GEO-CONGRESS 2026

Salt Lake City, Utah, March 9-12, 2026

Geo Tools, Technologies, and Techniques in an Environment of Change

CPT and Pile Foundations:

Past Insights, Present Trends, & Future Prospects

Short Course

Abolfazl Eslami, PhD & PE

Professor & Private Geotechnical Consultant

Measurements

- CPT/CPTu Measurements (qc, fs, u_2)
- Stratigraphy and Soil Variability
- Installation Records and Load Test Data

Interpretation

- Soil Behavior Type and Weak Layers
- Strength and Stiffness Profiling
- Axial Capacity and Settlement Indicators

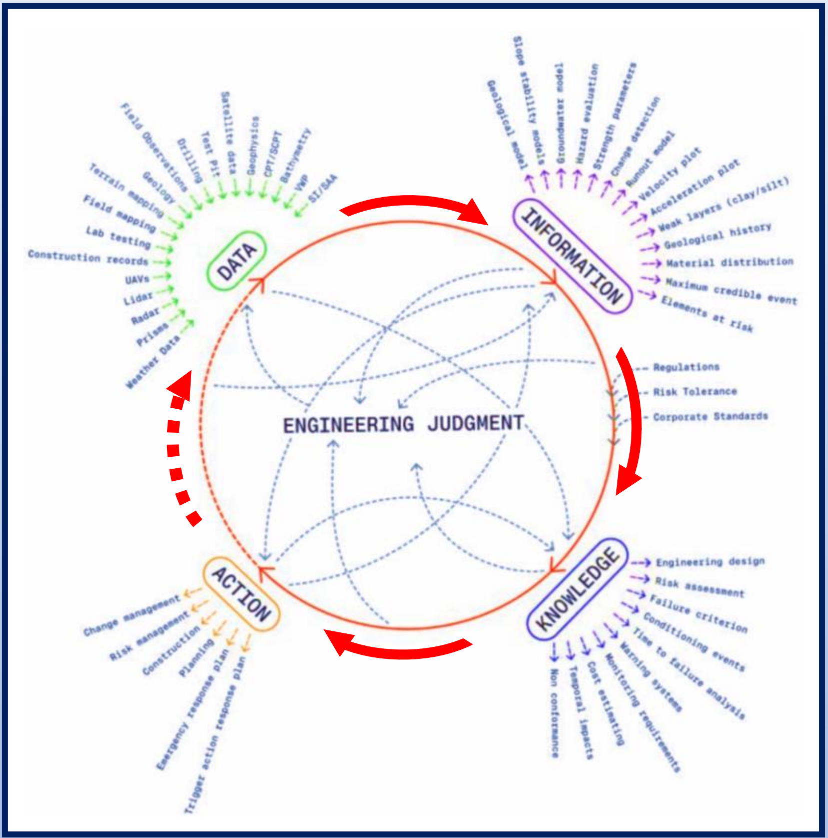
Methodology

- CPT-Based Axial Design Methodologies
- Load-Transfer Mechanisms
- Serviceability and Failure Criteria
- Uncertainty and Performance Assessment

Implementation

- Rational Pile Design; Axial Capacity
- Settlement Verification
- Risk-Informed Foundation Selection

Aims, Scope & Objectives

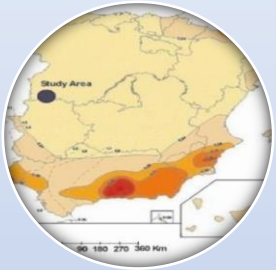


(Lato et al., Geostrata; February/March 2026)

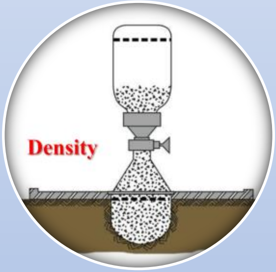
1. **CPT & CPTu in Geotechnical Engineering**
2. **Applications in Foundation Engineering**
3. **CPT-Based Methods for Deep Foundations**
4. **Worked Examples**
5. **Case Histories**
6. **Prospects**

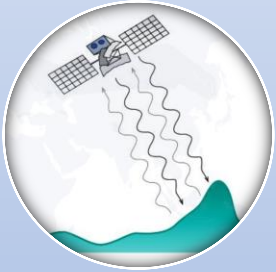
Data Sources

- **(1) Maps**
- **(2) Aerial Photos**
- **(3) Site Visit**
- **(4) Non Destructive Tests**
- **(5) Remote Sensing**
- **(6) On-Situ Testing**
- **(7) In-situ Penetration Testing**
- **(8) Boring and Sampling**
- **(9) Laboratory Testing**
- **(10) Physical Modeling**
- **(11) Full-Scale Tests**
- **(12) Instrumentation & Monitoring**





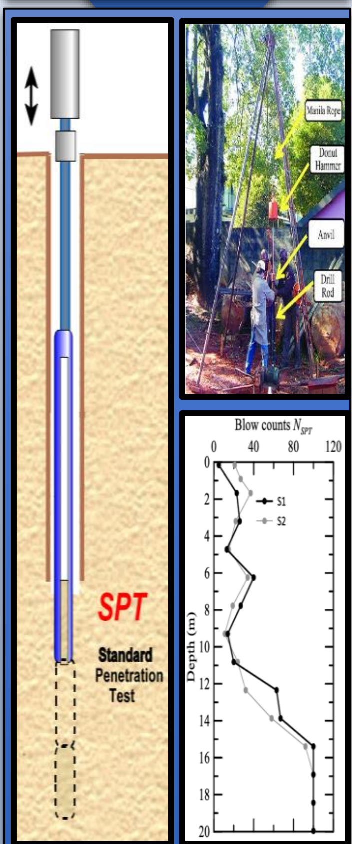






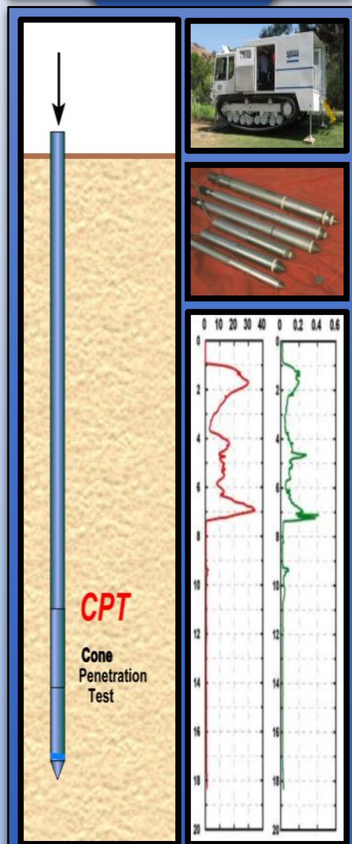
Major Approaches: In Situ Penetration Tests

SPT



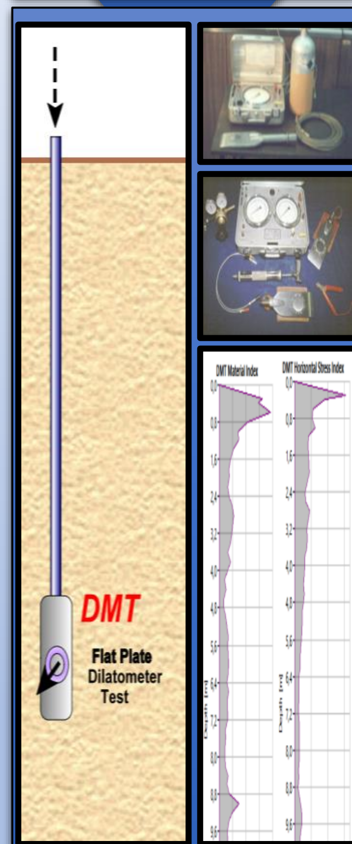
N

CPT



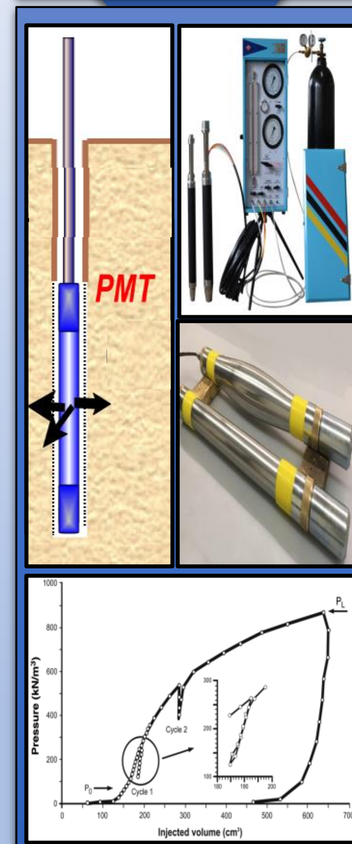
q_c, f_s & u_2

DMT



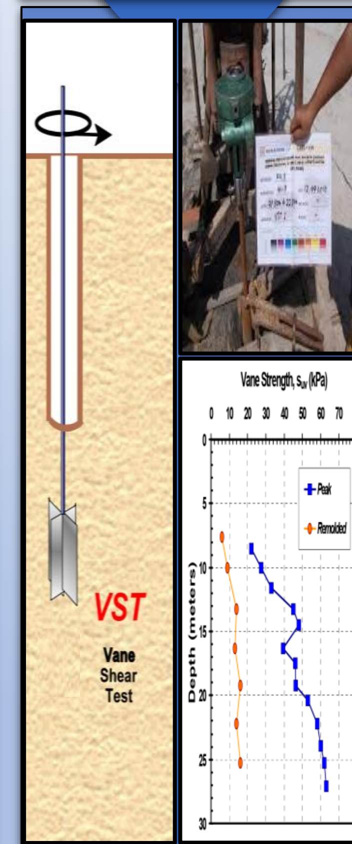
K_D, I_D & E_D

PMT



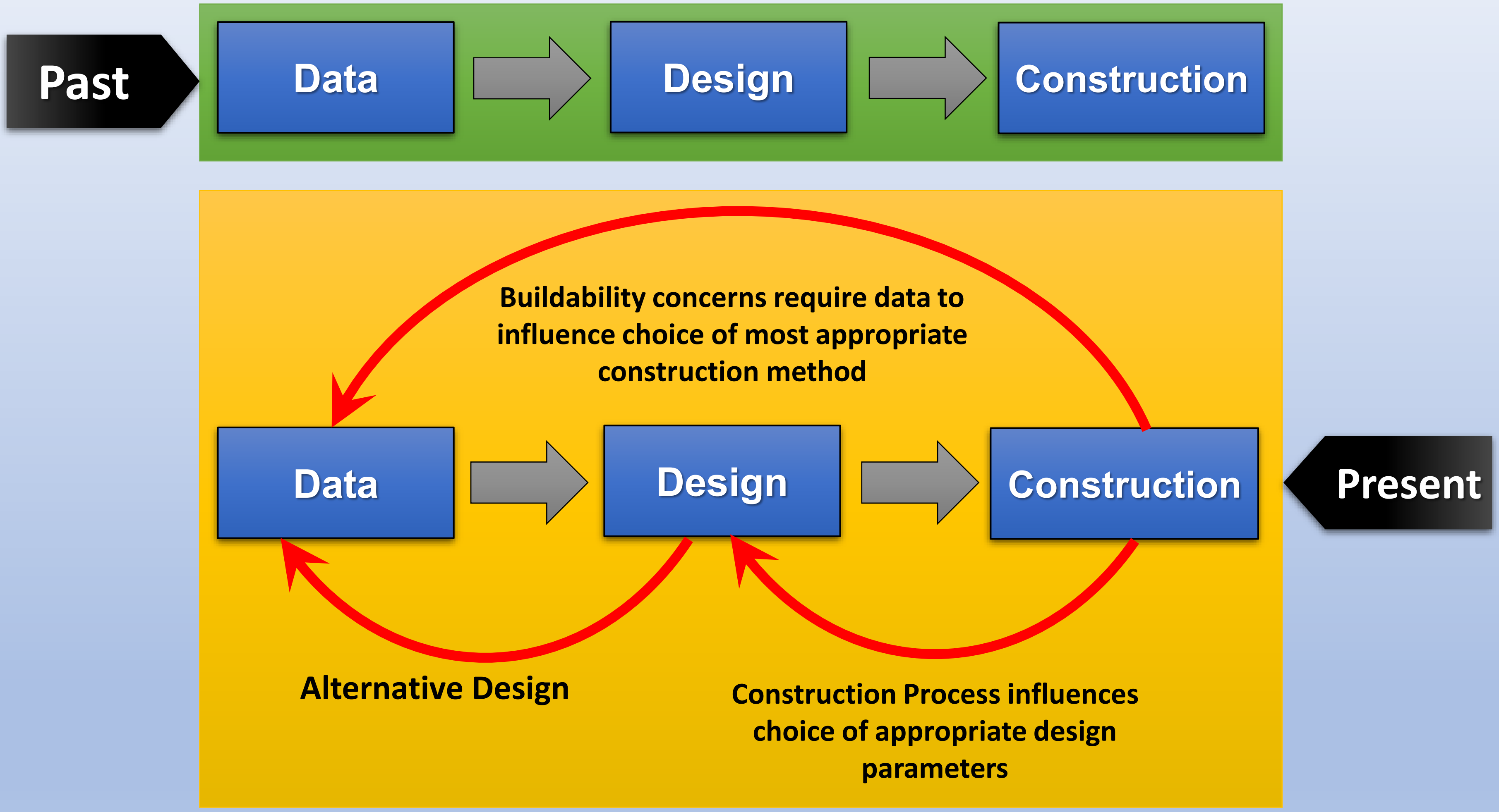
P_L & P_0

VST



S_u

Cycle of Data, Design, and Performance



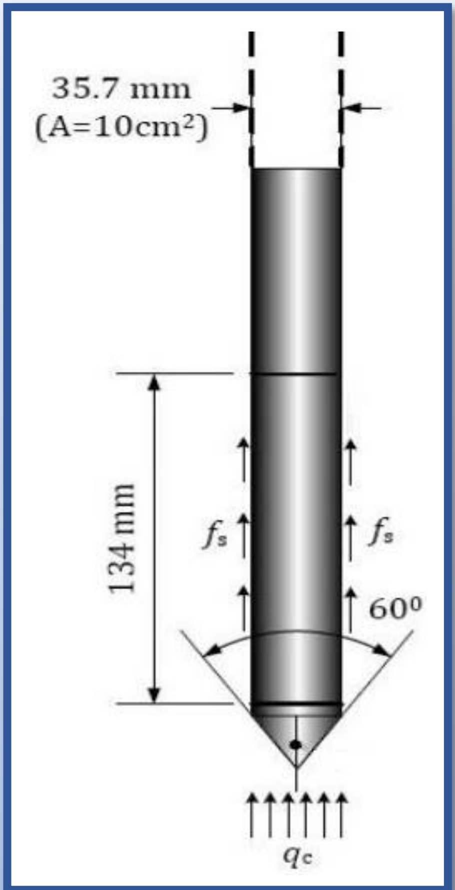
ICE, Geotechnical Manual (2012)

CPT Device

CPT involves driving a system of a steel cone and rods into the ground, and recording the mobilized resistance to penetration in the soil.

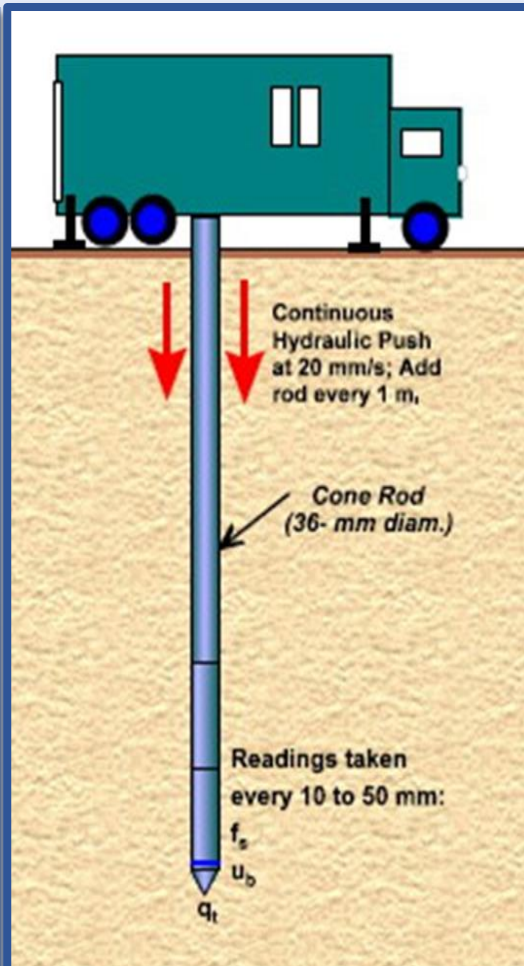
Penetration rate: 20 mm/s
Set of Data: Per 1 inch

- **Simple and relatively economical,**
- **Continuous records with depth,**
- **Interpretable on both empirical and analytical bases,**
- **Sensors can be incorporated with penetrometer,**
- **Availability of Profound experience-based knowledge**

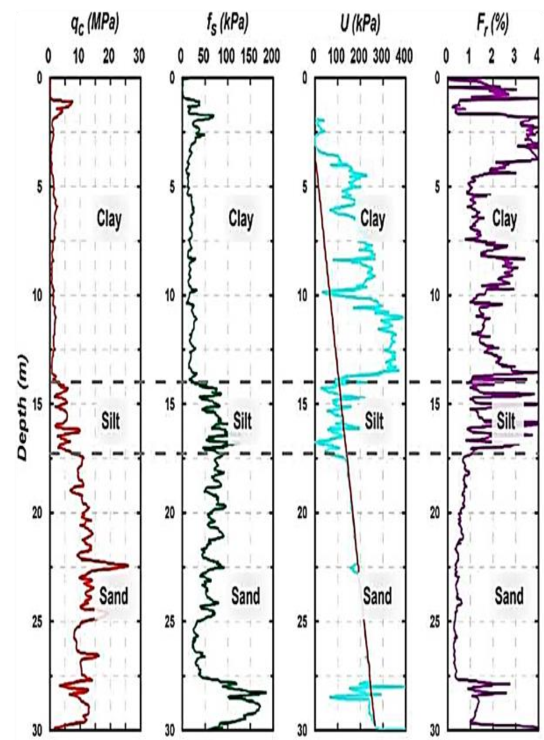


CPT; mostly applicable in soft to medium, compressible & problematic deposits

Equipment & Procedure



Penetration Rate: 20 mm/s
Set of Data: Per 25 mm or 1 inch



Data & Graphical Presentation

1. Measured Parameters

q_c, f_s, u

2. Corrected Parameters

- **Corrected tip resistance:**

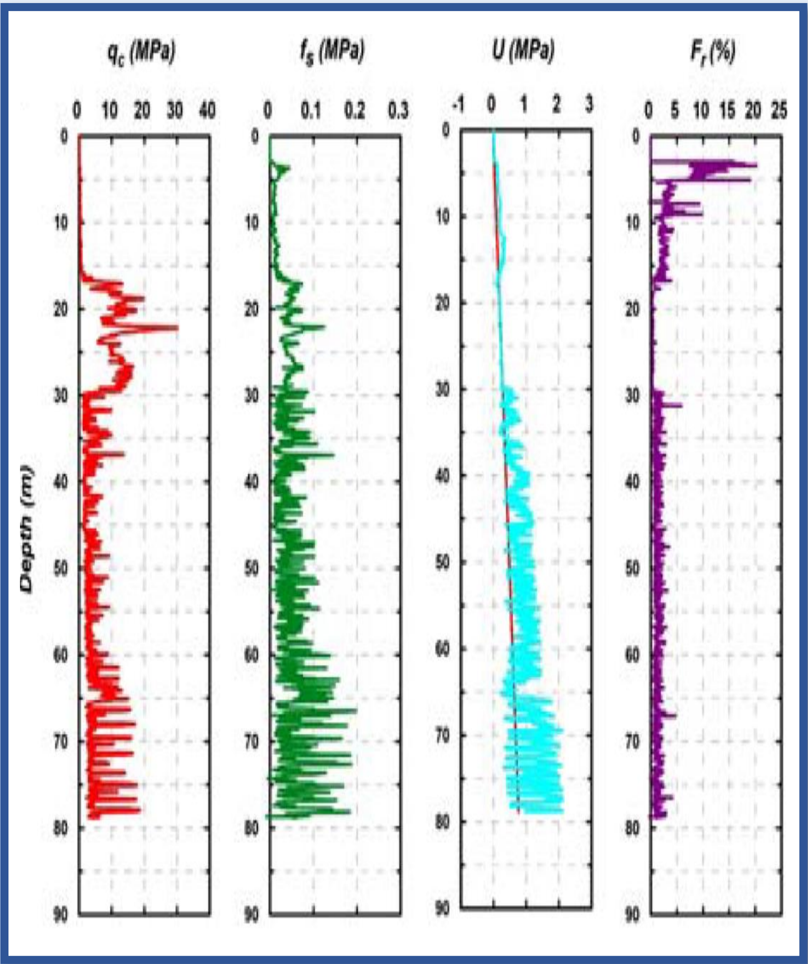
$$q_t = q_c + u_2(1 - a)$$

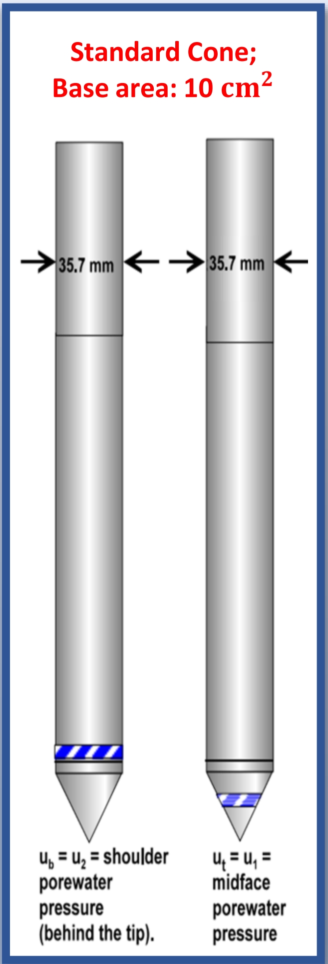
- **Friction ratio:**

$$R_f = {f_s}/{q_c}$$

- **Pore pressure coefficient:**

$$B_q = {\Delta u}/{(q_t - \sigma_{vo})}$$





Soil Characterization and Profiling

Soil Behavior Classification (SBC)

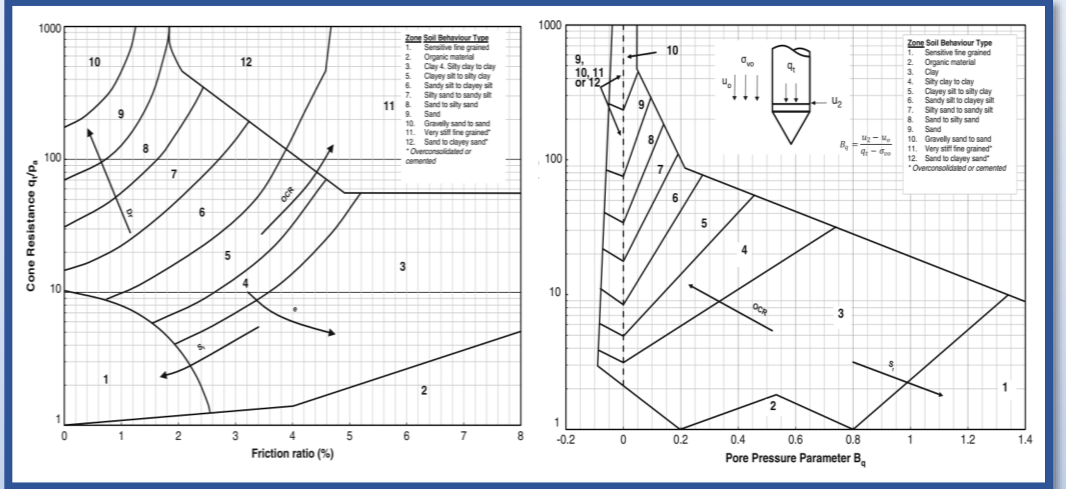
Estimating Engineering Parameters

Ground Improvement Practice

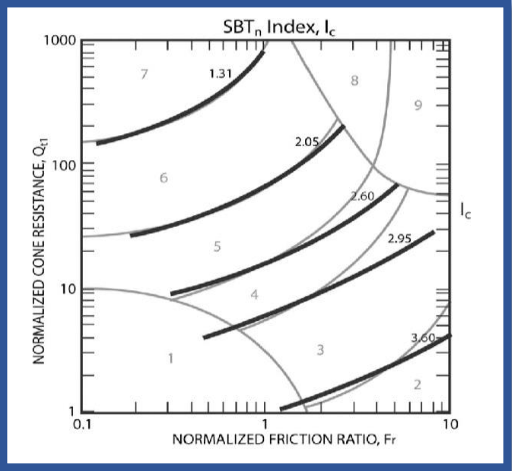
Foundation Engineering

Soil Behavior Classification: Charts & Diagrams

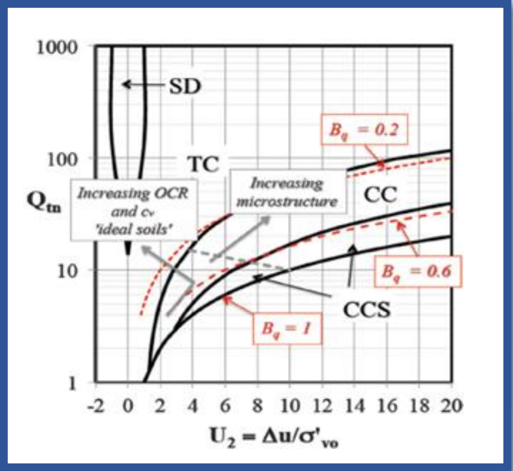
Robertson et al. (1986)



