83	9.13	8.10	0.40	0.940	3.05	20.67
4.20	62.979	42.38	30.55	10.40	9.89	0.40	0.940	3.72	24.38
4.60	61.277	48.60	33.76	11.49	11.50	0.40	0.940	4.32	28.71
5.00	59.574	40.62	33.72	11.48	10.91	0.40	0.940	4.10	32.81

Example 2.3

5.40	57.872	39.30	35.02	11.92	11.28	0.40	0.940	4.24	37.05
5.80	56.170	55.13	40.49	13.78	14.73	0.40	0.940	5.54	42.59
6.20	54.468	44.53	39.53	13.46	13.45	0.40	0.940	5.06	47.65
6.60	52.766	44.84	41.10	13.99	14.13	0.40	0.940	5.31	52.96
7.00	51.064	38.53	40.66	13.84	13.39	0.40	0.940	5.04	58.00
7.40	49.362	39.71	42.37	14.42	14.21	0.40	0.940	5.34	63.34
7.80	47.660	25.99	38.46	13.09	11.41	0.40	0.940	4.29	67.63
8.20	45.957	48.94	47.85	16.29	17.55	0.40	0.940	6.60	74.23
8.60	44.255	40.74	46.53	15.84	16.15	0.40	0.940	6.07	80.30
9.00	42.553	35.10	45.65	15.54	15.22	0.40	0.940	5.72	86.03
9.40	40.851	55.20	53.58	18.24	21.22	0.40	0.940	7.98	94.01
9.80	39.149	48.00	52.59	17.90	19.98	0.40	0.940	7.51	101.52
10.20	37.447	25.60	44.53	15.16	13.84	0.40	0.940	5.20	106.72
10.60	35.745	47.81	54.87	18.68	21.33	0.40	0.940	8.02	114.74
11.00	34.043	21.84	44.27	15.07	13.39	0.40	0.940	5.04	119.78
11.40	32.340	13.00	38.64	13.15	10.30	0.40	0.940	3.87	123.65
11.80	30.638	21.81	45.99	15.65	14.22	0.40	0.940	5.35	129.00
12.20	28.936	44.98	58.19	19.81	23.42	0.40	0.940	8.81	137.80

Example 2.3

12.60	27.234	57.69	63.80	21.72	28.95	0.40	0.940	10.88	148.69
13.00	25.532	41.89	58.95	20.07	23.90	0.40	0.940	8.99	157.68
13.40	23.830	42.44	60.16	20.48	25.01	0.40	0.940	9.40	167.08
13.80	22.128	50.12	64.25	21.87	29.25	0.40	0.940	11.00	178.07
14.20	20.426	61.82	69.49	23.66	35.60	0.40	0.940	13.39	191.46
14.60	18.723	54.07	67.76	23.07	33.60	0.40	0.940	12.63	204.09
15.00	17.021	43.92	64.59	21.99	30.11	0.40	0.940	11.32	215.41
15.40	15.319	66.09	74.05	25.21	43.14	0.40	0.940	16.22	231.63
15.80	13.617	75.55	78.14	26.60	50.75	0.40	0.940	19.08	250.72
16.20	11.915	62.93	74.97	25.52	46.59	0.40	0.940	17.52	268.23
16.60	10.213	75.98	80.36	27.36	58.24	0.40	0.940	21.90	290.13
17.00	8.511	44.78	69.45	23.64	41.07	0.40	0.940	15.44	305.57
17.40	6.809	33.07	64.20	21.86	35.42	0.40	0.940	13.32	318.89
17.80	5.106	48.65	72.96	24.84	54.27	0.40	0.940	20.41	339.30
18.20	3.404	64.35	80.28	27.33	81.99	0.40	0.940	30.83	370.13
18.60	1.702	52.89	76.57	26.07	87.08	0.40	0.940	32.74	402.87
19.00	0.000	42.29	72.41	24.65	72.27	0.40	0.940	27.18	430.05

Example 2.3

A summary of the bearing capacity values from CPT-based methods is presented in the below Table.

	UWA-05	NGI-05	ICP-05	German Method		
				Upper bound	Lower bound	Average value
R_s (kN)	430	408	419	519	266	392
R_t (kN)	171	223	234	266	181	223
R_u (kN)	601	631	653	784	447	615

1 Deep Foundations: Geotechnical Design

2 CPT & Pile Scale Effects

3 CPT- & CPTu-Based Methods

4 Worked Examples

5 Challenging Conditions

6 Case Histories

Challenging Conditions

- **Subsurface condition**
- **Superstructure loading combination**
- **Environmental aspects**
- **Construction constraints**
- **Serviceability requirements**
- **Natural and artificial disasters**

1 Deep Foundations: Geotechnical Design

2 CPT & Pile Scale Effects

3 CPT- & CPTu-Based Methods

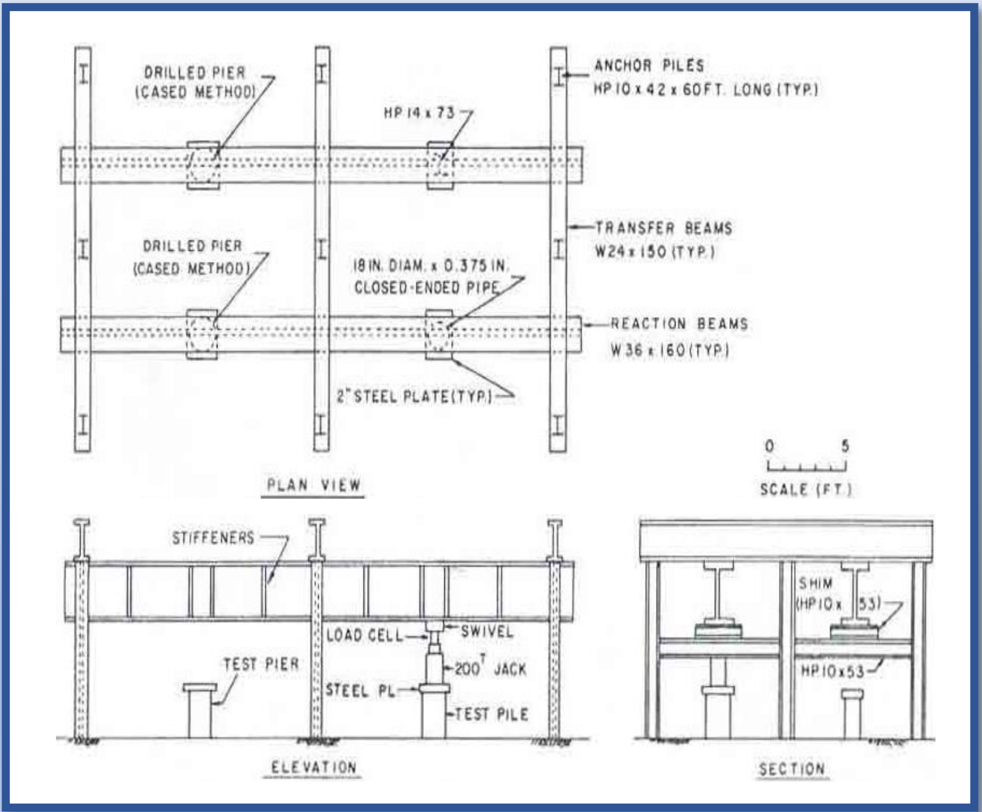
4 Worked Examples

5 Challenging Conditions

6 Case Histories

Case No. 1: Four Piles Prediction Symposium (Finno et al., 1989)

- **Lakefill site on the Evanston Campus of Northwestern University**
- **in conjunction with the 1989 Foundation Engineering Congress**
- **23 predictors were involved**

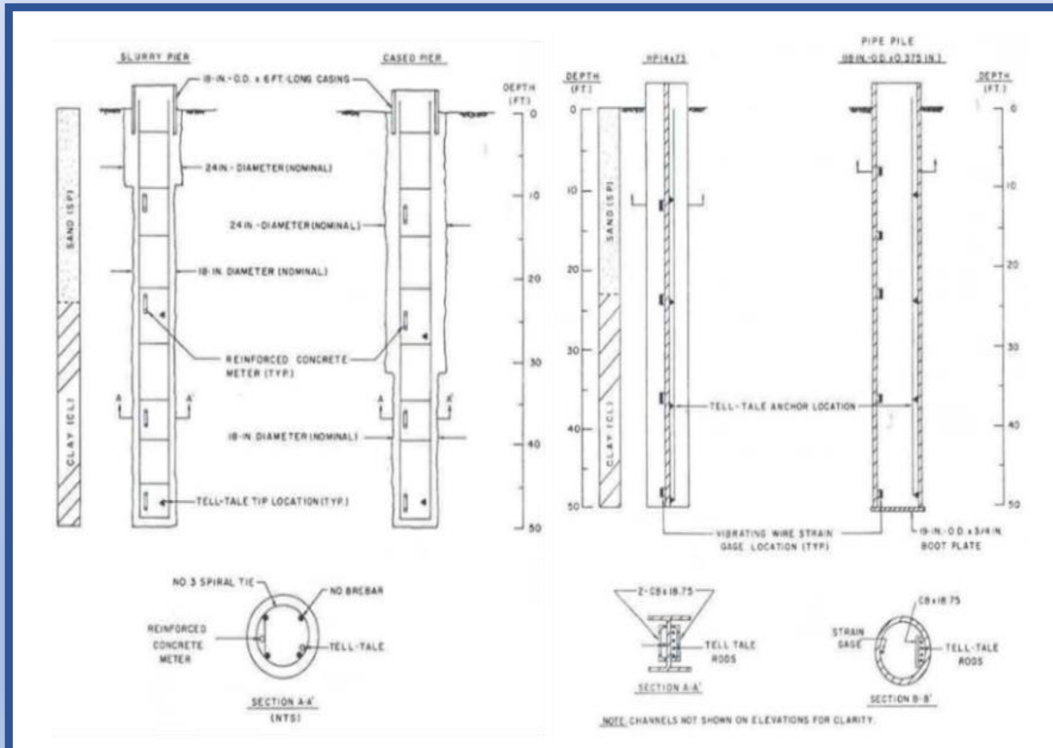


Test piles and reaction frames (Finno et al., 1989)

Case No. 1: Four Piles Prediction Symposium (Finno et al., 1989)

Piles ($\mathbf{D_f = 15.2\ m}$)

- **Two Driven**
  - ❖ **Pipe Pile: 450 mm closed-end, 9.5 mm wall, 480 mm toe-plate**
  - ❖ **H Pile: 355HP120 (14HP73) pile**
- **Two Bored (D = 450 mm)**
  - ❖ **bentonite slurry to the full depth**
  - ❖ **casing to a depth of 9.4 m**

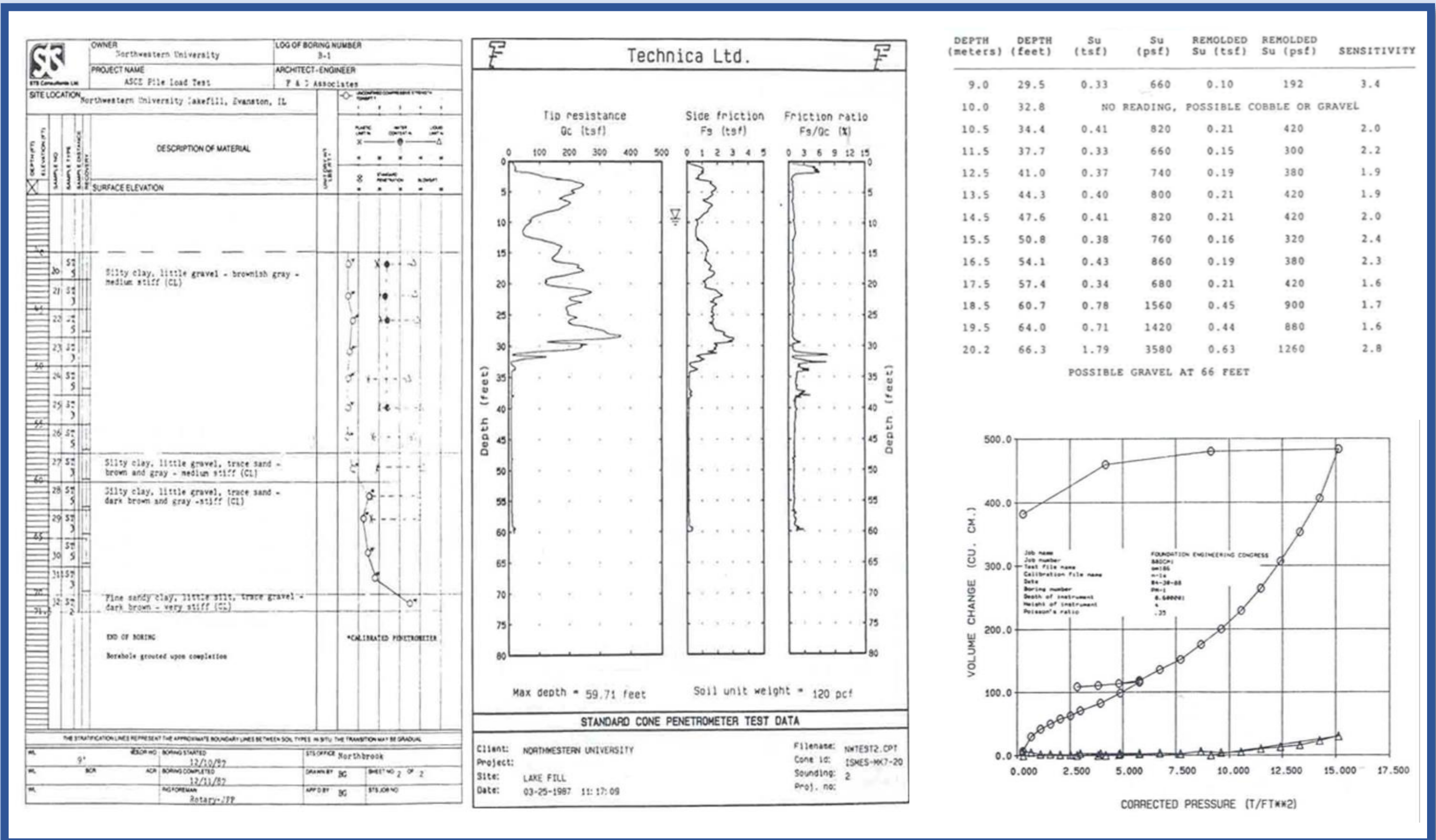


Details of test piles (Finno et al., 1989)

