Medro Engineering

Webinar

Performance-Based Deep Foundation Design:
Using CPT Records and Load–Displacement Response

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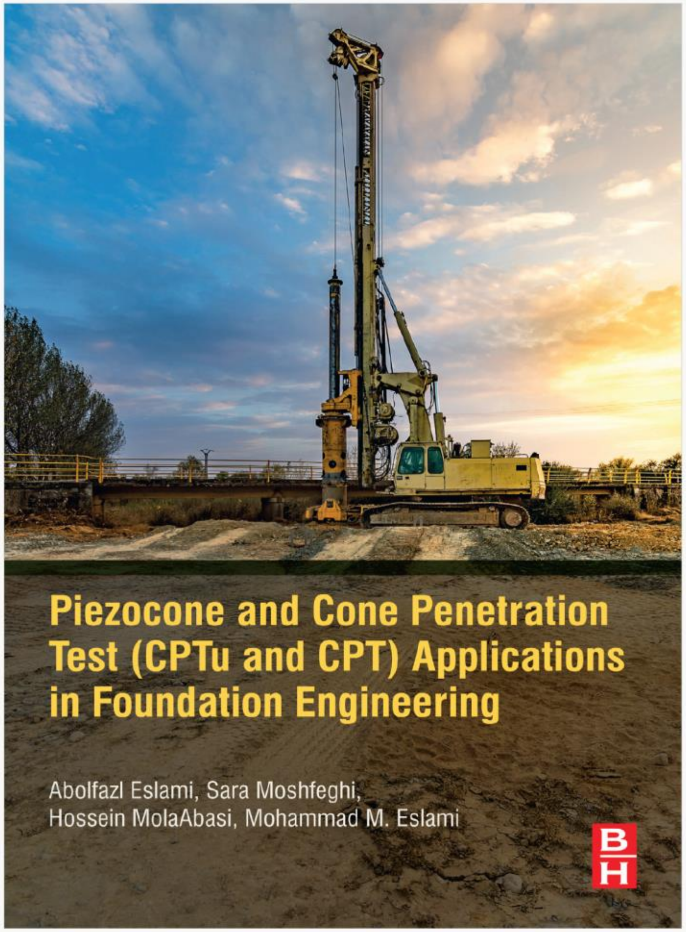
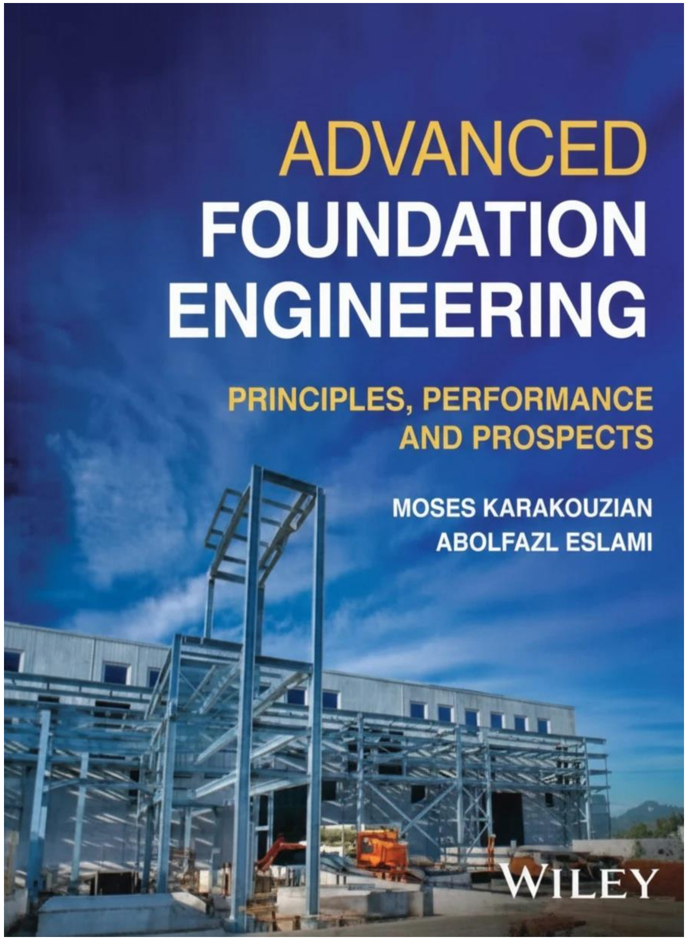
May 2026

1. Site Investigation & Interpretation (CPT-Based)
2. Foundation Systems & Design Methodology
3. Performance Verification & Validation
4. Load–Displacement Application
5. Summary & Concluding Remarks

Main Sources





The data interpretations, methodologies, and design frameworks presented in this webinar are sourced exclusively from the author's peer-reviewed publications.

Aim

To transition from the Art of Estimation to the Science of Certainty, unifying bearing capacity, settlement, and structural capability within a single load-displacement framework.

Scope

Data Acquisition & Interpretation:

- High-resolution stratigraphic intelligence (qc, fs, u_2)
- Integrating material properties and installation records
- SBT classification and stiffness profiling

Direct Design Methodology:

- Foundation system selection and performance-based framework
- Bearing capacity: direct and indirect CPT methods
- Load-transfer mechanisms and resistance distribution

Performance Verification & Implementation:

- Worked examples and case histories
- Installation records and construction control
- Load test vs. predicted performance

Load–Displacement Application:

- Load–displacement records and field calibration
- Relevant Data-Based Design (RDBD)
- Normalized load–displacement and stress–strain framework

Objective

To deliver a professional roadmap for Performance-Based Design, where behavioral response remains constant, through a design inherently adaptable to ground variability.

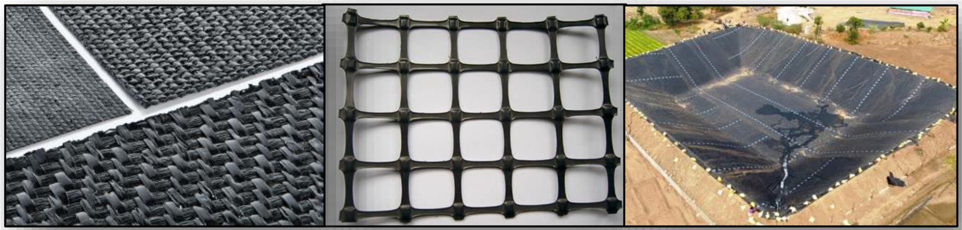
Geotechnical Engineering World

❖ **Geomaterials:**

Soil, Rock & Ground Water

❖ **Geosynthetics:**

Geotextile, Geogrid, Geomembrane, …



❖ **Geostructures:**

Foundation, Earth Retention Systems, …

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

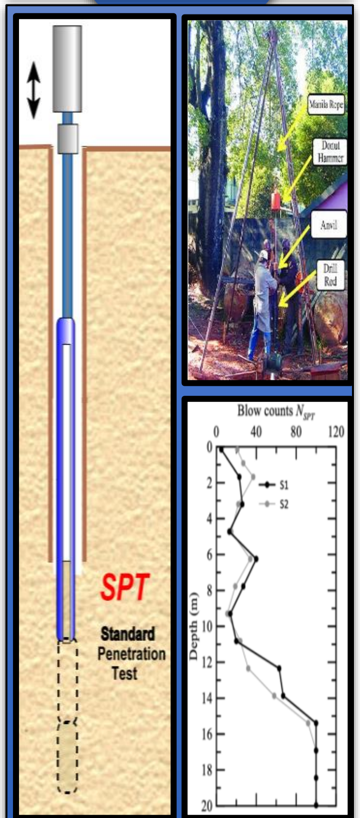
(11) Full-Scale Tests

(12) Instrumentation & Monitoring

In-Situ Tests; **more pronounced and applicable in Foundation Engineering**

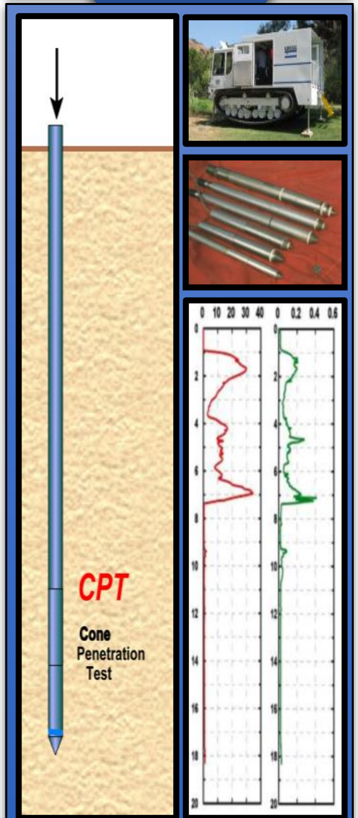
Major In Situ Penetration Tests: Schematic & Records

SPT



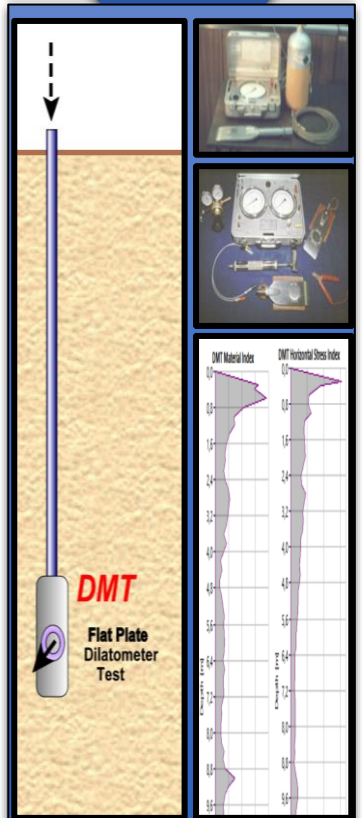
N
Blow count

CPT



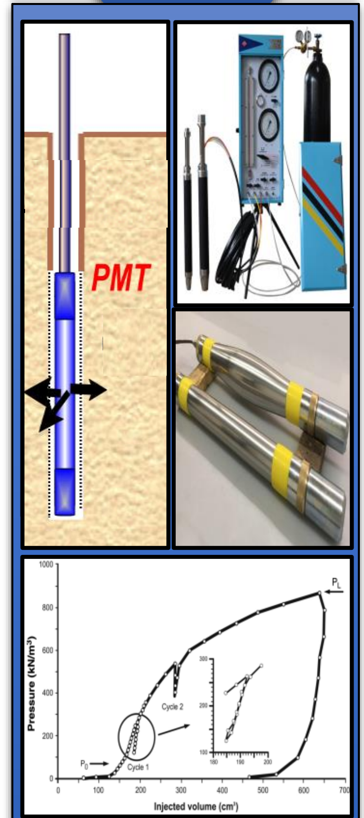
q_c, f_s & u_2
Tip resistance, sleeve friction & excess porewater pressure

DMT



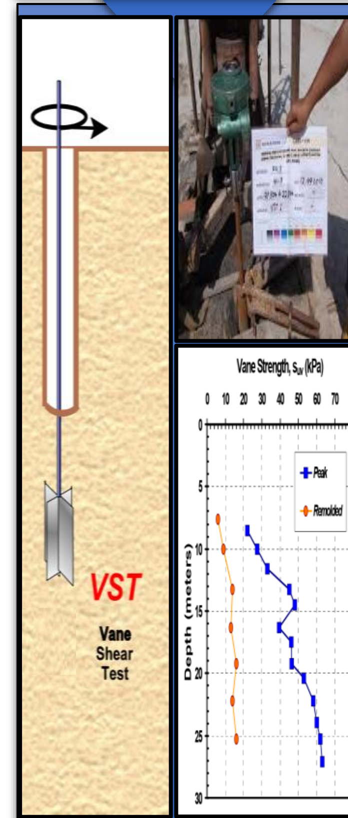
K_D, I_D & E_D
Horizontal stress index, material index & dilatometer modulus

PMT



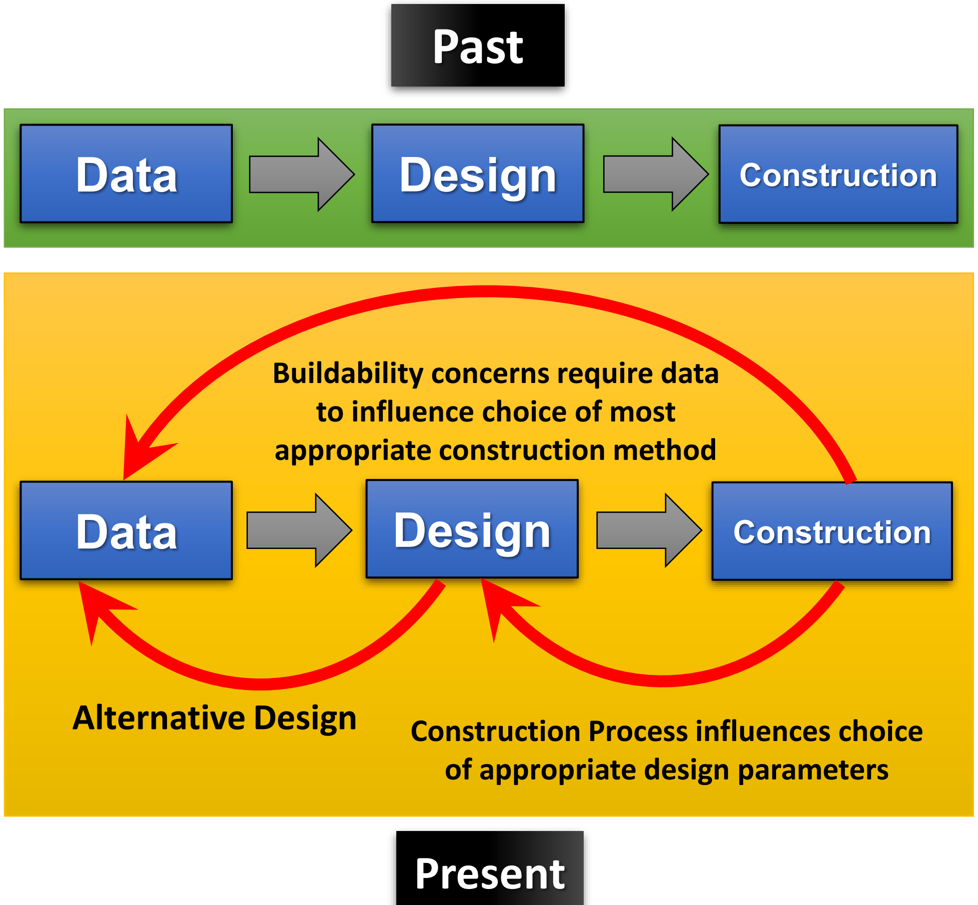
P_L & G
Limit pressure & shear modulus

VST

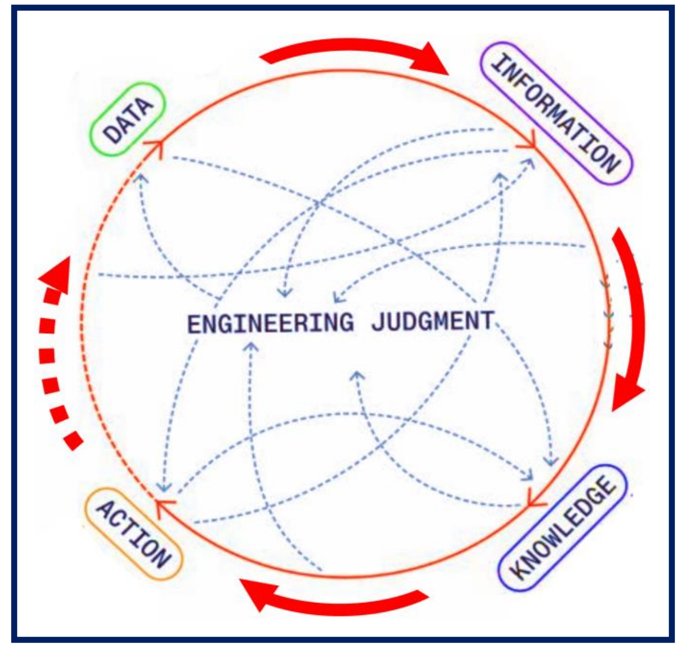


S_u
Undrained shear strength

Cycle of Data, Design, and Performance



ICE, Geotechnical Manual (2012)



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

Why In-Situ Testing?