Robertson (2010)



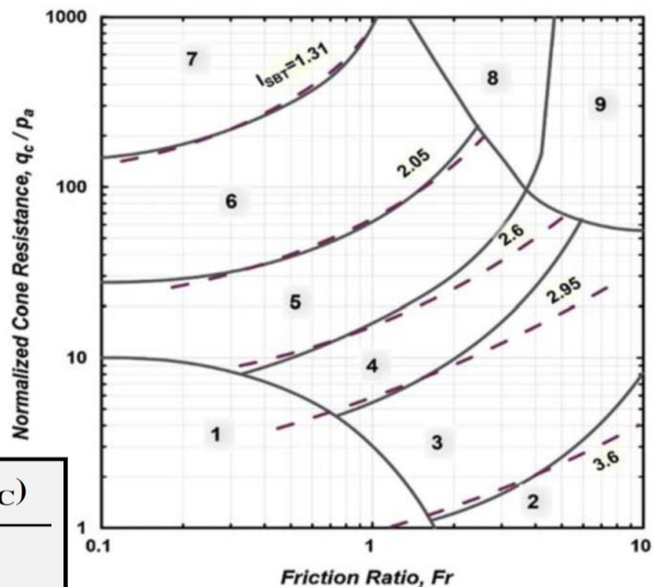
Robertson (2016)



Soil Behavior Classification: Charts & Diagrams

Jefferies and Davies (1991): I_{SBT} (I_C)

$$I_{SBT} = \left[\left(3.47 - \log\left({}^{q_c}/_{P_a}\right)\right)^2 + (\log R_f + 1.22)^2\right]^{0.5}$$



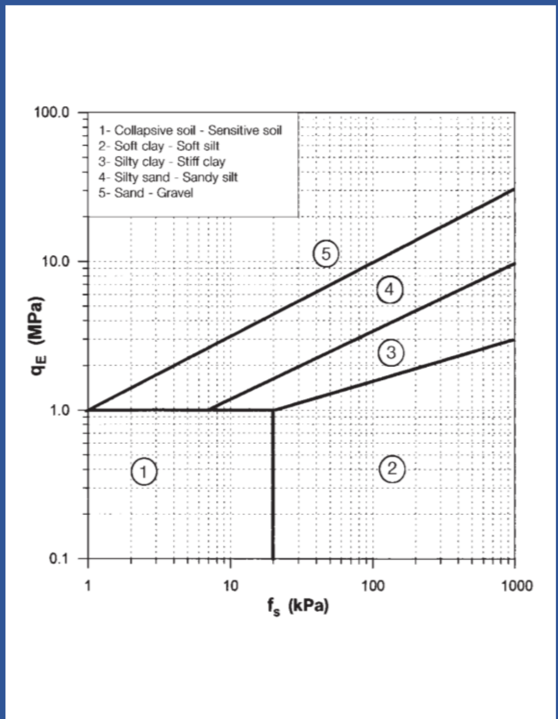
Values of I_c on the Robertson (1990) classification chart (Jefferies and Davies, 1991)

Soil classification based on I_{SBCT} (Li et al., 2015)

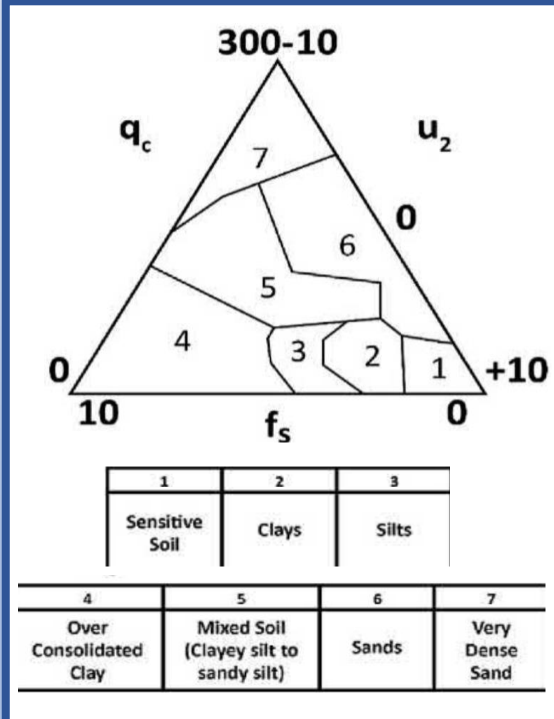
Soil classification	Zone	CPTu index (I_{SBC})
Gravelly sands	7	$I_{SBC}<1.25$
Sands—clean sand to silty sand	6	$1.25<I_{SBC}<1.90$
Sand mixture—silty sand to sandy silt	5	$1.90<I_{SBC}<2.54$
Silt mixture—clayey silt to silty clay	4	$2.54<I_{SBC}<2.82$
Clays	3	$2.82<I_{SBC}<3.22$
Organic clays	2	$I_{SBC}>3.22$

Soil Behavior Classification: Charts & Diagrams

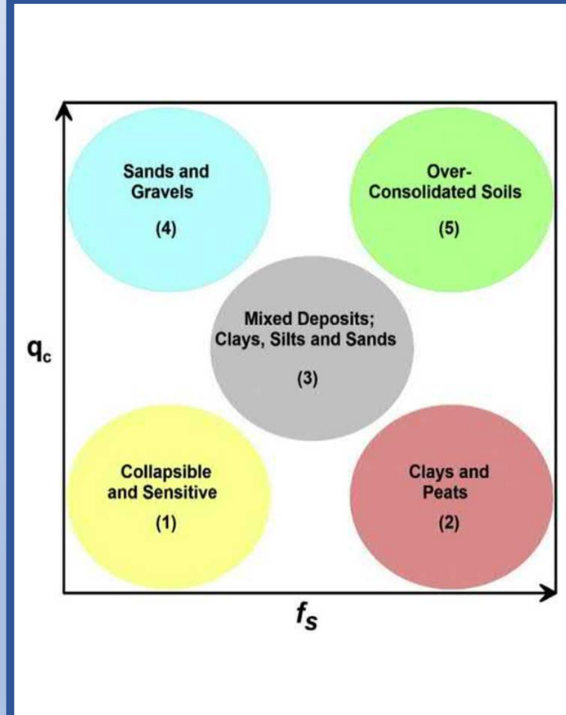
Eslami and Fellenius (1997)



Eslami et al. (2016 & 2022)

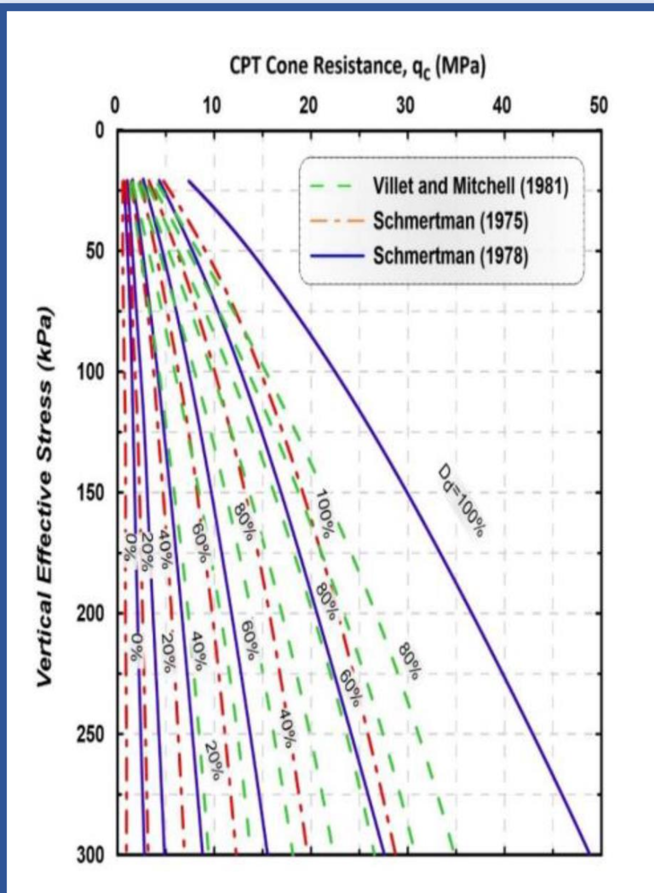


Eslami et al. (2018)

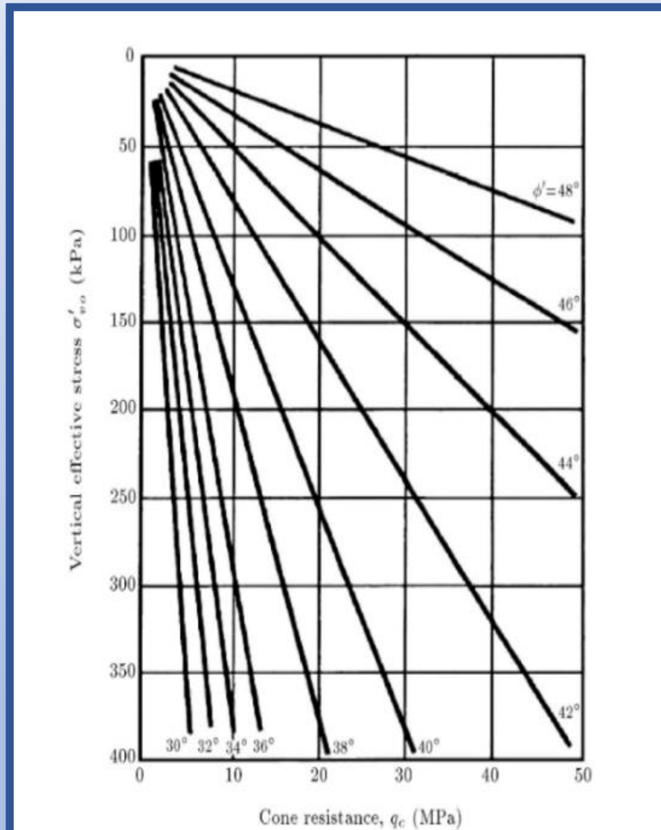


Estimating Soil Engineering Parameters

Relative Density ($\mathrm{D_r}$)



Friction Angle (φ)



(Mitchell & Durgunoglu, 1983)

Estimating Soil Engineering Parameters

Stiffness (E_s)

Soil Type	CPT
Sand	$E_s = (2-4)q_u$ $= 8000q_u$ - - - - - - - - - - - - - - - - - $E_s = 1.2(3D_r^2 + 2).q_c$ $E_s = (1 + D_r^2).q_c$
Saturated Sand	$E_s = F.q_c$ $e = 1.0$ $F = 3.5$ $e = 0.6$ $F = 7.0$
OCR Sand	$E_s = (6-30)q_c$
Clay Sand	$E_s = (3-6)q_c$
Silty Sand	$E_s = (1-2)q_c$ $q_c < 2500kPa$ $E_s' = 2.5q_c$ $2500 < q_c < 5000kPa$ $E_s' = 4q_c + 5000$
Soft Clay	$E_s = (3-8)q_c$

Undrained Shear Strength (S_u)

Correlations for undrained shear strength of the cohesion of soils

Reference	Correlations	Remarks
Lunne et al. (1997)	$S_u = {(q_c - \sigma_v)}/{N_c}$	N_c: cone factor
Risery (1974)	$S_u = {q_c}/{23}$	-
Kulhawy and Mayne (1990)	$S_u = \frac{\Delta u}{N_{\Delta u}}$	Δu = excess pore pressure measured at u_2 position = $u_2 - u_0$ $N_{\Delta u}$ = Pore pressure cone factor $N_{\Delta u} = N_{kt}$ Bq $N_{\Delta u}$ varies between 4 and 10
Naeini and Moayed (2007)	${S_u}/{\sigma'_v} = 0.107 + 0.111q_{cn1}$	q_{cn1}: normalized cone tip resistance; FC<30%
Rémai (2013)	The same as Kulhawy and Mayne (1990) method	$N_{\Delta u}$=24.3 Bq

Estimating Soil Engineering Parameters

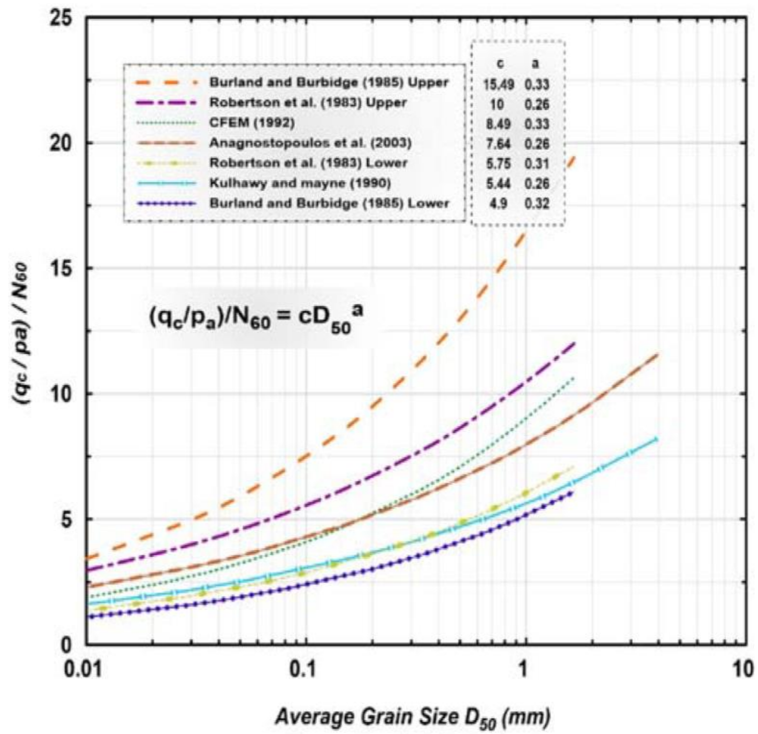
Unit Weight (γ)

Reference	Correlations	Parameter unit
Robertson and Cabal (2010)	$\gamma/\gamma_w = 0.27(log\, R_f) + 0.36(log\, q_t/p_a) + 1.236$	R_f (%)
Mayne (2010)	$\gamma_t = 1.95\gamma_w(\sigma'_{vo}/p_a)^{0.06}(f_s/p_a)^{0.06}$	f_s (MPa)
Mayne (2014)	$\gamma = 26 - \dfrac{14}{1 + (0.5\, log(f_s + 1))^2}$	f_s (kPa)
Baginska (2016)	$\gamma = 11 + 24\, ln(f_s + 0.7)$	f_s (MPa)
Lengkeek et al. (2018)	$\gamma_{sat} = 19 - 4.12\, log(5/q_t)/log(30/R_f)$	R_f (%), q_t (MPa)

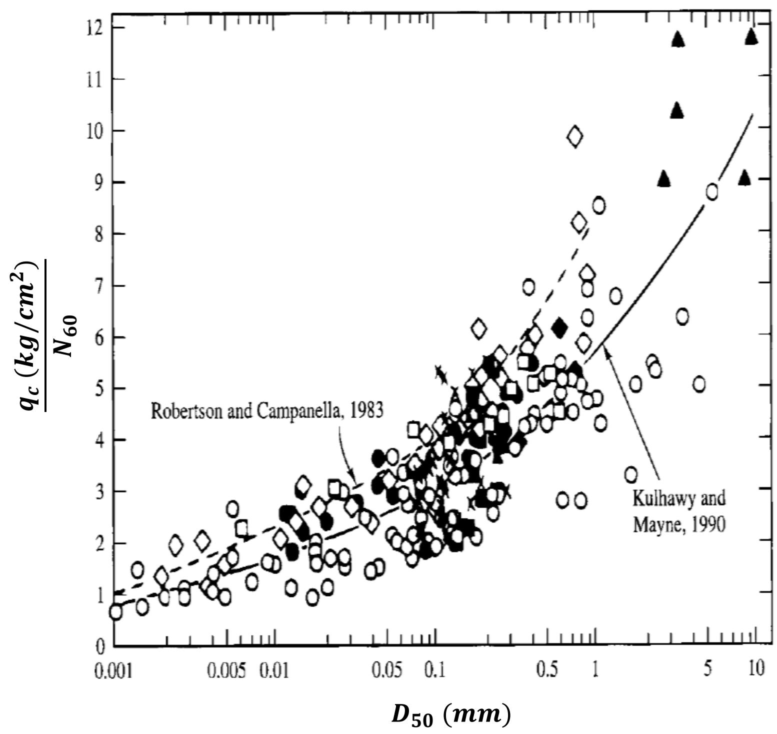
Pa: Atmosphere pressure; q_t: Corrected q_c ; σ'_v: Effective vertical stress; γ (kN/m³)

Estimating Soil Engineering Parameters

CPT (q_c) correlations with SPT (N)



(Adopted from Robertson & Campanella, 1983)



Estimating Soil Engineering Parameters

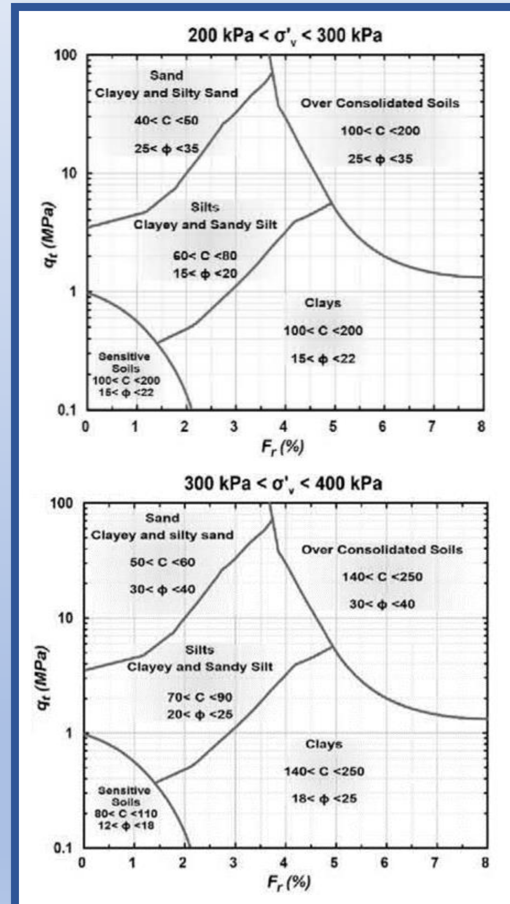
- **Eslami & Mohammadi (2016)**

q_c, f_s, u_2 → **C′ , ϕ′**

$$f_s = \tau = \sigma'_z tan\phi + C$$
$$q_c = q_u = CN_c + \bar{q}\, N_q + 0.5\gamma B N_\gamma$$
$$N_c, N_q \,\&\, N_\gamma \;\rightarrow f(\phi)$$

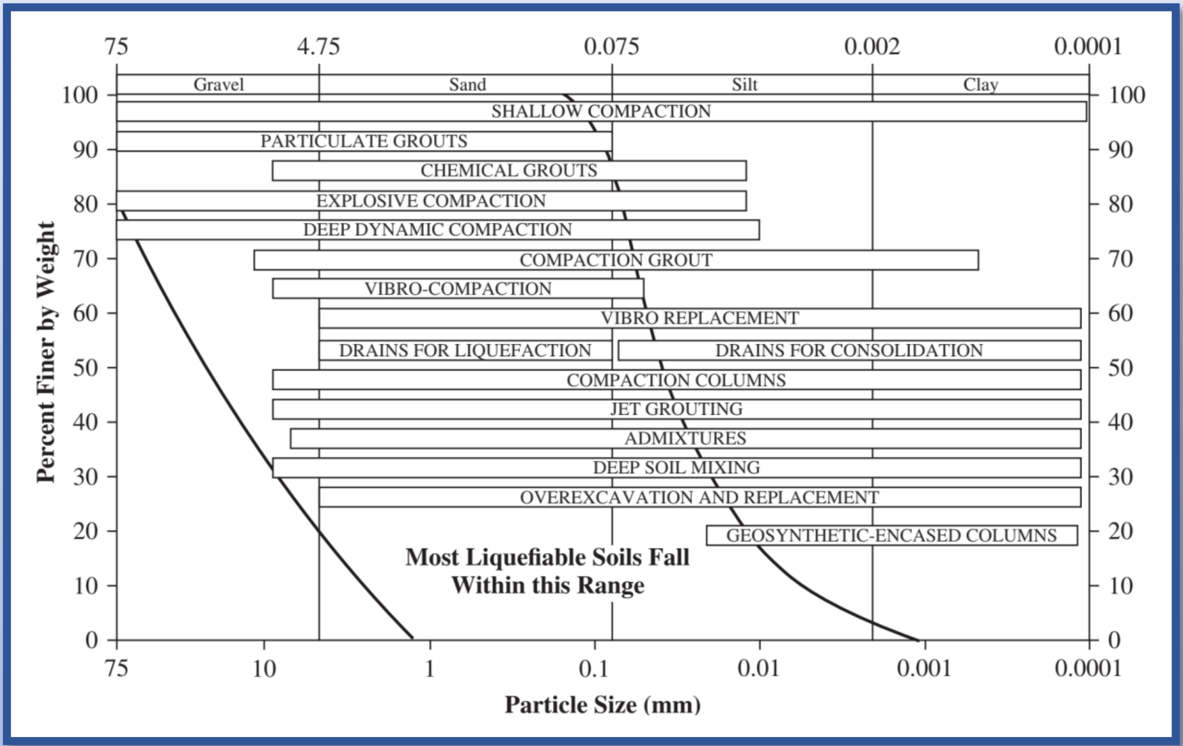
$$\begin{cases} C + 0.000789(1 - sin\phi)\sigma'_{v_0} tan\left(\frac{2}{3}\phi\right)\left[\dfrac{q_c - \left(\dfrac{\sigma_{v_0} - 2\sigma_{h_0}}{3}\right)}{\left(\dfrac{\sigma'_{v_0} - 2\sigma'_{h_0}}{3}\right)}\right]^{1.44} = f_s \\ \left(tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi tan\phi} - 1\right) C\, cot\phi + \bar{q}.\, tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi tan\phi} + \\ \gamma B \left[tan^2\left(\frac{\pi}{4} + \frac{\phi}{2}\right) e^{\pi tan\phi} + 1\right] tan\phi = q_E + N_u \Delta U \end{cases}$$

Variation range for C (kPa) and ϕ (Degree)



Role in Ground Improvement

- Identification
- Improvement Justification
- Design Procedure
- Method Selection
- Performance Assessment

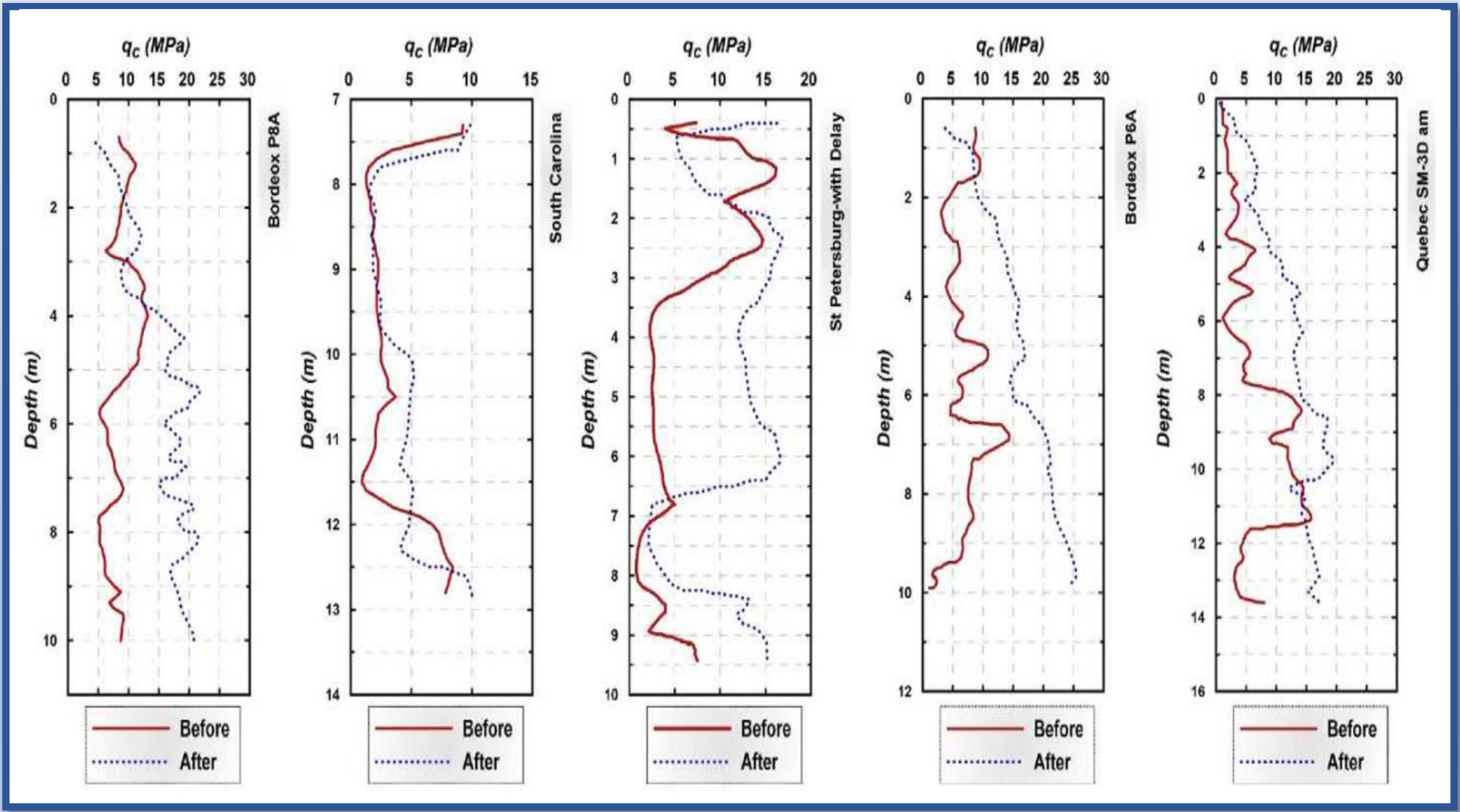


Available ground improvement methods for different soil types (modified from Schaefer et al., 2012)