Case No. 1: Four Piles Prediction Symposium (Finno et al., 1989)

Soil Profile

- Sand (7 m in thickness)
- Clay

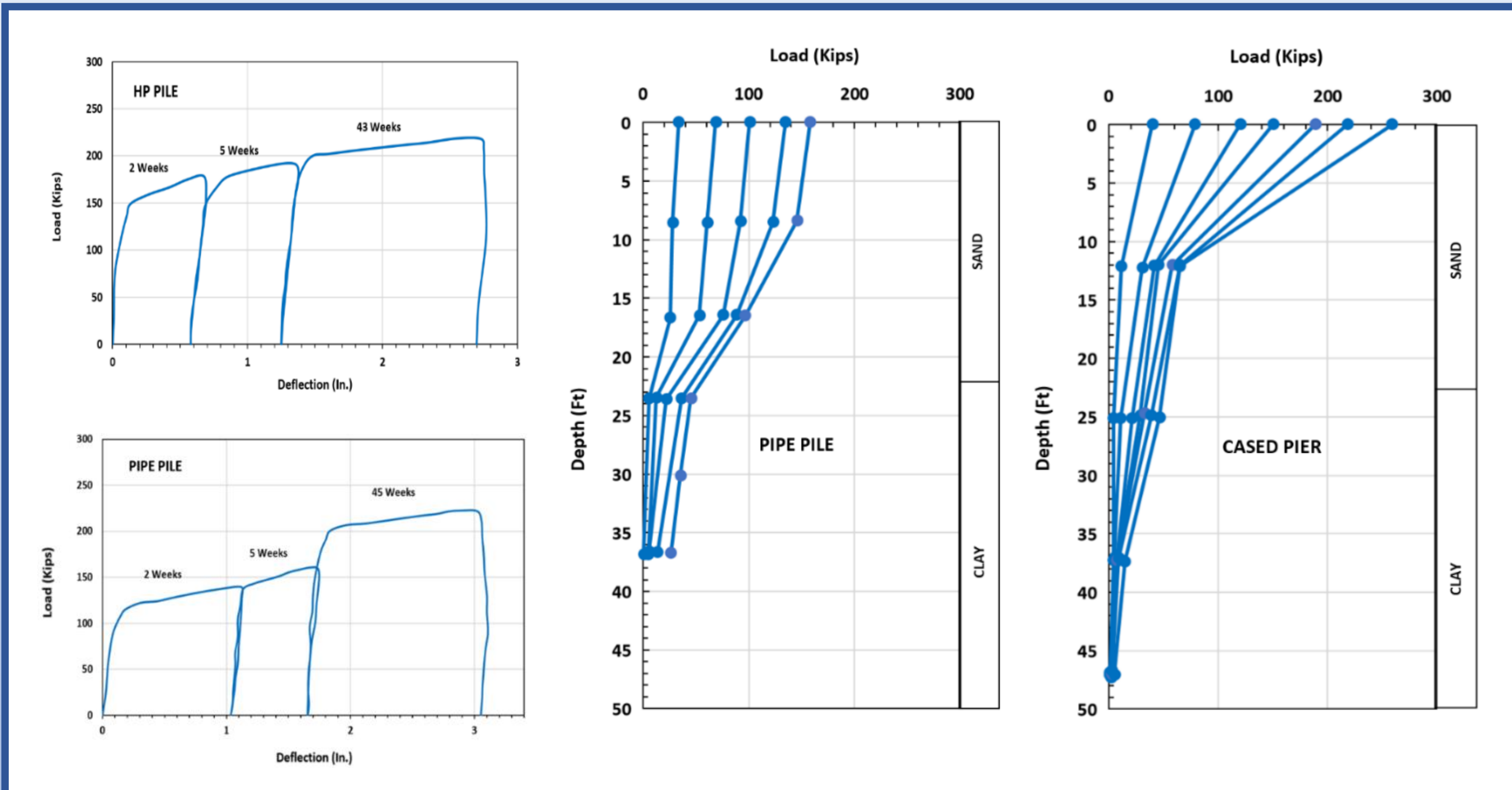


DEPTH (meters)	DEPTH (feet)	Su (tsf)	Su (psf)	REMOLDED Su (tsf)	REMOLDED Su (psf)	SENSITIVITY
9.0	29.5	0.33	660	0.10	192	3.4
10.0	32.8	NO READING, POSSIBLE COBBLE OR GRAVEL				
10.5	34.4	0.41	820	0.21	420	2.0
11.5	37.7	0.33	660	0.15	300	2.2
12.5	41.0	0.37	740	0.19	380	1.9
13.5	44.3	0.40	800	0.21	420	1.9
14.5	47.6	0.41	820	0.21	420	2.0
15.5	50.8	0.38	760	0.16	320	2.4
16.5	54.1	0.43	860	0.19	380	2.3
17.5	57.4	0.34	680	0.21	420	1.6
18.5	60.7	0.78	1560	0.45	900	1.7
19.5	64.0	0.71	1420	0.44	880	1.6
20.2	66.3	1.79	3580	0.63	1260	2.8

POSSIBLE GRAVEL AT 66 FEET

Summary of in situ tests results (Finno et al., 1989)

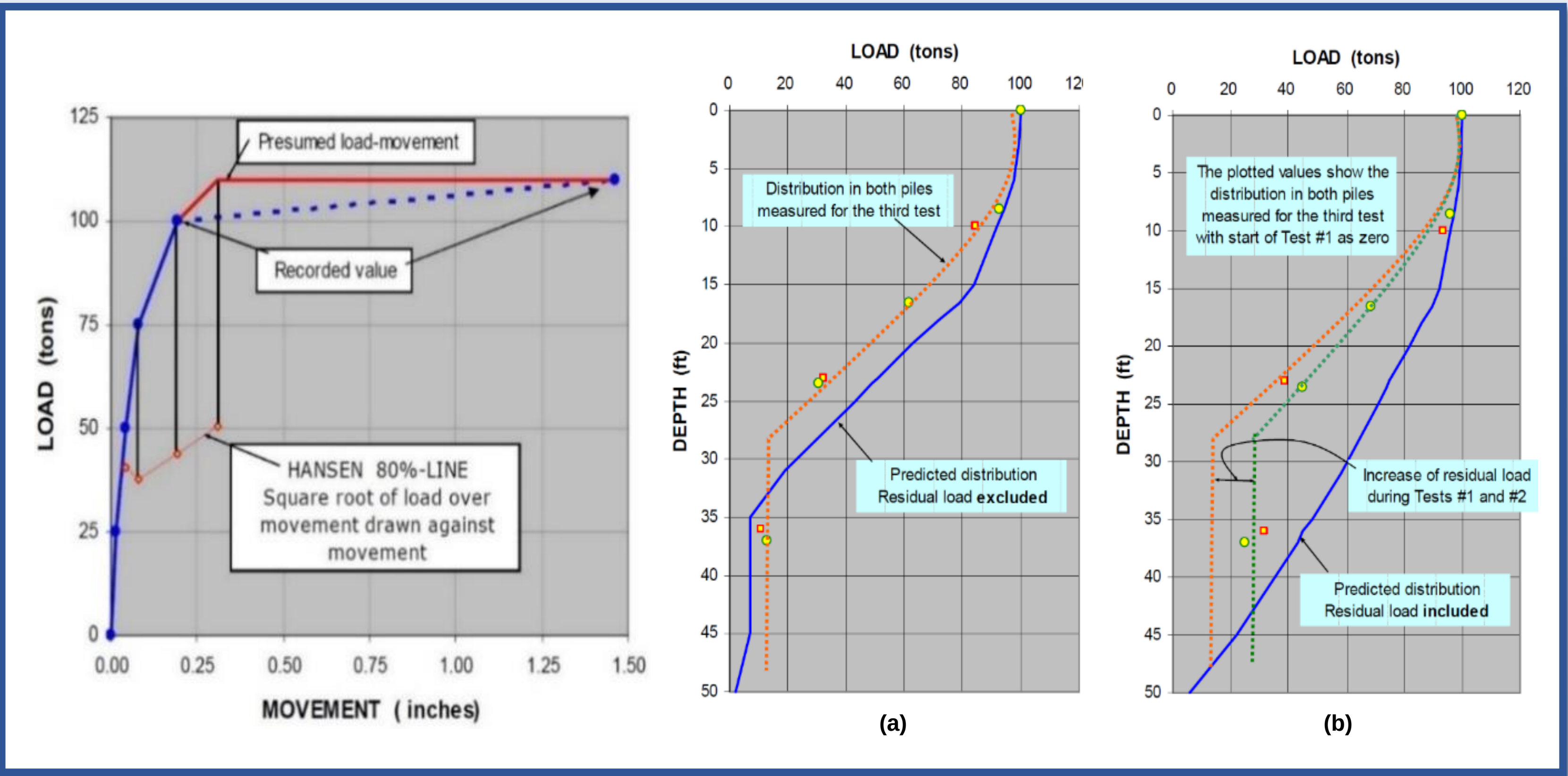
Case No. 1: Four Piles Prediction Symposium (Finno et al., 1989)



Axial load-deflection response of driven piles (Finno et al., 1989)

Typical load distribution in two-week tests (Finno et al., 1989)

Case No. 1: Four Piles Prediction Symposium (Finno et al., 1989)



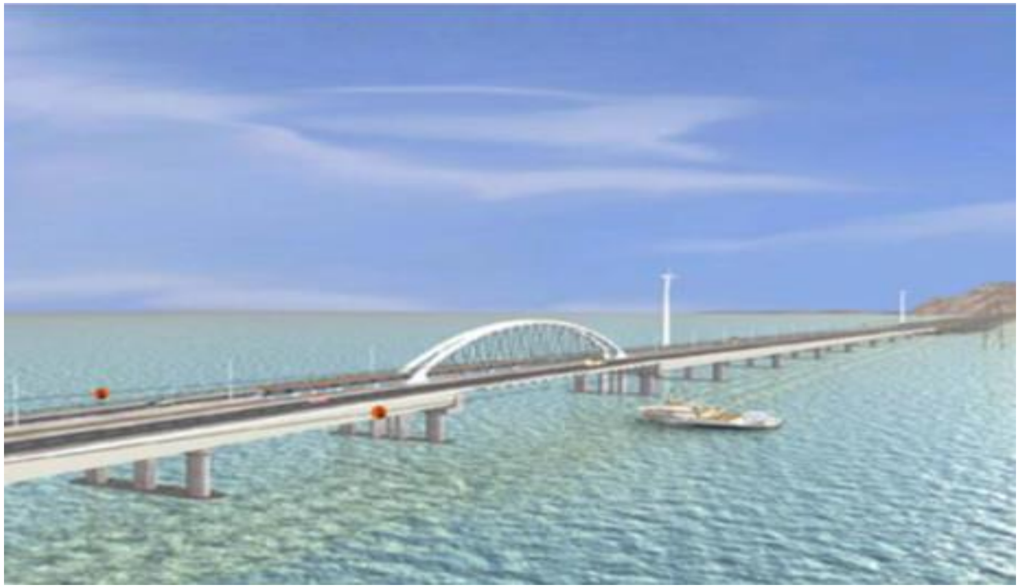
Presumed Pile-Head Load-Movement Curve with Correction Construed through the Brinch Hansen 80%-line. (Fellenius, 1991a)