Laboratory Test Limitations

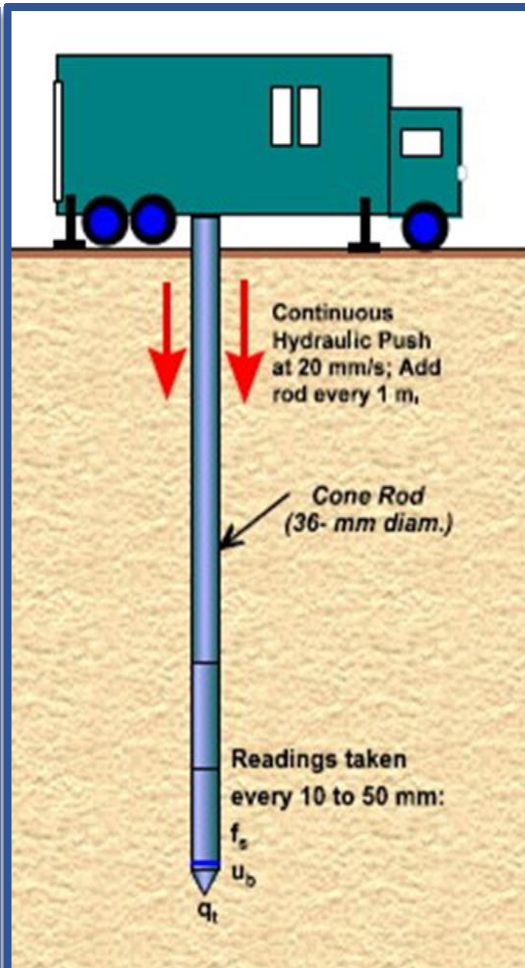
- ***Undisturbed Sampling Challenges:*** True in-situ soil conditions are hard to preserve.
- ***Soil Disturbance & Preservation:*** Handling and storage can alter natural properties.
- ***Volume Changes:*** Shrinkage or swelling may distort laboratory measurements.
- ***Inadequate Confinement:*** Lab conditions cannot fully replicate field stress states.
- ***Scale & Boundary Effects:*** Small specimens and boundaries can bias results.

Field Test Advantages

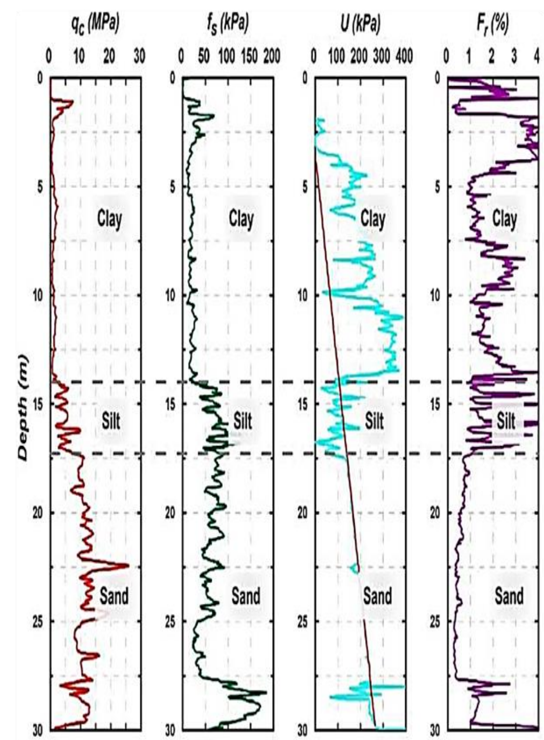
- ***Bypasses Sampling Difficulties:*** No need for extracting undisturbed samples.
- ***Maintains In-Situ Stress State:*** Preserves natural soil behavior during testing.
- ***Fast and Efficient:*** Simple execution with rapid data acquisition.
- ***Economical:*** Reduces laboratory handling and associated costs.
- ***Direct Foundation Relevance:*** Data immediately applicable to design and analysis.

Laboratory & In-Situ Tests; Complementary & Interactive

Equipment & Procedure



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



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

Data & Graphical Presentation

❖ Measured Parameters

q_c, f_s, u

❖ Corrected Parameters

- **Corrected tip resistance:**

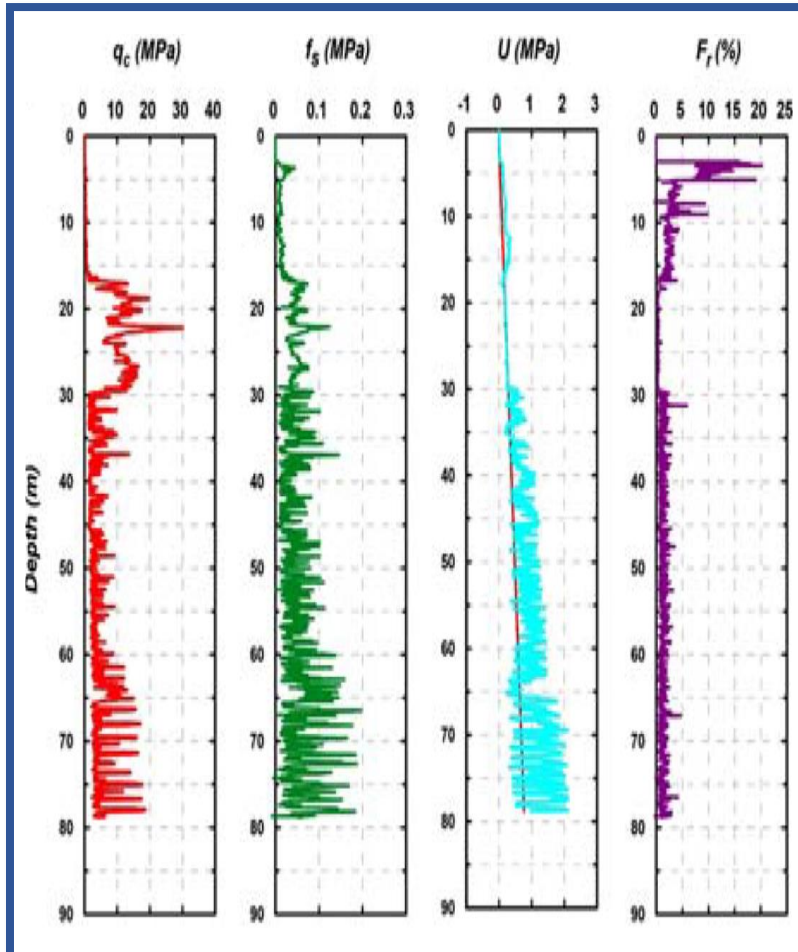
$$q_t = q_c + u_2(1-a)$$

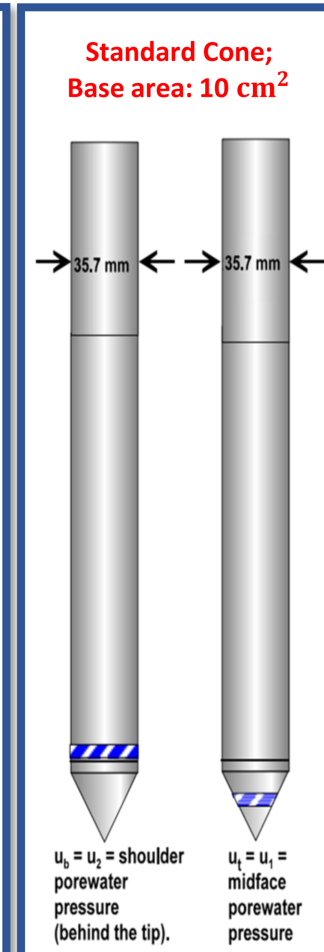
- **Friction ratio:**

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

- **Pore pressure coefficient:**

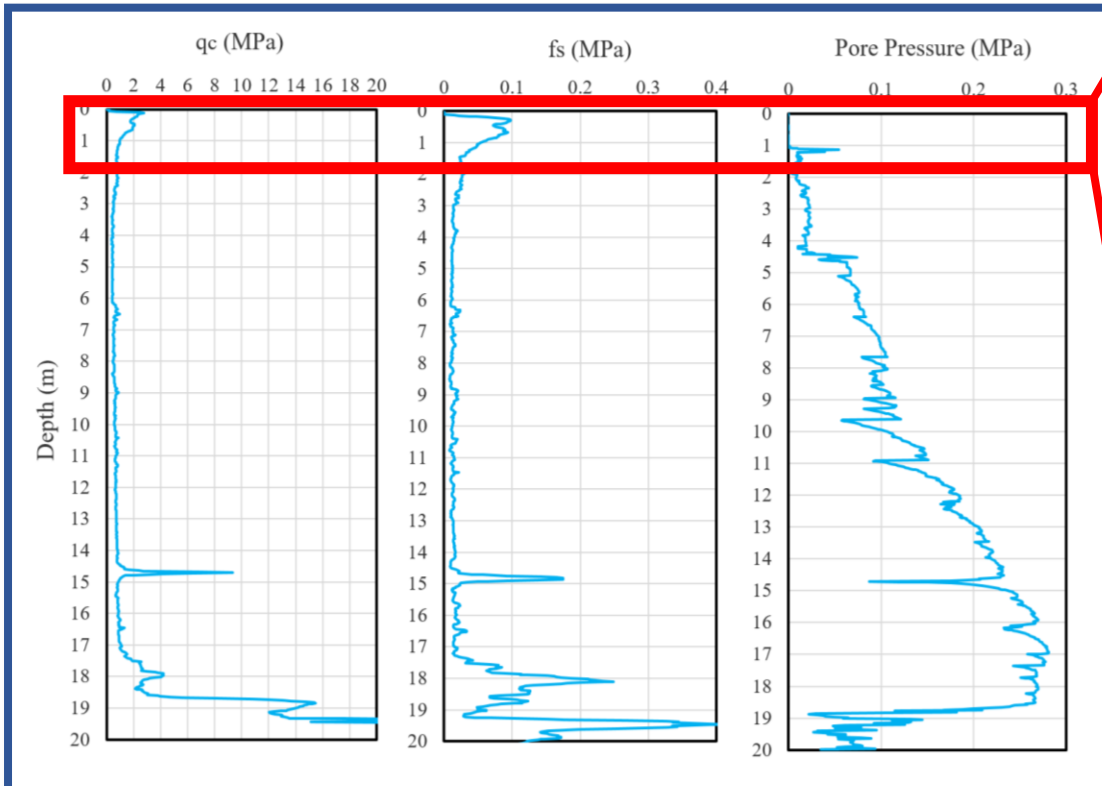
$$B_q = {\Delta u}/{(q_t - \sigma_{vo})}$$





Data & Graphical Presentation

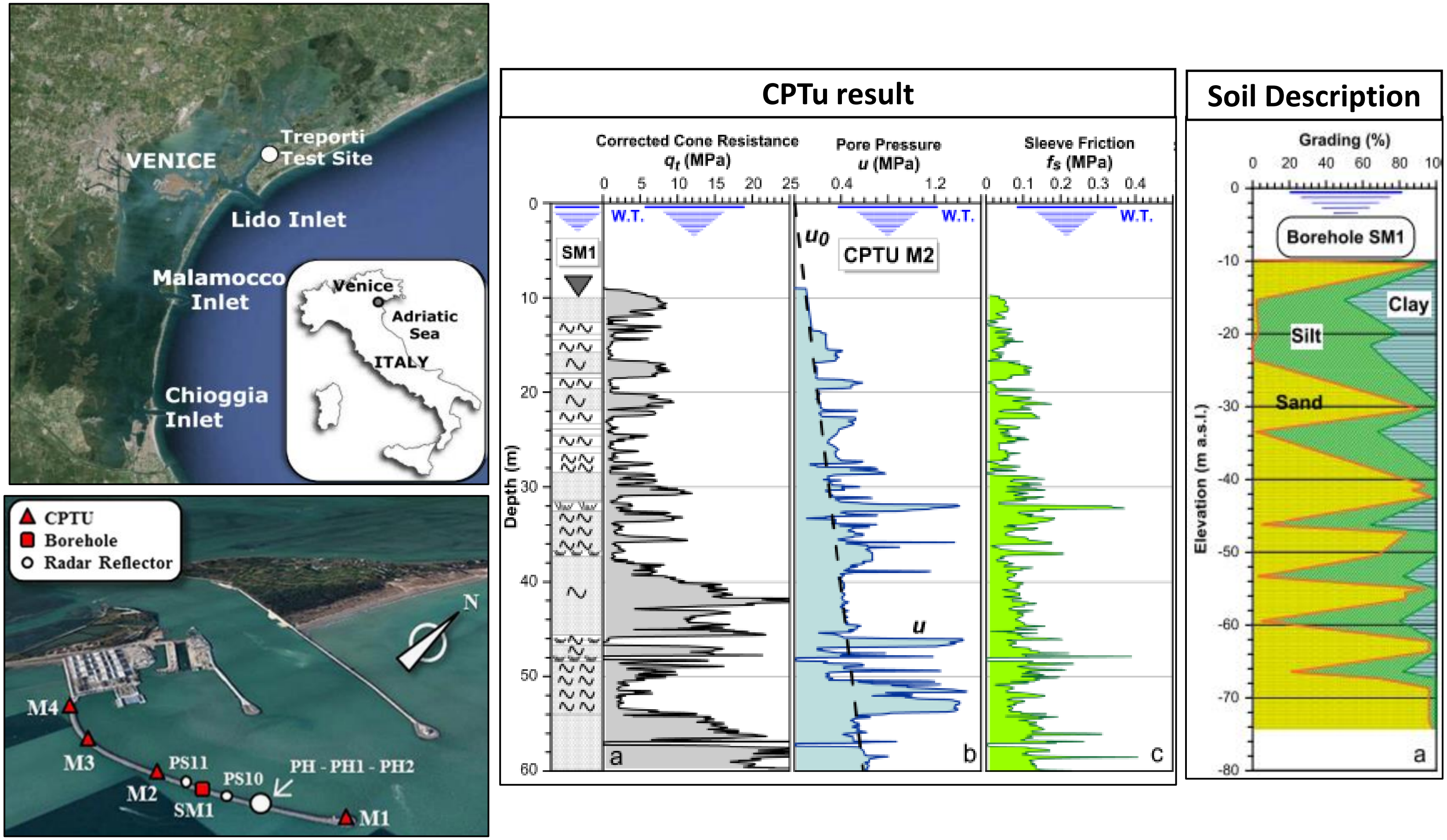
Tons of Data in 1 Meter !!!