Assessment of Soil Improvement Practice

- **Blast or Explosive Densification (Eslami, 2015)**

Pre & Post Soil Improvement Records Comparison



Applications in Foundation Engineering

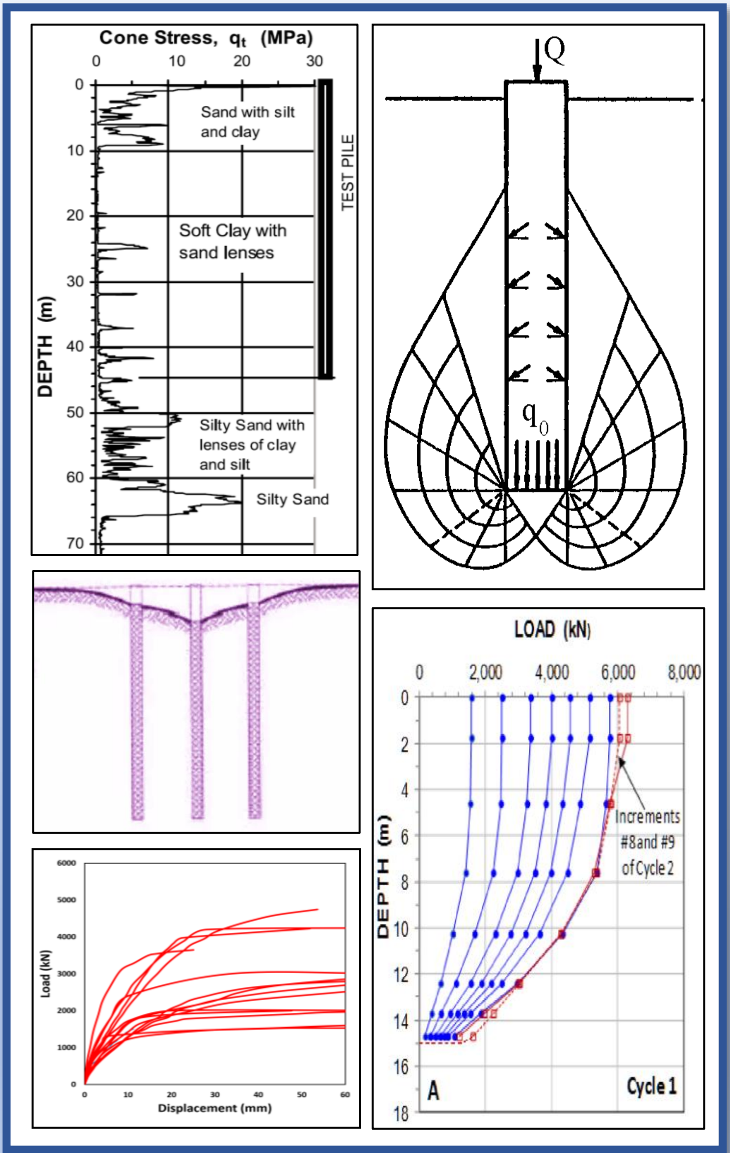
1. Selection & Installation

2. Bearing Capacity

3. Resistance Distribution

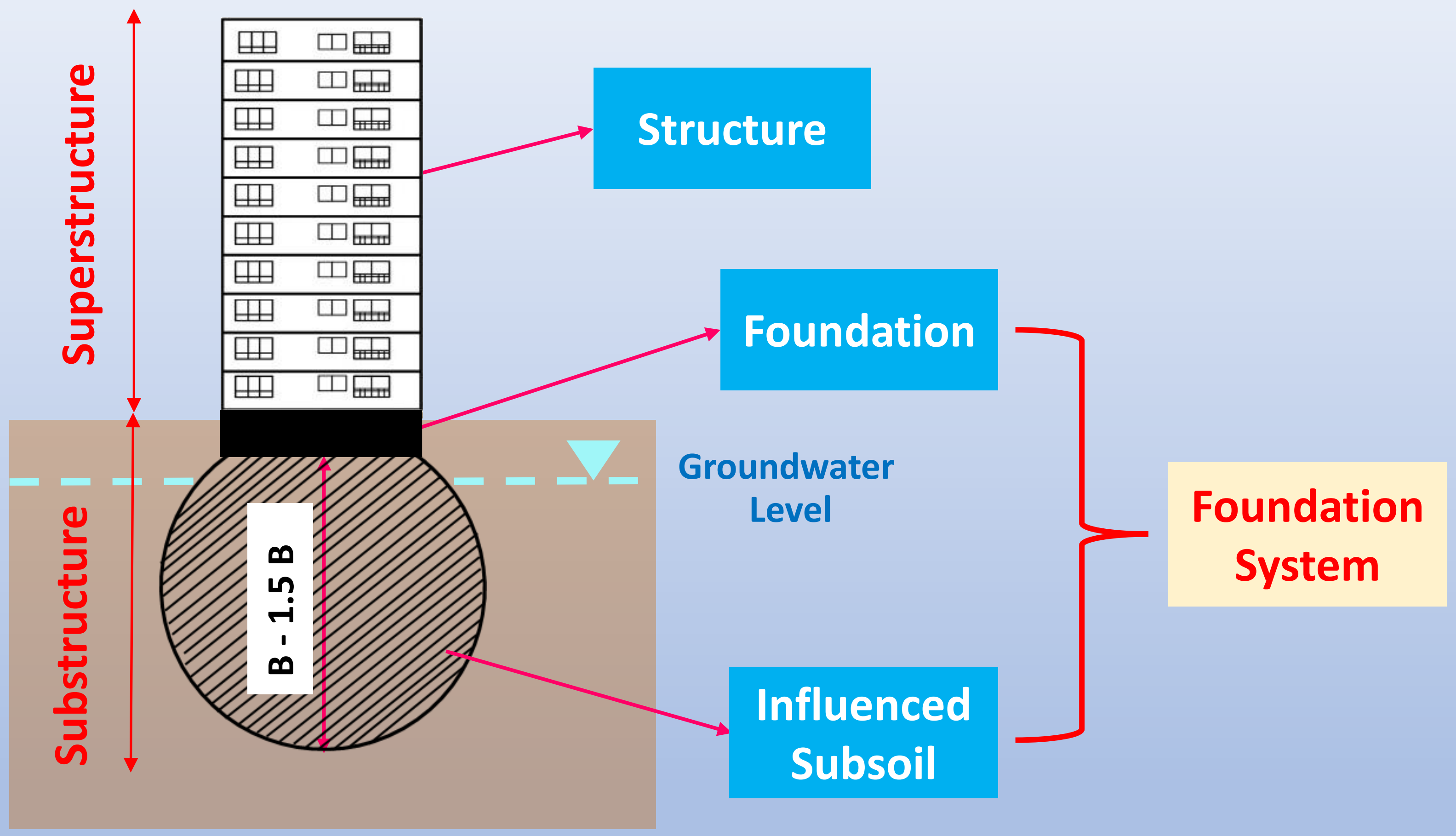
4. Settlement

5. Load - Displacement

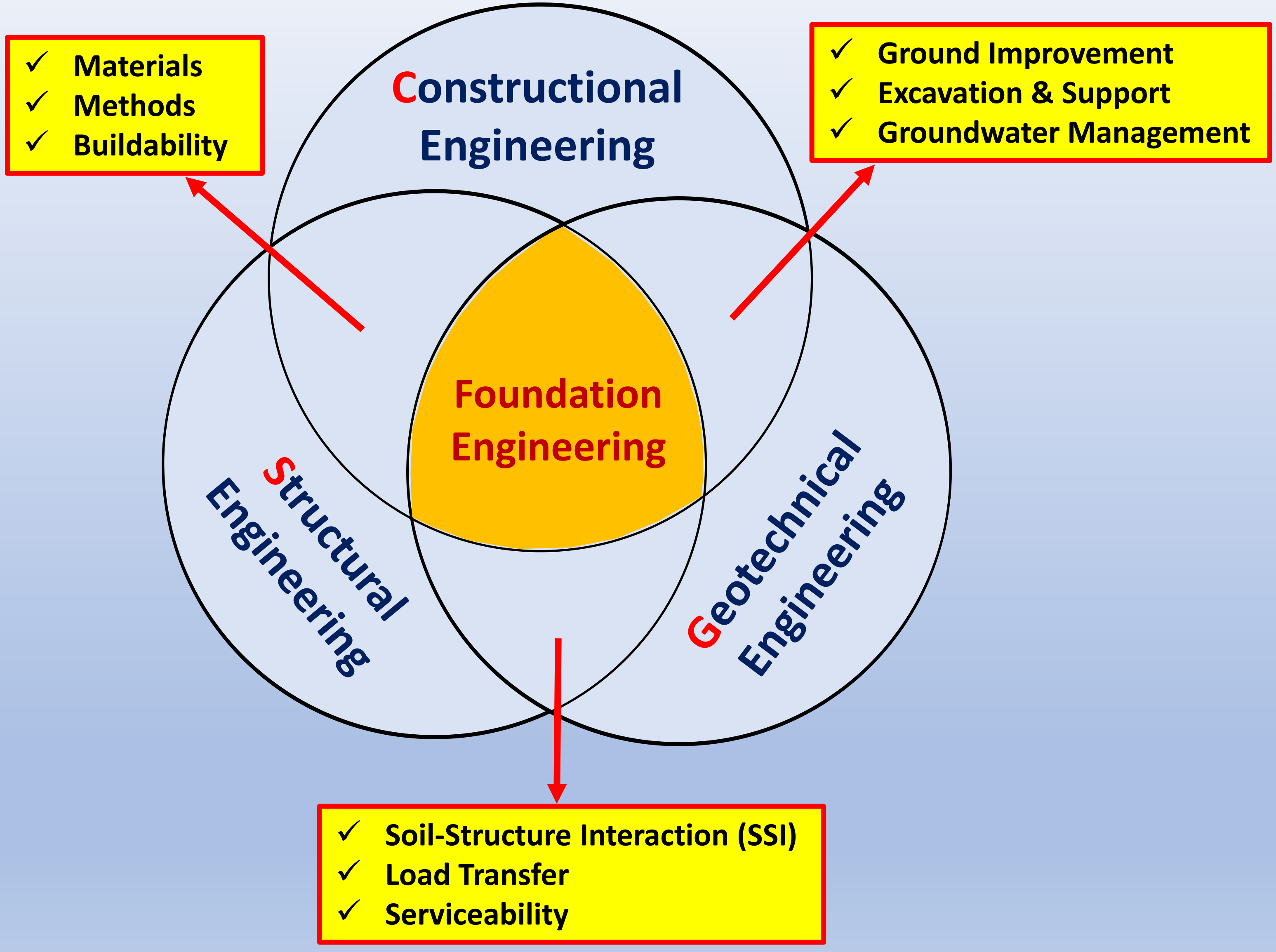


1. CPT & CPTu in Geotechnical Engineering
2. **Applications in Foundation Engineering**
3. CPT-Based Methods for Deep Foundations
4. Worked Examples
5. Case Histories
6. Prospects

Interaction of Superstructure & Substructure



Foundation elements or systems are structural units that transfer various load combinations from the superstructure to the underlying soils or rocks (i.e., geomaterials). Foundation units may tolerate the loads individually or by contribution of other elements such as basement walls, floors, or slabs (Eslami et al., 2020).


✓ Materials
✓ Methods
✓ Buildability
✓ Ground Improvement
✓ Excavation & Support
✓ Groundwater Management
Constructional Engineering
Foundation Engineering
Structural Engineering
Geotechnical Engineering
✓ Soil-Structure Interaction (SSI)
✓ Load Transfer
✓ Serviceability

Major Analysis & Design Requirements

Eslami et al. (2020):

- Bearing Capacity
- Serviceability
- Structural Design
- Stability Control
- Full or Model Scale Testing
- Constructional Aspects
- Durability
- Economic Requirements

Foundation Engineering Tools:

Salgado (2008):

- Soil & Rock Mechanics
- Codes & Standards
- Experience & Empiricism
- Publications: Where to Go for Help
- Conferences & Short Courses
- Computers

Fellenius (2015): The analysis and design of foundations are an iterative process since the amount of imposed loads, corresponding settlement, and foundation geometry are interactive, affected by geotechnical capacity, structural capacity and settlement requirements.

Foundations Classification

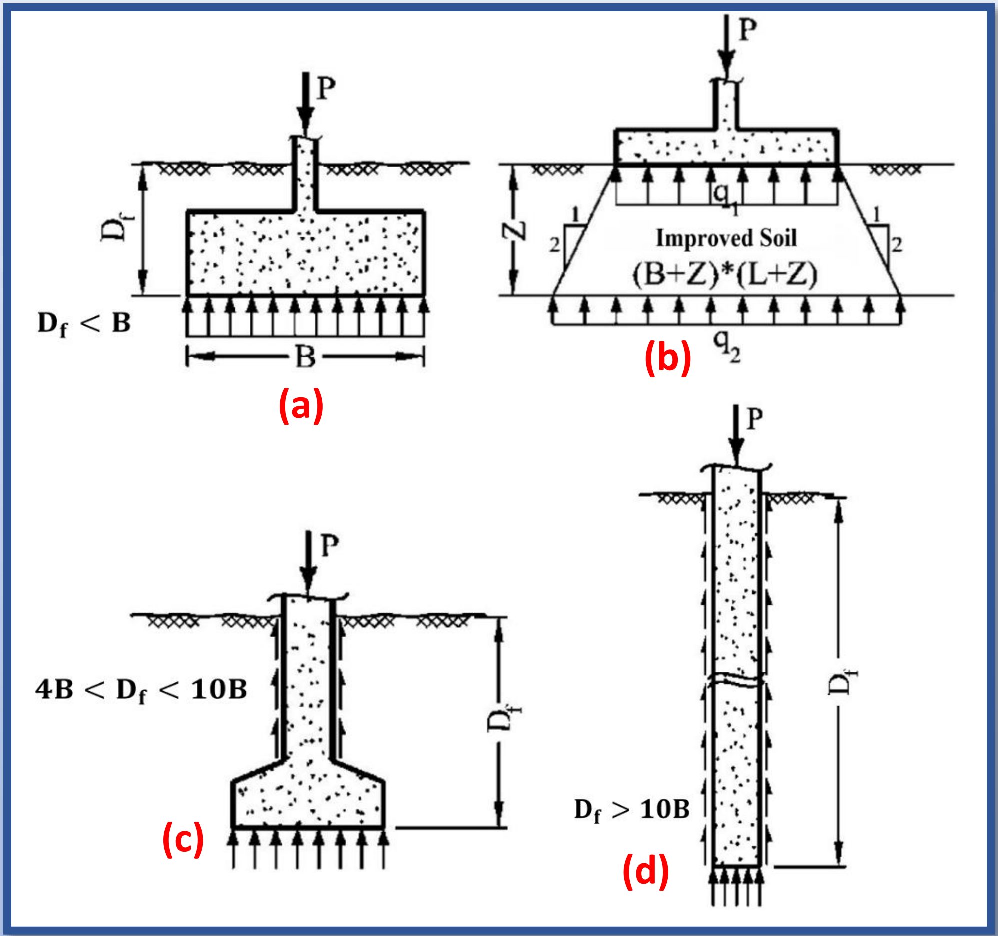
- **Embedment Depth**

✓ Shallow Foundations (a)

✓ Shallow + Soil Improvement (b)

✓ Semi-deep Foundations (c)

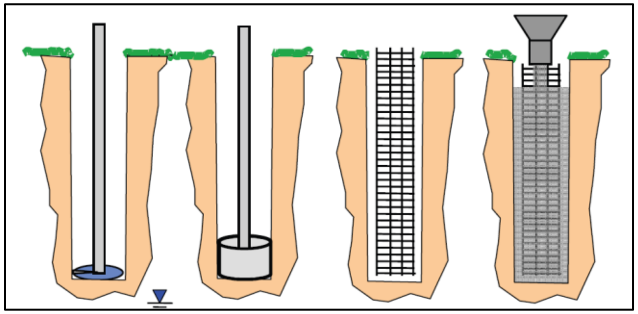
✓ Deep Foundations (d)



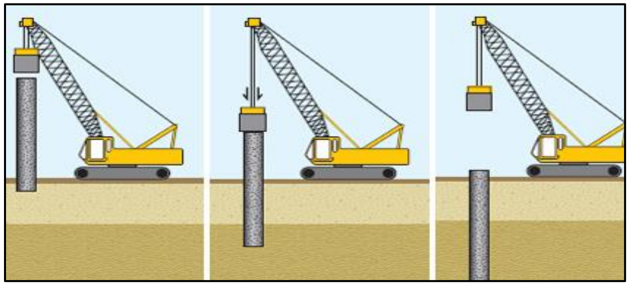
Current categories of foundations (Eslami et al., 2020)

Different Types of Deep Foundations

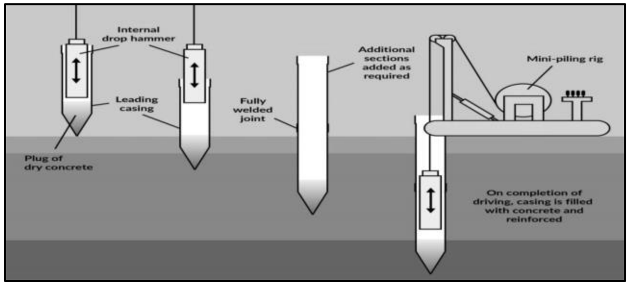
- **Replacement (non-displacement)**



- **Displacement**



- **Special**



Common Approaches for Bearing Capacity Appraisal

1. Static Analysis
2. In-Situ Tests Application
3. Static Load Test
4. Dynamic Analysis & Load Test
5. Numerical & Laboratory Modelings

Pile Bearing Capacity

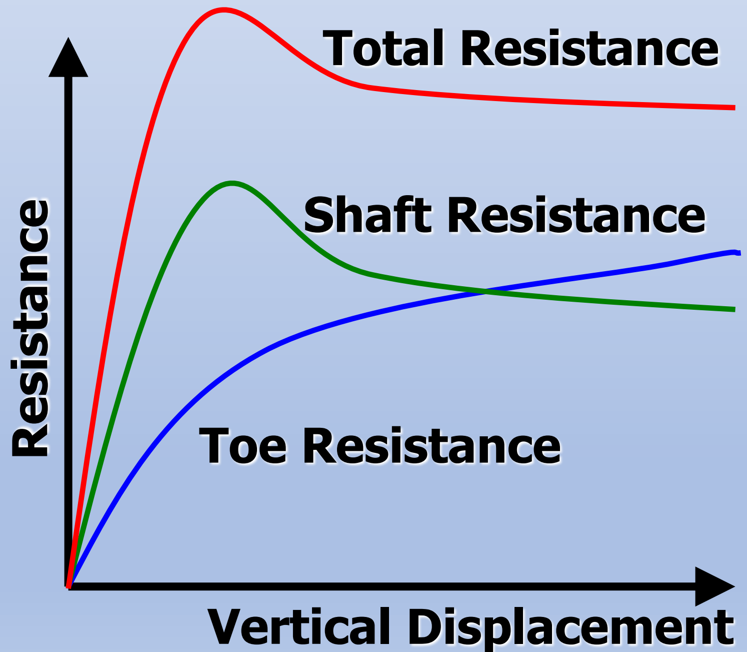
Ultimate Bearing Capacity = Toe Capacity + Shaft Capacity

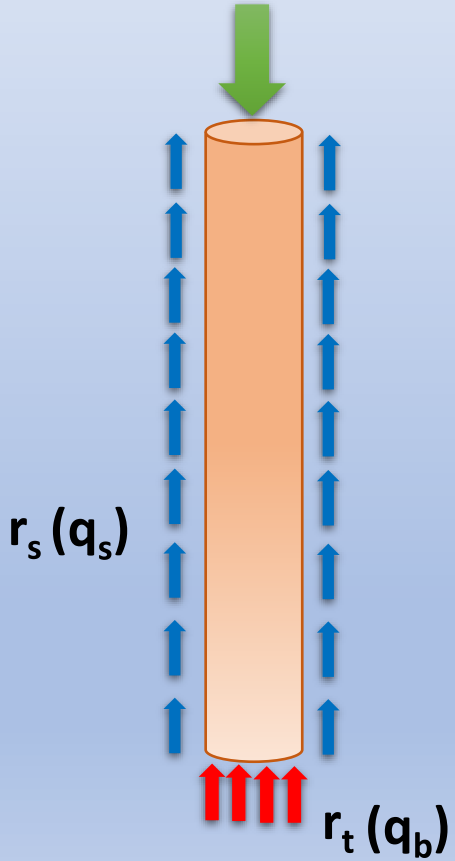
$$R_t = r_t . A_t$$

$$R_s = r_s . A_s . D_f$$

$$R_u = R_t + R_s$$

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





Static Analysis

Shaft Resistance

For $C - \varphi$ Geomaterials → $r_s = K_c(C + \sigma'_v tan\varphi)$

Toe Resistance

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

For $C - \varphi$ Geomaterials → $r_t = CN_C^* + \gamma D_F .N_q^*$

For Fine-Grained Soils (Undrained) → $r_t = CN_C^*$

For Coarse-Grained Soils (Drained) → $r_t = \overline{q}.N_q^*$

Static Analysis

Unified Pile Design (CFEM)

$$q_b = N_t \times \sigma'_{z=D_f}$$
$$q_s = \beta \times \sigma'_{z-avg}$$

$\sigma'_{Z=D_f}$ 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

Static Analysis

API (2011)

Cohesive Soils

$$q_b = 9S_u$$
$$q_s = \alpha S_u$$

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

Cohesionless Soils

$$q_b = N_q \times \sigma'_{z=D_f}$$
$$q_s = \beta \times \sigma'_{z-avg}$$

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.

Static Analysis

NAVFAC (2012)

Cohesive Soils

$\mathbf{p'_0}$ = Average effective vertical stress along the shaft

For $\frac{S_u}{p'_0} \leq 0.4 \rightarrow \text{NC Clay} \rightarrow q_s = p'_0[0.468 - 0.052\text{Ln}\left(\frac{L_P}{2}\right)] \leq S_u$; L_P = Embedment depth

For $0.4 < \frac{S_u}{p'_0} \leq 2 \rightarrow \text{OC Clay} \rightarrow q_s = S_u[0.458 - 0.155\text{Ln}\left(\frac{S_u}{p'_0}\right)]$

For $\frac{S_u}{p'_0} > 2 \rightarrow q_s = 0.351S_u$

$$q_b = 9S_u$$

Cohesionless Soils

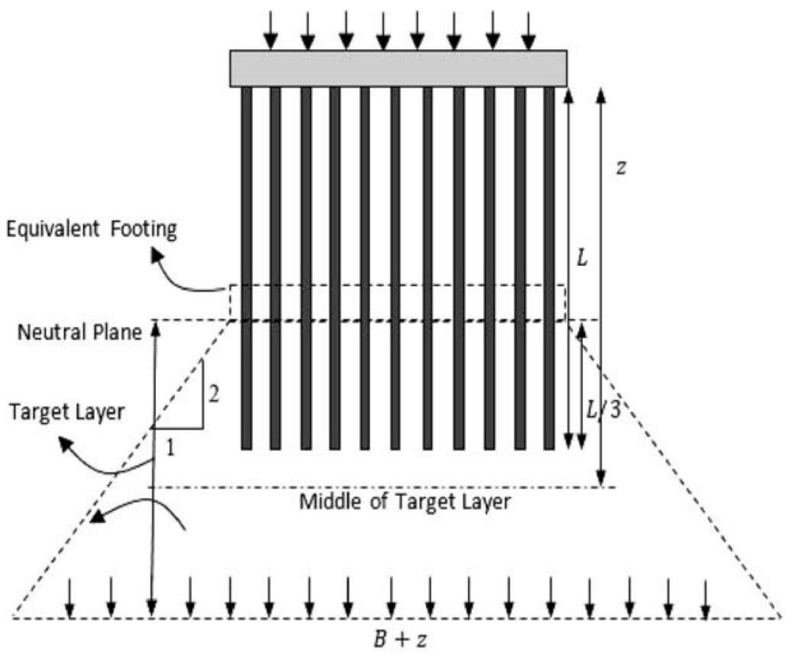
$$q_b = N_q \times \sigma'_{z=D_f}$$

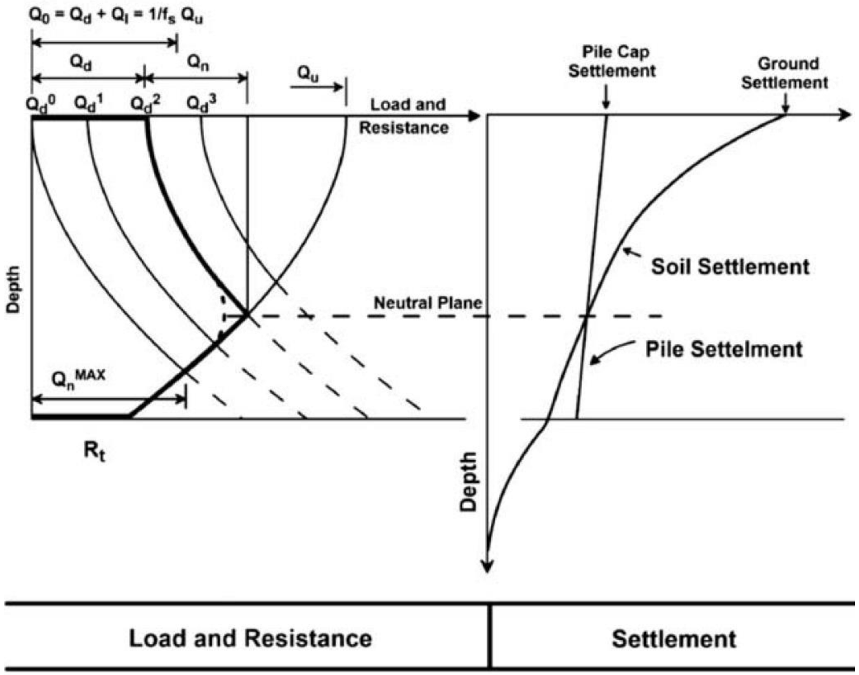
$$q_s = K\tan(\varphi - 5) \times \sigma'_z$$