Loads during the third tests as calculated from observed strain data a) residual loads excluded, b) including residual loads (Fellenius, 1991b)

Case No. 2: Urmia Lake Causeway (Eslami et al., 2011)

- Decreasing the porposed route from 300 km to 120 km,
- Total length of 1260 m
- 19 spans
- 100 m in lengh for the main span

(a)

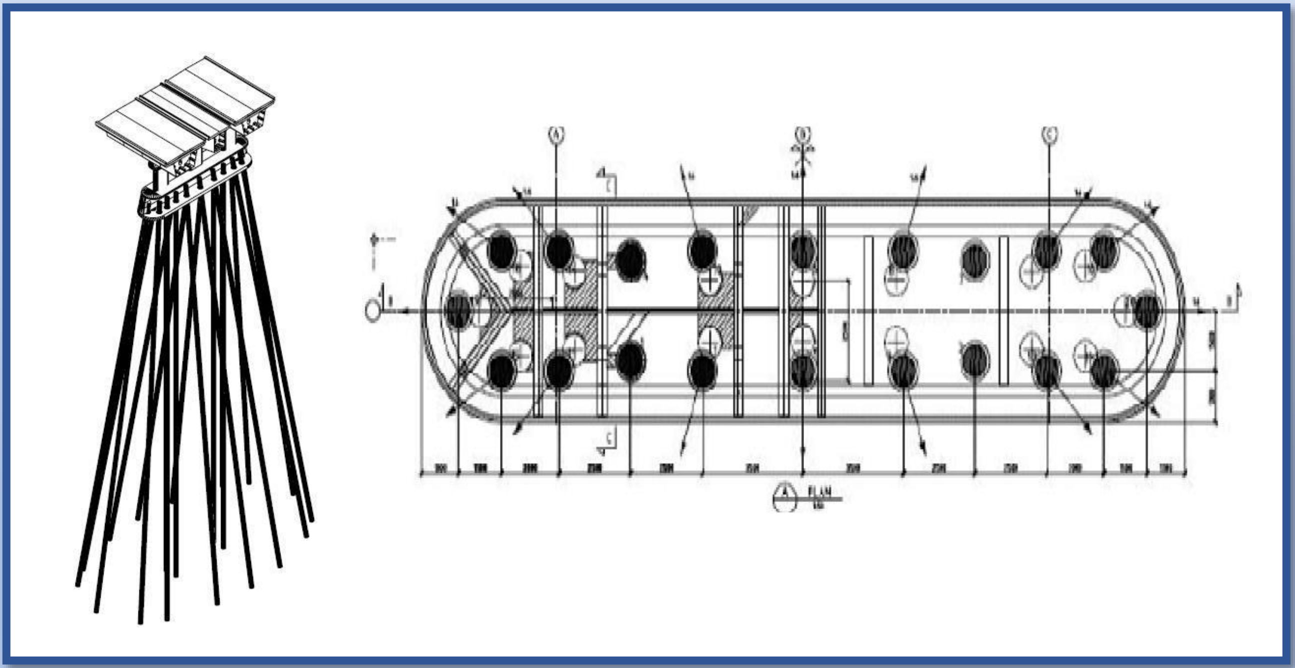


(b)

a) Location, b) longitudinal view of the causeway

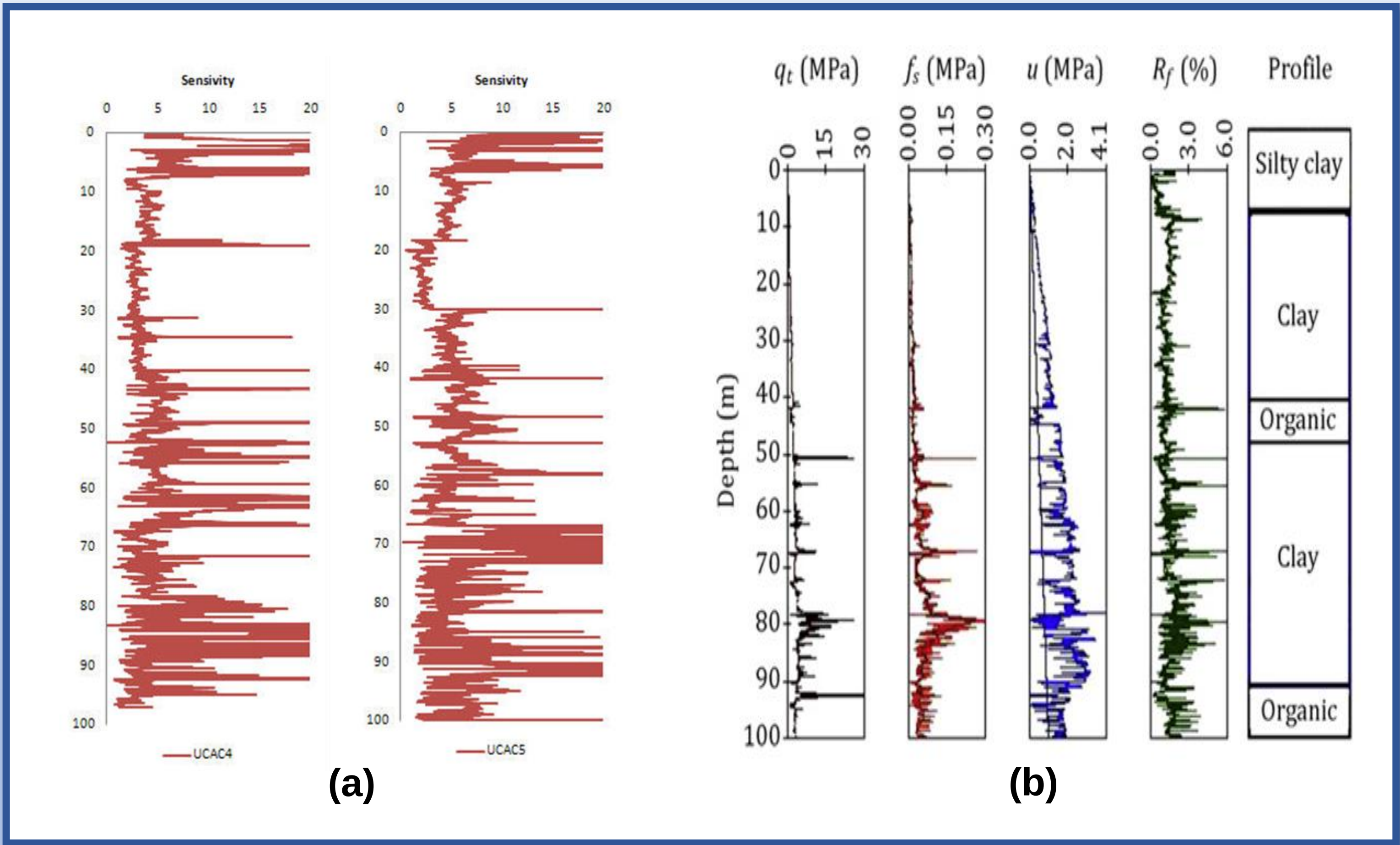
Case No. 2: Urmia Lake Causeway (Eslami et al., 2011)

- More than 400 pipe piles
- Total installed length of 32 km
- Piles 813 mm in diameter and 66 to 75 m in depth
- Total 800 m piles applied for static & dynamic tests



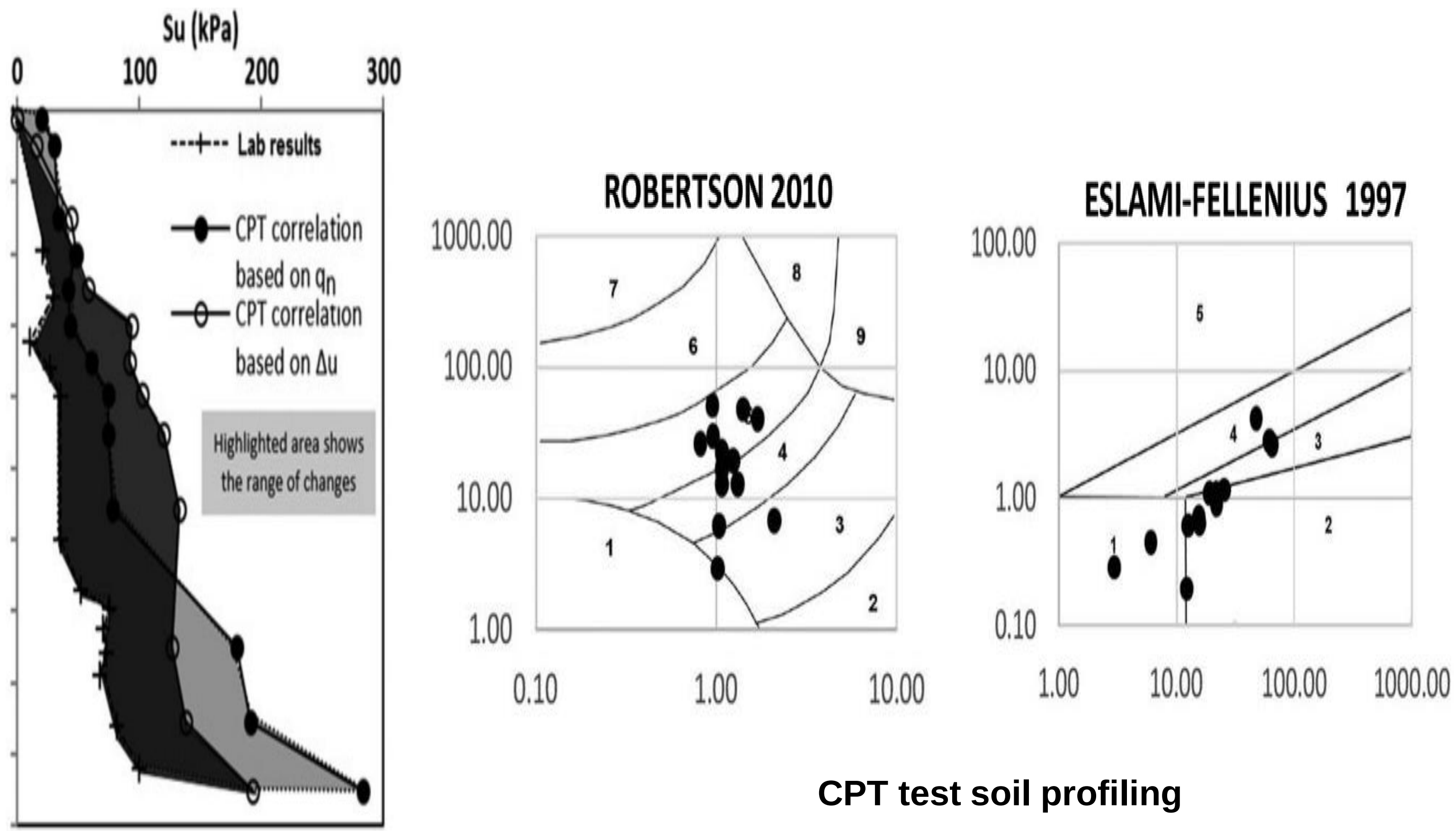
Configuration of installed piles

Case No. 2: Urmia Lake Causeway (Eslami et al., 2011)



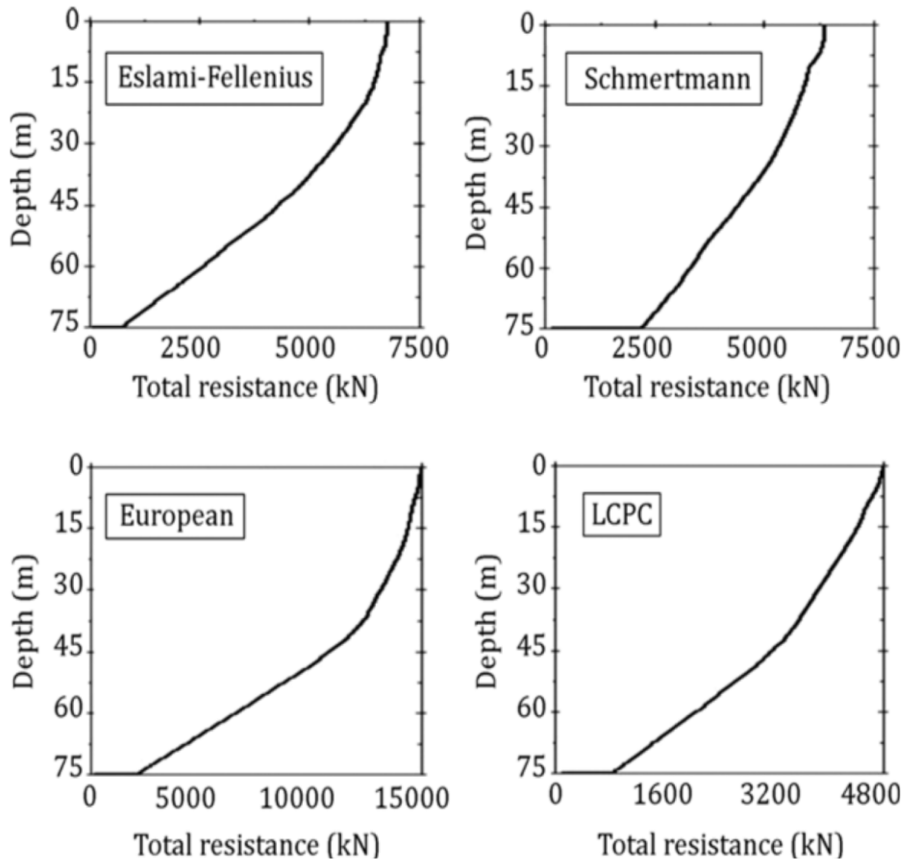
a) Sensivity log, b) typical CPT logs

Case No. 2: Urmia Lake Causeway (Eslami et al., 2011)