z	qc	fs	u2
m	MPa	MPa	MPa
0	0	0	0
0.02	0.06	-0.0003	0
0.04	0.24	-0.0003	0
0.06	0.16	0.001	-0.0005
0.08	0.71	0	-0.0005
0.1	2.47	0.0004	-0.001
0.12	2.79	0.0147	-0.001
0.14	2.48	0.0215	-0.001
0.16	2.31	0.0386	-0.001
0.18	2.32	0.0479	-0.001
0.2	2.27	0.0639	-0.001
0.22	2.27	0.0759	-0.0005
0.24	2.22	0.0878	-0.0005
0.26	2.1	0.0957	-0.001
0.28	2.07	0.0976	-0.001
0.3	1.99	0.0981	-0.0005
0.32	1.88	0.096	-0.0005
0.34	1.81	0.0926	-0.0005
0.36	1.79	0.092	-0.0005
0.38	1.75	0.0873	0
0.4	1.75	0.083	0
0.42	1.83	0.0764	0
0.44	1.92	0.0728	0
0.46	1.96	0.072	0
0.48	2.06	0.0743	0
0.5	2.08	0.0762	0
0.52	2.05	0.0812	-0.0005
0.54	1.99	0.0863	-0.0005
0.56	1.97	0.0879	-0.001
0.58	1.91	0.0897	-0.001
0.6	1.92	0.0888	-0.0005
0.62	1.91	0.0865	-0.0005
0.64	1.9	0.0885	-0.0005
0.66	1.83	0.0903	-0.0005
0.68	1.73	0.0935	0
0.7	1.68	0.0918	-0.0005
0.72	1.5833	0.0877	0
0.74	1.4867	0.0836	0
0.76	1.39	0.0795	0
0.78	1.36	0.0806	0
0.8	1.33	0.0792	0
0.82	1.28	0.0775	0
0.84	1.2	0.0772	0
0.86	1.18	0.0739	-0.0005
0.88	1.16	0.0704	-0.0005
0.9	1.14	0.0648	0
0.92	1.09	0.0634	-0.0005
0.94	1.04	0.0619	0
0.96	1.02	0.0586	-0.0005
0.98	0.98	0.0558	0
1	0.99	0.0536	0

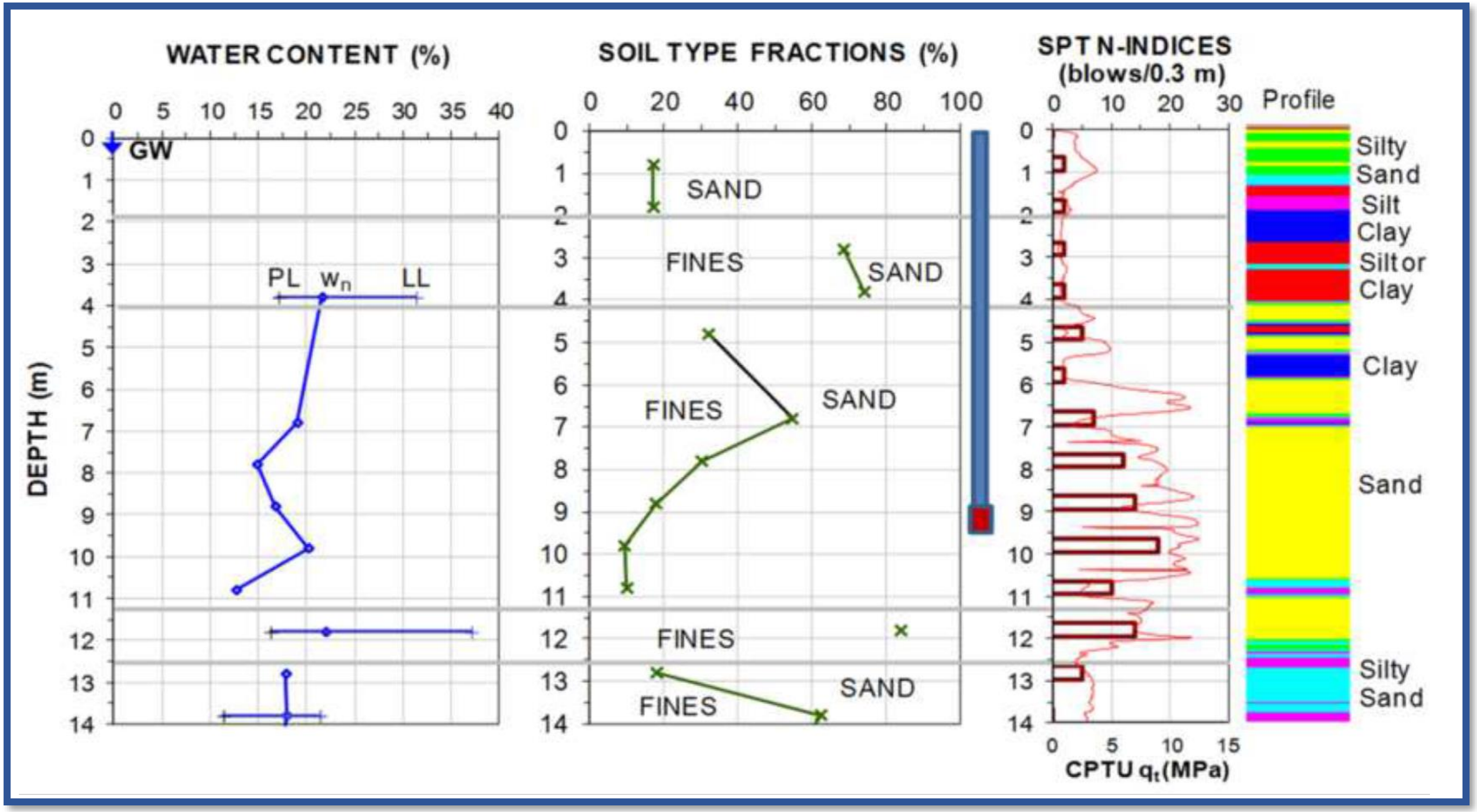
Soil Profiling

❖ **Malamocco Breakwater Offshore Project, Venetian Lagoon, Italy (AUT:GMD; Eslami et al., 2022)**



Soil Profiling: Typical & In-Situ Tests

❖ **Bolivian Experimental Site for Testing (B.E.S.T.) (Fellenius et al., 2017)**



Borehole data and SPT N values for BH-E2 (Fellenius et al., 2017)

Soil Profiling: Typical & In-Situ Tests

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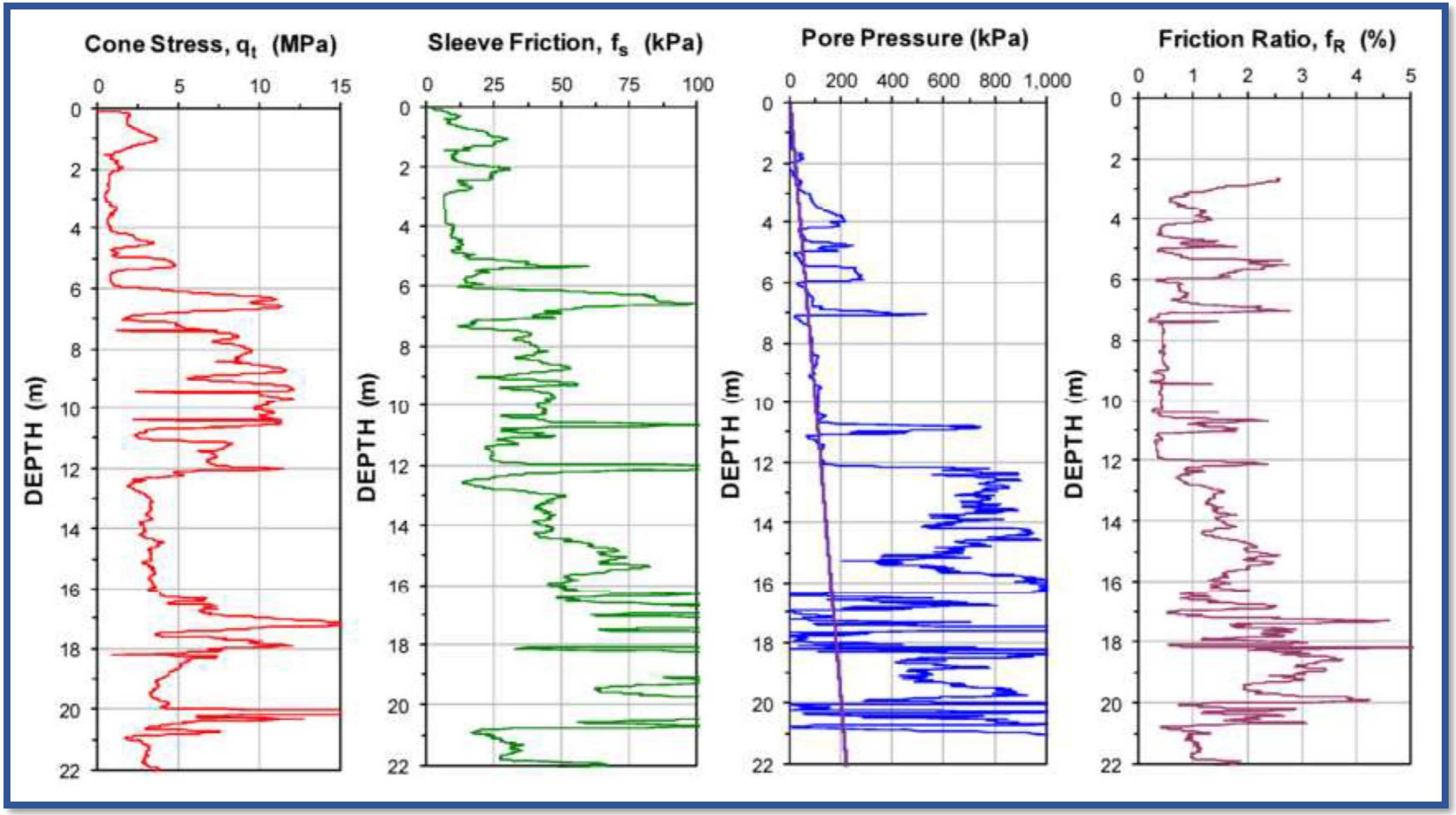

CPTU sounding diagram for BH-E2 (Fellenius et al., 2017)

Soil Profiling: Typical & In-Situ Tests

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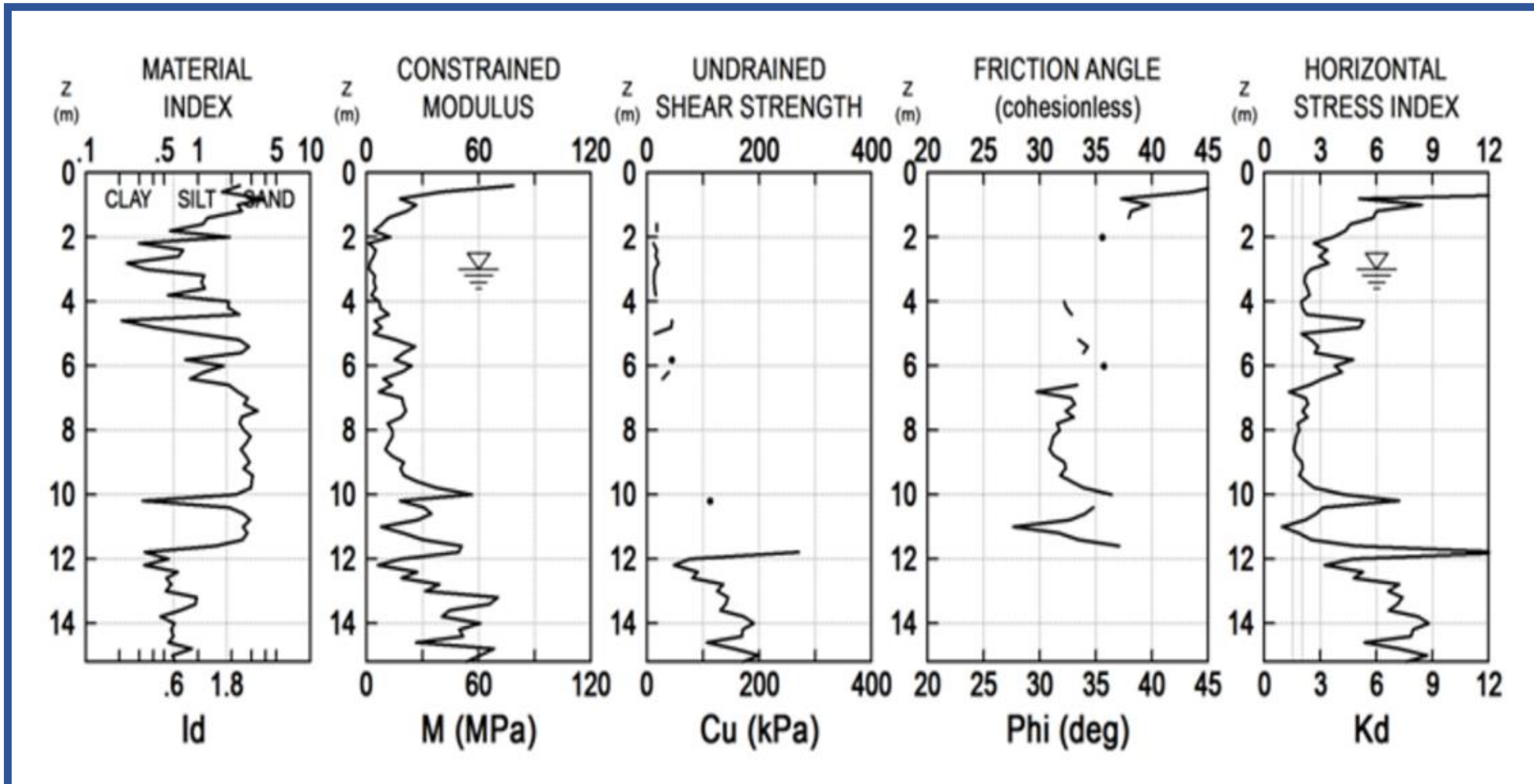
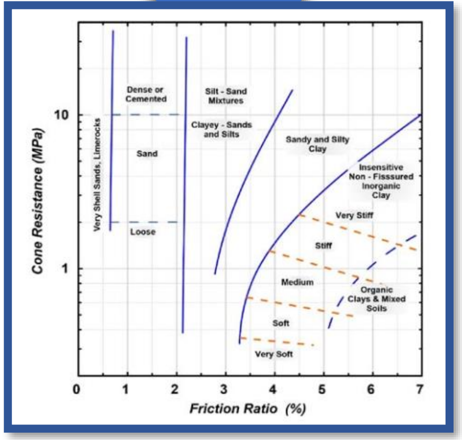