K: Compression=0.7, Uplift=0.5

Soil Type	ϕ (deg)	δ (deg)	N_q	f_s (max) (ksf)	q_p (max) (ksf)
Noncalcareous Soils					
Sand	35	30	40	2.0	200
Silty Sand	30	25	20	1.7	100
Sandy Silt	25	20	12	1.4	60
Silt	20	15	8	1.0	40
Calcareous Soils					
Uncemented calcareous sand (easily crushed)	30	20	20	0.3[a]	60

Settlement & Resistance Distribution





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

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

Direct Application for Settlement

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

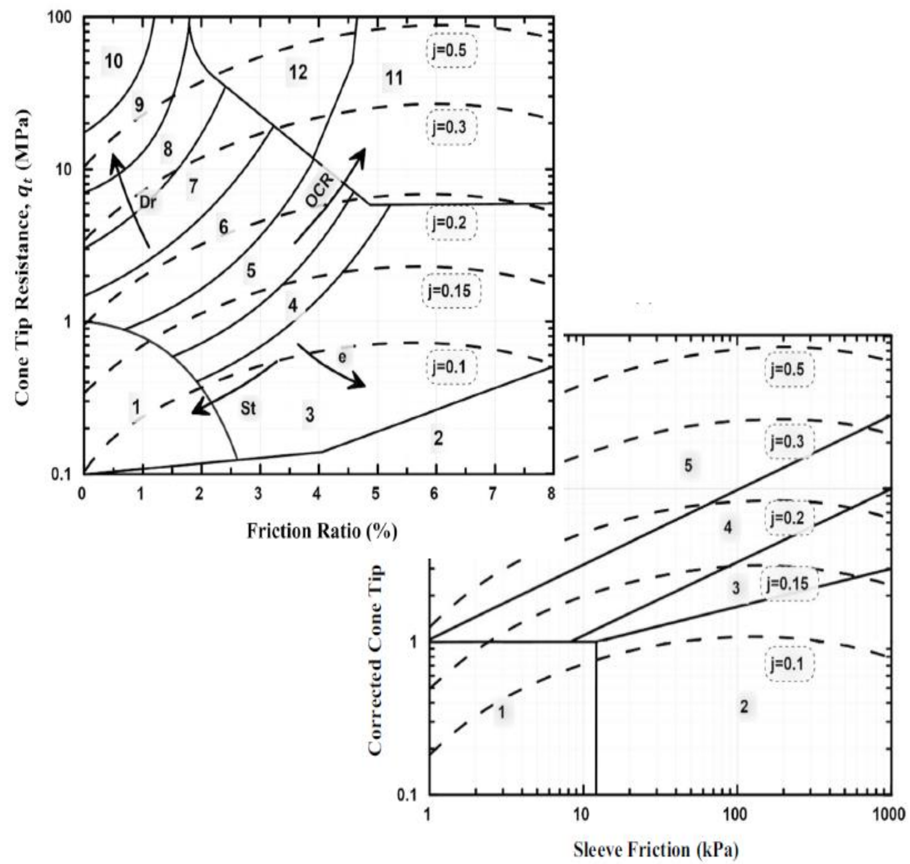
(b and B are in m and q_c is in kPa)

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

$$x = 0.02R_f + 0.5$$

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

(q_c and σ_0' are in kPa)



Integrated Settlement Estimation: New Approach

$$\Delta L = \frac{PL}{EA} = \frac{\sigma L}{E}$$

$$S = k\frac{qB}{E}$$

q = Applied Pressure,
B = Breadth,
$k = f$ (Shape, Confinement & Time)~ $0.5 - 1.5$
E = Elastic Modulus → From q_c Variation

Load-Displacement & Dominant Factors

Foundation

Geometry & Embedment

Installation Effects

Interface Properties

Structural Material

Geomaterial

Strength & Stiffness

Drainage Condition

Time History & Aging

Sensitivity & Thixotropy

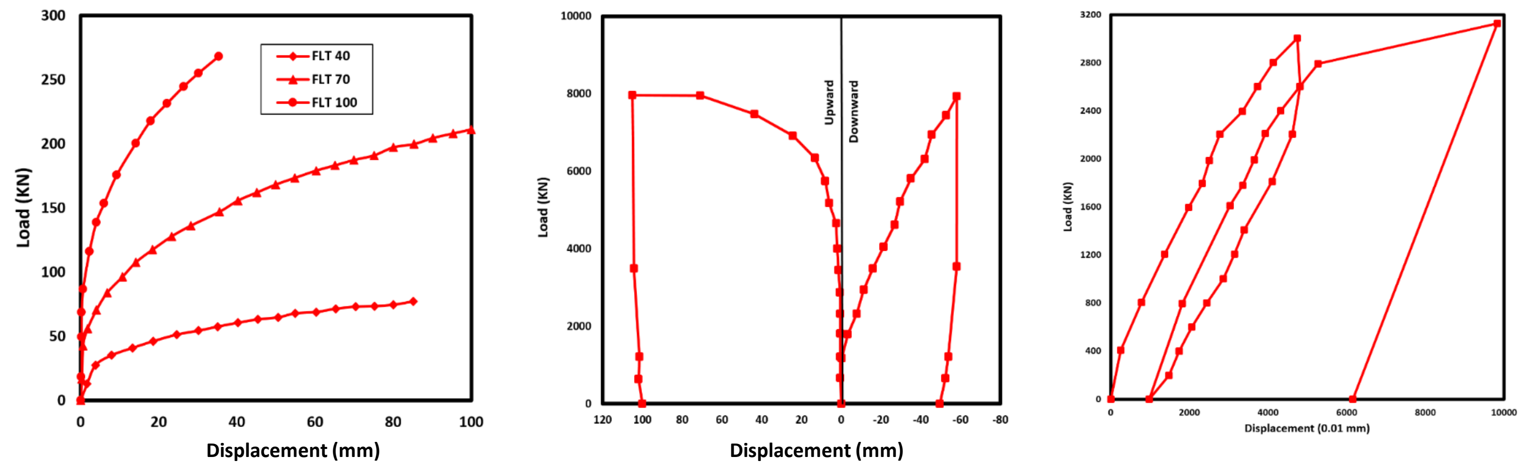
Loading

Pattern:

Slow or Rapid

Type:

Compressive, Tensile, Cyclic

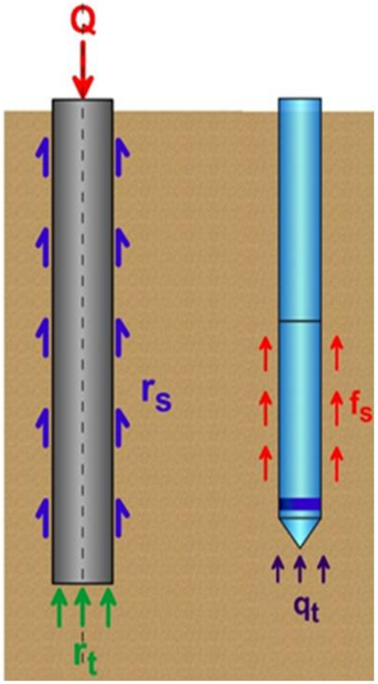


Square Concrete Footings (Consoli et al., 1998)

Bidirectional test on a bored pile (Fellenius, 2023)

Driven pile, pull out test (Eslami et al., 2011)

Direct & Indirect Approach



Indirect Approach { $\mathbf{q_c}$, $\mathbf{f_s}$, $\mathbf{u_2}$ } → C, $\boldsymbol{\varphi}$ → r_t, r_s

Direct Approach
- q_c → r_t ($\mathbf{q_b}$)
- f_s → r_s ($\mathbf{q_s}$)

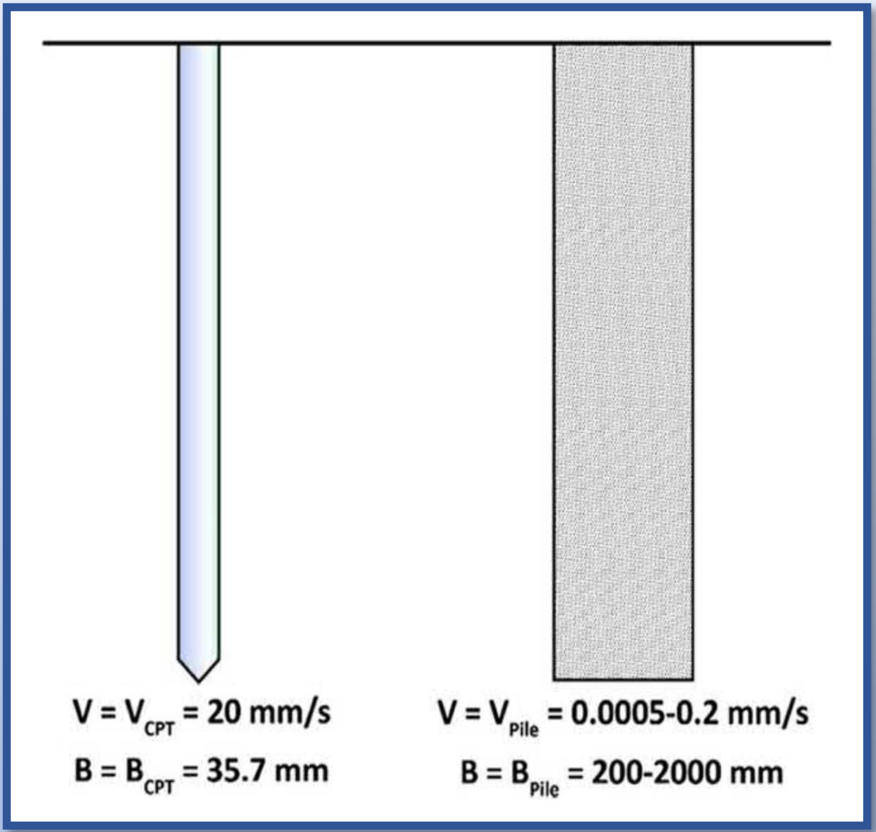
1. CPT & CPTu in Geotechnical Engineering
2. Applications in Foundation Engineering
3. **CPT-Based Methods for Deep Foundations**
4. Worked Examples
5. Case Histories
6. Prospects

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

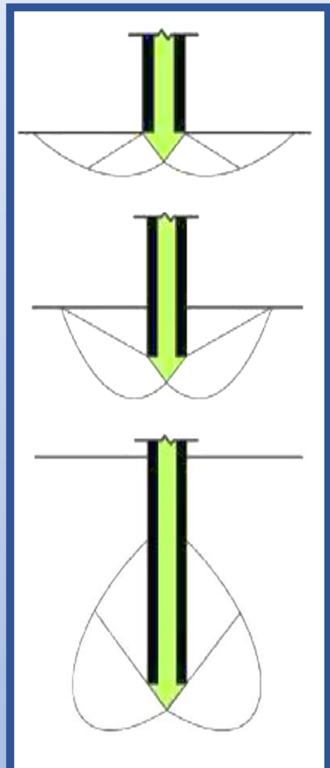
Penetrometers can be realized as a *model pile*



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

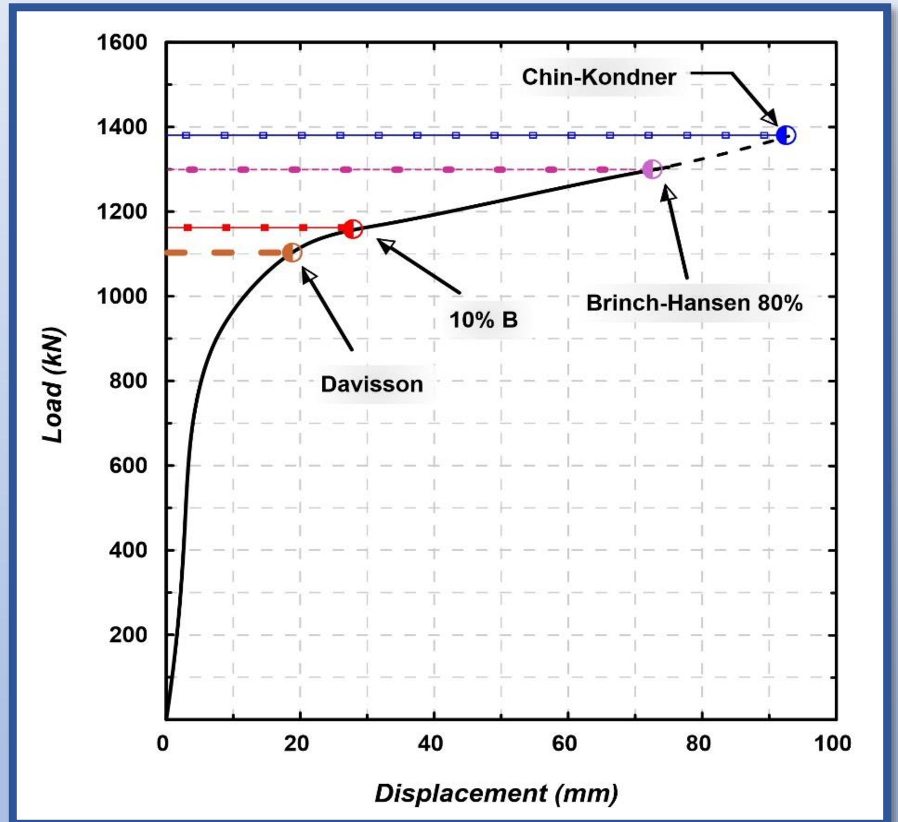
Scale Effect Correlations

Embedment Depth



Schematic view of transformation of shear failure from shallow to deep (Nottingham, 1975)

Ultimate Capacity Condition

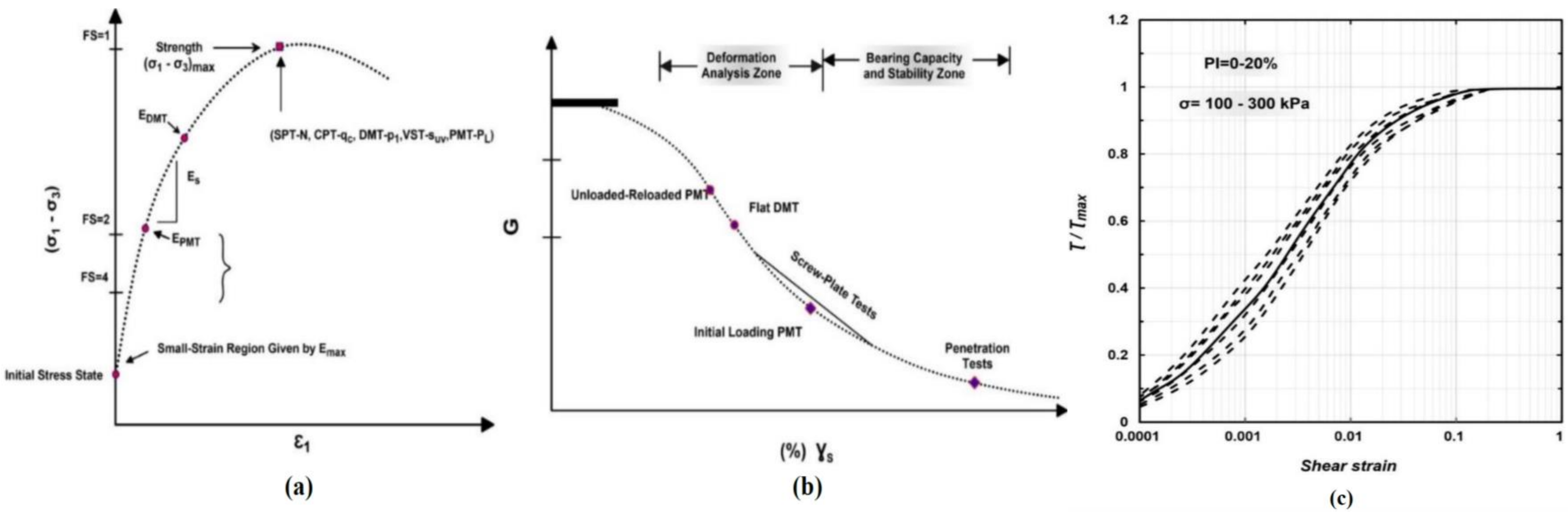


Interpretation of load displacement diagram for Case 001-L&D31 (Moshfeghi & Eslami, 2016)

Scale Effect Correlations

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

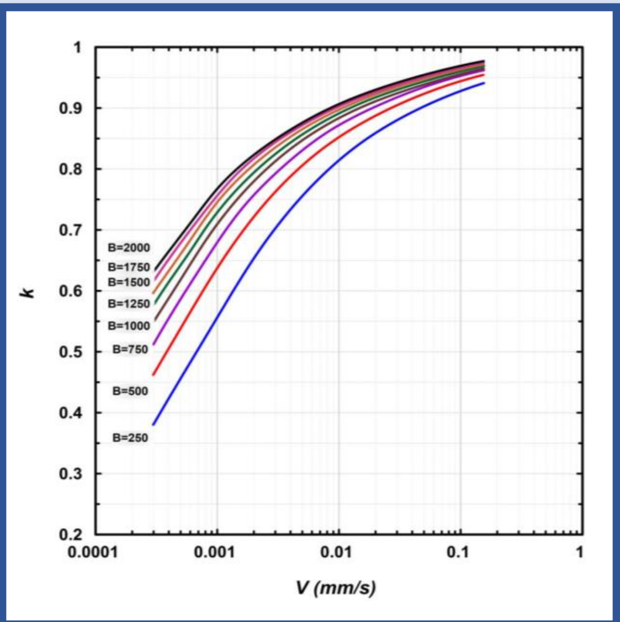
$$\gamma_{pile} = 0.0255\,(V_{pile})^{0.61} . (D_{pile})^{0.5}$$



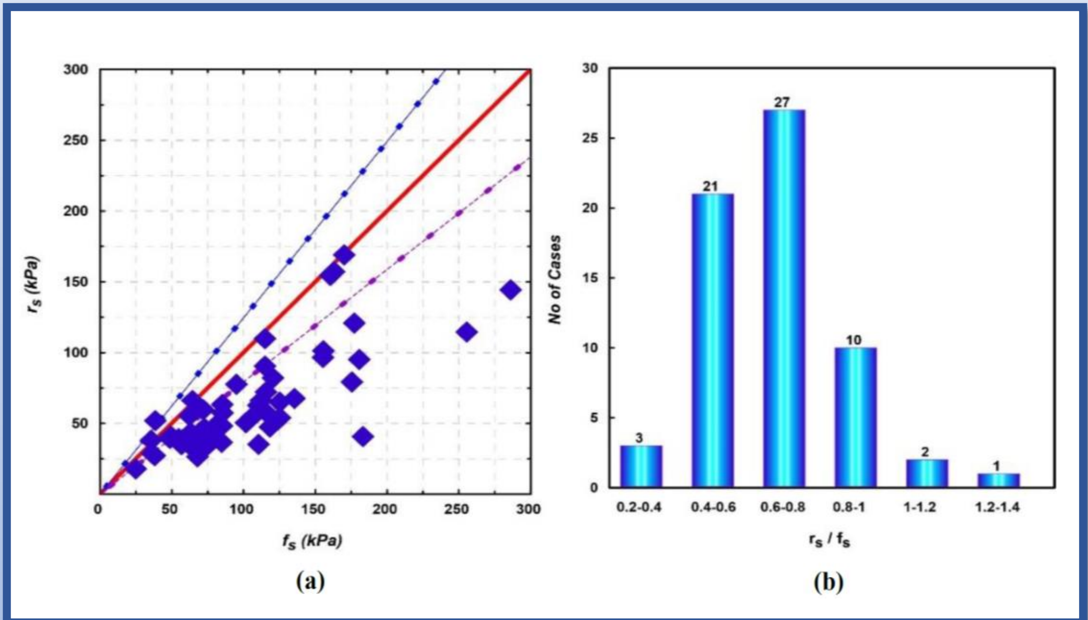
Stress strain Strength curves for different in situ tests; (a) strength measured by in situ tests at the peak of the stress strain curve, (b) variation of shear modulus with strain level (c) Variation of shear stress with shear strain (Sabatani et al., 2002)

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)

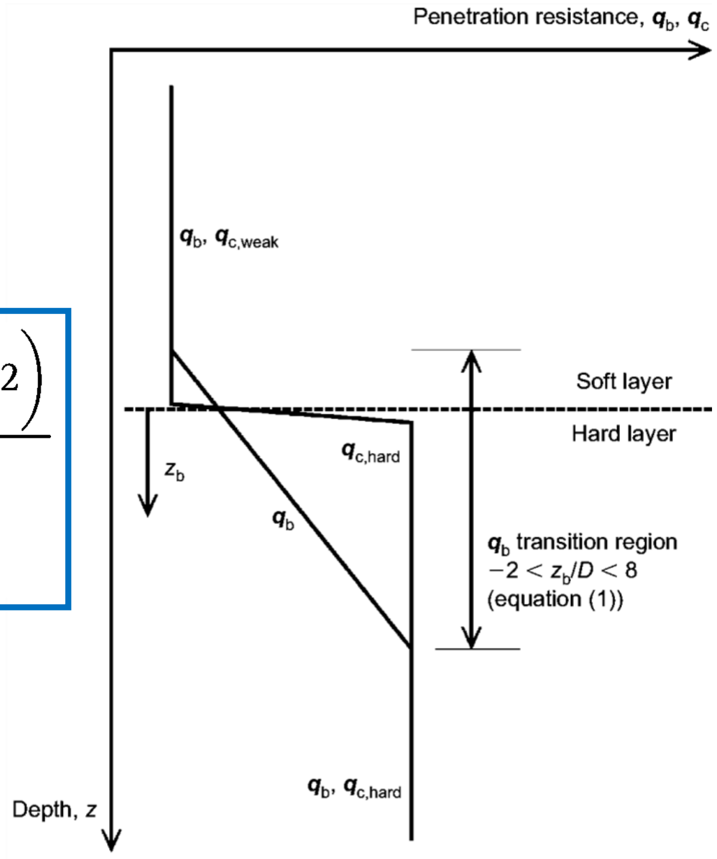
Scale Effect Correlations

White and Bolton (2005):

Partial embedment reduction factor on toe resistance:

$$q_{c,\text{corrected}} = q_{c,\text{weak}} + \frac{(q_{c,\text{hard}} - q_{c,\text{weak}})\left(\frac{z_b}{D} + 2\right)}{10}$$

$$\text{for } -2 < \frac{z_b}{D} < 8$$



Partial embedment reduction factor on base resistance (White & Bolton, 2005)

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/Reference	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)	$r_s = C_s q_c$ $r_s = K f_s$ $C_s = 0.8{\sim}1.8\%$, $K = 0.8{\sim}2(sand)$	$r_t = q_{ca}$
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: $Plugged$: $\frac{r_{t0.1}}{q_{c,tip}} = \frac{0.7}{1+3{D_r}^2}$ $Unplugged$: $r_{t0.1} = r_{t,ann}A_r + r_{t,plug}(1 - A_r)$ $r_{t,ann} = q_{c,tip}$, $r_{t,plug} = \frac{12r_{s,avg}L}{\pi D_i}$, $r_{t0.1} = min(r_{t0.1,plugged}, r_{t0.1,unplugged})$

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

UniCone Method, Eslami & Fellenius (1997)

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

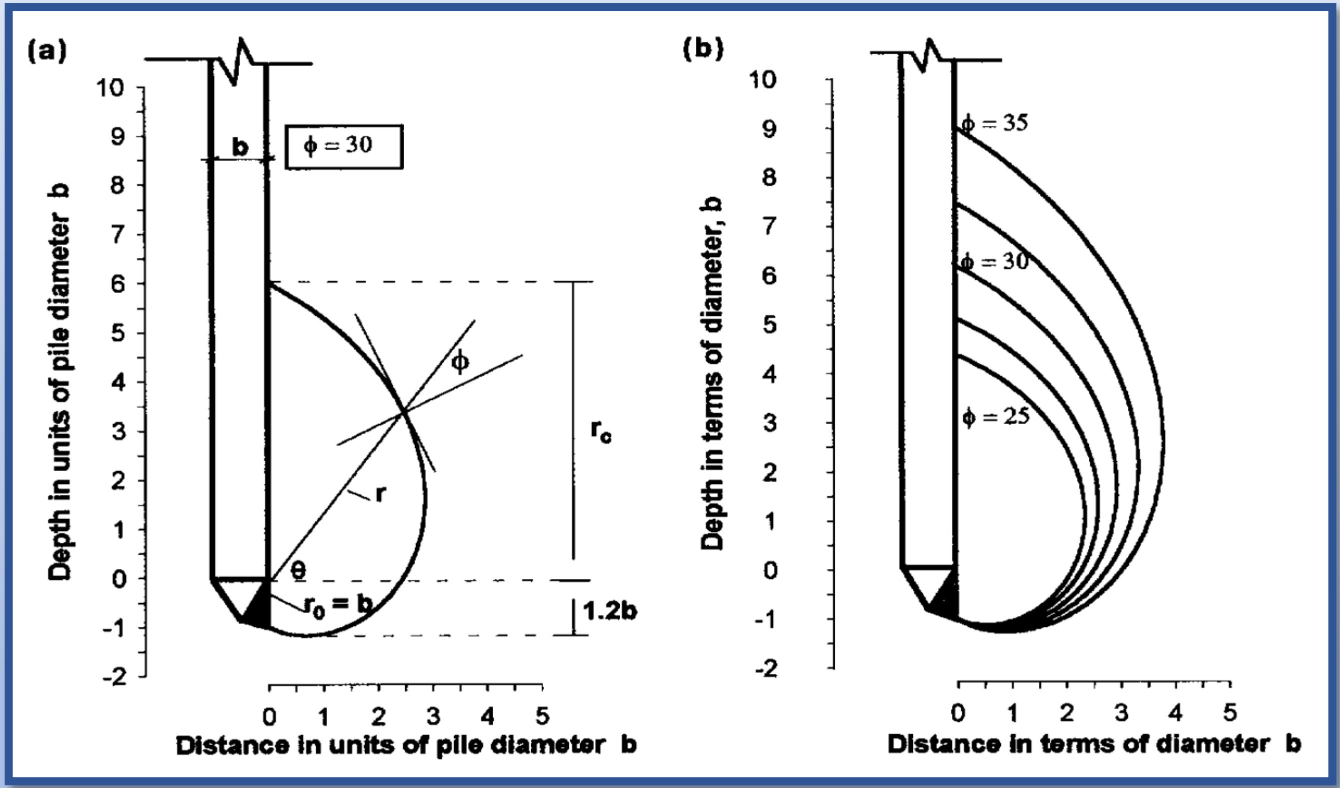
Abolfazl Eslami and Bengt H. Fellenius

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.