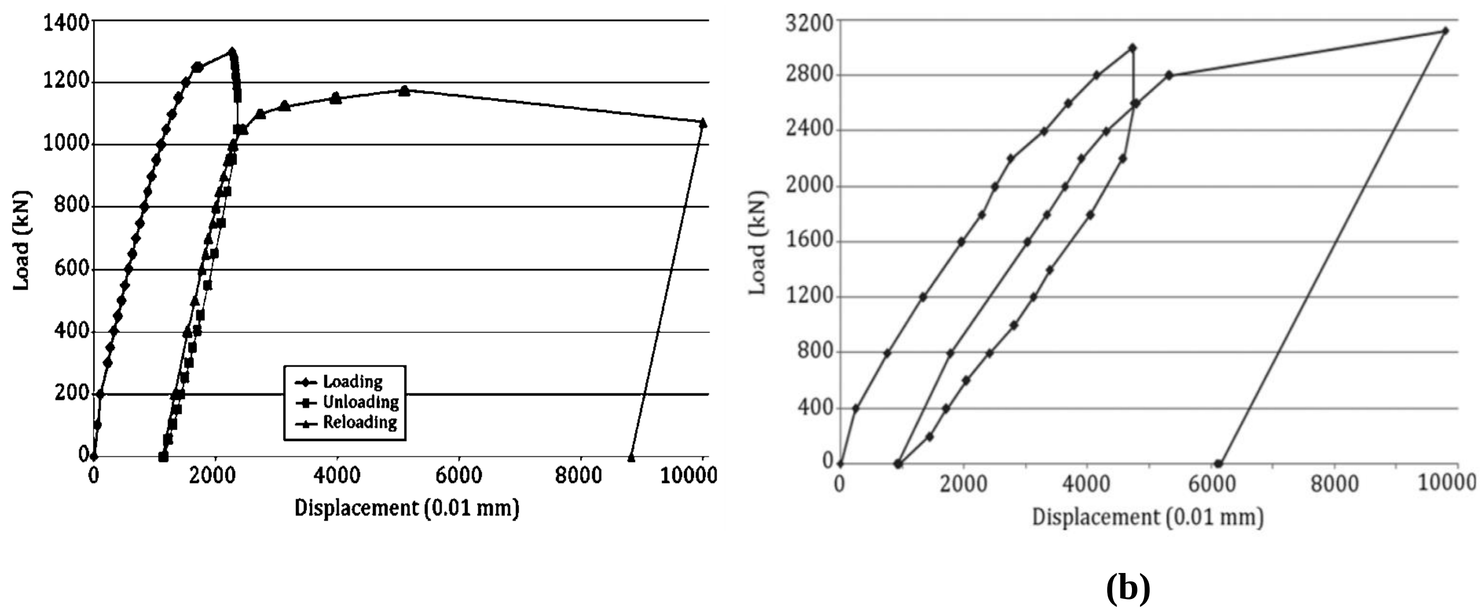
CPT test soil profiling

Case No. 2: Urmia Lake Causeway (Eslami et al., 2011)



Pile total capacity distribution (length of 75 m, diameter of 813 mm & wall thickness of 38.1 mm) (Eslami et al., 2011)

Case No. 2: Urmia Lake Causeway (Eslami et al., 2011)

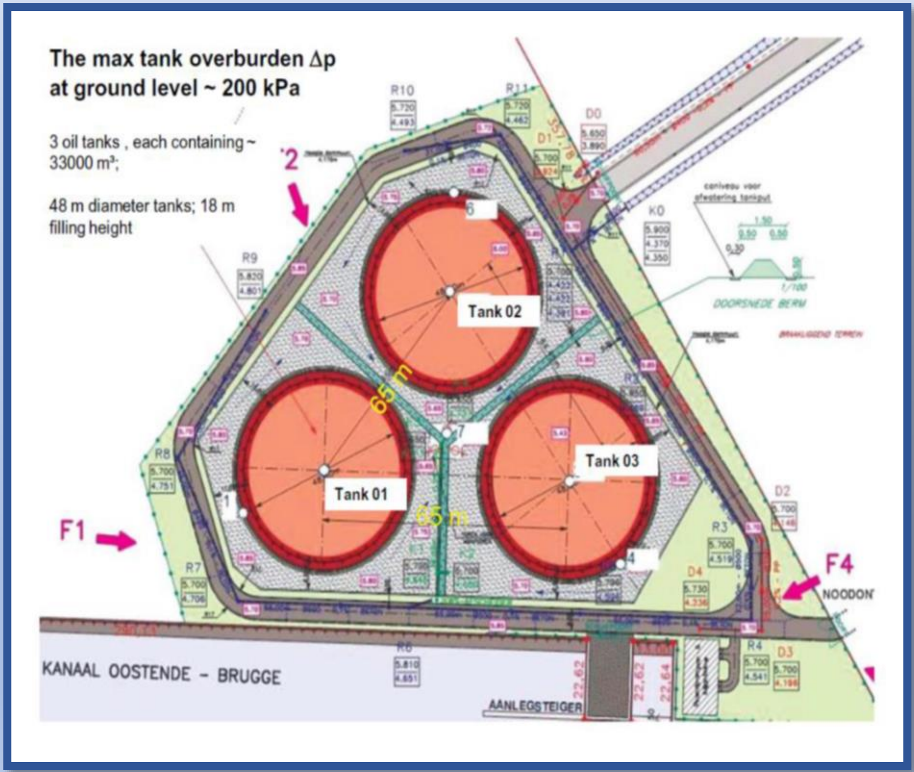


Static load test result; a) compressive (length of 30 m, diameter of 356 mm & wall thickness of 12 mm), b) tension (length of 70 m, diameter of 305 mm & wall thickness of 16 mm) (Eslami et al., 2011)

Case No. 3: Three oil tanks - Belgium (Van Impe et al., 2013 & 2015)

Tanks Characteristics

- **48 m in diameter and 19 m in height,**
- **Each holding 33000 m^2**
- **Triangle configuration - 65 m center to center**

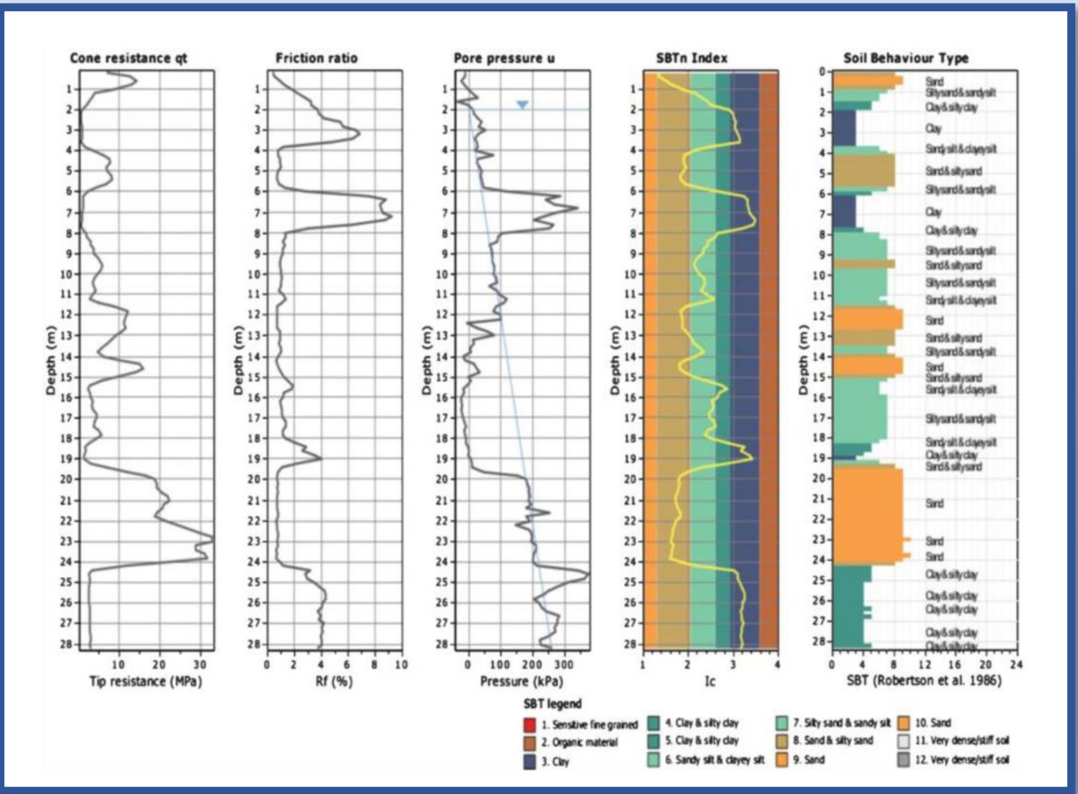


General view of oil tanks at Ostend Belgium (Van Impe et al., 2013)

Case No. 3: Three oil tanks - Belgium (Van Impe et al., 2013 & 2015)

Soil Profile

- **0–12 to 15 m depth: Old fill (GWT at 2 m)**
- **12-15 to 18 m depth: Sand**
- **18 to 100-120 m depth: OC Clay**

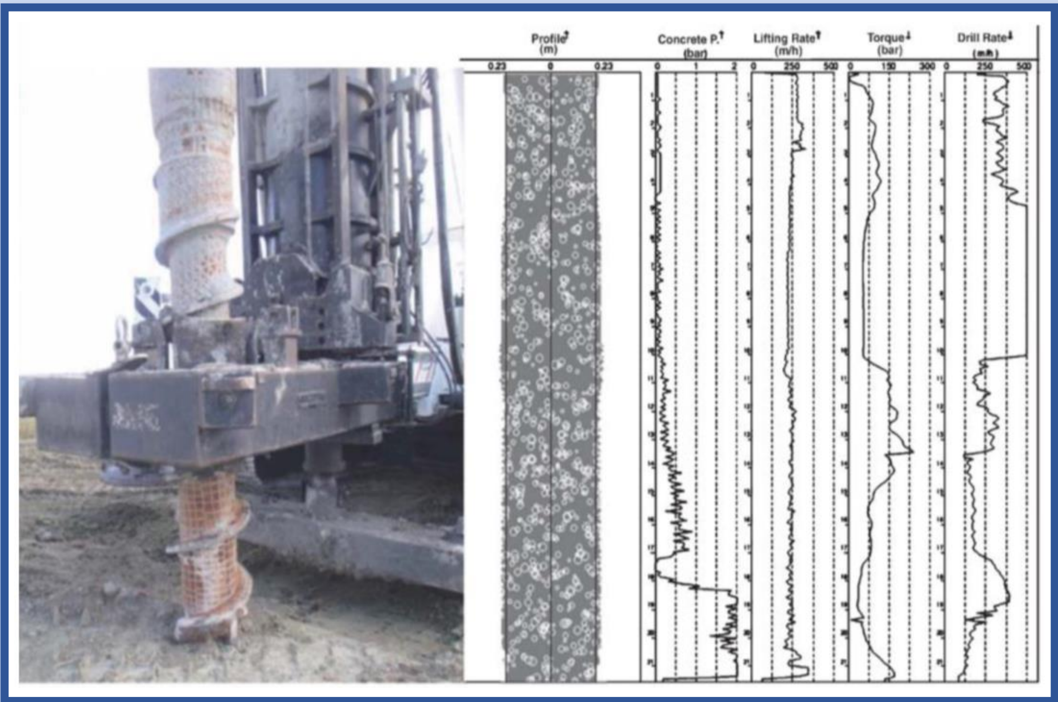


Relevant CPTu log in Ostend (Van Impe et al., 2013)