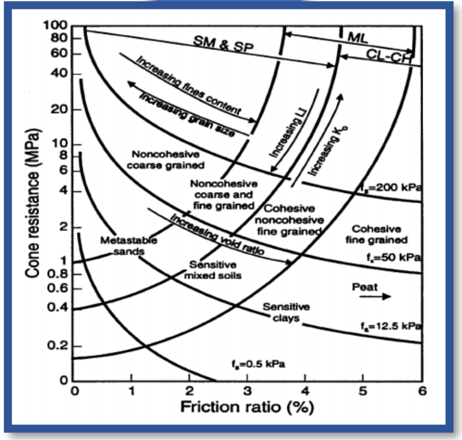
DMT results for BH-B2 (Fellenius et al., 2017)

Soil Behavior Classification: Charts & Diagrams

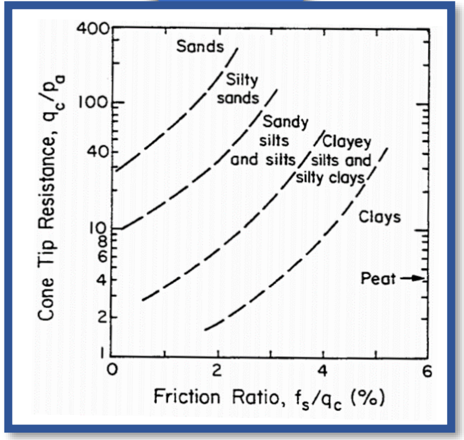
Schmertmann (1978)



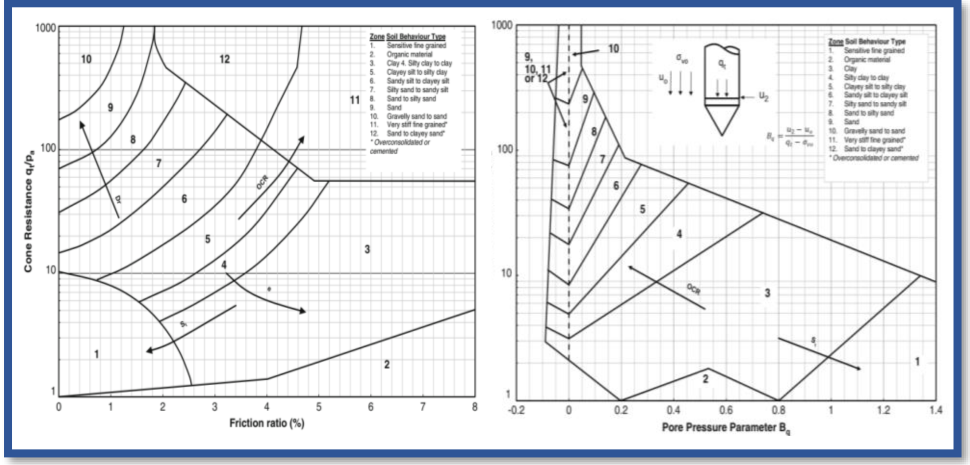
Douglas & Olsen (1981)



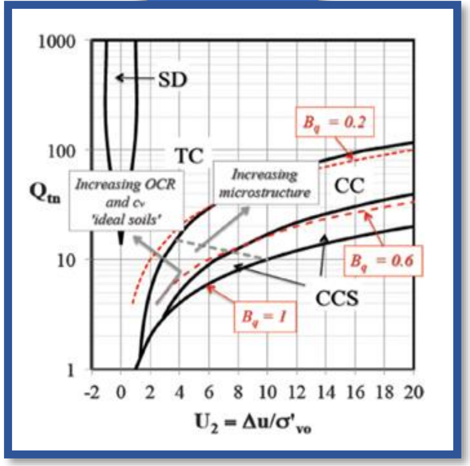
Campanella et al. (1983)



Robertson et al. (1986)



Robertson (2016)



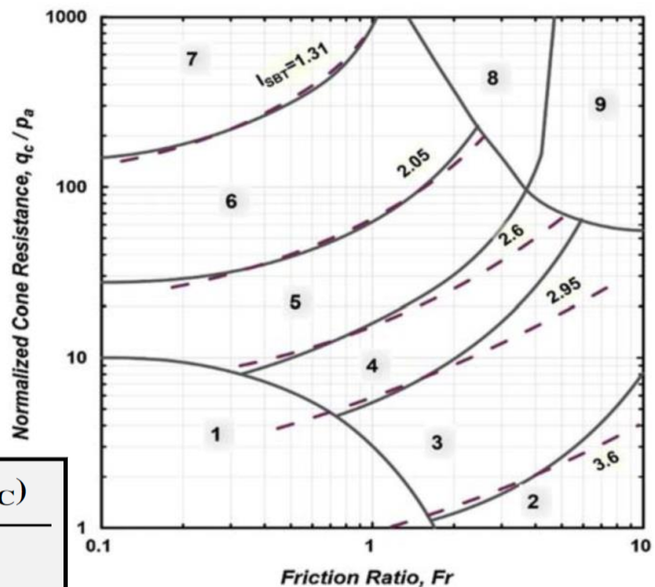
Soil Behavior Classification: Charts & Diagrams

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

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

Soil classification based on ISBCT (Li et al., 2015)

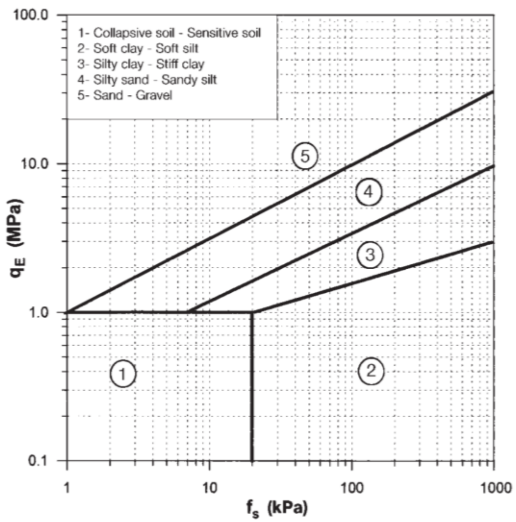
Soil classification	Zone	CPTu index (I_{SBC})
Gravelly sands	7	$I_{SBC}<1.25$
Sands—clean sand to silty sand	6	$1.25<I_{SBC}<1.90$
Sand mixture—silty sand to sandy silt	5	$1.90<I_{SBC}<2.54$
Silt mixture—clayey silt to silty clay	4	$2.54<I_{SBC}<2.82$
Clays	3	$2.82<I_{SBC}<3.22$
Organic clays	2	$I_{SBC}>3.22$



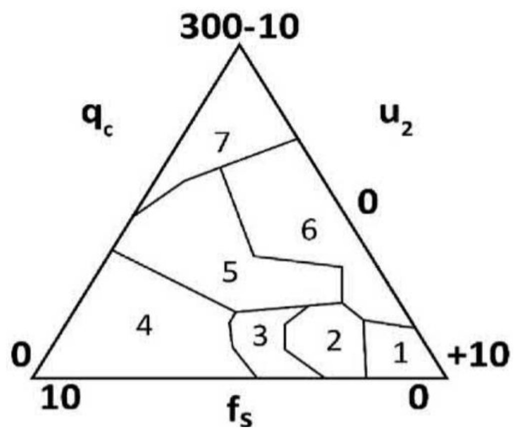
Values of Ic on the Robertson (1990) classification chart (Jefferies and Davies, 1991)

Soil Behavior Classification: Charts & Diagrams

Eslami and Fellenius (1997)



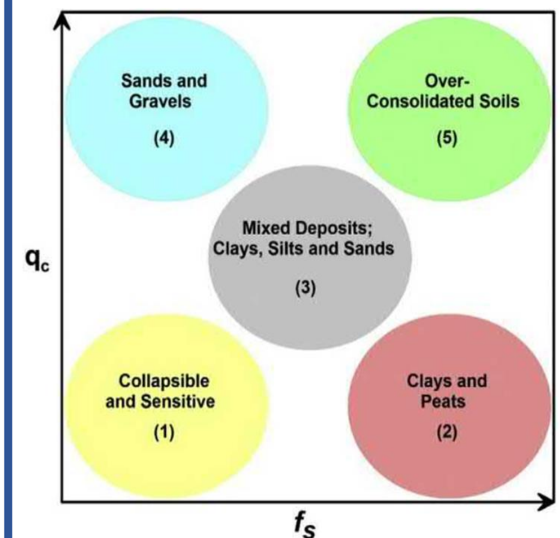
Eslami et al. (2016 & 2022)



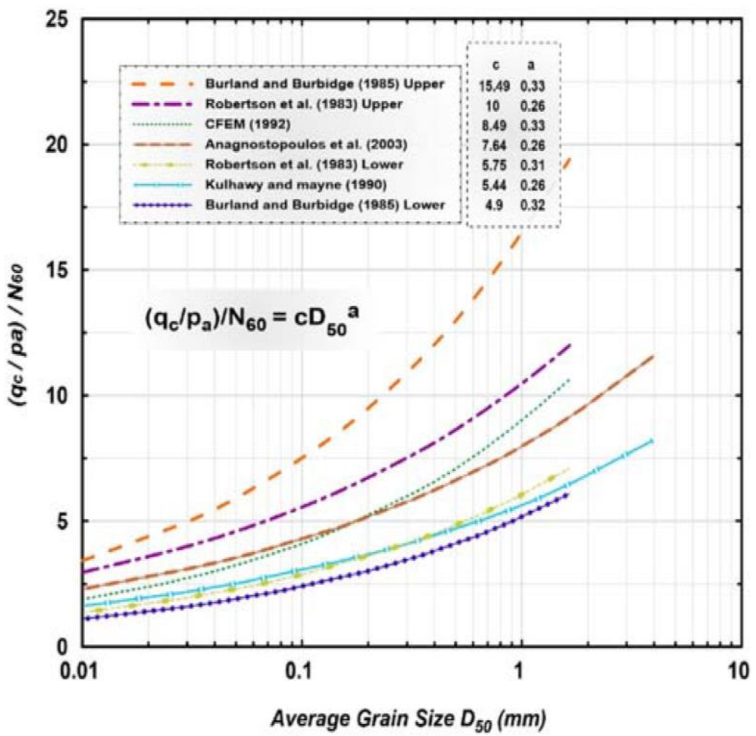
1	2	3
Sensitive Soil	Clays	Silts

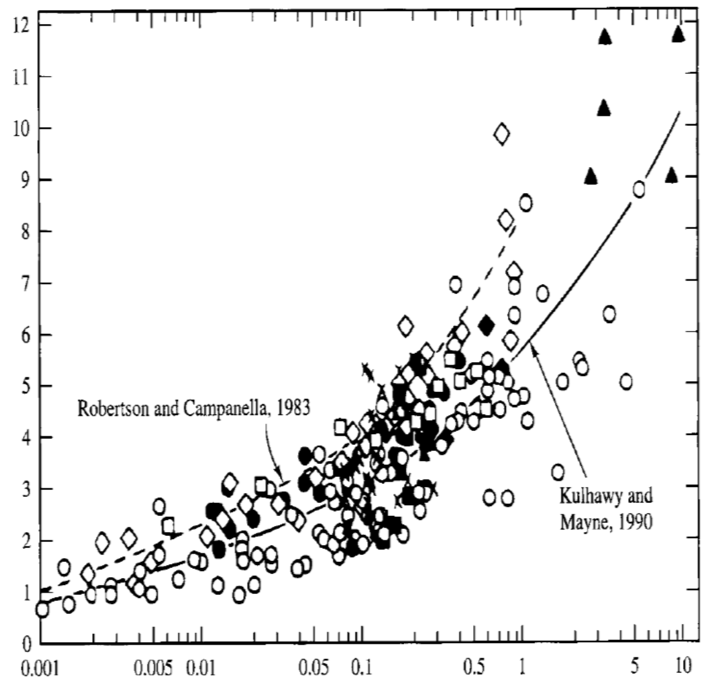
4	5	6	7
Over Consolidated Clay	Mixed Soil (Clayey silt to sandy silt)	Sands	Very Dense Sand

Eslami et al. (2018)



CPT (q_c) correlations with SPT (N)





(Adopted from Robertson & Campanella, 1983)