UniCone Method, Eslami & Fellenius (1997)

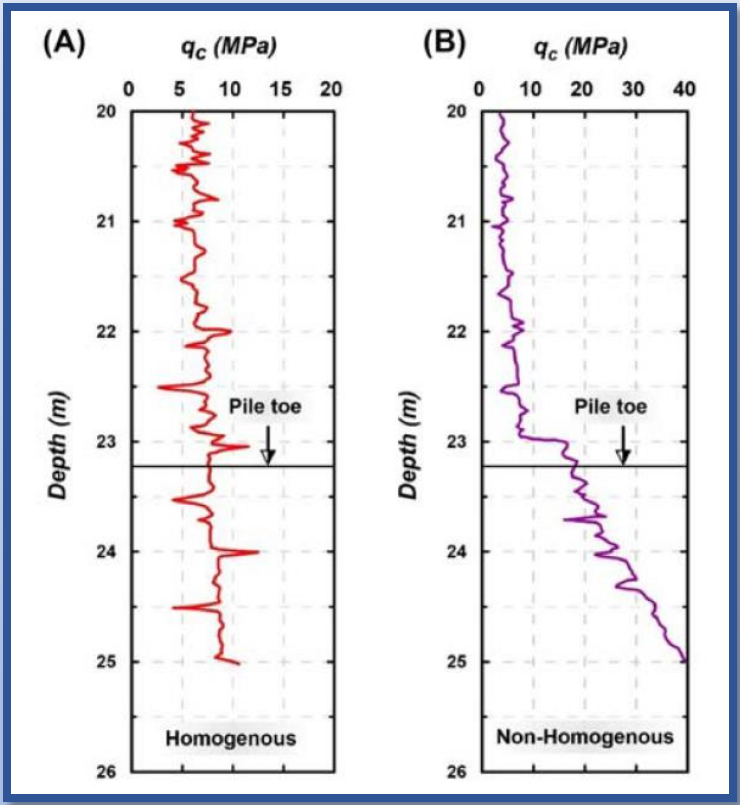
Toe Failure Zone



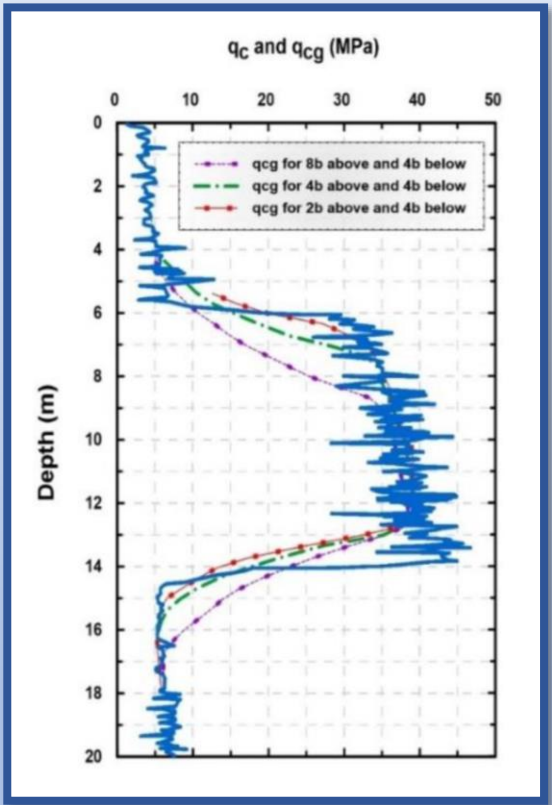
a) Principle of a logarithmic spiral rupture, b) rupture surfaces around pile toe for different soils (Eslami & Fellenius, 1997)

UniCone Method, 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

UniCone Method, Eslami & Fellenius (1997)

Toe Capacity

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

$$c_t = 1$$

$$q_E = q_t - u_2$$

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

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

Fellenius (2002):

Adjustment factor:

$$c_t = \frac{1}{3B} \text{ (B in meter)}$$

$$c_t = \frac{12}{B} \text{ (B in inches)}$$

UniCone Method, Eslami & Fellenius (1997)

Shaft Capacity

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

q_{Eg} = Geometrical – mean within proposed layer

$$r_s = \beta \times \sigma'_{z-avg} \quad \text{Eq. 1}$$

$$r_t = N_t \times \sigma'_{z=D_f} \quad \text{Eq. 2}$$

$$r_t = q_c \quad \text{Eq. 3}$$

Eq. 1 & Eq. 2 → $\frac{r_s}{r_t} = \frac{\beta}{N_t}$ **Eq. 4**

Eq. 3 & Eq. 4 → $r_s = \left(\frac{\beta}{N_t}\right) \times (r_t \text{ or } q_c)$

$$c_s = f\left(\frac{\beta}{N_t}\right)$$

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%

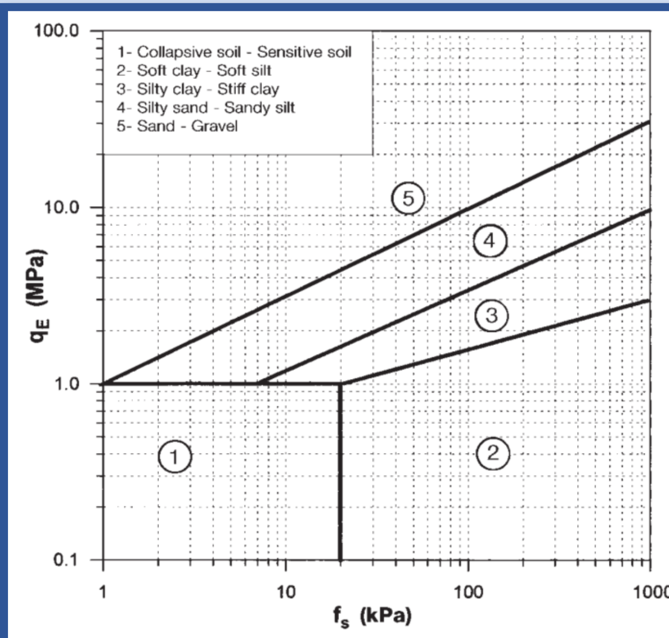


Chart for soil classification (Eslami & Fellenius, 1997)

Comments on the Current CPT-Based Methods for Pile Design

- **Old Technology:** Developed mechanically in the 1970s–1980s before the introduction of the piezocone (CPTu), these approaches do not reflect the accuracy and capabilities of modern electronic CPT measurements.

- **Uncorrected Measurements:** Early formulations did not correct cone resistance for pore pressure effects (u_2), leading to minor discrepancies in sands but significant errors in clays.

- **Total Stress Framework:** Several methods rely on total stress parameters, although long-term pile behavior is governed primarily by effective stress conditions.

- **Data Processing Practices:** Representative soil resistance must be based on geometric averaging instead of adjusted arithmetic means; resulting in variability in interpretation.

- **Locally Derived Databases:** Many methods were calibrated using limited regional pile load tests, without sufficient diversity in soil profiles and pile types.

Comments on the Current CPT-Based Methods for Pile Design

- **Arbitrary Resistance Limits:** Certain approaches impose upper limits on unit toe resistance that may not reflect the higher values recorded in very dense sands.

- **Influence Zone Considerations:** Scale effects between the small-diameter cone and the larger pile are not consistently quantified.

- **Soil Classification Approach:** Most methods lack CPT-based soil typing and instead depend on conventional borings and laboratory classifications that misalign with CPT Data.

- **Capacity Interpretation Criteria:** While some methods define clear benchmarks for interpreting static load tests, others do not explicitly establish ultimate capacity criteria.

- **Limited Soil Applicability:** Methods such as NGI, ICP, Fugro, and UWA primarily address displacement piles in sand, with broadly similar formulations and restricted soil coverage.

These limitations highlight the need for a unified CPT-based framework that integrates corrected measurements, soil behavior typing, and consistent capacity criteria.

1. CPT & CPTu in Geotechnical Engineering
2. Applications in Foundation Engineering
3. CPT-Based Methods for Deep Foundations
4. **Worked Examples**
5. Case Histories
6. Prospects

Example No. 3:
In-Situ Based Methods versus Static Load Test (SLT); Single Pile

A square concrete pile with an embedment depth of 15.2 m and a width of 0.305 m is driven in VA, USA. The variation of CPT and SPT records with depth is illustrated and also tabulated. Predict the axial bearing capacity of this pile according to the following methods:

a) UniCone (Eslami & Fellenius, 1997)

b) Schmertmann (1978)

c) LCPC (Bustamante & Gianesselli, 1983)

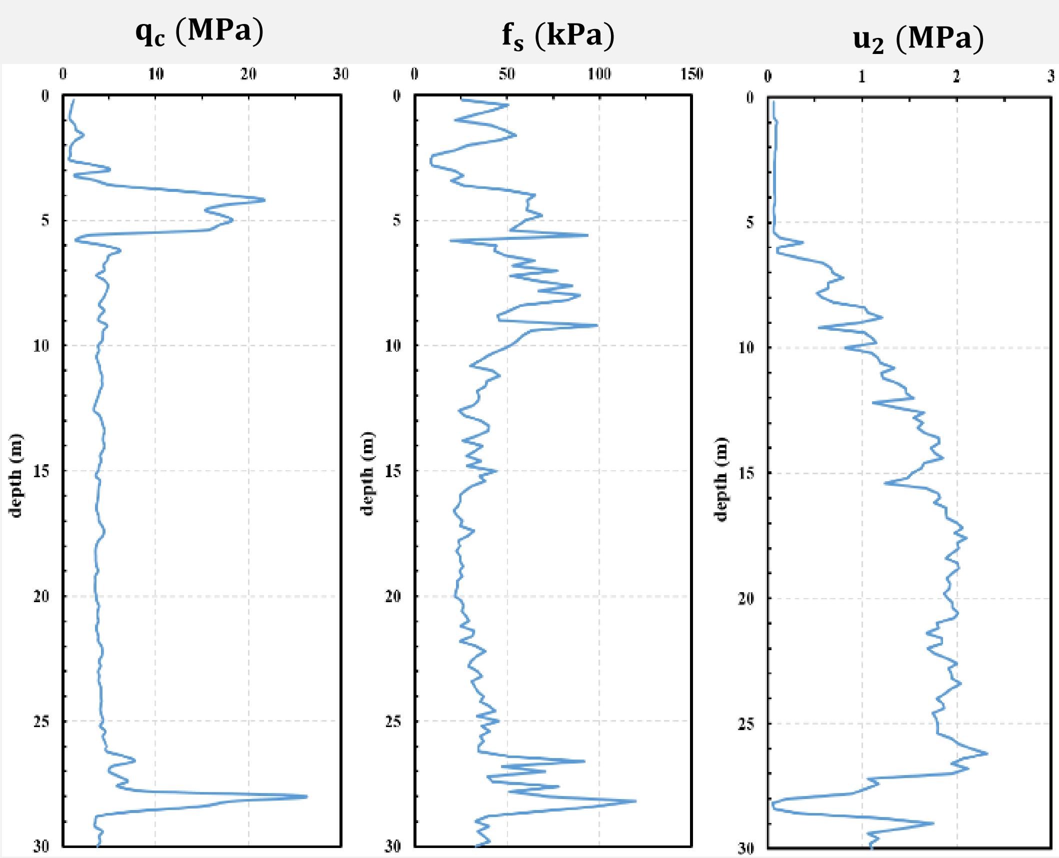
d) Bazaraa & Kurkur (1986)

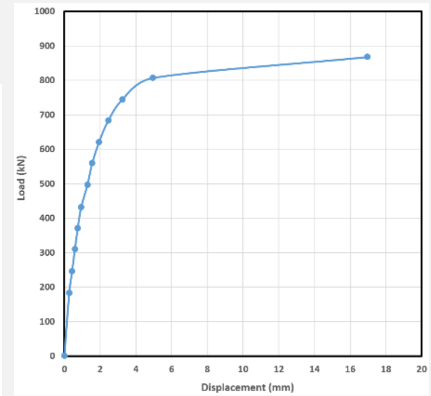
e) Briaud & Tucker (1988)

f) Decourt (1992)

g) Compare the results with the given static pile load test

Example No. 3:
In-Situ Based Methods versus Static Load Test (SLT); Single Pile





Depth (m)	N	Depth (m)	N
1	3	11	9
2	2	12	8
3	6	13	10
4	27	14	8
5	24	15	9
6	10	16	8
7	10	17	10
8	9	18	8
9	10	19	8
10	8	20	9

Example No. 3: Solution

a) Eslami & Fellenius (1997)

$$q_{E-toe} = 2.23\ \mathrm{MPa}\,,\ r_t = 2230\ \mathrm{kPa}\,,$$

$$R_t = 2230 \times 0.305^2 = 207.5\ \mathrm{kN}$$

$$R_s = 707.1\ \mathrm{kN}$$

$$R_u = R_t + R_s = 914.6\ \mathrm{kN}$$

b) Schmertmann (1978)

$$r_t = 3260\ \mathrm{kPa}\,,$$

$$R_t = 3260 \times 0.305^2 = 303\ \mathrm{kN}$$

$$R_s = 710.1\ \mathrm{kN}$$

$$R_u = R_t + R_s = 1013.1\ \mathrm{kN}$$

c) LCPC (Bustamante & Gianesselli, 1983)

$$r_t = 1760\ \mathrm{kPa}$$

$$R_t = 1760 \times 0.305^2 = 163.8\ \mathrm{kN}$$

$$R_s = 776.5\ \mathrm{kN}$$

$$R_u = R_t + R_s = 940.3\ \mathrm{kN}$$

Summary of SPT-Based Methods

	Method	Pile Type	Soil	n_s	n_t	Relevant Zone of End Bearing Capacity	Remark
1	Meyerhof (1956)		Clay	2	22.5		N<=50
2	Aoki & Velloso (1975)	Bored & Driven	Clay Silt Sand	$\frac{\alpha.K}{F_2}$	$100\times\frac{K}{F_1}$	Average of the 3 N-Values Closest to Pile Tip.	For α, K, F_1, F_2 See Fakharian & Eslami (2006)
3	Decourt & Quaresma (1982)		Clay Clayey Silt Sandy Silt Sand	$r_s = 10 + 3.33N_s$	120 200 250 400	[D-1m , D+1m]	3<=N<=50
4	Shioi & Fukui (1982)	Bored	Clay Sand	5 1	$=q_d$ (See Fig. 1)		
		Driven	Clay Sand	10 2	100		
5	Meyerhof (1983)	Bored Driven	Sand	1 2	$120\,.C_1.C_2$ $400\,.C_1.C_2$	[D-8B , D+3B]	Correction of Ns Due to Depth effect. $C_1 = \left(\frac{B+0.5}{2B}\right)^n \le 1$ n for Loose , Medium and Dense Sand is 1, 2 and 3. $C_2 = \frac{L_b}{10B} \le 1$
6	Briaud & Tucker (1984)	Driven	Sand			[D-4B , D+4B]	See Fakharian & Eslami (2006)
7	Bazaraa & Kurkur (1986)	Bored	Cohesive	2	40	[D-3.75B , D+1B]	N<50
			Cohessionless	0.67 (B<=0.5m) 1.34B (B>0.5m)	135 (B<=0.5m) 270 (B>0.5m)		
		Driven	Cohesive	3.3	60		
			Cohessionless	2.2 (B<=0.5m) 4.4B (B>0.5m)	200 (B<=0.5m) 400 (B>0.5m)		
8	Lopes & Laprpvitera (1988)	Bored		Same as 2nd Method	Same as 2nd Method	[D-1B , D+1B]	
9	Resse & O' Neill (1989)	Bored		β – Method	60 (0.52<B<1.27m) $\frac{76.2}{B}$ (B>1.27m)	[D , D+2B]	
10	Hirayama (1990)	Bored	Clay	10	100	[D-1B , D+1B]	For Clay $r_s \le 150$ For Sand $r_s \le 200$
			Silt		250		
			Sand	5	400		
			Clay		600		
11	Robert (1997)	Bored Driven	Sand	1.9 1.9	115 190		Correction of N_s Due to Depth effect.
12	Bouafia & Yaich Achour (2004)	Bored	Sand	4.14	120	[D-8B , D+3B]	

Example No. 3: Solution

d) Bazaraa & Kurkur (1986)

$\mathbf{N_s = 10.2, n_s = 3 \rightarrow r_s = 30.6\ kPa}$

$\mathbf{R_s = 30.6 \times 15.24 \times 4 \times 0.305 = 568.9\ kN}$

$\mathbf{N_b = 8.5, n_b = 0.13 \rightarrow r_t = 1110\ kPa\ and\ R_t = 1110 \times 0.305^2 = 102.8\ kN}$

$\mathbf{R_u = R_t + R_s = 671.7\ kN}$

e) Briaud & Tucker (1988)

$\mathbf{N_b = 8.5, K_t = 1894571.86, r_{t,max} = 4267.35, r_{t,res} = 1667.92}$

$\mathbf{r_t = 4232.16\ kPa, R_t = 4232.16 \times 0.305^2 = 393.7\ kN}$

$\mathbf{N_s = 10.2, K_s = 37441.40, r_{s,max} = 43.93, r_{s,res} = 8.33}$

$\mathbf{r_s = 43.21\ kPa, R_s = 43.21 \times 15.24 \times 4 \times 0.305 = 803.4\ kN}$

$\mathbf{R_u = R_t + R_s = 393.7 + 803.4 = 1197.1\ kN}$

f) Decourt (1992)

$\mathbf{r_t = 1837.5\ kPa, R_t = 1837.5 \times 0.305^2 = 170.9\ kN}$

$\mathbf{r_s = 33.8\ kPa, R_s = 33.8 \times 15.24 \times 4 \times 0.305 = 628.4\ kN}$

$\mathbf{R_u = R_t + R_s = 170.9 + 628.4 = 799.3\ kN}$

Example No. 3: Solution

Capacity	CPT-Based Methods			SPT-Based Methods			Load-Displacement	
	Eslami & Fellenius (1997)	Schmertmann (1978)	LCPC (1983)	Bazaraa & Kurkur (1986)	Briaud & Tucker (1988)	Decourt (1992)	Davisson Offset Limit (1972)	w/d=1 0%
R_t (kN)	207.5	303	163.8	102.8	393.7	170.9	-	-
R_s (kN)	707.1	710.1	776.5	568.9	803.4	628.4	-	-
R_u (kN)	914.6	1013.1	940.3	671.7	1197.1	799.3	840.75	888.2

Example No. 4:
CPT & SPT Based Methods versus Static Load Test (SLT); Single Pile

The records of SPT & CPT tests in a piling site are given below. The installed piles are closed-end steel pipe piles with a diameter of 324 mm and an embedded length of 14 m. The existing soil type is recognized as sand based on site investigation. Determine the axial bearing capacity utilizing the given method and compare the results with the load-displacement diagram.

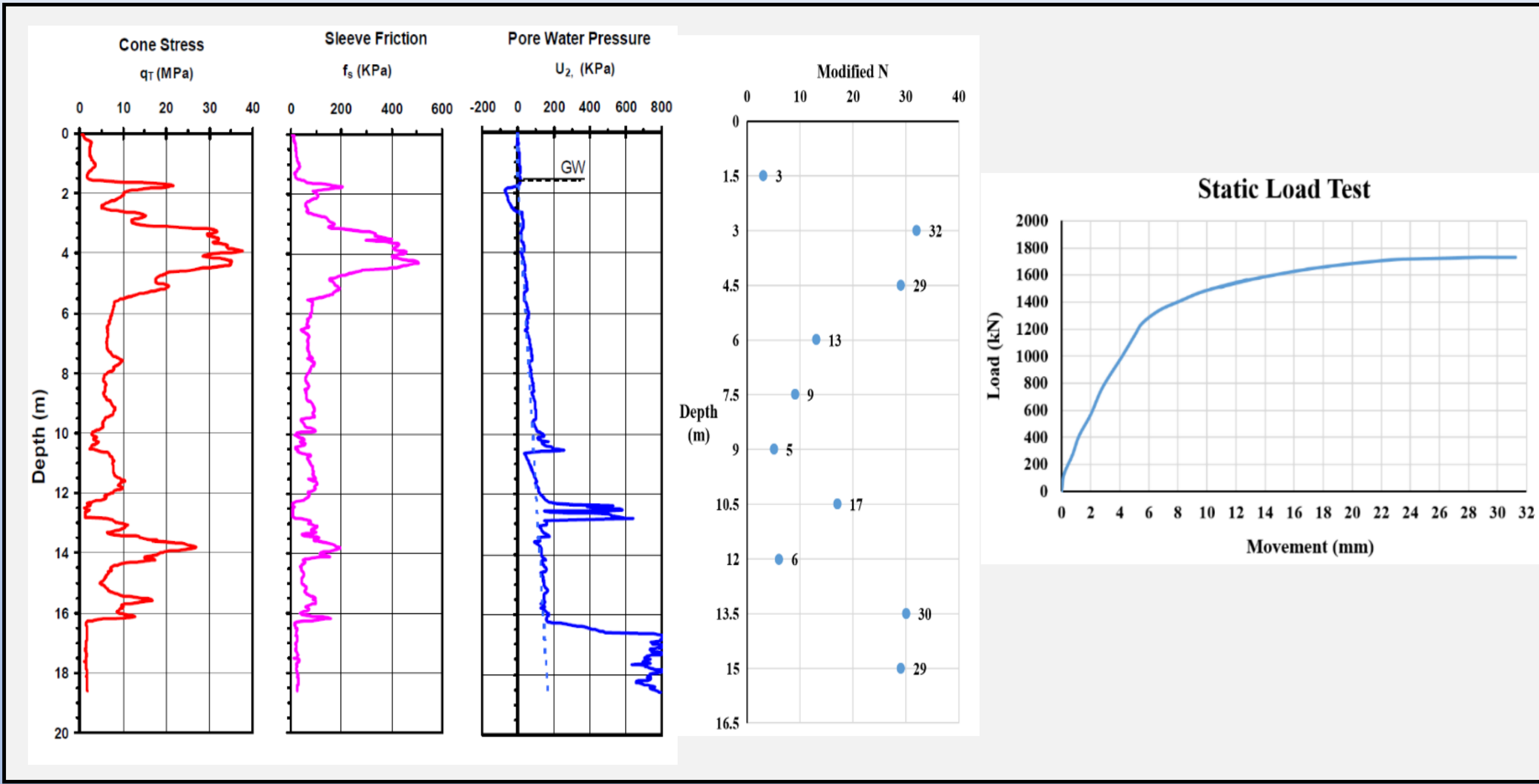
SPT-based:

- ***Meyerhof (1976)***
- ***Shariatmadari et al. (2008)***

CPT-based:

- ***Eslami & Fellenius (1997)***
- ***German (2010)***

Example No. 4:
CPT & SPT Based Methods versus Static Load Test (SLT); Single Pile



Example No. 4: Solution

SPT-based Methods

Meyerhof (1976)

$\mathbf{N}_{avg-\text{shaft}} = \mathbf{16}$

$\mathbf{n_s = 2 \rightarrow r_s = 2 \times 16 = 32\ KPa} \quad \mathbf{A_s = 14.25} \quad \mathbf{R_s = 456\ KN}$

$\mathbf{n_t = 400\ C_1\ C_2} \quad \mathbf{, C_1 = 1} \quad , \quad \mathbf{C_2 = 1}$

$, \quad \mathbf{n_t = 400} \quad \mathbf{[11.4, 14.97] \rightarrow N_{avg} = 21.67}$

$\mathbf{r_t = 400 \times 21.67 = 8668} \quad \mathbf{A_t = 0.082\ m^2} \quad , \quad \mathbf{R_t = 8668 \times 0.082 = 710.77}$

$\mathbf{R_u = 1166.77\ KN}$

Shariatmadari et al. (2008)

$\mathbf{N}_{avg-shaft-\text{geometrical}} = \mathbf{12}$

$\mathbf{n_s = 3.65 \rightarrow r_s = 3.65 \times 12 = 43.8\ KPa} \quad \mathbf{A_s = 14.25} \quad \mathbf{R_s = 624.15\ KN}$

$\mathbf{n_t = 385} \quad \mathbf{[11.4, 15.3] \rightarrow N_{avg} = 17.35}$

$\mathbf{r_t = 385 \times 17.35 = 6680} \quad \mathbf{A_t = 0.082\ m^2} \quad , \quad \mathbf{R_t = 6680 \times 0.082 = 547.76}$

$\mathbf{R_u = 1171.91\ KN}$

Example No. 4: Solution

CPT-based Methods

German (2010)