Case No. 3: Three oil tanks - Belgium (Van Impe et al., 2013 & 2015)

Foundation System

- **Pile Cap: 0.6 m thick and 49 m diameter**
- **422 Omega Piles as pile group**
- **Piles: 460 mm in diameter and 22 in depth**
- **Total ultimate capacity: 1950 kN**

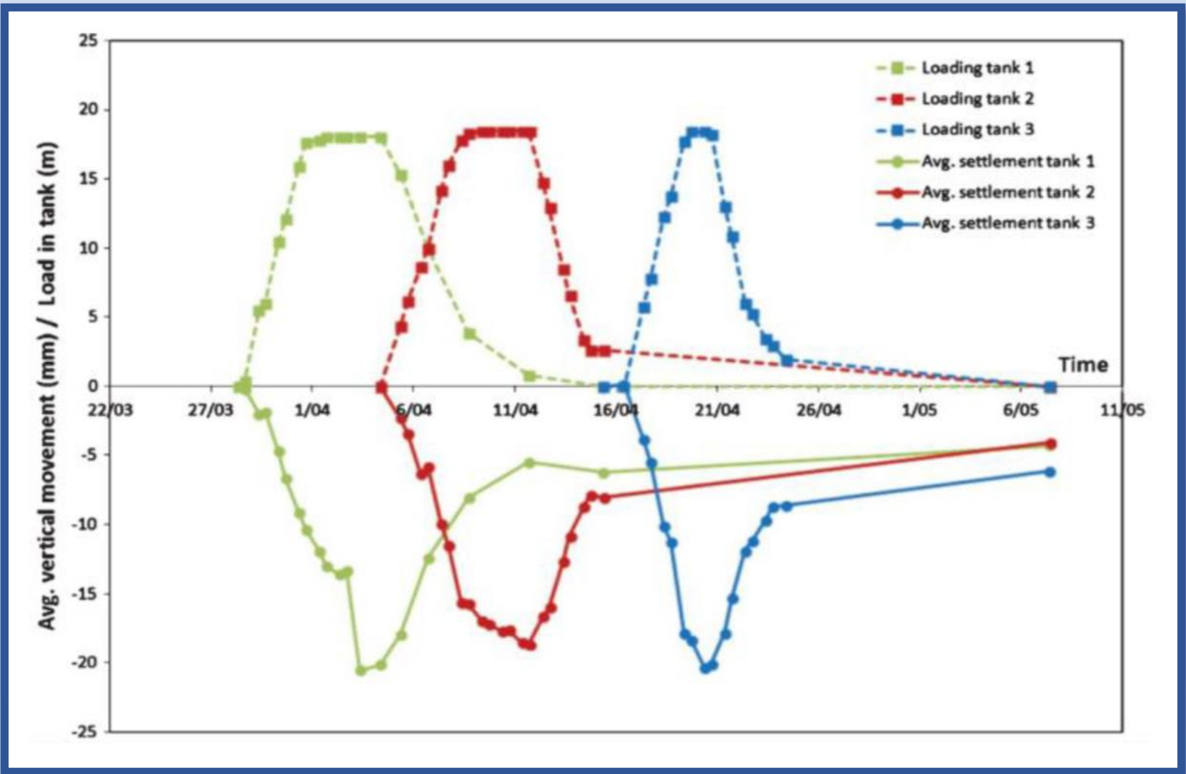


Test pile installation parameters (Van Impe et al., 2013)

Case No. 3: Three oil tanks - Belgium (Van Impe et al., 2013 & 2015)

Hydro – Load Testing

- Filled to a height of around 18 m
- Steady water level for 4 days
- Emptying by 3 days

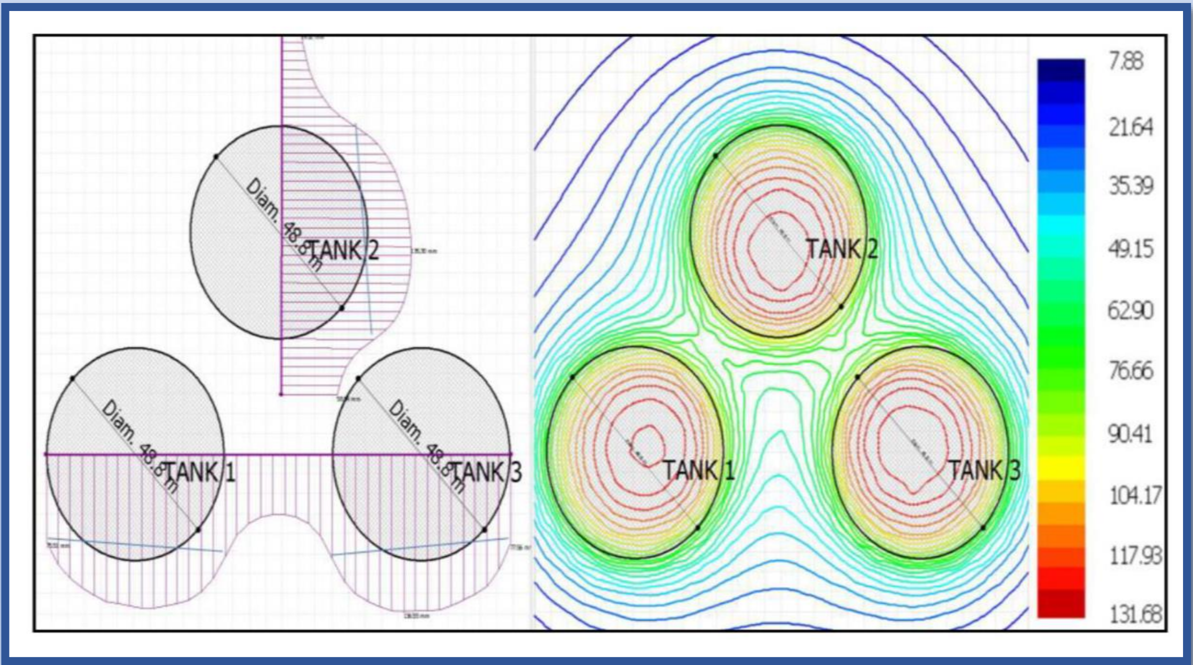


Load and movement history
(Van Impe et al., 2013)

Case No. 3: Three oil tanks - Belgium (Van Impe et al., 2013 & 2015)

3D Numerical Simulation

- **final average settlement: 87 to 90 mm**
- **Settlement at center: 132 to 136 mm**
- **long-term tilt: 19 to 21 mm**



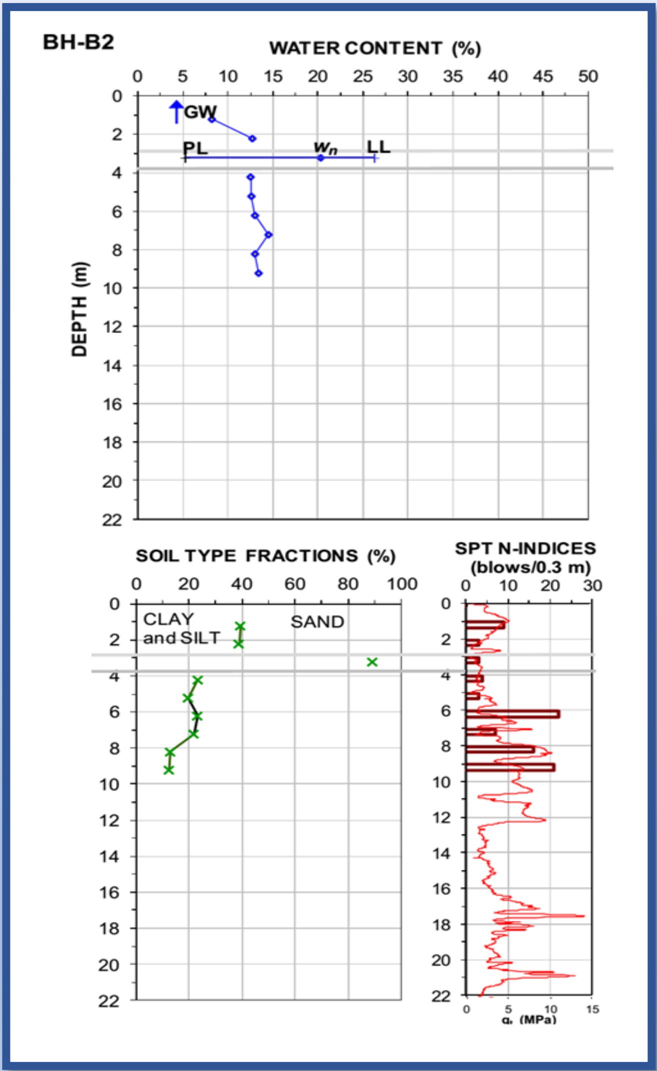
Settlement analysis under operational load (Van Impe et al., 2015)

Case No. 4: Bolivian Experimental Site for Testing (B.E.S.T.) (Fellenius et al., 2017)

- 24 kms north-east of Santa Cruz de la Sierra, Bolivia
- Aiming to provide well-documented site for different pile tests
- Conjuncted with the 3rd International Conference on Deep Foundations (C.F.P.B)
- 71 predictors were involved

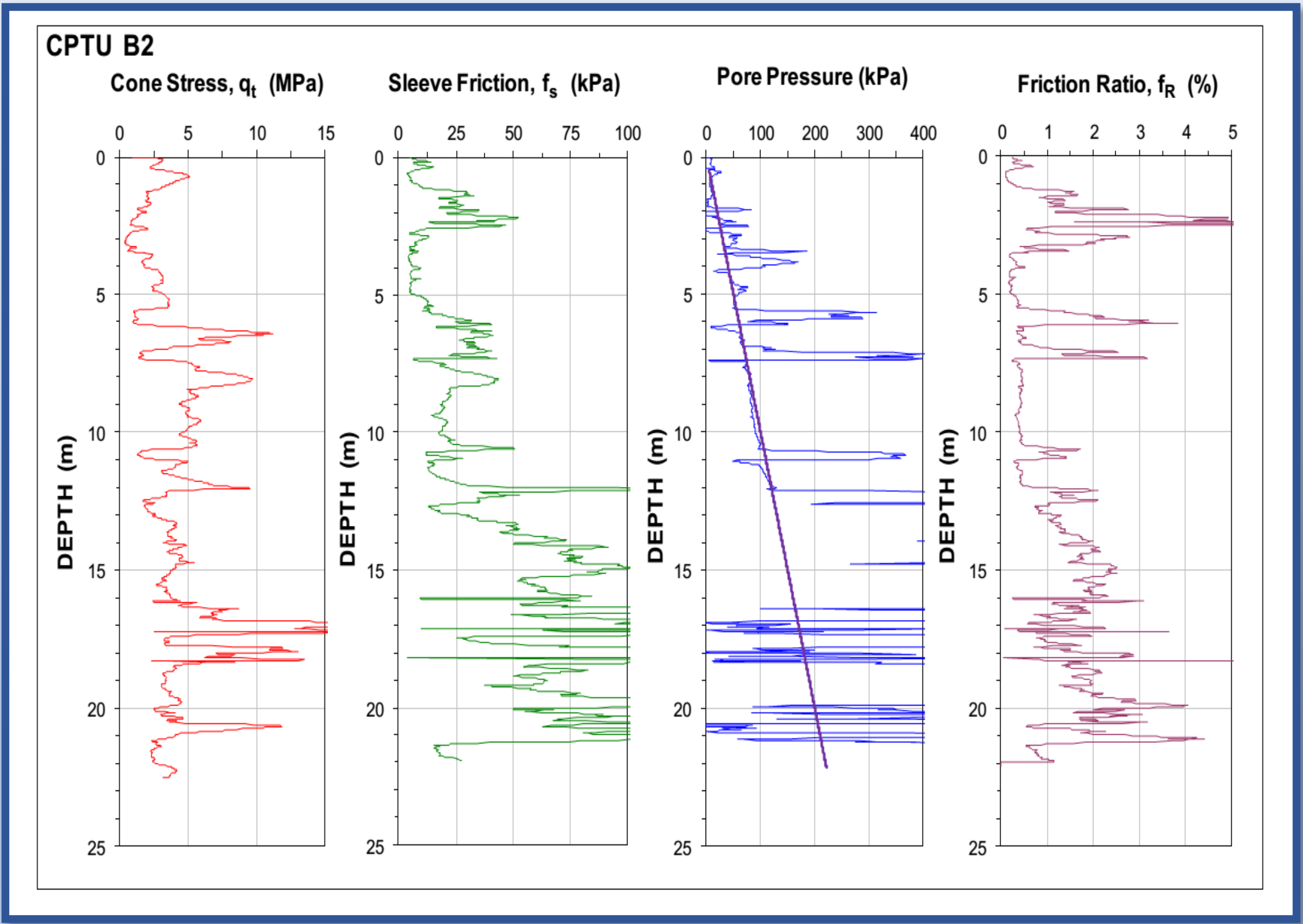
Case No. 4: Bolivian Experimental Site for Testing (B.E.S.T.) (Fellenius et al., 2017)