Coupled Geotechnical Parameters

Sands

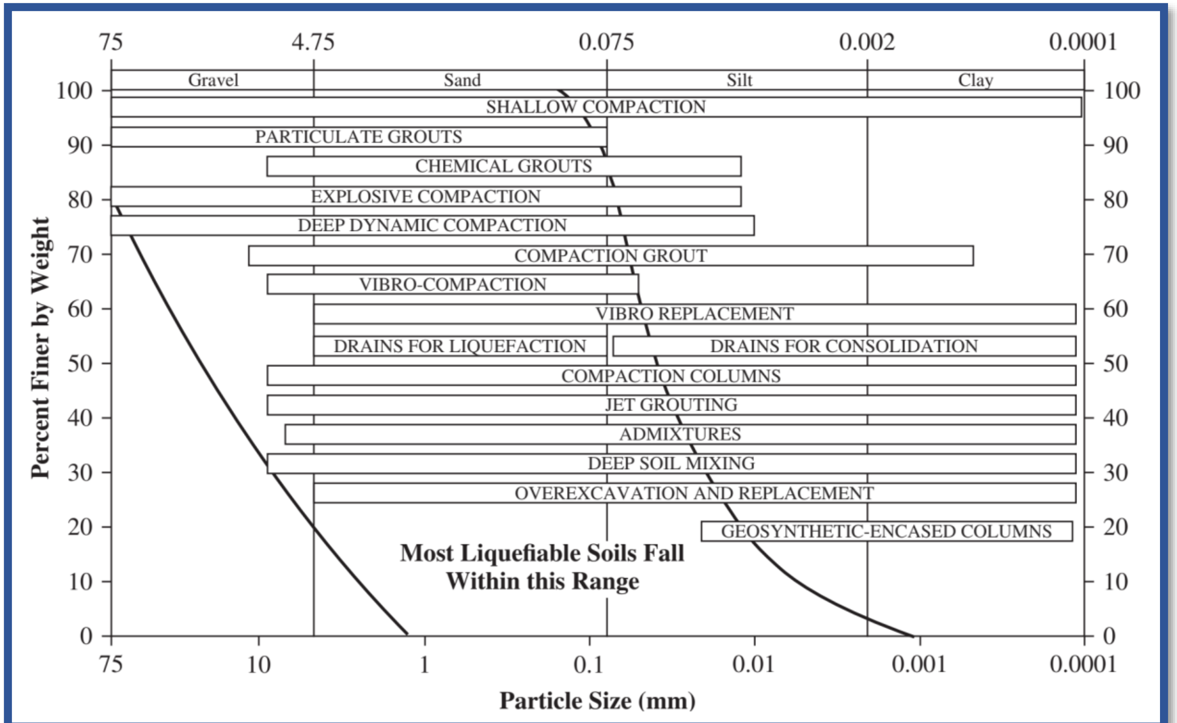
Soil condition / Geotechnical parameters	Very loose	Loose	Medium	Dense	Very dense
N_{SPT}	<4	4–10	10–30	30–35	>50
$q_c\left(\frac{kg}{cm^2}\right)$	<50	50–100	100–150	150–200	>200
$D_r(\%)$	<15	15–35	35–65	65–85	85–100
$\varphi(°)$	<30	30–32	32–35	35–38	>38
$\gamma_{dry}\left(\frac{ton}{m^3}\right)$	1.4	1.4–1.6	1.6–1.8	1.8–2	>2
$\frac{\tau}{\sigma'}$	<0.04	0.04–0.1	0.1–0.35	>0.35	–

Clays

Soil condition / Geotechnical parameters	Soft	Medium to stiff	Stiff to hard
N_{SPT}	<5	5–15	>15
$q_c\left(\frac{kg}{cm^2}\right)$	<10	10–50	>50
$S_u\left(\frac{kg}{cm^2}\right)$	<0.5	0–1	1–2
$\gamma_{dry}\left(\frac{ton}{m^3}\right)$	0.9–1.4	1.4–1.7	1.7–2.1
ω (%)	>30	20–40	<20

Role in Ground Improvement

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

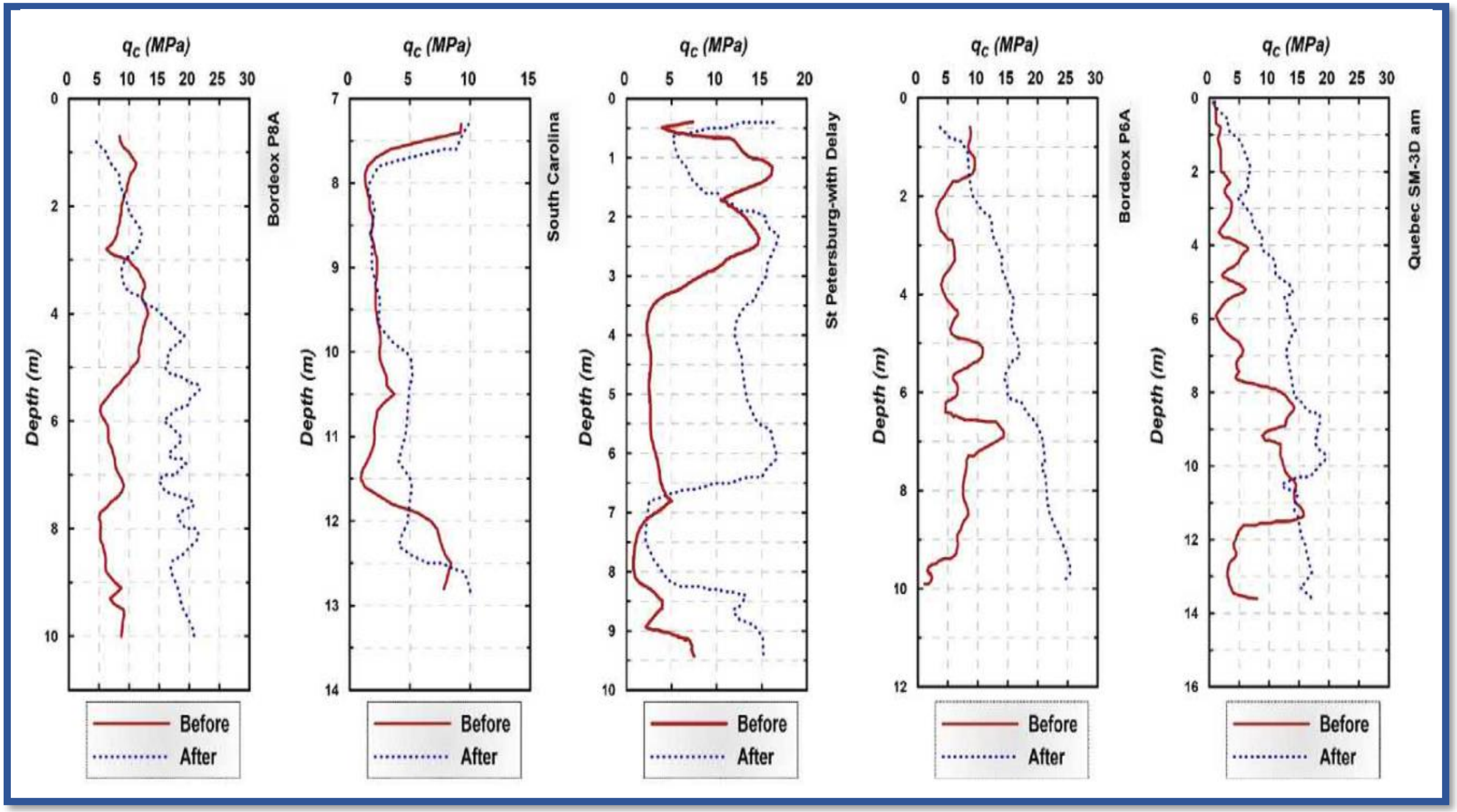


Available ground improvement methods for different soil types (modified from Schaefer et al., 2012)

Role in Ground Improvement

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

Pre & Post Soil Improvement Records Comparison



Case No. 1: Torre Latino Americana, Mexico City (Coduto et al., 2016)

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

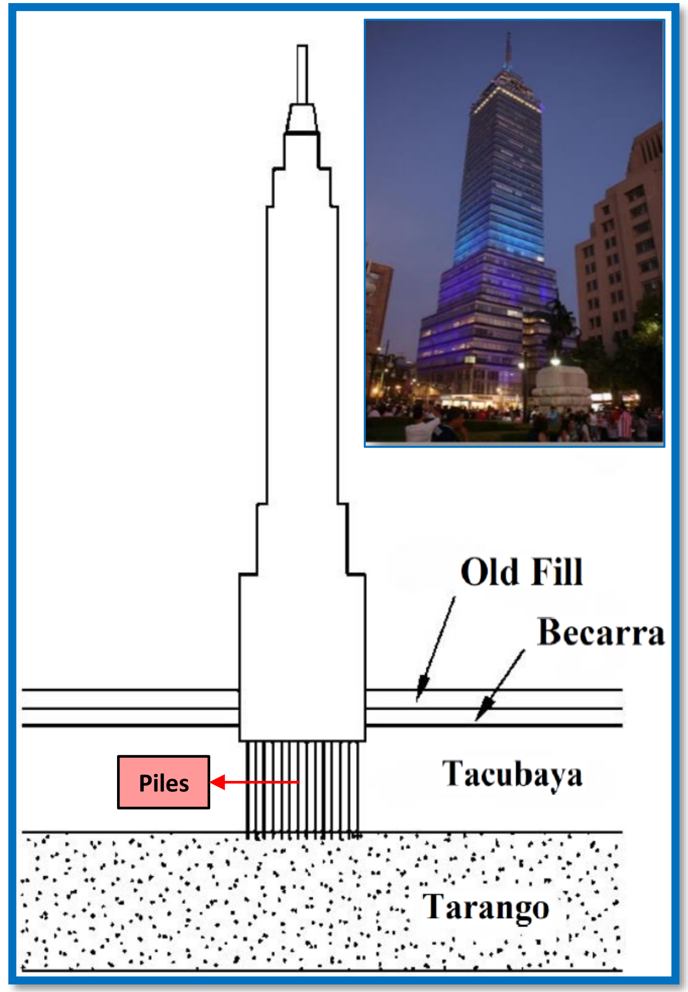
The soil profile:

- **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

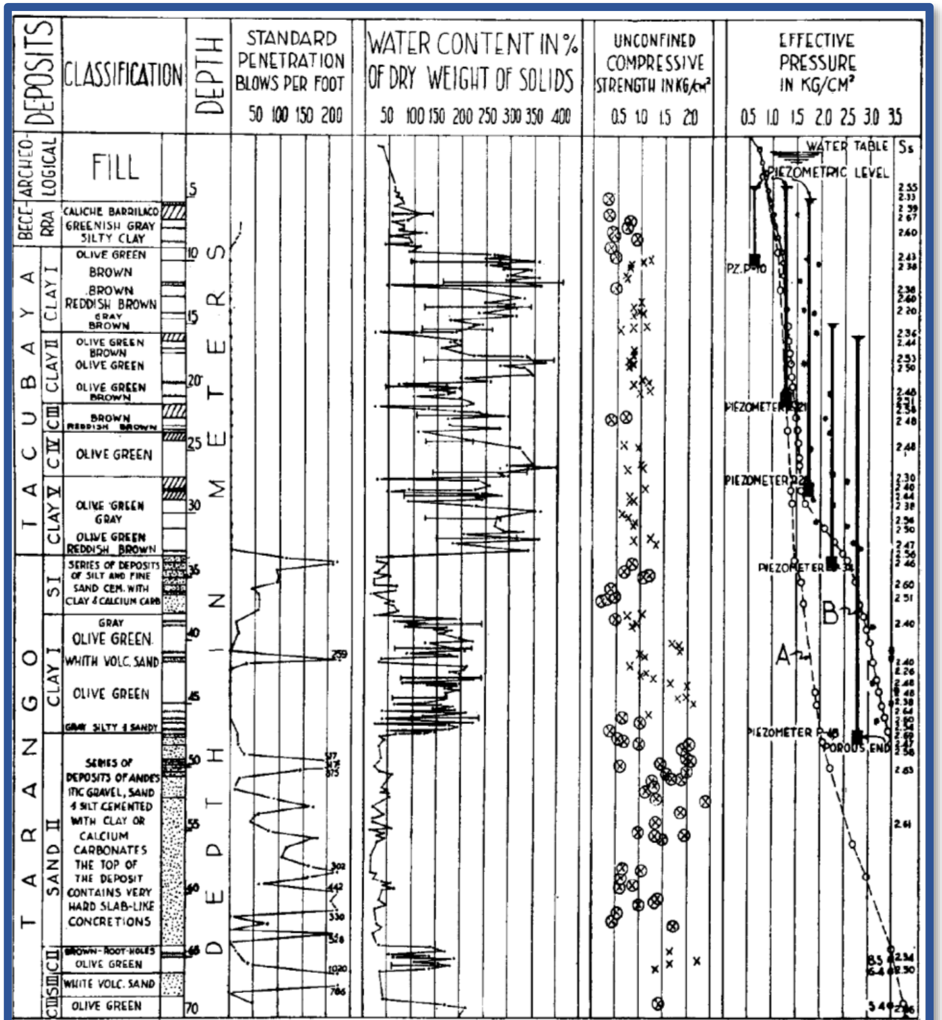
- **33.5–70.0 m depth: Tarango sands**

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

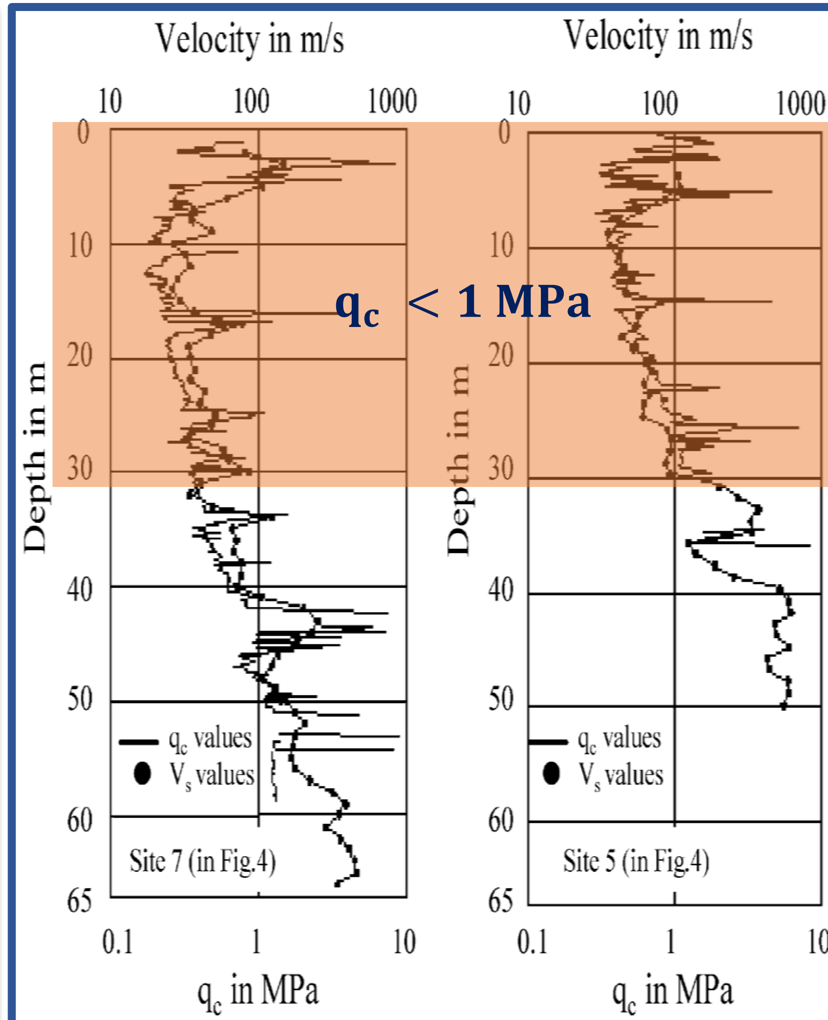


The foundation and the sublayer profile (Coduto et al., 2016)