$q_{c-shaft} \rightarrow 12.5$ $r_s = 80\ kPa$

$q_{c-toe} = 17\ MPa$ $r_t = 9\ MPa$

$R_s = 80 \times 14.25 = 1140\ kN$ $R_t = 9000 \times 0.082 = 738\ kN \rightarrow R_u = 1878\ kN$

Eslami & Fellenius (1997)

$q_{E-shaft} = 9.11\ MPa$ $q_{E-toe} = 8.98\ MPa$

$r_s = 0.01 \times 9110 = 91.1\ KPa$ $r_t = 8980$

$R_s = 91.1 \times 14.25 = 1298.18\ kN$ $R_t = 8980 \times 0.082 = 736.36\ kN \rightarrow R_u = 2034.54\ kN$

Load-Displacement Diagram

Davisson Offset Limit $(1972) \rightarrow$ $offset = 4 + \frac{344}{120} = 6.7\ mm$ $K = 90$ $\rightarrow Q_u = 1725\ kN$

Brinch – Hansen (1963) $Q_u = 1767.77\ kN,$ $C_1 = 0.00005,$ $C_2 = 0.00016$

1. CPT & CPTu in Geotechnical Engineering
2. Applications in Foundation Engineering
3. CPT-Based Methods for Deep Foundations
4. Worked Examples
5. Case Histories
6. Prospects

I. Supplementary Cases

7. Four Piles Prediction Symposium; ASCE (Finno et al., 1989)

8. The Hong Kong Bridge (Duan et al., 2021)

9. AUT:Geo-CPT&Pile Database (Eslami et al., 2016 - 2024)

10. Three oil tanks -Belgium (Van Impe et al., 2013 & 2015)

11. James River bridge, Virginia (Batten & Keaney, 2021)

12. Pile Prediction Event, Alabama (Fellenius, 2025)

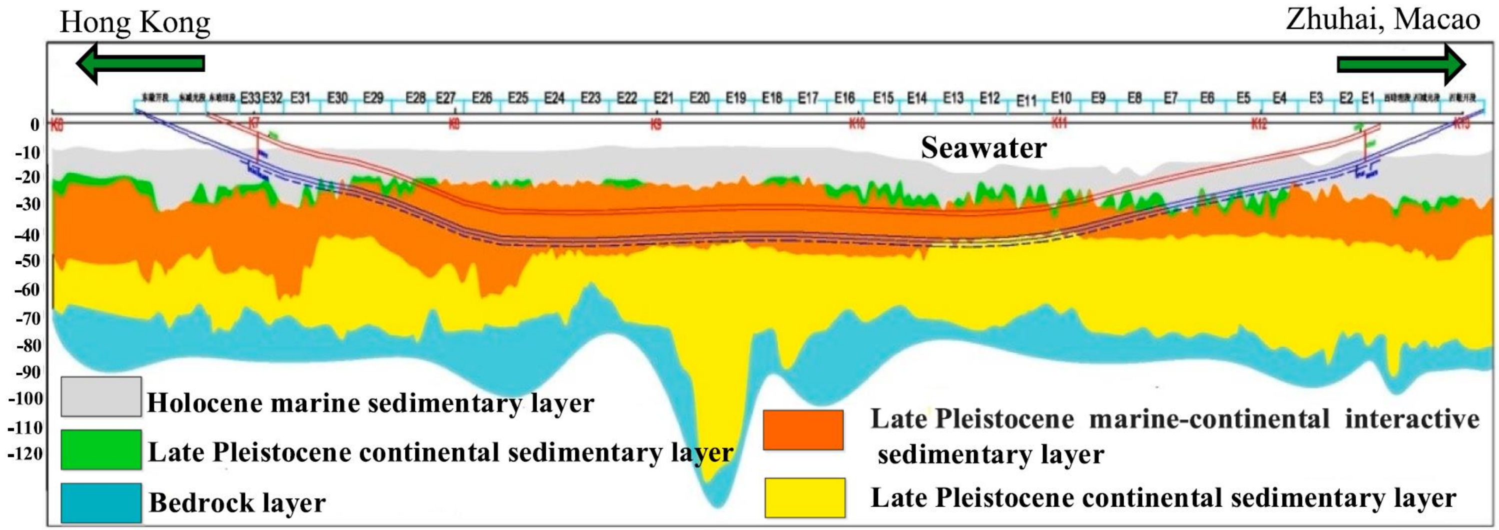
13. Pile Instrumentation, Louisiana (Abu-Farsakh et al., 2016)

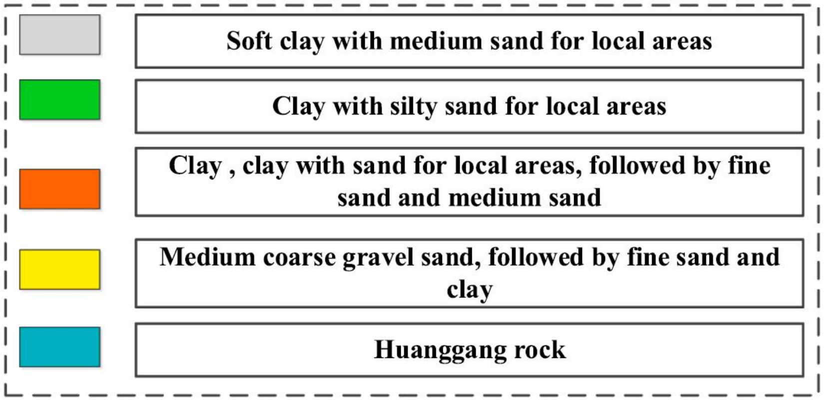
Case History No. 8: The Hong Kong Bridge (Duan et al., 2021)

- **Soft marine clay located on the bedrock**
- **Project Cost: Over $12 billion**
- **The Longest bridge-Tunnel Combo (50 km)**
- **The max span = 180m**
- **700 drilled shafts**
- **660 of piles in the marine environment**
- **The piles length: 7 m to 107 m**
- **Average diameter of 2.5 m**
- **About 400 CPTu & geotechnical data**

Geographical location of HZMB project (Duan et al., 2021)

Case Study No. 8: Continued

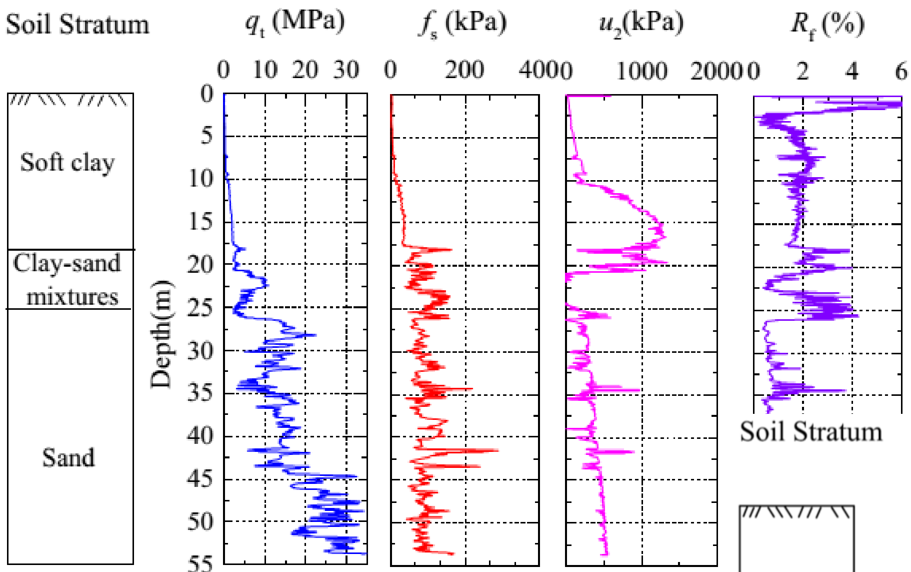





Offshore hydraulic lifting drilling platform

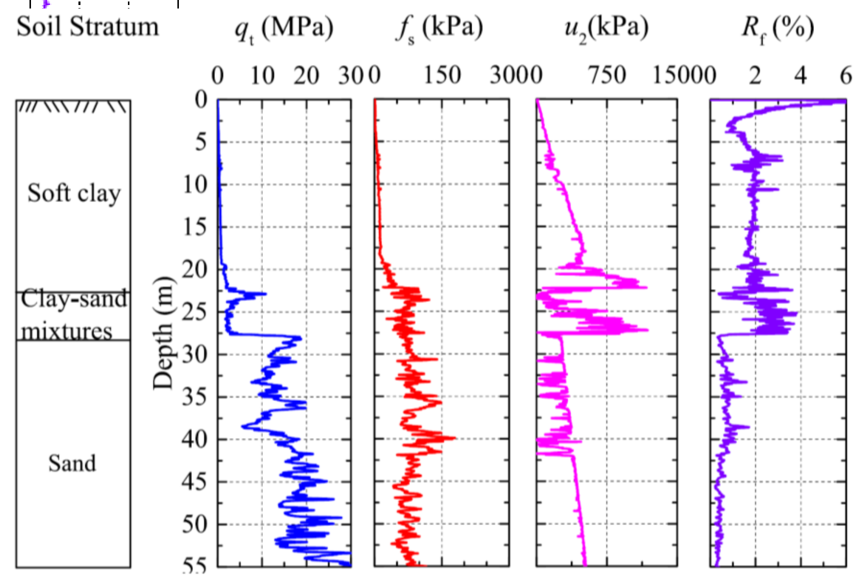
Simplified comprehensive geological longitudinal profile of island and tunnel (Duan et al., 2021)

Case Study No. 8: Continued



Typical CPT records of HZMB project
(Duan et al., 2021)

Offshore piezocone testing in this study
(Duan et al., 2021)

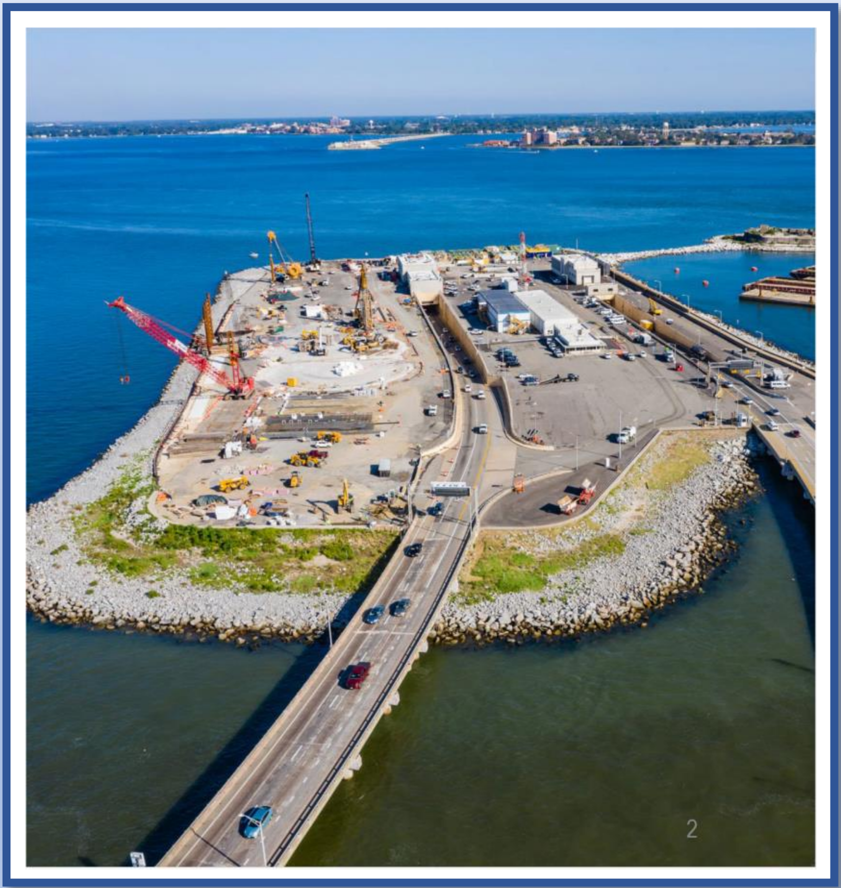


Case History No. 11: James River bridge, Virginia (Batten & Keaney, 2021)

Optimization Large Diameter Concrete Piles for Marine Bridges

Project Overview

- $3.8 Billion Design-Build project, awarded by VDOT to HRCP Construction Joint Venture in 2019
- Design Joint Venture of HDR and Mott MacDonald
- 2.7 miles marine trestle bridges over the James River on 1-64
- Connect Hampton to Norfolk, VA
- North and South Trestles connect shoreline to man made island expansions



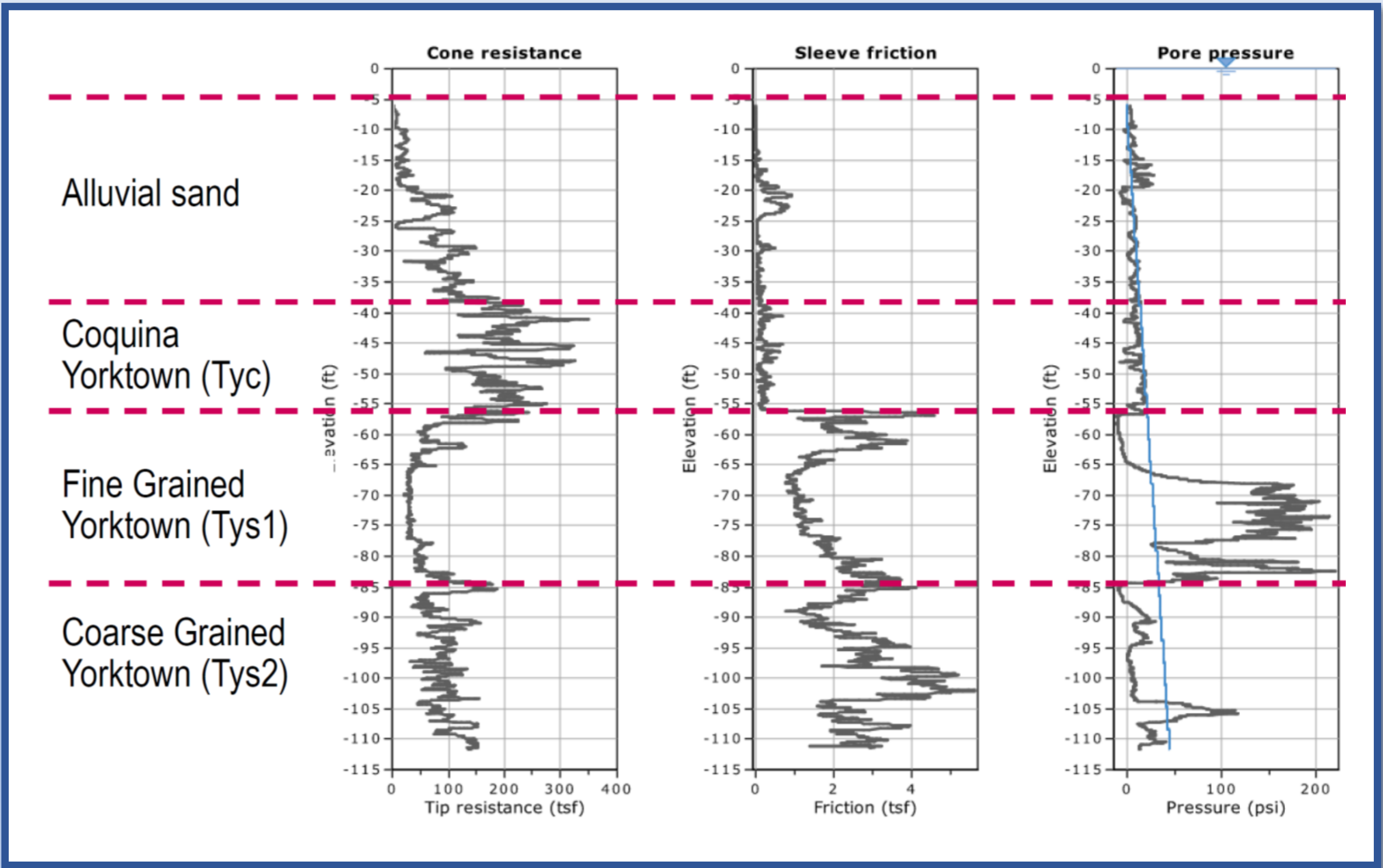
Case History No. 11: Continued

Subsurface Exploration

- Historic VDOT Marine Trestle Data: 17 SPT and 35 CPTu
- Supplemental Marine Trestle Data: 135 SPT and 149 CPTu
    - *Jack up barges for deep water*
    - *Spud barges for shallow water*

Case History No. 11: Continued

Typical CPT Sounding



Case History No. 11: Continued

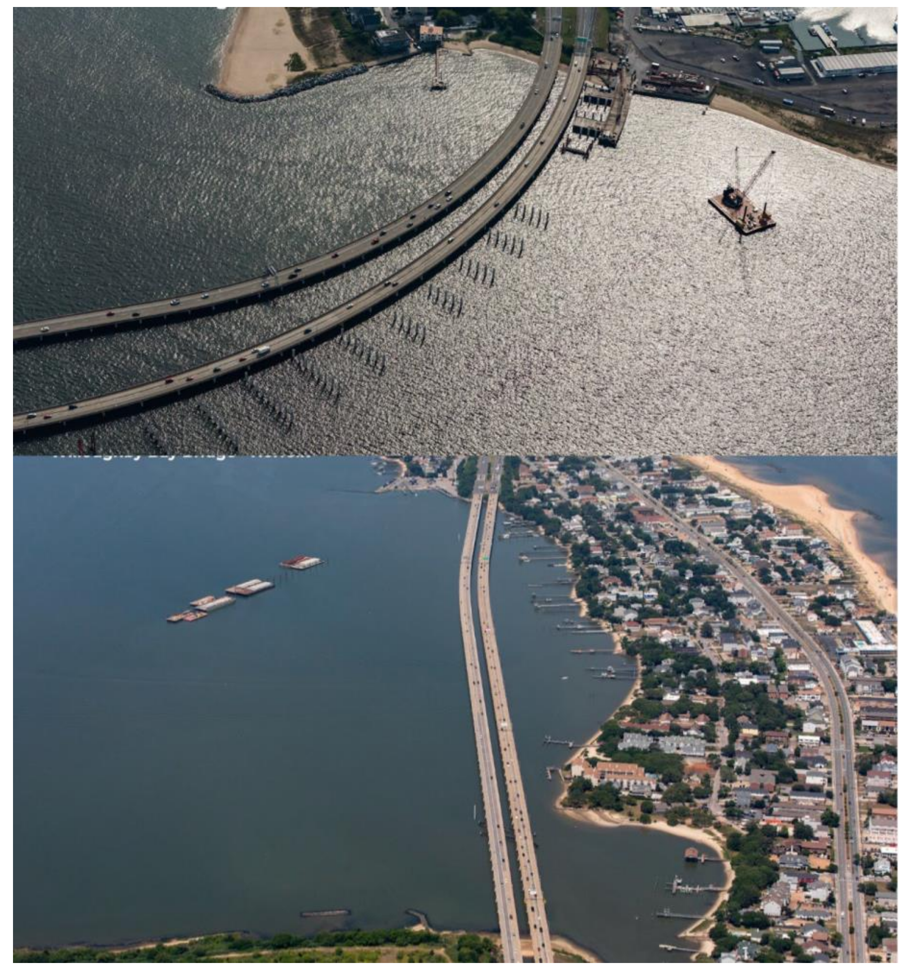
Pile Types

- **North & South Trestles:**

***54-inch** precast prestressed concrete **cylinder piles**, 6.75-inch wall thickness, bed cast, NOT spun cast*

- **Willoughby Bay Trestle:**

24-inch** precast prestressed concrete **square piles



Case History No. 11: Continued

Pile Load Tests

- **North Trestle**

  2 Dynamic Load Test Piles

  1 Static Load Test Pile

- **South Trestle**

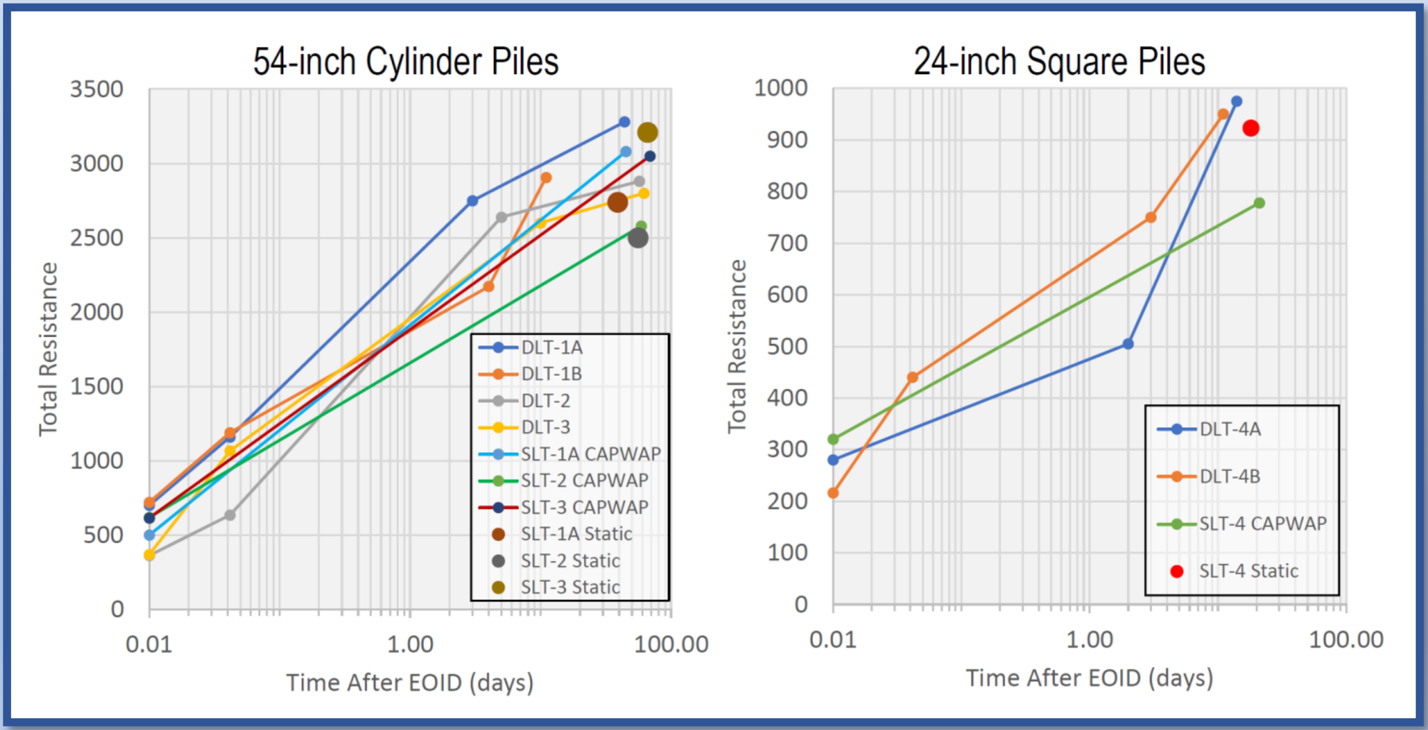
  2 Dynamic Load Test Piles

  2 Static Load Test Piles

- **Willoughby Bay Bridge.**

  2 Dynamic Load Test Piles

  1 Static Load Test Pile



Project-Based Calibration of Parameters

Case History No. 11: Continued

Soil Type	Soil Description	Original C_s (Fellenius 2021)	Calibrated Against HRBT Static Load Tests: C_s	Calibrated Against HRBT Static Load Tests: Limiting q_{skin} (ksf)
1	Soft sensitive soils	0.08	0.150	None
2	Clay	0.05	0.066	1.5
3	Silty clay, stiff clay and silt	0.025	0.066	3.0
4a	Sandy silt and silt	0.015	0.030	3.0
4b	Fine sand or silty sand	0.010	0.017	3.0
5	Sand to sandy gravel	0.004	0.015	2.1

Pile ID	Calculated Axial Compression Total Resistance[1]				Max Static Axial Test Load		
	Nordlund (1963; 1979) / Tomlinson (1987)	Martin et al (1987)[2]	Original Eslami and Fellenius (1997)[3]	Calibrated Eslami and Fellenius (1997)	Total Load (kips)	Skin Friction (kips)	Toe Resistance (kips)
SLT-1	2,954	1,895	2,495	2,642	2,738	2,117	621
SLT-2	3,025	1,624	2,113	2,495	2,491	2,097	394
SLT-3	2,961	2,650	2,276	2,954	3,209	2,607	602
SLT-4	1,445	834	962	884	875	663	212

1 All axial resistance methods assumed an unplugged toe area for the open ended 54-inch cylinder piles.
2 The Martin et al (1987) method was only applied to pile embedment within the Yorktown strata. Conventional effective stress beta analysis was used to calculate unit skin friction for the Quaternary soils above the Yorktown.
3 The Original Eslami and Fellenius (1997) method calculations used a C_t coefficient of 1.0 for both pile sizes.