- Upper about 5 to 6 m: loose silt and sand
- Hereunder a 6 to 7 m layer of compact silt and sand
- At about 11 m: an about 1 m thick layer of soft silty clay
- Followed by an about 1 m thick layer of compact sand
- Below about 12 m, alternating between about 2 m thick layers of compact to dense silty sand or loose sand
- The groundwater table ranging between the ground surface and about 0.5 m



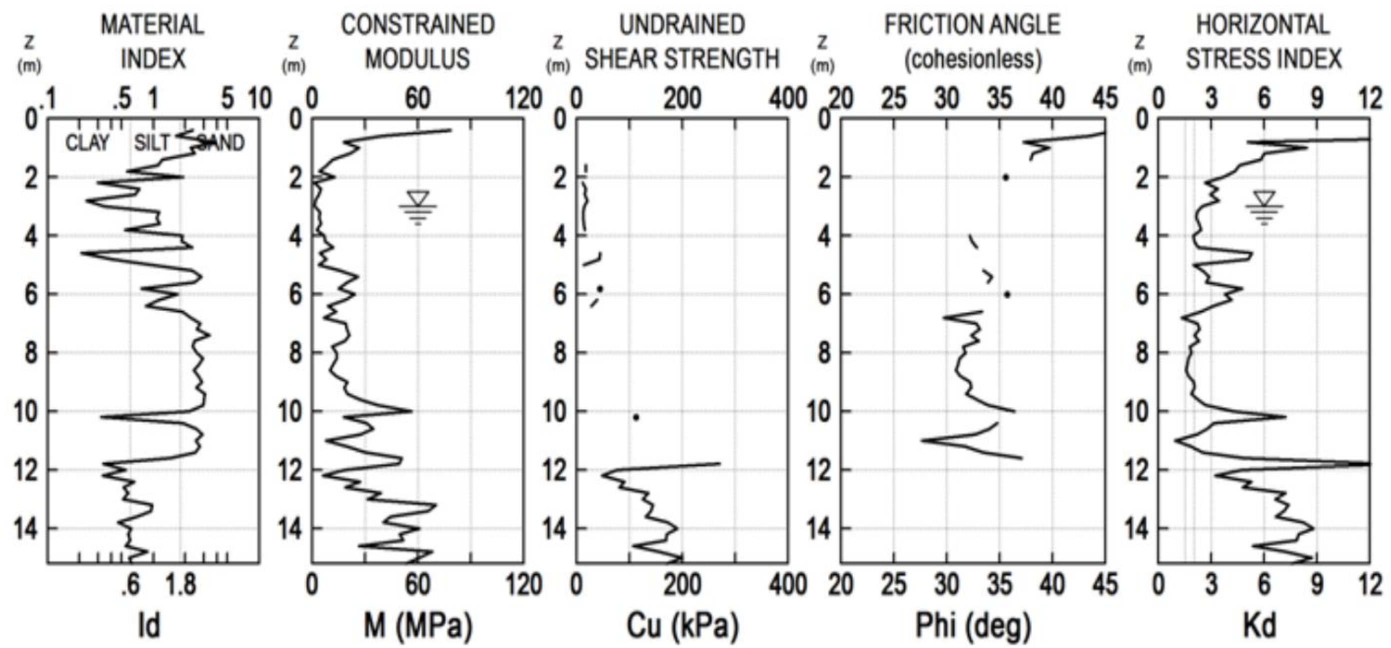
Borehole data and SPT N values for BH-B2 (Fellenius et al., 2017)

Case No. 4: Bolivian Experimental Site for Testing (B.E.S.T.) (Fellenius et al., 2017)



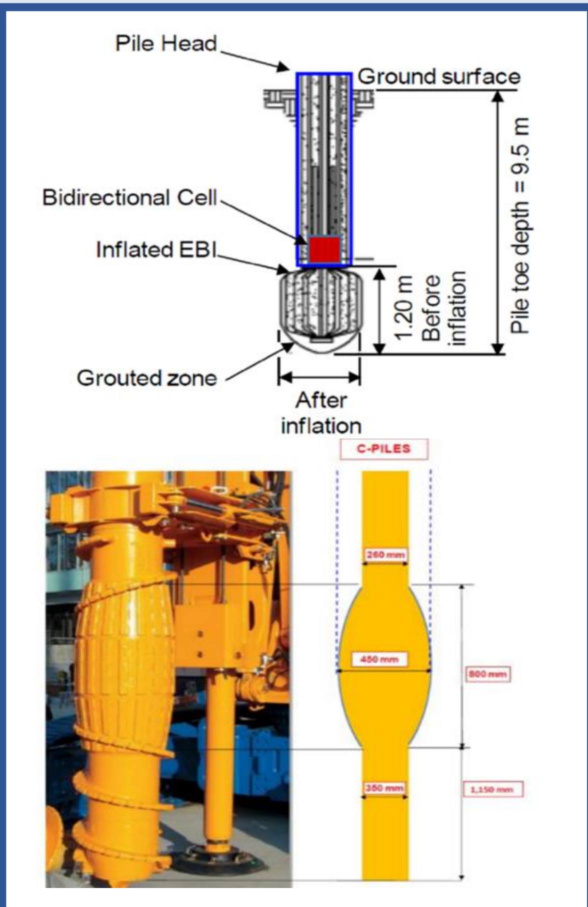
CPTu records for B2 (Fellenius et al., 2017)

Case No. 4: Bolivian Experimental Site for Testing (B.E.S.T.) (Fellenius et al., 2017)



DMT results for B2 (Fellenius et al., 2017)

Case No. 4: Bolivian Experimental Site for Testing (B.E.S.T.) (Fellenius et al., 2017)



Equipment and geometry of the 450 mm-full displacement pile (C2) (Fellenius et al., 2017)

PILE ID	PILE TYPE	PILE DIAMETER (mm)	ATTACHED DEVICE	TEST and SEQUENCE	THERMAL INTEGRITY PROFILER (TIP)	CROSS-HOLE CHECK pvc or steel pipe	GAGE TYPES and LEVELS	
							VW	RESISTIVE
A-1	Drilled with slurry	620	EB (800)	BD•HD•DT	2 wires		L1, L2, L3	L1, L2, L3
A-2			TB • SB	BD•HD•DT			L1, L2, L3	L1, L2, L3
A-3*				HD•DT				L1, L2, L3
B-1	CFA	450	EB (600)	BD•HD•DT			L1, L2, L3	L1, L2, L3
B-2*				HD•DT				L1, L2, L3
C-1	FDP	450	EB (600)	BD•HD•DT			L1, L2, L3	L1, L2, L3
C-2*				HD•DT	2 wires			L1, L2, L3
D-1	Self boring Micropile	150	EB (500)	HD			L1, L2, L3	L1, L2, L3
D-2				HD				L1, L2, L3
E-1*	FDP	220	EB (300)	BD•HD			L1, L2, L3	L1, L2, L3
E-2 to E-14			EB (300)	BD•HD			L1, L3	L2
F-1	Drilled with slurry	450	EB (400)	BD•HD•DT	2 wires	1 pvc	L1, L2, L3	L1, L2, L3
F-2		600	EB (600)	BD•HD•DT	4 wires	3 pvc	L1, L2, L3	L1, L2, L3
F-3		1,200			6 wires	5 pvc		L1, L2, L3
G-1	Helical	300		HD			L1, L2, L3	L1, L2, L3
DC1200-1	Bored Pile with retrievable casing	1,200			6 wires	5 steel	L1, L2, L3	L1, L2, L3
DC620-1		620			4 wires		L1, L2, L3	L1, L2, L3
DC620-2		620			4 wires	3 steel	L1, L2, L3	L1, L2, L3
DC620-3		620			4 wires	3 pvc	L1, L2, L3	L1, L2, L3
CFA450-1	CFA	450			2 wires		L1, L2, L3	L1, L2, L3
CFA450-2		450			2 wires	1 pvc	L1, L2, L3	L1, L2, L3
FDP450-1	FDP	450			2 wires		L1, L2, L3	L1, L2, L3
FDP360-1	FDP	360			2 wires		L1, L2, L3	L1, L2, L3

NOTES:

Length: All piles intended for static testing will be installed with pile toe at 9.5 m below grade
For pile with EBI unit, the depth is measured to bottom of the EBI before expansion
* PREDICTION PILE

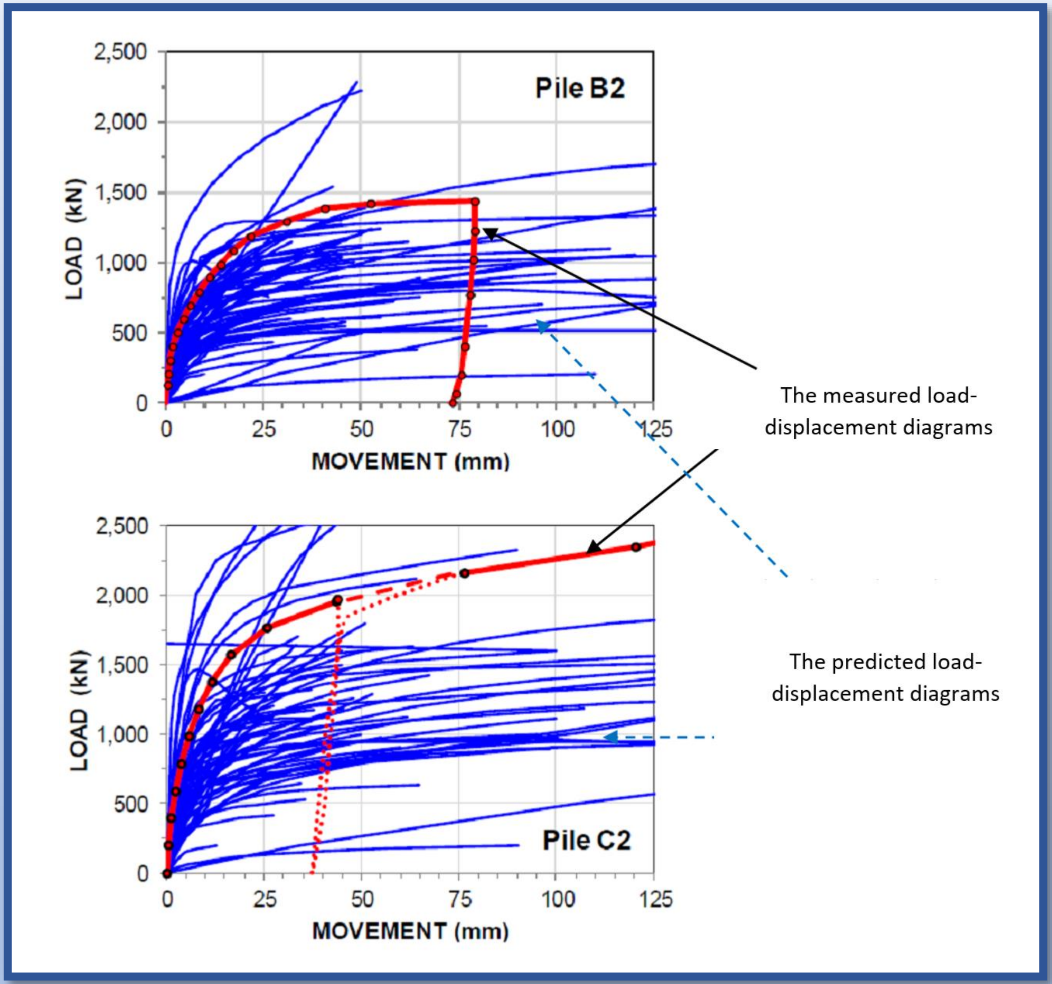
BD: BIDIRECTIONAL STATIC LOADING TEST
HD: HEAD-DOWN STATIC LOADING TEST
SG: INSTRUMENTED WITH STRAIN GAGES
DT: DYNAMIC TEST
PILES FOR LOADING AND INTEGRITY TESTS
PILES FOR INTEGRITY TESTS ONLY

EB: EXPANDER BASE
TB: TOE BOX
SB: SHAFT BOX

L: GAGE LEVEL
L1 at 2.0 m depth
L2 at 5.0 m depth
L3 at 7.5 m depth

Summary of tested piles (Fellenius et al., 2017)

Case No. 4: Bolivian Experimental Site for Testing (B.E.S.T.) (Fellenius et al., 2017)