Case Study No. 1: Continued - Profiling



Classic log & SPT result (Zeevaert, 1957)



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

Case Study No. 1: Continued - Major Investigation, Design and Performance Aspects

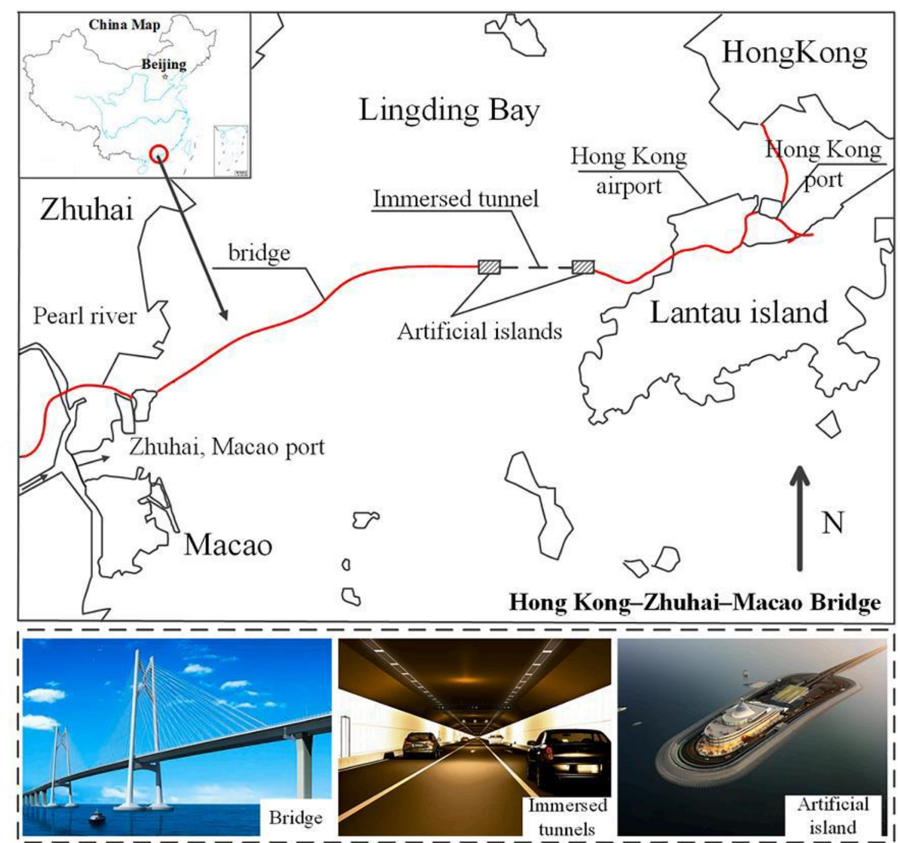
- In-situ testing: recognition of challenging sublayers
- Significancy of end-bearing deep foundations
- Optimized application of floating foundations
- Restrict the settlement within the allowed range

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

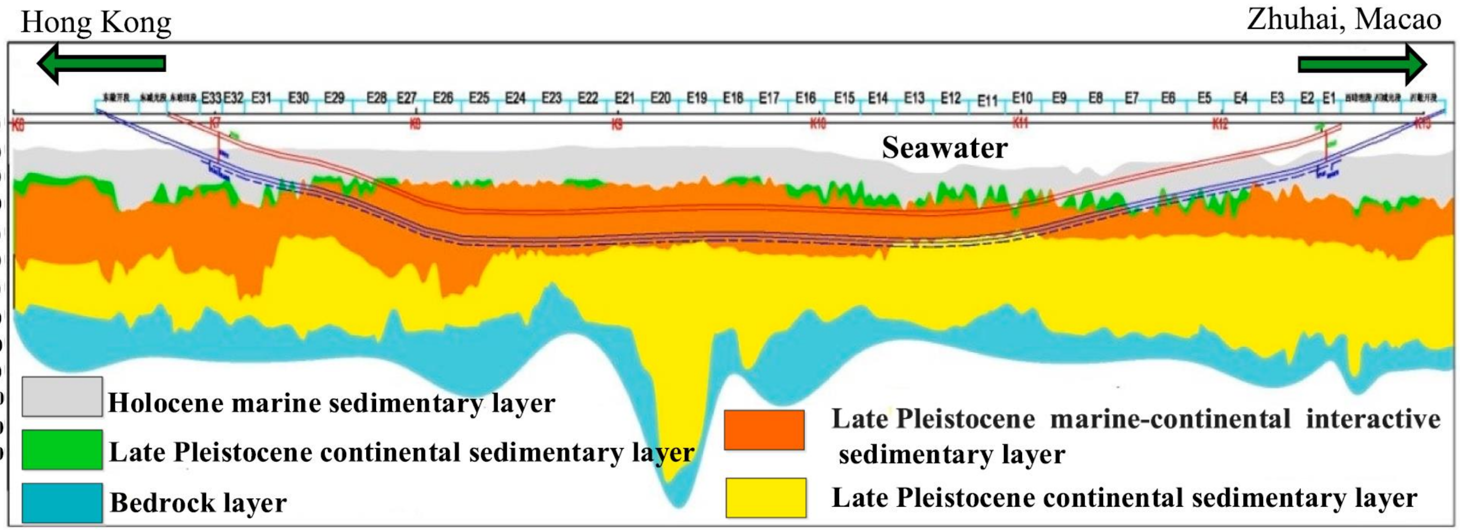
Case No. 2: The Hong Kong Bridge (Duan et al., 2021)

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



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

Case Study No. 2: Continued – Soil Profiling

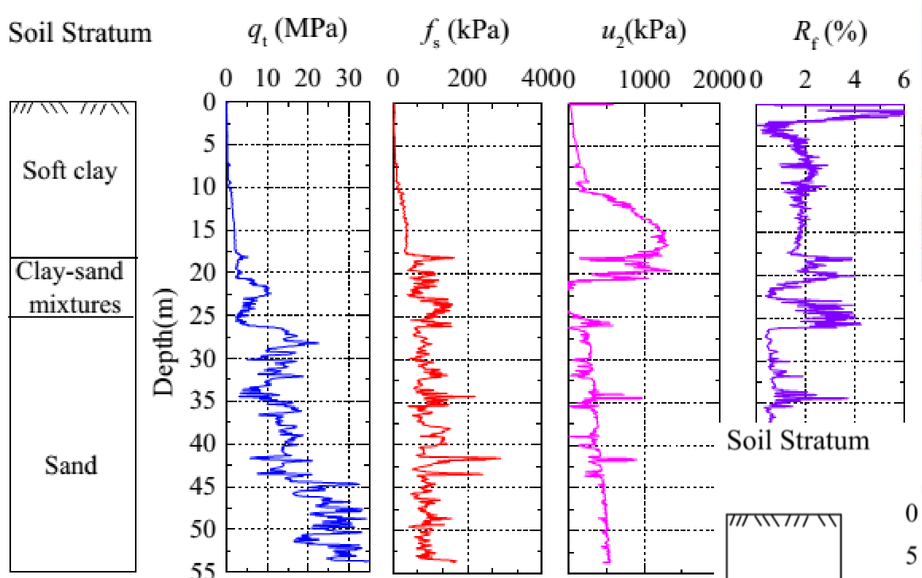


- Soft clay with medium sand for local areas
- Clay with silty sand for local areas
- Clay , clay with sand for local areas, followed by fine sand and medium sand
- Medium coarse gravel sand, followed by fine sand and clay
- Huanggang rock

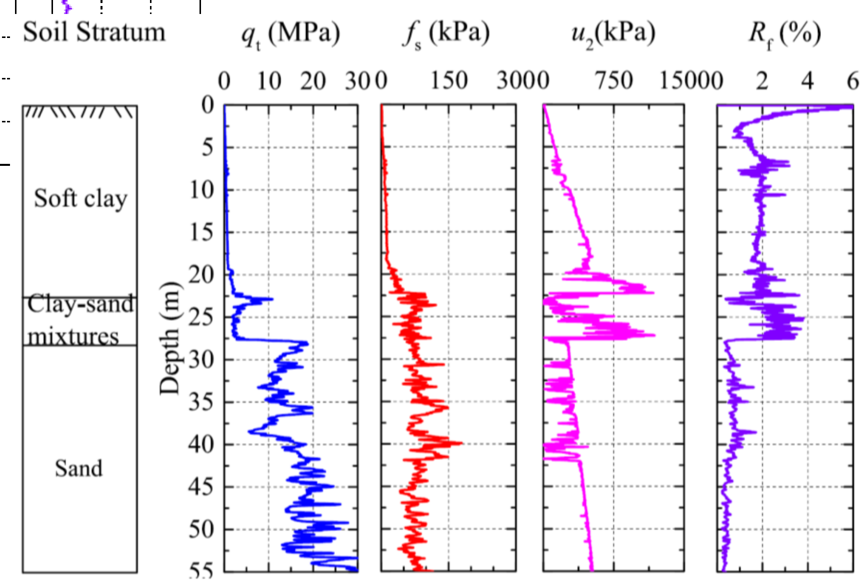
Offshore hydraulic lifting drilling platform

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

Case Study No. 2: Continued – CPT Records



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



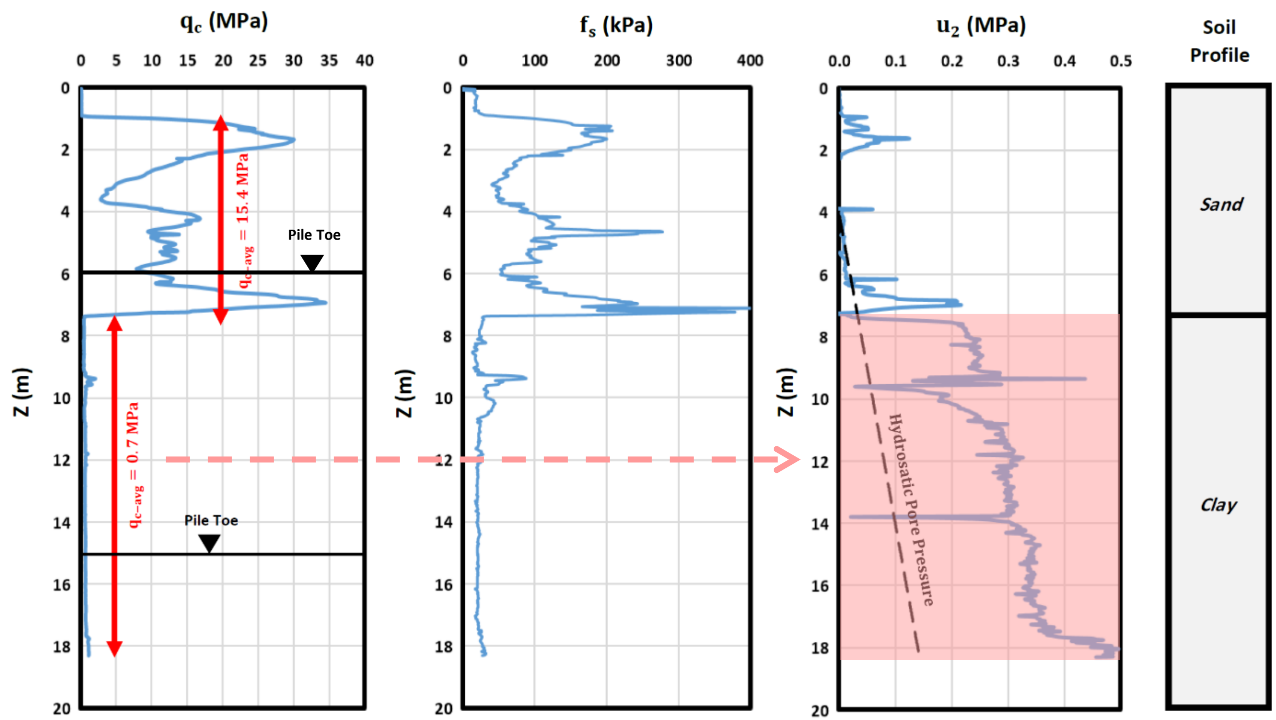
Typical CPT records of HZMB project (Duan et al., 2021)

Case No. 3-a: CPT Records for Characterization & Installation Depth (Finno, 1989)