1. CPT & CPTu in Geotechnical Engineering
2. Applications in Foundation Engineering
3. CPT-Based Methods for Deep Foundations
4. Worked Examples
5. Case Histories
6. **Prospects**

Topic Subtitles

I. AUT: Geo-Marine Database (AUT: GMD)

II. Relevant Data-Based Design (RDBD)

III. Piling in Marine Challenging Conditions

IV. Capacity Loss Under Dynamic Conditions

V. Load-Displacement (Stress-Strain) Approach: FELADD

VI. AI & ML Application for Helical Piles

VII. Summary & Conclusions

Piling in Marine Challenging Conditions (Ebrahimipour & Eslami, 2024)

Contents lists available at ScienceDirect

Ocean Engineering

journal homepage: www.elsevier.com/locate/oceaneng

ELSEVIER

Research paper

Analytical study of piles behavior for marine challenging substructures

Amirhossein Ebrahimipour [a], Abolfazl Eslami [a,b,*]

[a] *Civil and Environmental Engineering Department, Amirkabir University of Technology, Tehran, Iran*
[b] *Structural Engineering Department, University of California San Diego, USA*

A R T I C L E I N F O

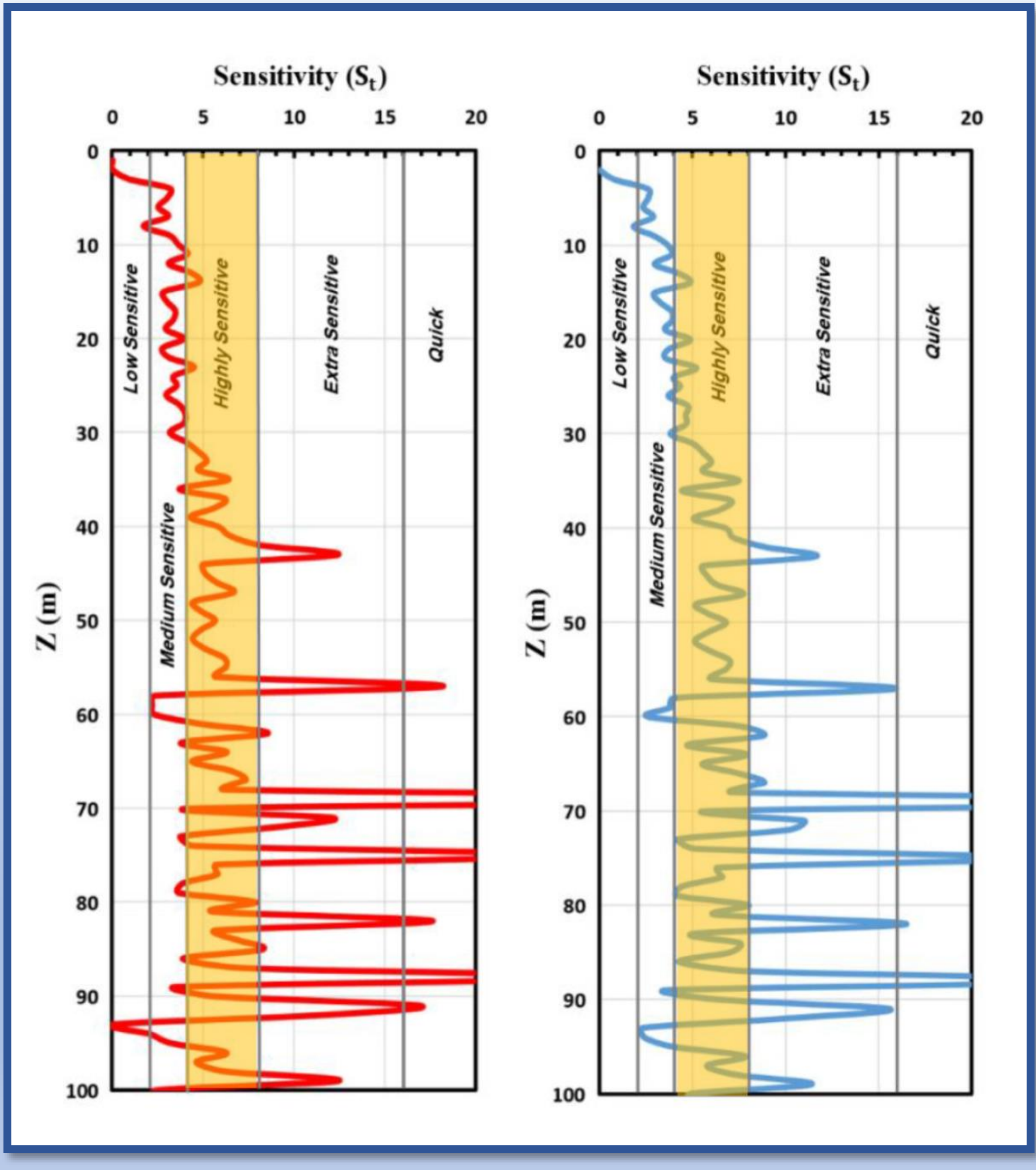
Handling Editor: Prof. A.I. Incecik

Keywords:
Analytical study
Pile behavior
Challenging deposits
CPT & CPTu
Neutral plane
Downdrag force

A B S T R A C T

Based on a new categorization, significancy and feasibility of vector-acting foundations for offshore challenging substructures are emphasized. A marine challenging status is typically related to loading, investigations, subsurface, environment, construction and sustainability requirements. The issue of the interaction of piles and complex subsoil is discussed in this paper. Three scenarios are studied comprising sensitive deposits at the full depth, the most impressive one, compressible deposits at the top layer and soft soil underlain by strong deposits. The role of CPTu records is addressed in recognition of problematic layers, geotechnical design and embedment depth designation. The effect of soil sensitivity on shaft and toe resistance is quantified through static analysis and reduction factors are suggested. Since bearing capacity, settlement and axial load are interactive, the neutral plane concept is applied, unifying the mentioned aspects. Through the neutral plane approach and considering the downdrag force due to the collapse of sensitive soil, primary design criteria incorporating sensitivity are discussed and quantified. Applying ground modification as a supplement for deep foundations is realized through two case studies, leading to enhancement in geotechnical performance. Overall, the mentioned phenomena must be discovered comprehensively in extraordinary conditions, aiming for a safe, reliable, knowledge-based and optimum design.

Piling in Marine Challenging Conditions (Ebrahimipour & Eslami, 2024)



Sensitivity profiles of Urmia Lake deposit via two approaches (Ebrahimipour & Eslami, 2024)

Piling in Marine Challenging Conditions (Ebrahimipour & Eslami, 2024)

$$\Psi = \frac{S_u}{\sigma'_v} \qquad S_t = \frac{S_{u-\text{undisturbed}}}{S_{u-\text{remolded}}}$$

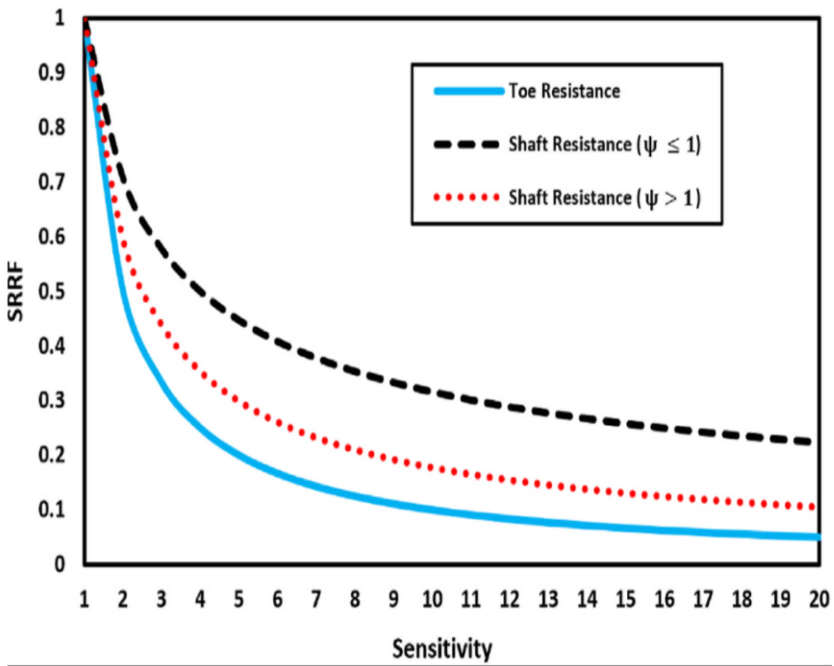
Shaft Resistance:

$$\text{For } \Psi \leq 1 \rightarrow \frac{r_{s-\text{undisturbed}}}{r_{s-\text{disturbed}}} = S_t^{0.5}$$

$$\text{For } \Psi > 1 \rightarrow \frac{r_{s-\text{undisturbed}}}{r_{s-\text{disturbed}}} = S_t^{0.75}$$

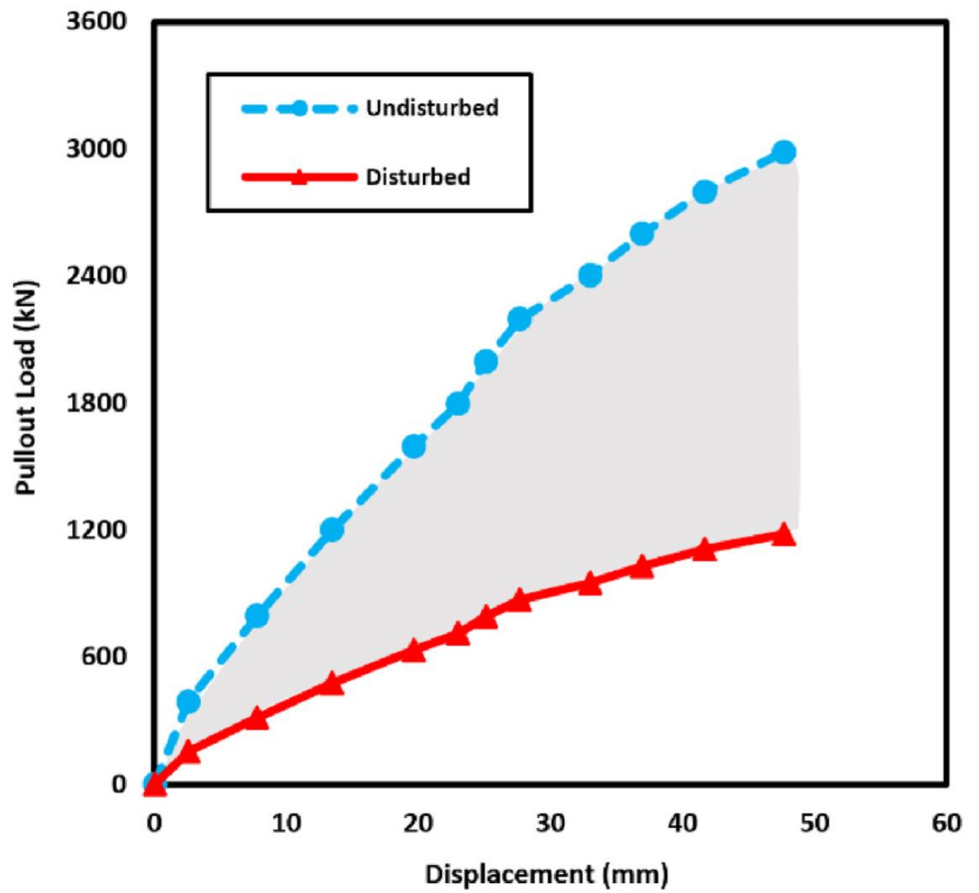
Toe Resistance:

$$\frac{r_{t-\text{undisturbed}}}{r_{t-\text{disturbed}}} = S_t$$



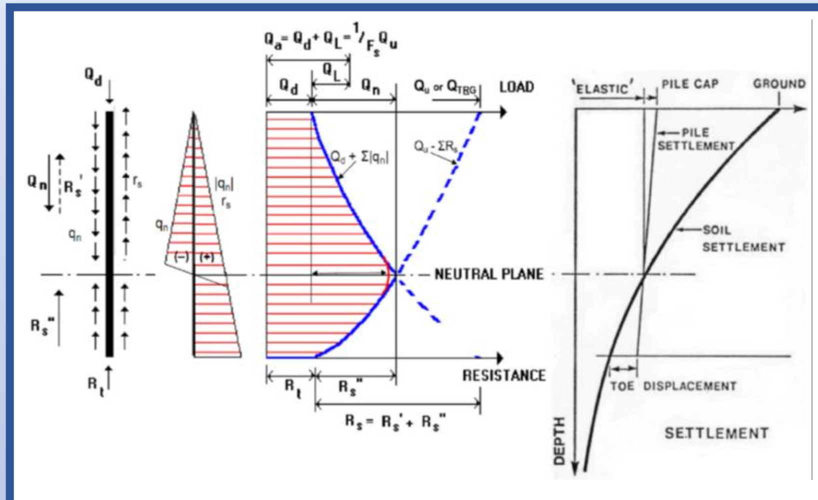
Sensitivity-based reduction factors (Ebrahimipour & Eslami, 2024)

Piling in Marine Challenging Conditions (Ebrahimipour & Eslami, 2024)

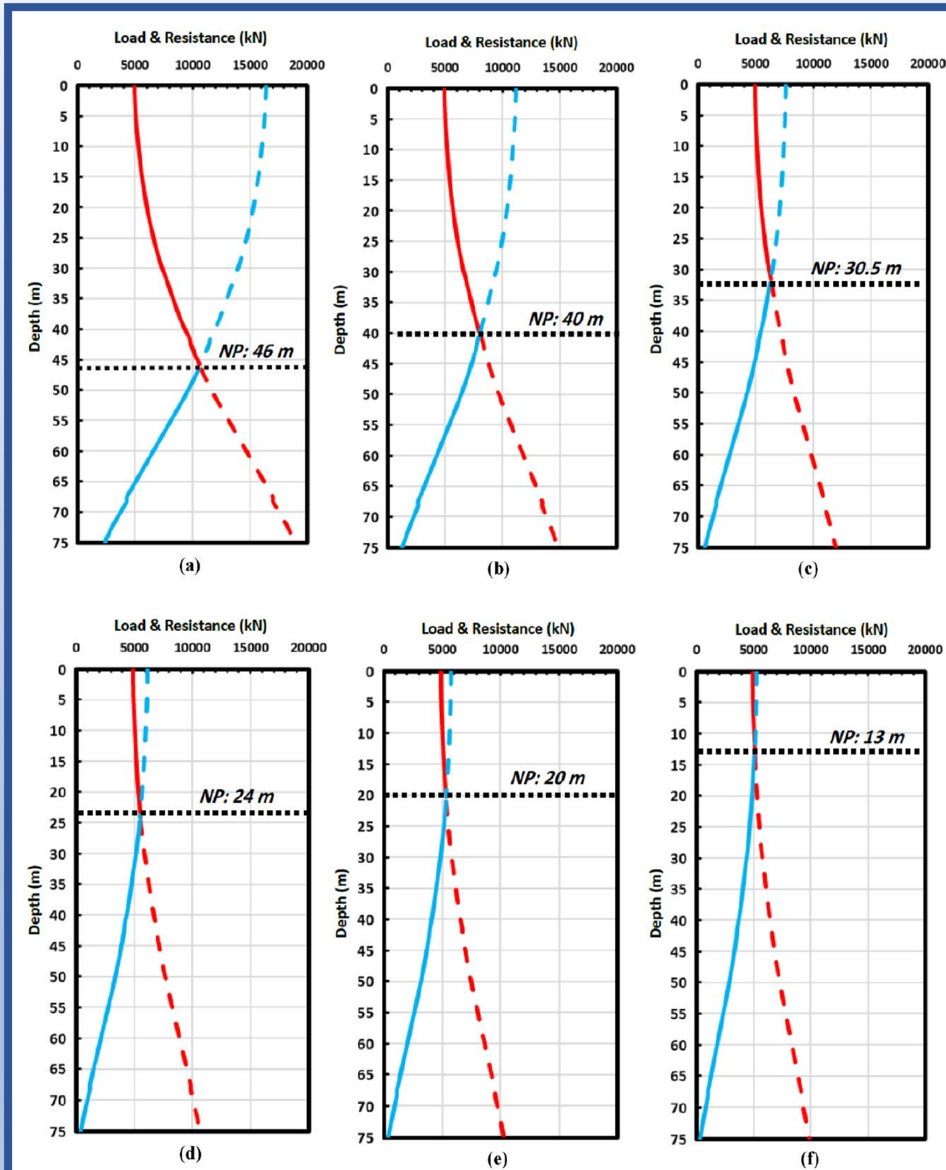


Pullout load-displacement of 70 m pile in disturbed and undisturbed condition (Ebrahimipour & Eslami, 2024)

Piling in Marine Challenging Conditions (Ebrahimipour & Eslami, 2024)



Neutral plane concept (Fellenius, 2023)



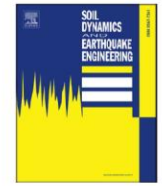
Variation of load, resistance & neutral plane depth through altering the sensitivity for a typical pipe pile in the Urmia lake: a) reference condition, b) St = 2, c) St = 4, d) St = 6, e) St = 6.75, f) St = 8 (Ebrahimipour & Eslami, 2024)

Capacity Loss Under Dynamic Conditions (Eslami et al., 2025)

Contents lists available at ScienceDirect

Soil Dynamics and Earthquake Engineering

journal homepage: www.elsevier.com/locate/soildyn



Pore water pressure generation and sensitivity aspects for pile dynamics and capacity loss: CPTu records and case studies

Abolfazl Eslami *, Dorsa Shadlou , Amirhossein Ebrahimipour

Civil and Environmental Engineering Department, Amirkabir University of Technology, Tehran, Iran

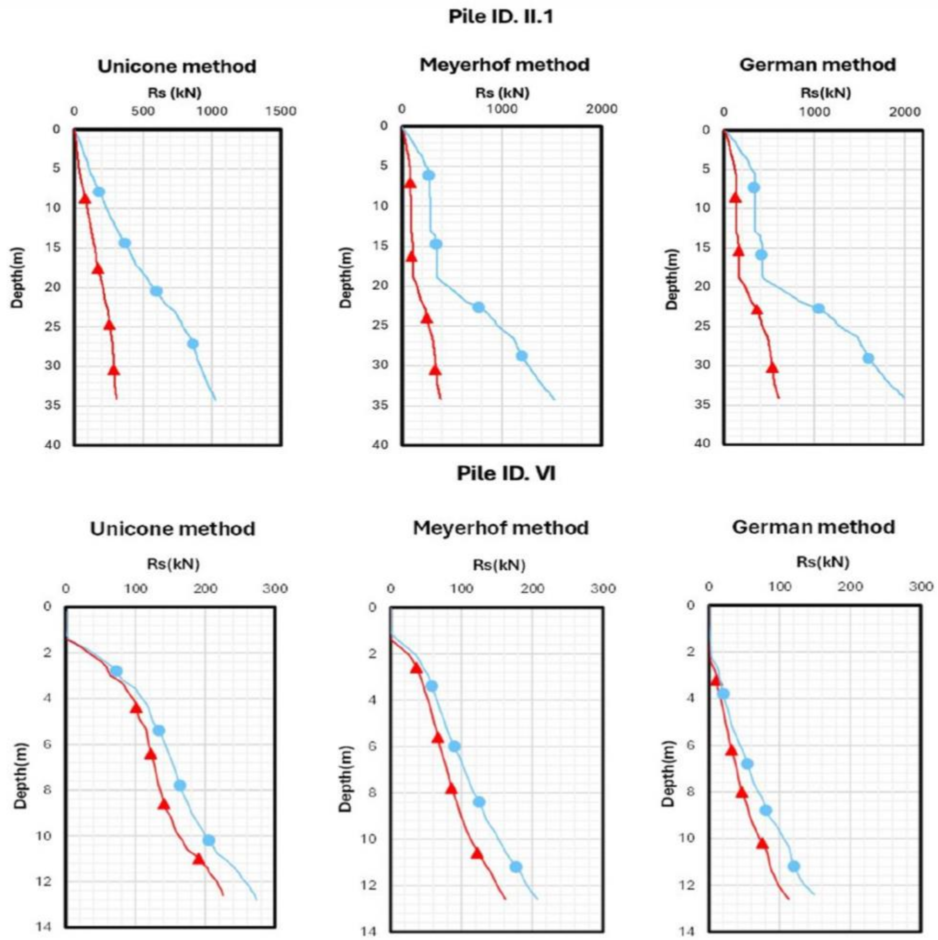
A R T I C L E I N F O

Keywords:
Pore water pressure
Sensitivity
Pile dynamics
Capacity loss
CPTu
Case studies

A B S T R A C T

Soil disturbance and excess pore water pressure generation, induced by dynamics and transient excitations such as pile driving, seismic loading, and impact effects, can significantly degrade the geotechnical strength and stiffness. Given the critical importance of axial capacity in sustaining superstructures, it is essential to recognize and mitigate potential damages. This research investigates the piles reduction of axial capacity through CPTu records, which offer rapid and reliable data. Aiming to quantify the consequences of soil sensitivity and excess pore water pressure, a comprehensive dataset has been compiled comprising CPTu and pile performance records from 11 diverse sites worldwide, focusing on soft and loose deposits. The research identifies problematic sub-layers and, by incorporating an analytical approach, evaluates the intact and reduced shaft and toe resistance through three distinct methods. Results indicate a substantial reduction in bearing capacity due to dynamic loading on piles. Four levels of capacity loss concern are recognized quantitively. Through case studies, the response of the problematic deposit under dynamic loading is more apprehended, conforming with the findings. The current research addresses and emphasizes the necessity of realizing pile dynamics and problematic deposit interaction. It can lead to safe, reliable, and optimal design practices based on a comprehensive understanding of soil-pile interaction.

Capacity Loss Under Dynamic Conditions (Eslami et al., 2025)



$$\Psi = \frac{S_u}{\sigma'_v} \qquad r_u = \frac{\Delta u}{\sigma'_v}$$

Shaft Resistance:

For $\psi \le 1 \Rightarrow \mathrm{SRRF} = \frac{1}{St^{0.5}}$

For $\psi > 1 \Rightarrow \mathrm{SRRF} = \frac{1}{St^{0.75}}$

Toe Resistance:

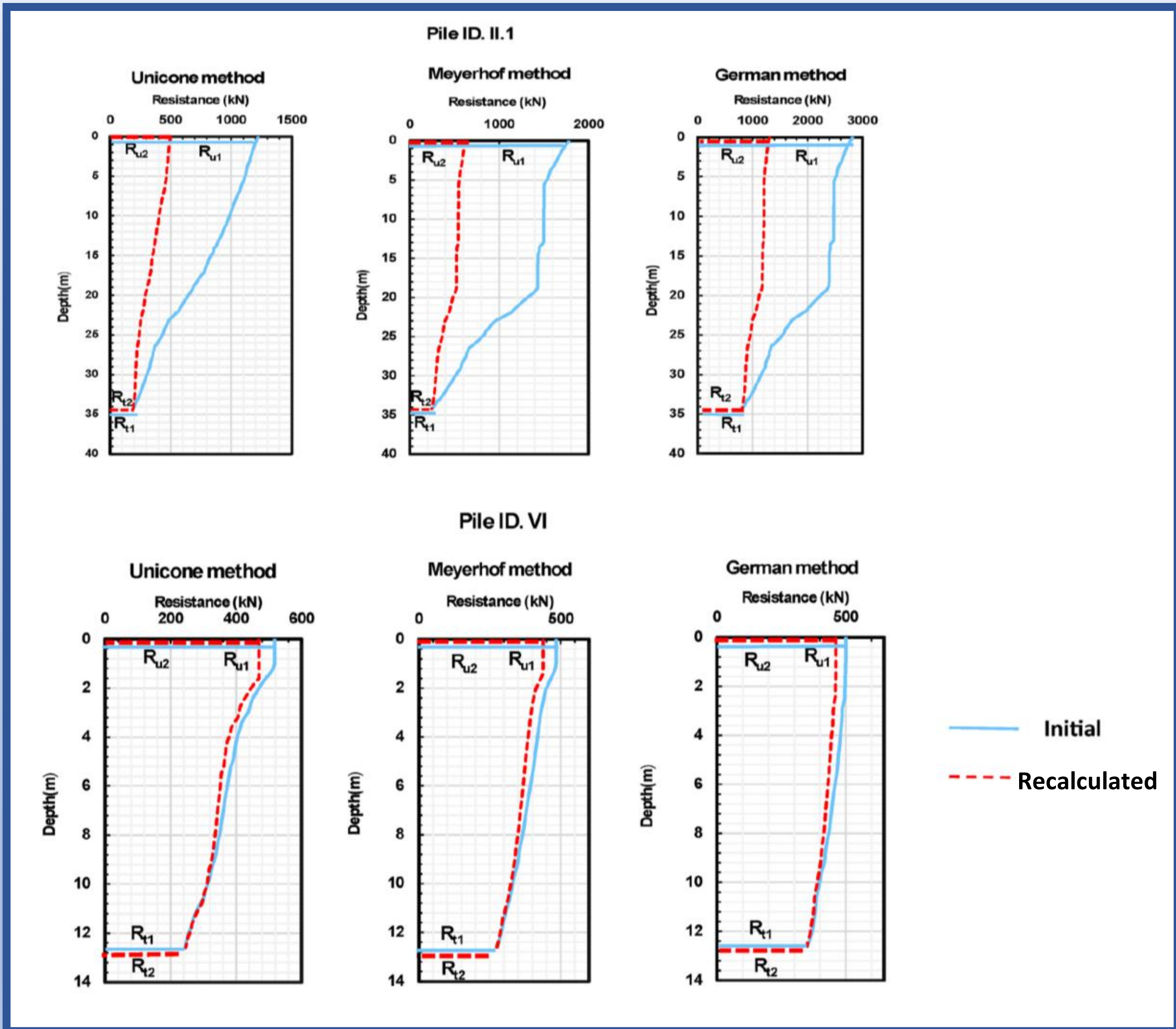
$$\frac{q_E}{q_t} = \frac{q_t - u_2}{q_t} = 1 - \frac{u_2}{q_t}$$

Initial

Recalculated

Shaft distribution before and after revision based on SRRF

Capacity Loss Under Dynamic Conditions (Eslami et al., 2025)



Total resistance before and after revision in pile cases II.1, VI

FELADD (Eslami et al., 2023 - Ongoing)