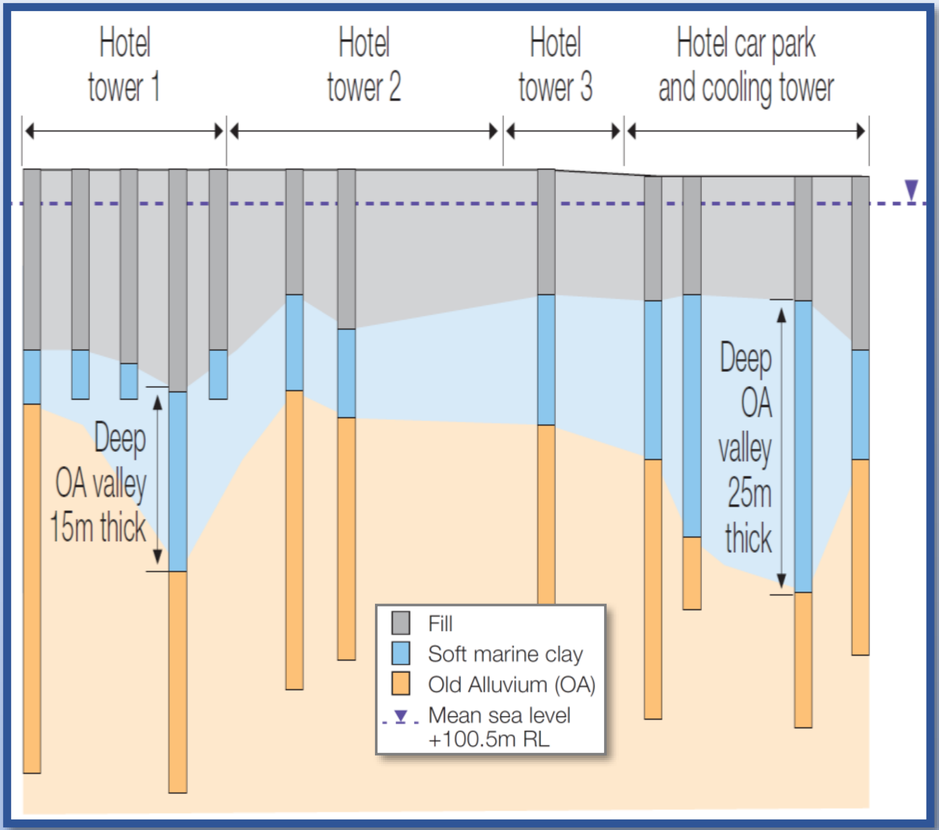
Predicted load-displacement diagrams vs. measured one (Fellenius et al., 2017)

Case No. 5: Marina Bay Sands, Singapore (Arup Group, 2018)

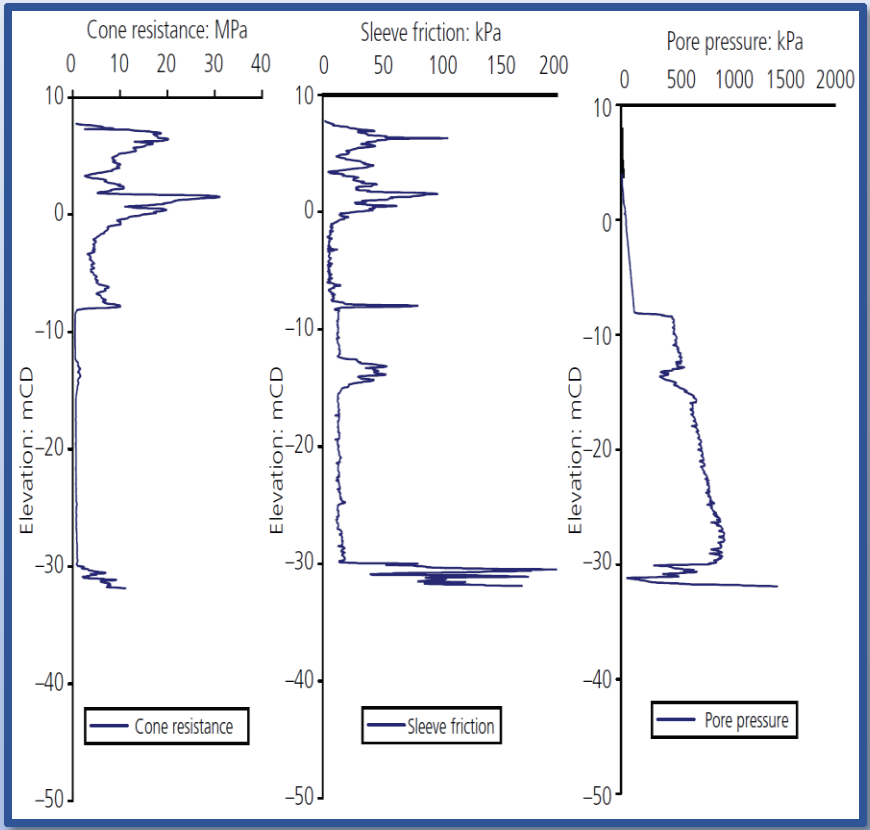
- **Year of Completion: 2010**
- **Height: 207 m**
- **Number of Storeys: 57**
- **Gross floor area: 581,400 m^2**
- **Primary use: Hotel, Conference, Retail, Leisure**

General View of the complex

Case No. 5: Marina Bay Sands, Singapore (Arup Group, 2018)



Typical Soil Profile of the site

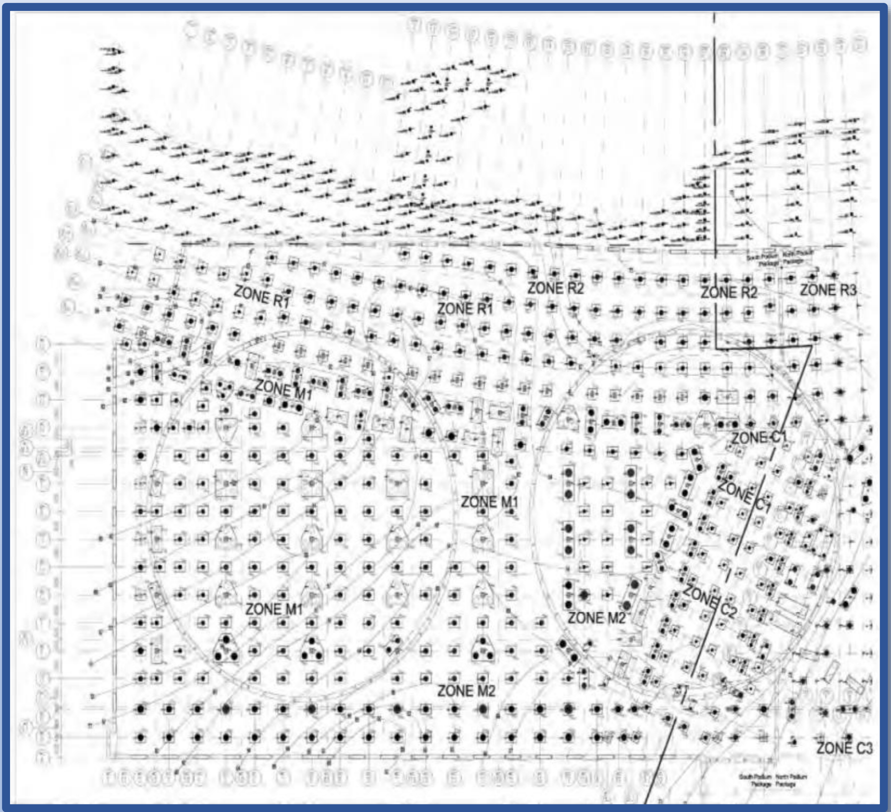


Typical CPTu result of marine clay in Singapore (Bo et al., 2019)

Case No. 5: Marina Bay Sands, Singapore (Arup Group, 2018)

Test Piles:

Diameter: 1.8 - 3 m, Length: 70 - 80 m, Treshold of Loading : 2200 – 5500 ton



A forest of drilled shafts
(Foundation Drilling, 2012)

O-Cell implementation in Marine Bay Sands project
(Foundation Drilling, 2012)

Case No. 6: Torre Latino Americana, Mexico City (Coduto et al., 2016)

- **43-story Building**
- **Milestone in floating foundations technology**

The soil profile:

- **0–5.5 m depth: Old fill (GWT at 2 m)**
- **5.5–9.1 m depth: Becarra sediments**
- **9.1–33.5 m depth: Tacubaya clays;**

moisture content = 100 – 400%, C_c = 8; S_u = 35–70 kPa.

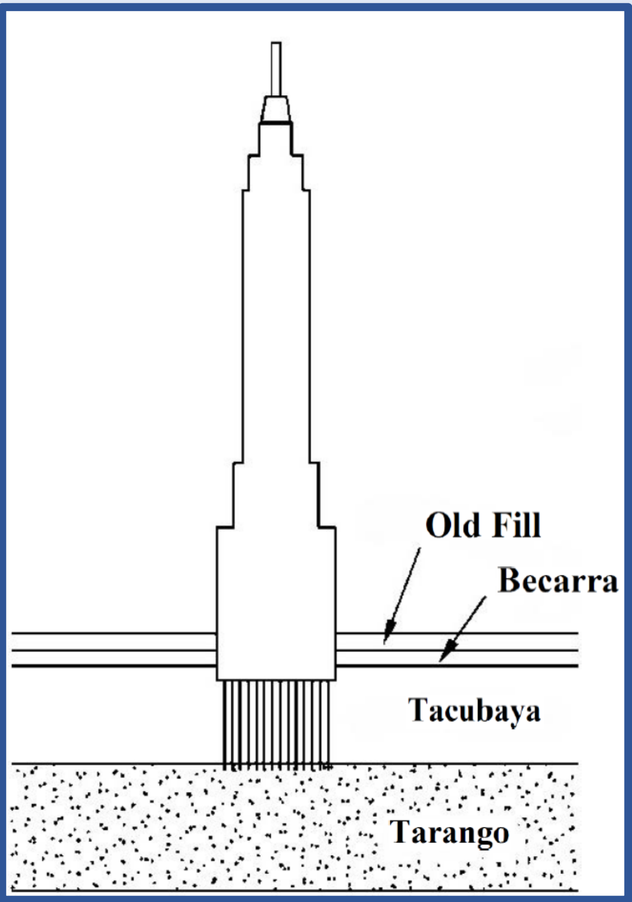
- **33.5–70.0 m depth: Tarango sands**

The Palace of Fine Arts, located across the street from the Tower, settled over 3 m (10 ft) from 1904 to 1962 (Zeevaert, 1957).

Case No. 6: Torre Latino Americana, Mexico City (Coduto et al., 2016)

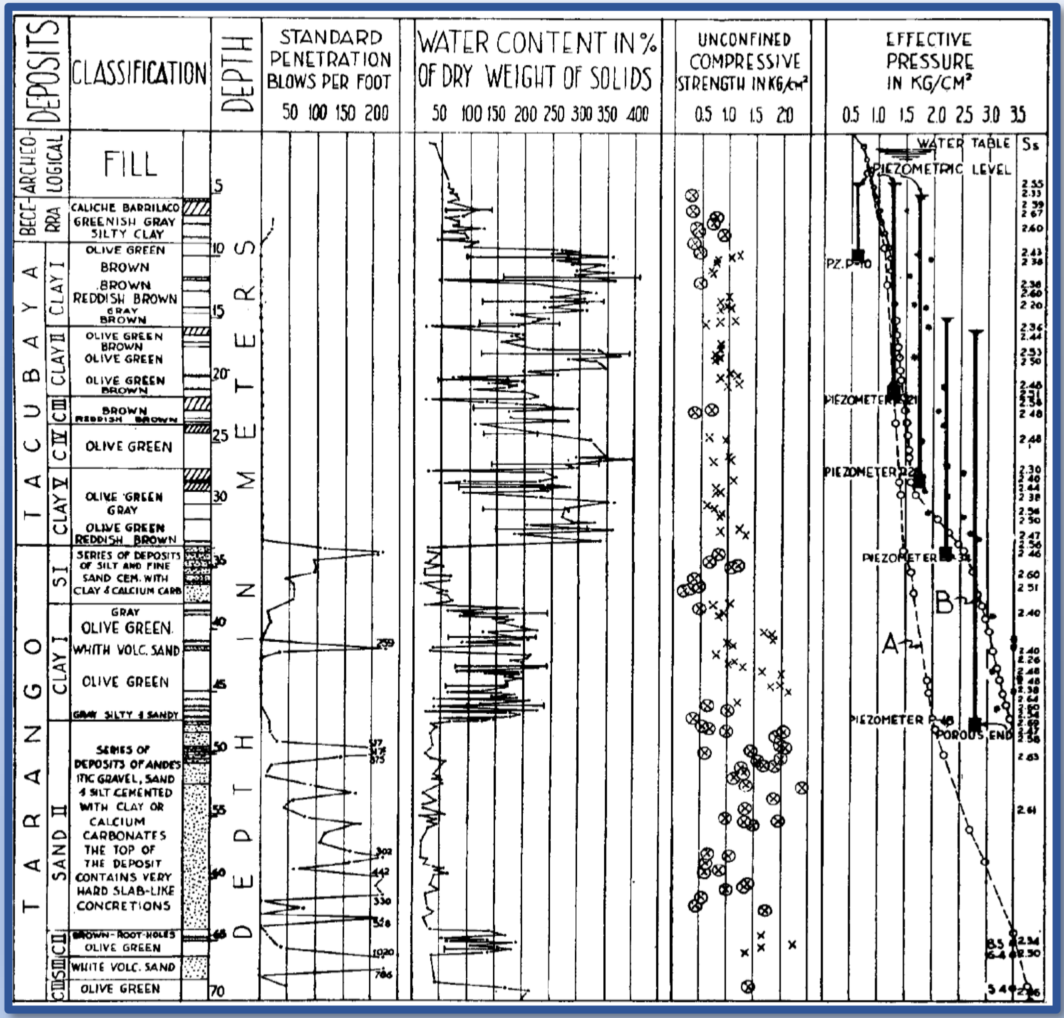
Typical compression Index C_c values (Holtz et al., 2023)