Compare and contrast two pile installation depths based on the given CPTu profile:

a) At the depth of 6 m

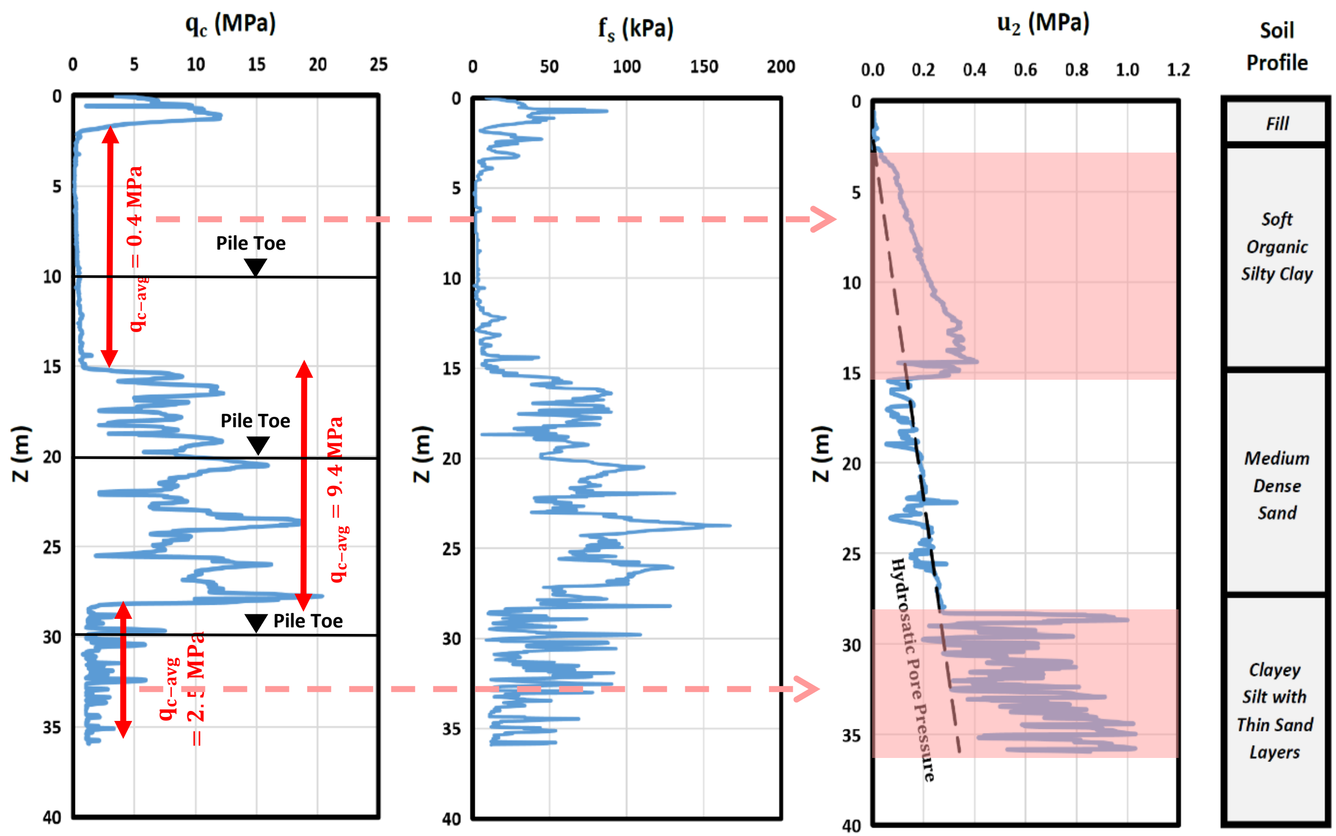
b) At the depth of 15 m



CPT profiles and pile alternatives: NWU Site (CPT data from Finno, 1989)

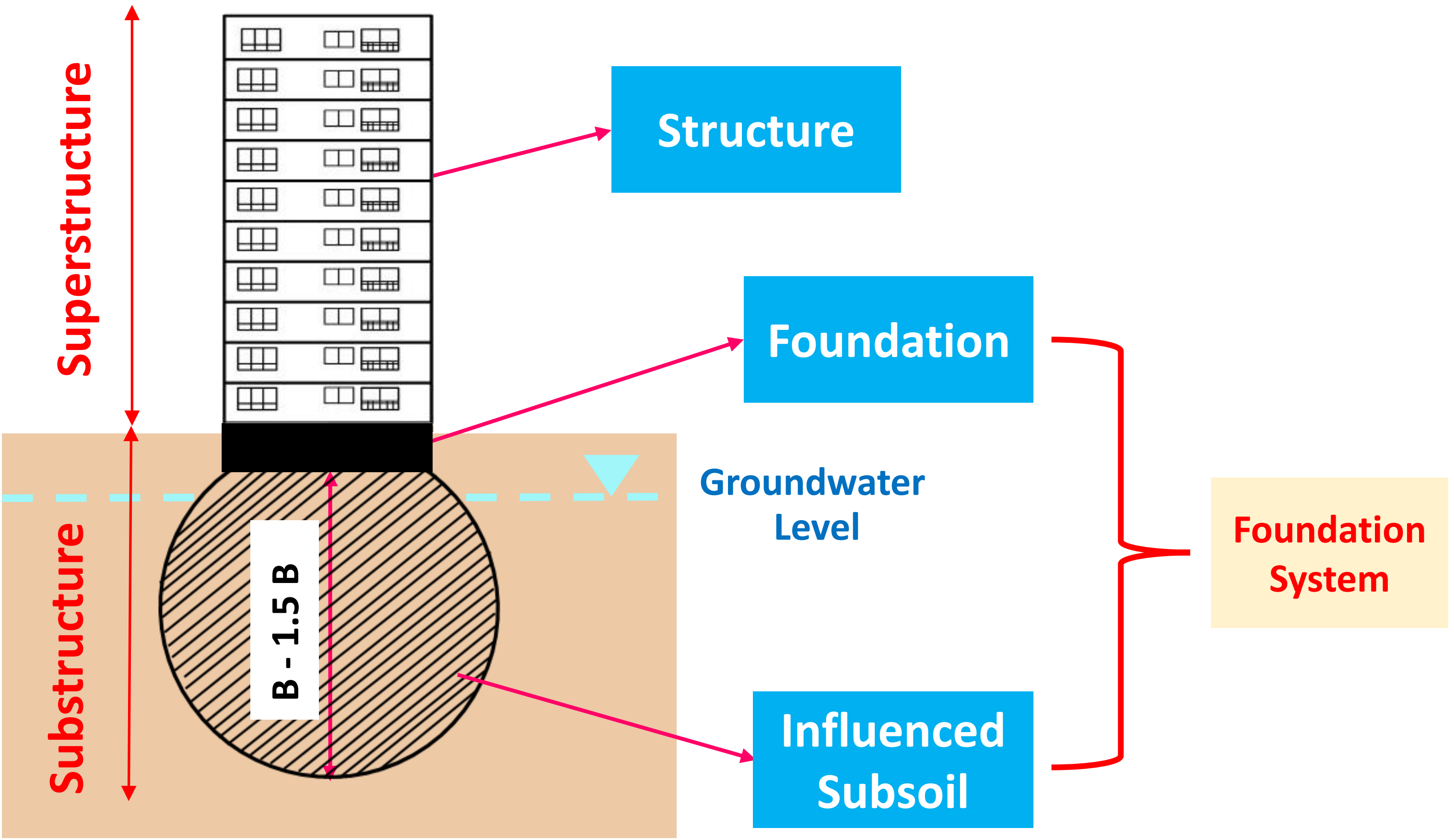
Case No. 3-b: CPT Records for Characterization & Installation Depth (Campanella et al., 1989)

Differentiate the toe (base) resistance of three pile installation depths based on the given CPTu: a) around 10 m; b) around 20 m & c) around 30 m



CPT profiles and pile alternatives: UBC research site (CPT data from Campanella et al., 1989)

Interaction of Superstructure & Substructure

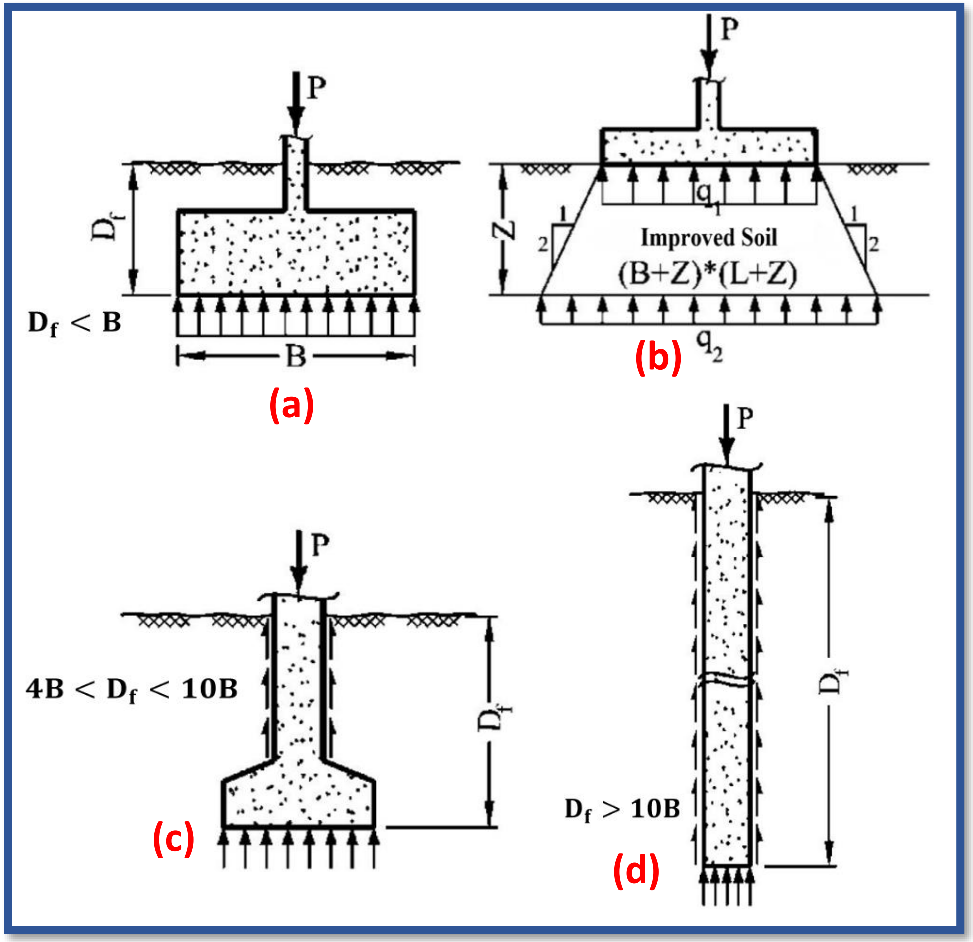


Foundation elements or systems are structural units that transfer load combinations from the superstructure to the underlying geomaterials. Foundation units may tolerate the loads individually or by contribution of other elements such as basement walls, floors, or slabs (Eslami et al., 2020).

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)

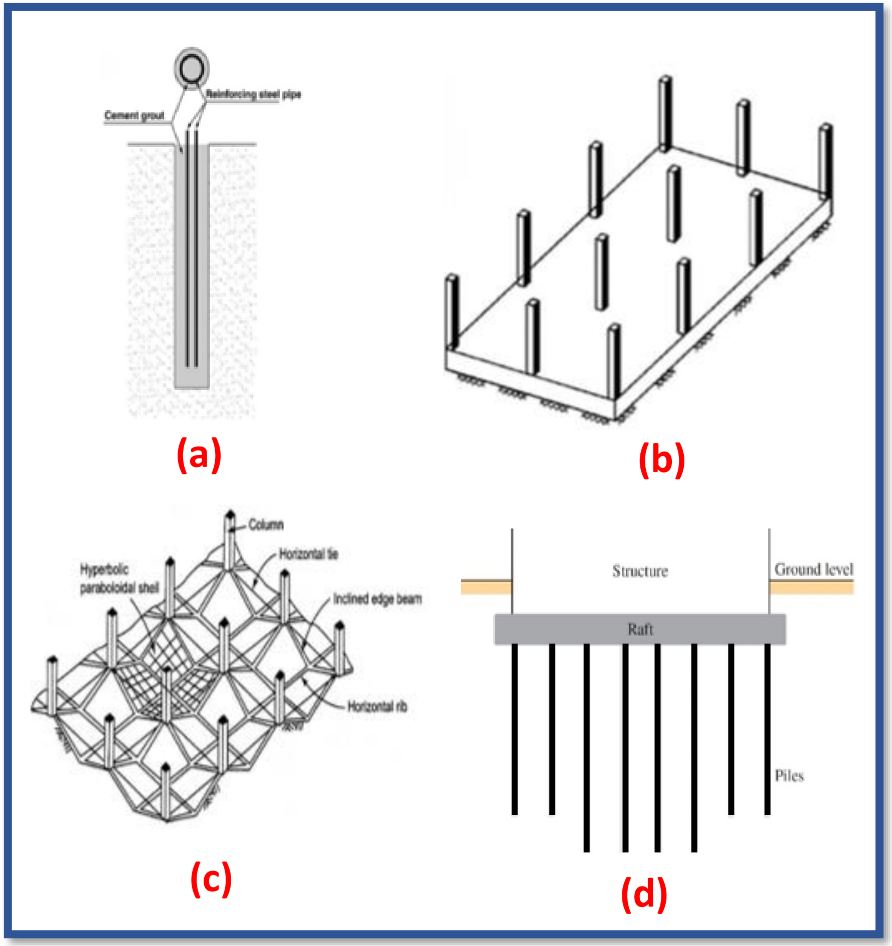
Foundations Classification

❖ Form and Function

- Linear (1D) Foundations
- Planar (2D) Foundations
- Volumetric (3D) Foundations

❖ Load Transfer System

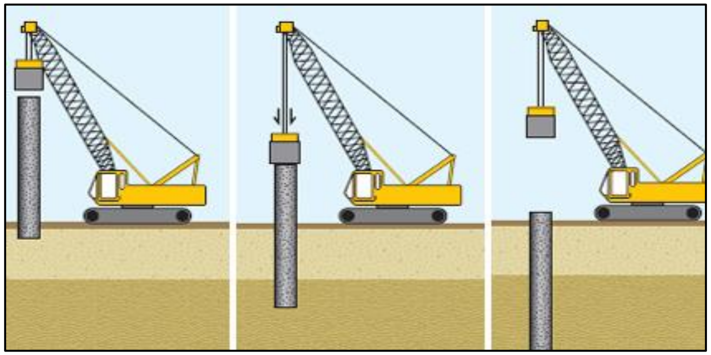
- Vector-acting Foundations (a)
- Section-acting Foundations (b)
- Surface-acting Foundations (c)
- Block-acting (Hybrid) Foundations (d)