Foundation Engineering Load And Displacement Database

Foundation Categories for Database Arrangement

Categories		Foundation Types		Acronym
A	Physical Modeling	I	Model-Scale	MS
		II	Plate Load Test (PLT)	PLT
B	Shallow & Semi-deep Foundation	III	Spread Footings	SF
		IV	Semi-Deep	SD
C	Deep Foundations	V	Driven Piles	DP
		VI	Drilled Shafts	DS
		VII	Rock Socketed	RS
D	Special Deep Foundations	VIII	Micro Piles	MP
		IX	Helical & Expanded Piles	HP & ExP
		X	Drilled Displacement Piles	DD
E	Block & Massive Foundations	XI	Pile Groups & Piled Raft Foundations	PG & PRF
F	Rigid Intrusions	XII	Stone Columns, Deep Mixing & Jet Grouting	RISC, RIDM or IRJG

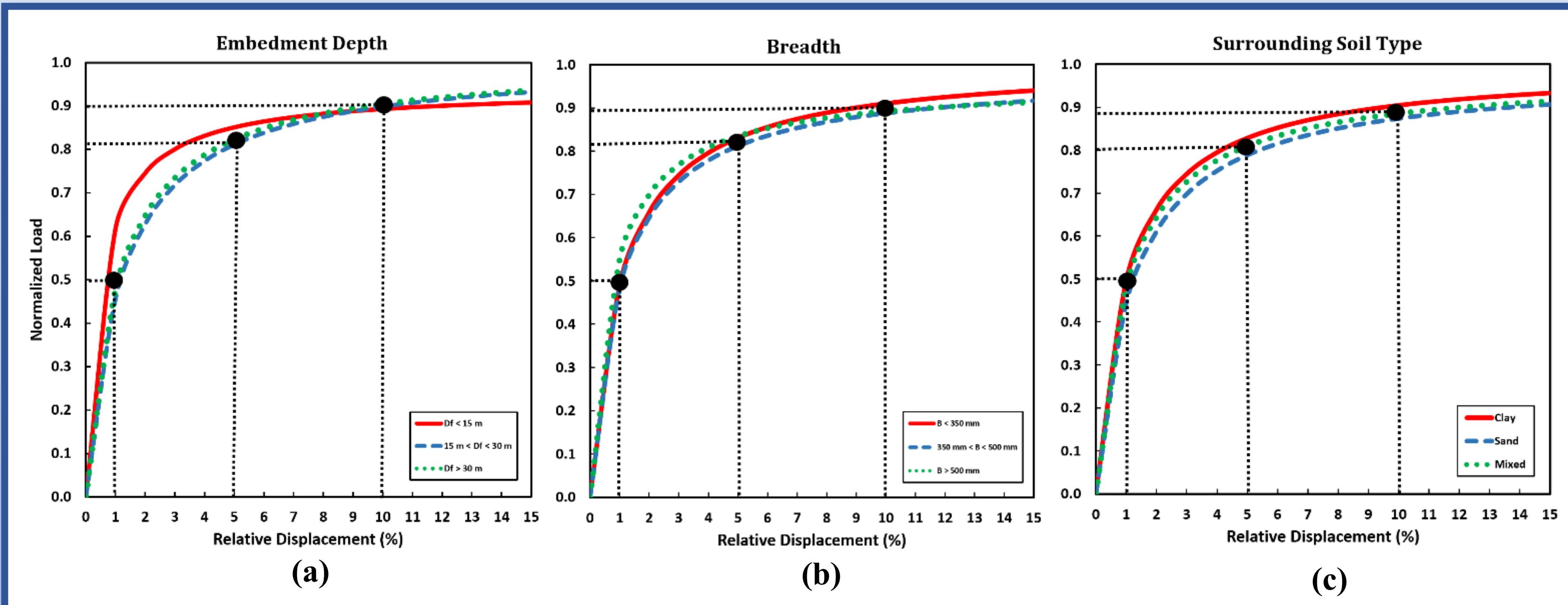
Driven Piles

Normalization Approach:

- Load: Brinch-Hansen 80% (1963)
- Displacement: Breadth

Relative Displacement & Normalized Load:

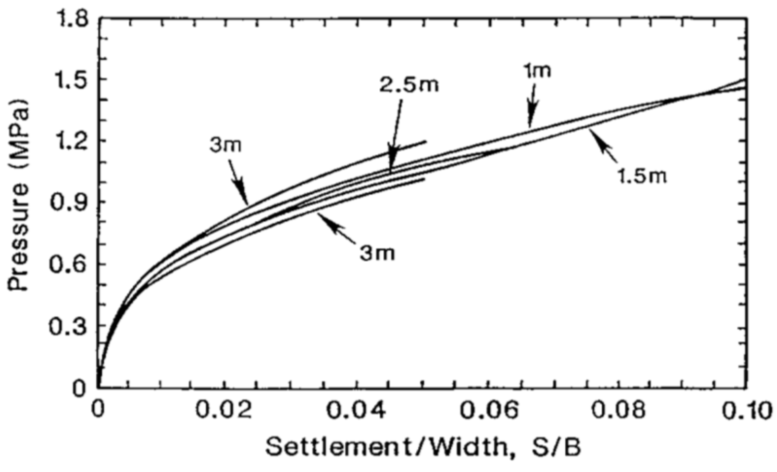
- 1 % → 0.5 Pu (FS=2)
- 5 % → 0.8 Pu
- 10 % → 0.9 Pu



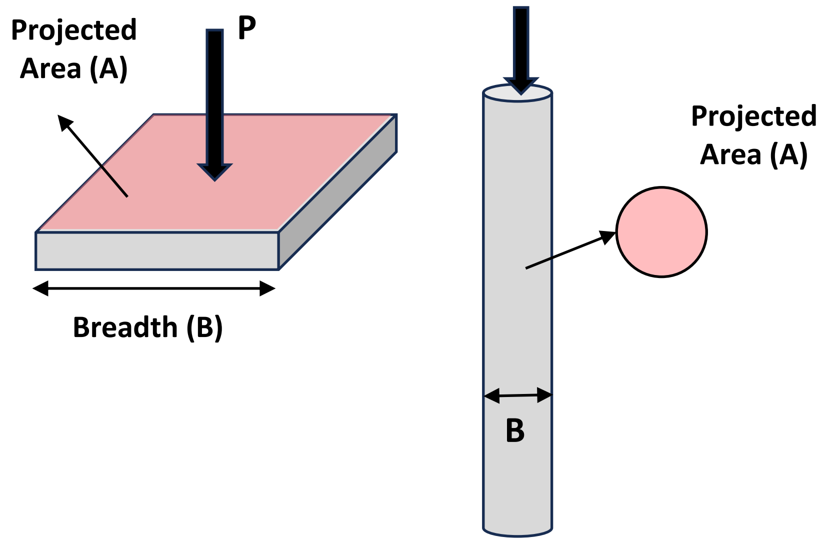
Normalized hyperbolic trending of load-displacement for dominant factors: a) embedment depth, b) breadth, c) surrounding soil type (Eslami & Ebrahimipour, 2024)

Stress – Strain Approach in Foundation Engineering

- ❖ Providing a Unitless Framework
- ❖ Facilitating Broader Practical Application
- ❖ Enhancing Foundation Selection
- ❖ Aligning with Established Engineering Principles



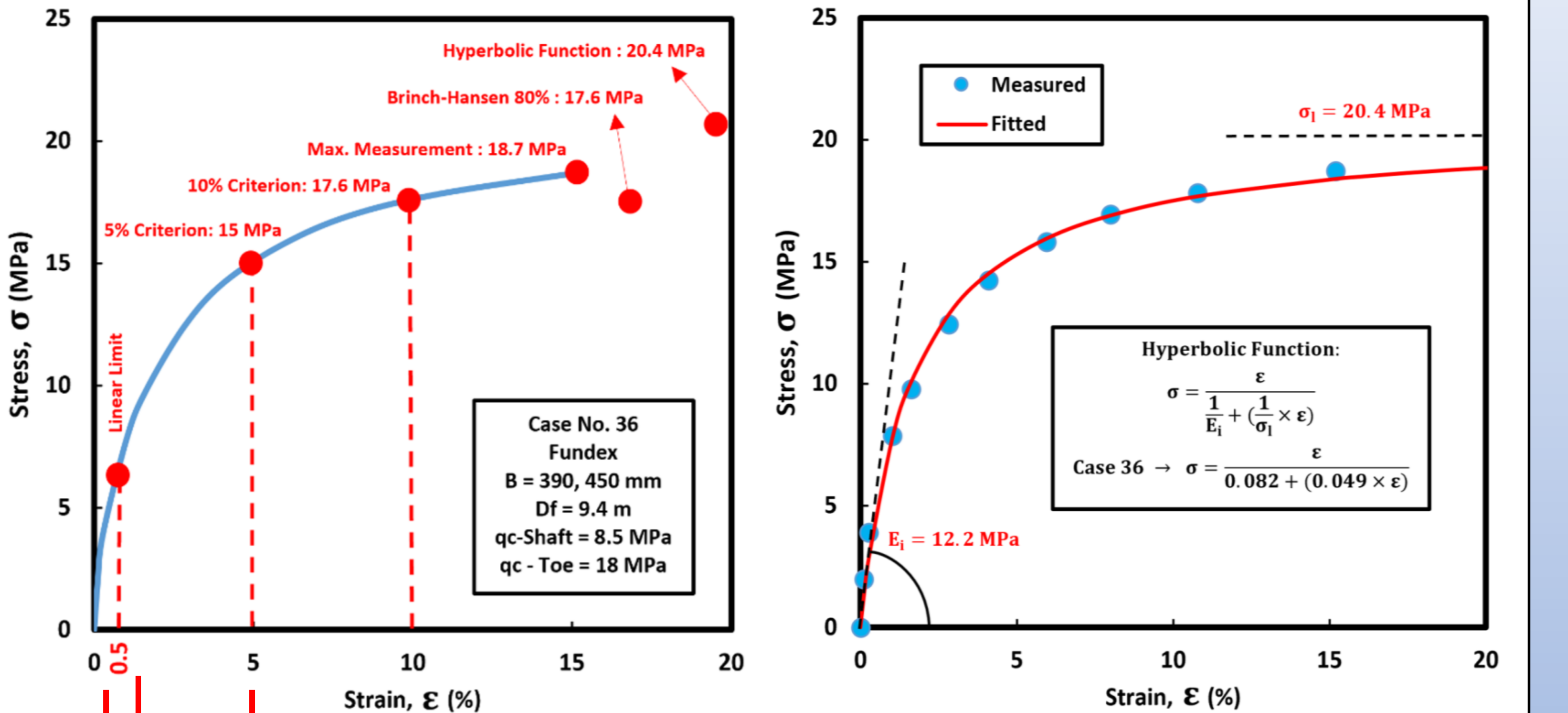
Footings stress - strain (Briaud & Gibbens, 1999)



$$\text{Stress } (\sigma) = \frac{\text{Axial Load } (P)}{\text{Projected Area } (A)}$$

$$\text{Strain } (\varepsilon) = \frac{\text{Displacement}}{\text{Breadth } (B)}$$

Dominant Criteria & Hyperbolic Fitting



Ultimate Limit State (ULS), Plastic: 1 → 5 to 10 %

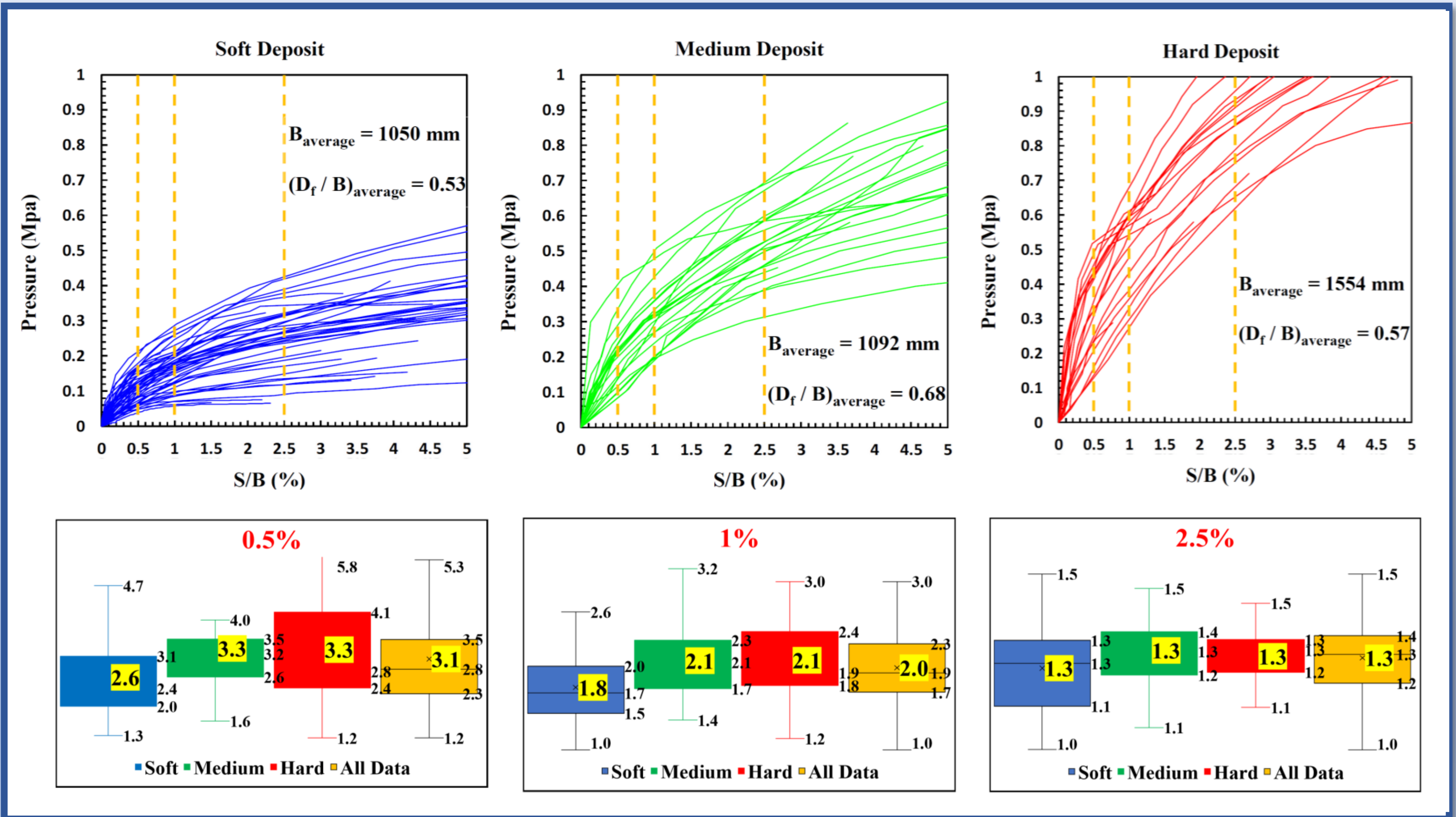
Serviceability Limit State (SLS), Non-Linear Elastic: 0.1 → 0.5 to 1 %

Operation State, Linear Elastic: 0.01 → 0.1 %

Spread Footings

- 70 Cases of Spread Footings
- Implemented on Sand, Clay and Mixed Deposits
- Embedment Depths between 0 to 3 m
- Breadth between 0.5 to 2.2 m

Assuming Ultimate Load at S/B = 5%



Evaluating factor of safety for spread footings based on displacement ratios

Energy-Based Evaluation of DD Piles (Eslami et al., 2025)

Toughness and Resilience

$$\frac{Energy}{Volume} = \int_0^{\varepsilon} \sigma \, d\varepsilon$$

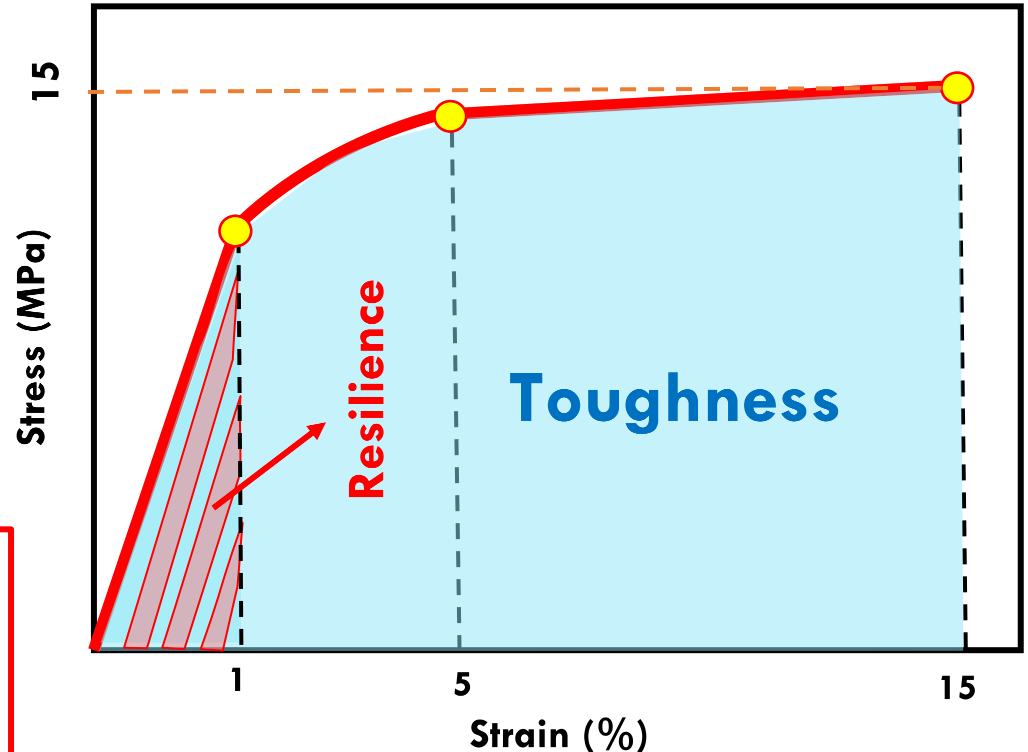
ε = Strain,

σ = Stress

Energy Absorption;

Resilience: Elastic Region

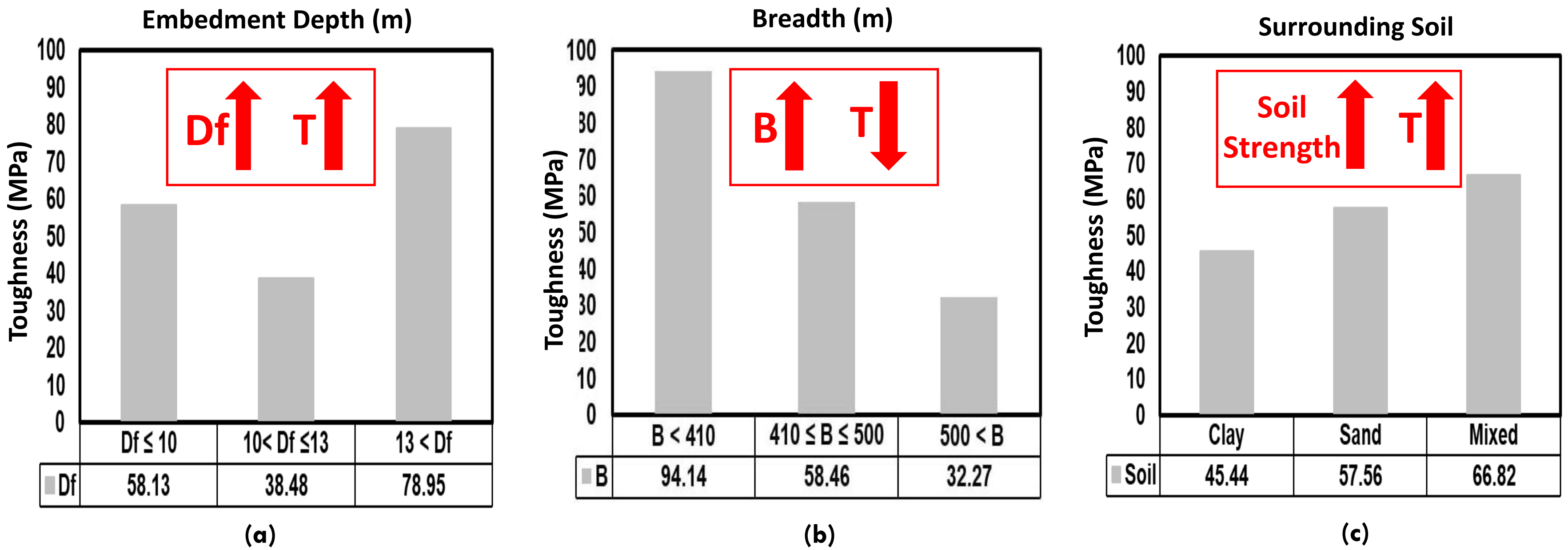
Toughness: Fracture Threshold



Resilience and toughness Concept

Energy-Based Evaluation of DD Piles (Eslami et al., 2025)

Dominant Factors on Toughness (T) Value



Average toughness values up the displacement of 1 inch: a) Embedment depth, b) Breadth, c) Surrounding soil type

Longer, Narrower piles in Mixed soil condition lead to Higher toughness value

- **Characterization: CPT & CPTu Records**
  - **Continuous, high-resolution in-situ measurements (q_t, f_s, u_2)**
  - **Reducing uncertainty, supplementing lab tests**
  - **Soil behavior captured efficiently and comprehensively**

- **Quantification: Interpretation & Parameters**
  - **Translate CPT data into soil behavior classifications (SBC)**
  - **Identify installation depth, bearing capacity, settlement, and resistance distribution**
  - **Support reliable load-displacement predictions**

- **Validation: Databases & Method Development**
  - **CPT-based pile capacity methods: scale-aware, data-driven**
  - **Recalibration and validation using case histories and field comparisons**
  - **Screening methods for efficient design: less testing, more modeling**

- **Performance: Enhanced Pile Foundation Design**
  - ❖ **Confident, advanced CPT-based design approaches**
  - ❖ **Risk-aware, resilient, and sustainable foundation solutions**
  - ❖ **Integration with soft computing, AI/ML, and challenging site applications**

- ***Takeaways***

  Participants can confidently:

  - ❖ **Interpret CPT data,**
  - ❖ **Predict pile performance,**
  - ❖ **Apply advanced methods,**
  - ❖ **Manage and mitigate risk,**
  - ❖ **Deliver resilient and sustainable solutions.**

- Abu-Farsakh, M.Y., Haque, N., & Chen, Q. (2016). Field instrumentation and testing to study set-up phenomenon of piles driven into Louisiana clayey soils. Report No.: FHWA/LA.15/562

- 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).

- Campanella, R. (1990). Current status of the piezocone test. In Proc. 1st Int. Symp. on Penetration Testing (1, 93-116). ISOPT.

- Ebrahimipour, A., and Eslami, A. (2024). Analytical study of piles behavior for marine challenging substructures. Ocean Engineering, 292, 116514.

- 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., Moshfeghi, S., Heidarie, S., Valikhah, F., (2019). AUT: Geo-CPT &Pile Database Updates and Implementations for Pile Geotechnical Design, Geotechnical Engineering Journal of the SEAGS & AGSSEA, Vol. 50 No. 2.

- 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

- Eslami, A., Heidarie Golafzani, S. & Naghibi, M.H. (2022). Developed triangular charts; deltaic CPTu-based soil behavior classification using AUT: CPTu-Geo-Marine Database, Probabilistic Engineering Mechanics

- Eslami, A., & Ebrahimipour, A. (2024). Load-displacement appraisal and analysis for driven piles; a data-centric approach. Journal of Computers and Geotechnics, 171, 106377

- Eslami, A., Rahimi, A. & Nobahar, M. (2024). Ultimate load bearing of helical piles prediction and evaluation using machine learning-based algorithms, Geomechanics and Geoengineering, 20(3)
- Eslami, A., Ebrahimipour, A., Imani, M., Imam, R. and Mo, P.Q. (2025). Form and Load Transfer Aspects of Foundation Systems; Case-Based Implementation and Adaptation for Buildings, Deep Underground Science and Engineering
- Eslami, A., Shadlou, D., & Ebrahimipour, A. (2025). Pore water pressure generation and sensitivity aspects for pile dynamics and capacity loss: CPTu records and case studies. Soil Dynamics and Earthquake Engineering, 190
- 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.
- Heidari, S., Eslami, A., & Jamshidi Chenari, R. (2017). Reliability based assessment of pile foundation bearing capacity: static analysis, SPT and CPT-based methods, Probabilistic Engineering Mechanics, submitted.
- Heidarie Golafzani, S., Jamshidi Chenari, R., and Eslami, A. (2020). Reliability based assessment of axial pile bearing capacity: static analysis, SPT and CPT-based methods, Georisk: Assessment and Management of Risk for Engineered Systems and Geohazards
- Karakouzian, M. & Eslami, A. (2026). Advanced Foundation Engineering, Principles, Performance and Prospect, Wiley (Under Publication)
- Kempfert, H.G., & Becker, P. (2010). Axial pile resistance of different pile types based on empirical values. Proceedings of Geo-Shanghai, 149-154
- Mayne, P.W. (2007). Cone penetration testing State-of-Practice. In: NCHRP Synthesis. Transportation Research Board Report Project 20 - 05, 118 pp.

- 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.
- Nobahar, M., Rahimi, A., & Eslami, A. (2025). Predicting and Evaluating Load-Displacement Trends of Helical Piles Using Machine Learning–Based Algorithms. Transportation Research Record: Journal of the Transportation Research Board
- Robertson, P.K. (2009). Interpretation of cone penetration tests e a unified approach. Canadian Geotechnical Journal 46 (11), 1337 – 1355
- 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.

Special Thanks to:

Amirhossein Ebrahimipour; Research Fellow

For His Productive Contribution

Appreciation to:

Masoud Nobahar

Sara Heidarie Golafzani

Amirhossein Rahimi

For Their Informative Support

Gratitude For Your Attention

[*https://abolfazleslami.com/*](https://abolfazleslami.com/)

LinkedIn