Soil	C_c
Normally consolidated medium sensitive clays	0.2 to 0.5
Chicago silty clay (CL)	0.15 to 0.3
Boston blue clay (CL)	0.3 to 0.5
Vicksburg buckshot clay (CH)	0.5 to 0.6
Swedish medium sensitive clays (CL-CH)	1 to 3
Canadian Leda clays (CL-CH)	1 to 4
Mexico City clay (MH)	**7 to 10**
Organic clays (OH)	10 to 15
Peats (Pt)	Long, short
Organic silt and clayey silts (ML-MH)	1.5 to 4
San Francisco Bay mud (CL)	0.4 to 1.2
San Francisco Old Bay clays (CH)	0.7 to 0.9
Bangkok clay	0.4



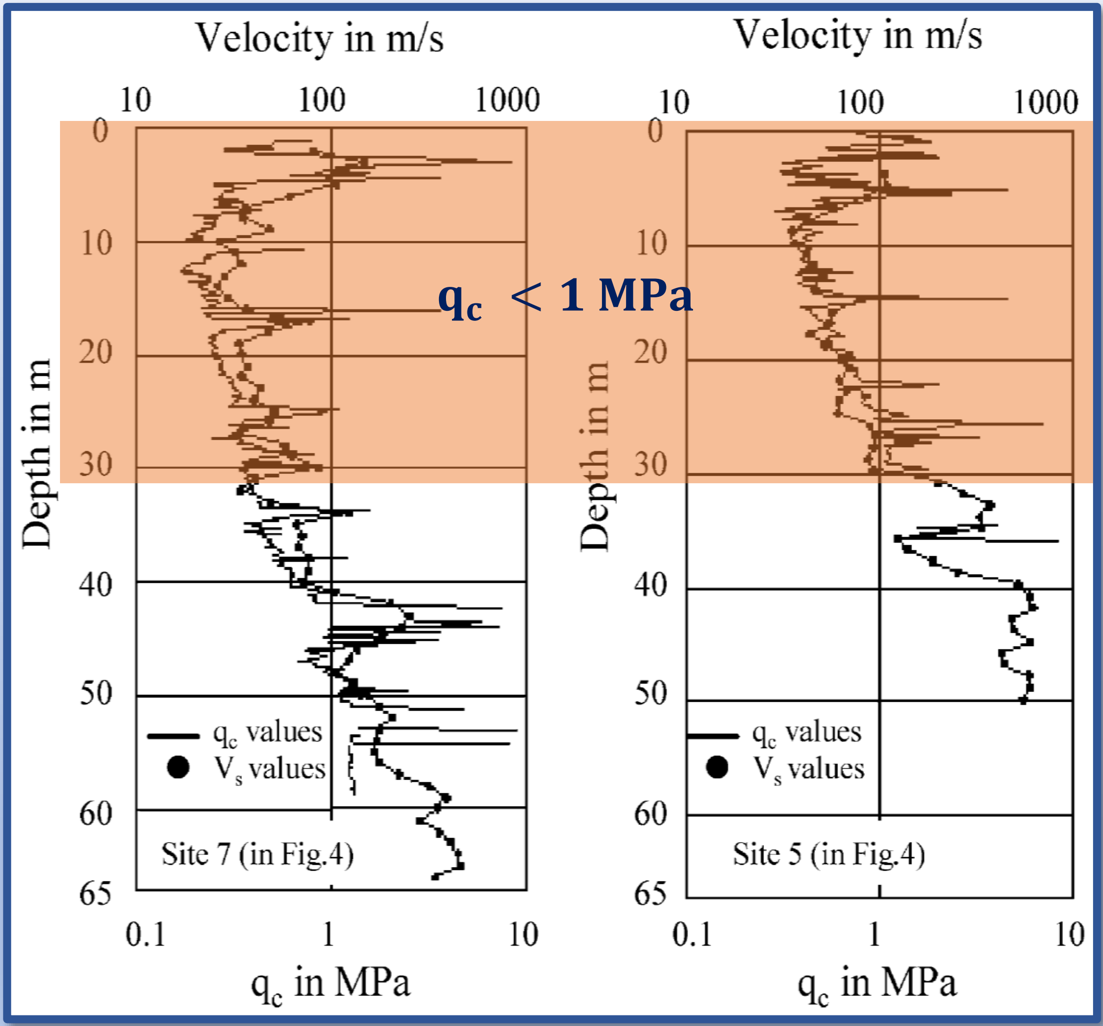
The foundation and the sublayer profile (Coduto et al., 2016)

Case No. 6: Torre Latino Americana, Mexico City (Coduto et al., 2016)



Classic log & SPT result (Zeevaert, 1957)

Case No. 6: Torre Latino Americana, Mexico City (Coduto et al., 2016)

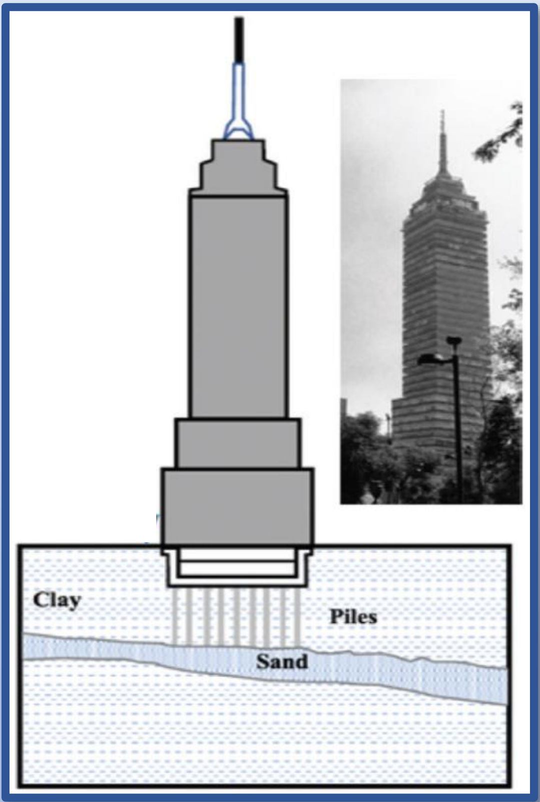


Stratigraphic characteristics of Mexico City soil deposits (Romo & Garcia, 2003)

Case No. 6: Torre Latino Americana, Mexico City (Coduto et al., 2016)

Major design aspects:

- Role of in-situ testing in recognition of challenging sublayers
- Significancy of end-bearing deep foundations
- Optimized application of floating foundations
- Controlling the settlement within the allowed range



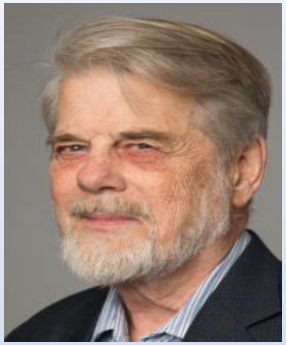
Major References

- **Bustamante, M., & Gianeselli, L. (1982, May). Pile bearing capacity prediction by means of static penetrometer CPT. In Proceedings of the 2nd European symposium on penetration testing, Amsterdam (Vol. 2, pp. 493-500).**
- **Clausen, C.J.F., Aas, P.M., & Karlsrud, K. (2005). Bearing capacity of driven piles in sand, the NGI approach. Proceedings of international symposium on frontiers in offshore geomechanics (ISFOG 2005), Perth, Taylor & Francis, London, 677–681.**
- **Coduto, D.P., Kitch, W.A., & Yeung, M.R. (2016). Foundation design principles and practices (3rd ed.). Upper Saddle River, NJ: Prentice Hall, Inc.**
- **Eslami, A., & Fellenius, B.H. (1997). Pile capacity by direct CPT and CPTu methods applied to 102 case histories. Canadian Geotechnical Journal, 34(6), 886-904.**
- **Eslami, A., Aflaki, E., & Hoseini, B. (2011). Evaluating CPT and CPTu based pile bearing capacity estimation methods using Urmiyeh lake Causeway piling records, Scientia Iranica transaction a-civil engineering, 19 October. Vol.18, No.5, pp.1009 - 1019.**
- **Eslami, A. (2013). Foundation Engineering, Design, and Construction (2nd ed.). Tehran, Building and Housing Research center**
- **Eslami, A., Akbarimehr, D., Aflaki, E. & Hajitaheriha, M. M. (2019). Geotechnical site characterization of the Lake Urmia super-soft sediments using laboratory and CPTu records. Marine Georesources & Geotechnology**
- **Eslami, A., Moshfeghi, S., Molaabasi, H., & Eslami, M. (2020). Piezocone and Cone Penetration Test (CPTu and CPT) Applications in Foundation Engineering. Elsevier, 1st edition, 2019**
- **Fellenius, B.H., (1989). Prediction of pile capacity. Proceedings of ASCE, Geotechnical Engineering Division, the1989 Foundation Engineering Congress, Symposium on Predicted and Observed Behavior of Piles, R. J. Finno, Editor, ASCE Geotechnical Special Publication No. 23, pp. 293-302.**
- **Fellenius, B. H., Infante, J. L. & Eslami, A. (2002). “UniCone Software”, for Processing and Reporting of Cone Penetration Tests (CPT and CPTu), Soil Profiling, and Pile Capacity Analysis.**
- **Fellenius, B. H. (2017), Report on the B.E.S.T. prediction survey of the 3rd CBFP event. Proceedings of the 3rd Bolivian International Conference on Deep Foundations, Santa Cruz de la Sierra, Bolivia, Vol. 3, pp. 7-25.**
- **Holtz, R.D., Kovacs, W.D. & Sheahan, T.C. (2023) An Introduction to Geotechnical Engineering (3rd ed.). Pearson**

Major References

- **Jardine, R. J., Chow, F. C., Overy, R., & Standing, J. (2005). ICP design methods for driven piles in sands and clays. Thomas Telford Publishing, London, 105 p.**
- **Kempfert, H.G., & Becker, P. (2010). Axial pile resistance of different pile types based on empirical values. Proceedings of Geo-Shanghai, 149-154**
- **Lehane, B. M., Schneider, J. A., & Xu, X. (2005). The UWA-05 method for prediction of axial capacity of driven piles in sand. Frontiers in Offshore Geotechnics: ISFOG, 683-689.**
- **Meyerhof, G.G. (1983). Scale effects of pile capacity. Journal of Geotechnical Engineering, ASCE 108 (GT3), 195 - 228.**
- **Niazi, F. S., & Mayne, P. W. (2013). Cone penetration test based direct methods for evaluating static axial capacity of single piles. Geotechnical and Geological Engineering, 31(4), 979-1009.**
- **Nottingham, L.C. (1975). Use of Quasi-static Friction Cone Penetrometer Data: To Predict Load Capacity of Displacement Piles (Doctoral dissertation, University of Florida).**
- **Schmertmann, J.H. (1978). Guidelines for cone penetration test.(performance and design) (No. FHWA-TS-78-209 Final Rpt**
- **Van Impe, P.O., Van Impe, W.F., & Seminck, L. (2013). Discussion of an instrumented screw pile load test and connected pile group load settlement behavior. Journal of Geo-Engineering Sciences. 1, 13–36**
- **Zeevaert, L. (1957). Foundation Design and Behavior of Tower Latino Americana in Mexico City. Géotechnique, 7, 115–133.**

Mentors



The Legendary
Prof. B.H. Fellenius

The Late Beloved
Prof. R.G. Campanella

Colleagues

Dr. S. Heidarie Golafzani

Engr. A. Ebrahimipour

Dr. M. Nobahar

Engr. A. NikoueiNahali

Thanks For Your Attention