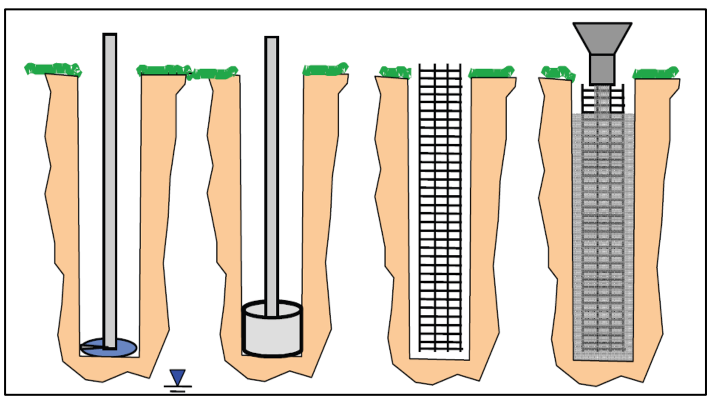
(Eslami et al., 2025)

Different Types of Deep Foundations

❖ Conventional

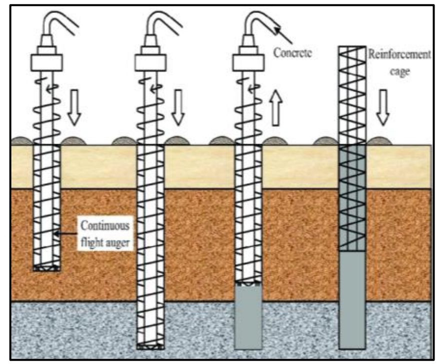


Displacement (Driven Piles)

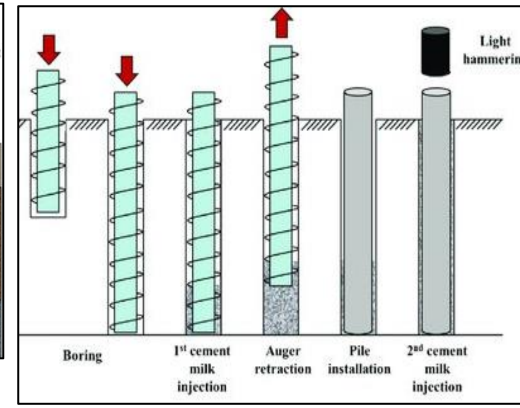


Replacement (Drilled Shafts)

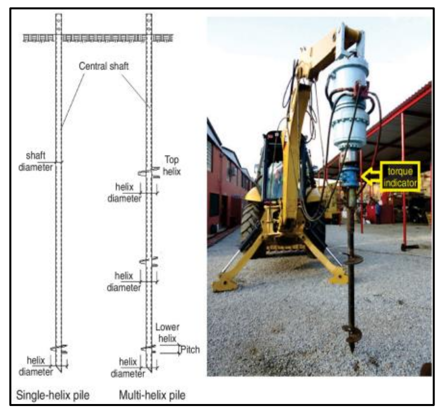
❖ Special



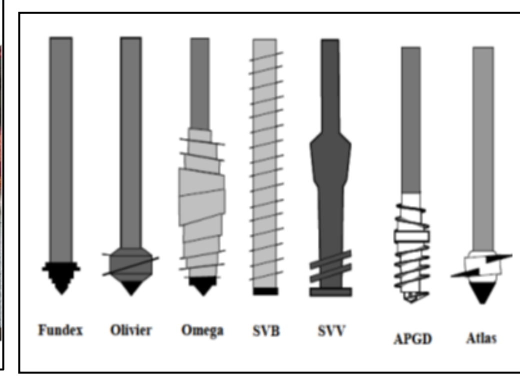
CFA



Prebored & Precast Piles (PPP)



Helical Pile



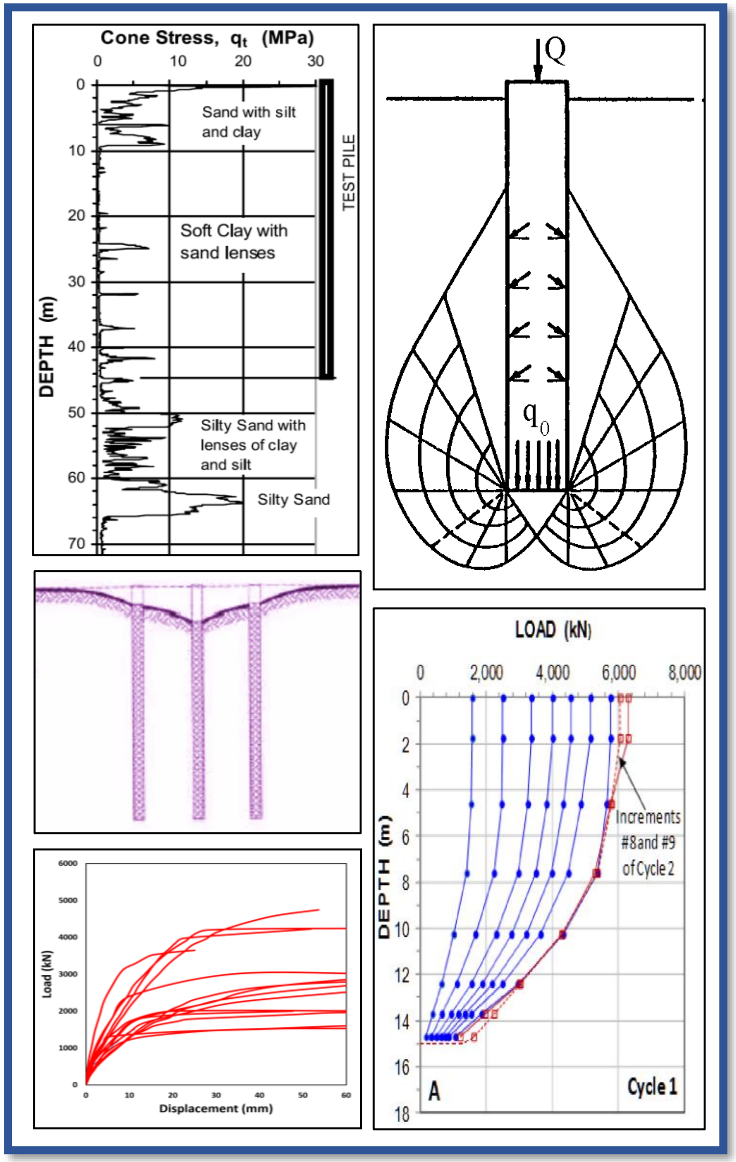
Drilled Displacement Piles (DDP)

When, Where and Why Deep Foundations Are Required

- **Near-surface soils deficient in bearing capacity or vulnerable to scour in riverine, coastal, and offshore environments**
- **To overcome excessive settlement and differential movement of shallow foundations on compressible, soft, or consolidating soil strata**
- **Transfer of large concentrated structural loads to competent bearing strata beyond the shallow founding horizon**
- **Mobilization of axial resistance against uplift, overturning, lateral forces, seismic actions, and combined V–M–H loading**
- **Enhancement of subgrade stiffness and settlement control for dynamic, machine, and high-groundwater construction conditions**
- **Rehabilitation and performance recovery of existing shallow foundations where settlement or capacity has reached critical limits**

Role in Foundation Engineering: Geotechnical Design

1. Selection & Installation
2. Bearing Capacity
3. Resistance Distribution
4. Settlement
5. Load - Displacement



Common Approaches for Bearing Capacity of Deep Foundations

1. Static Analysis

2. In-Situ Tests Application

3. Static Load Test

4. Dynamic Analysis & Load Test

5. Numerical & Laboratory Modelings

Pile Bearing Capacity

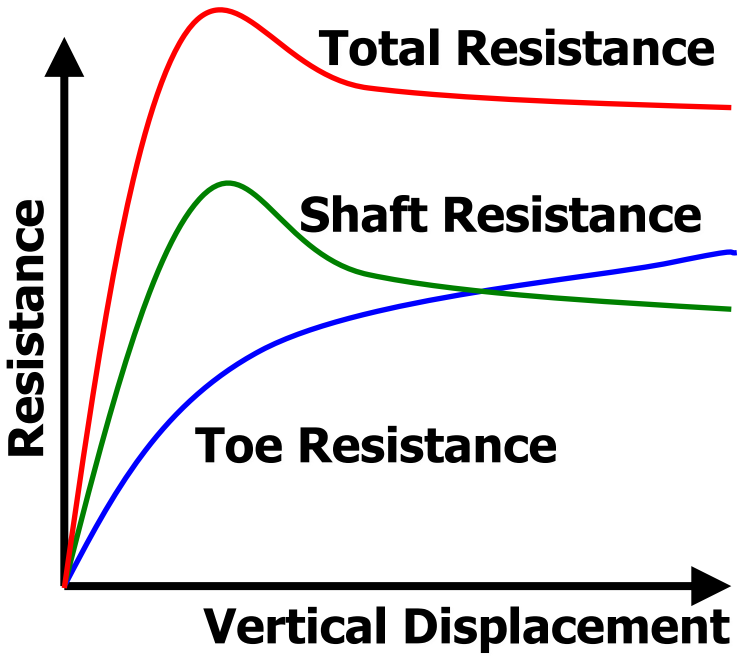
Ultimate Bearing Capacity = **Toe Capacity** + **Shaft Capacity**

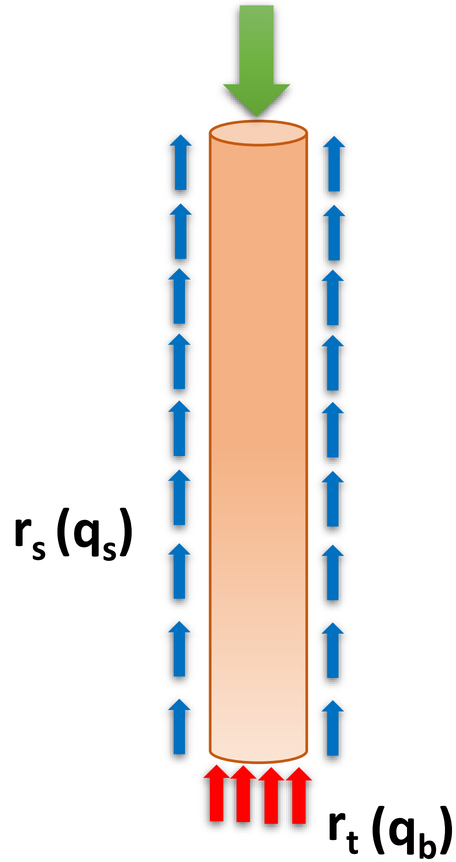
$$R_t = r_t . A_t$$

$$R_s = r_s . A_s . D_f$$

$$R_u = R_t + R_s$$

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





Static Analysis

Unified Pile Design (CFEM)

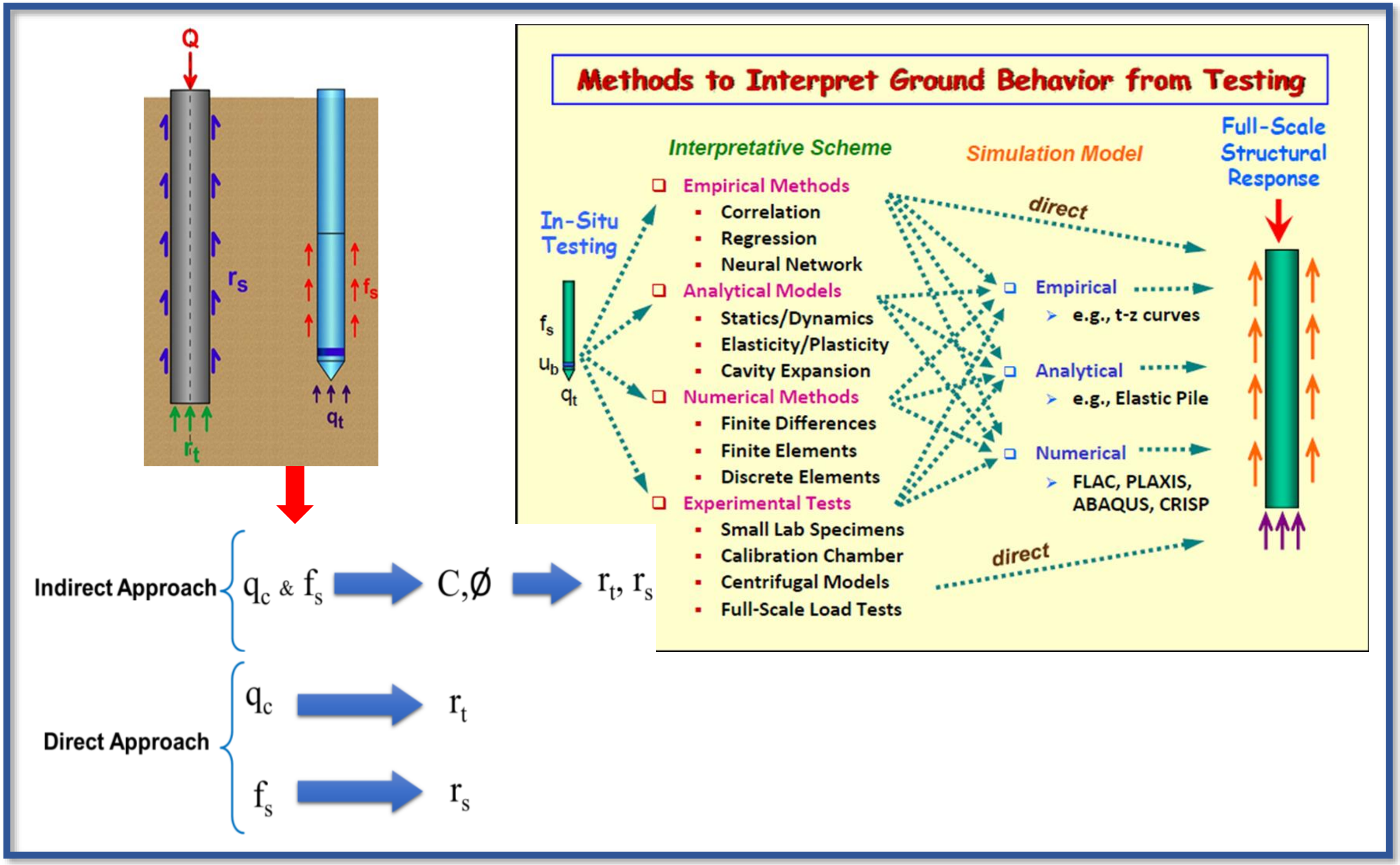
$$q_b = N_t \times \sigma'_{z=D_f}$$
$$q_s = \beta \times \sigma'_{z-avg}$$

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

The values of β and N_t are as given in Table

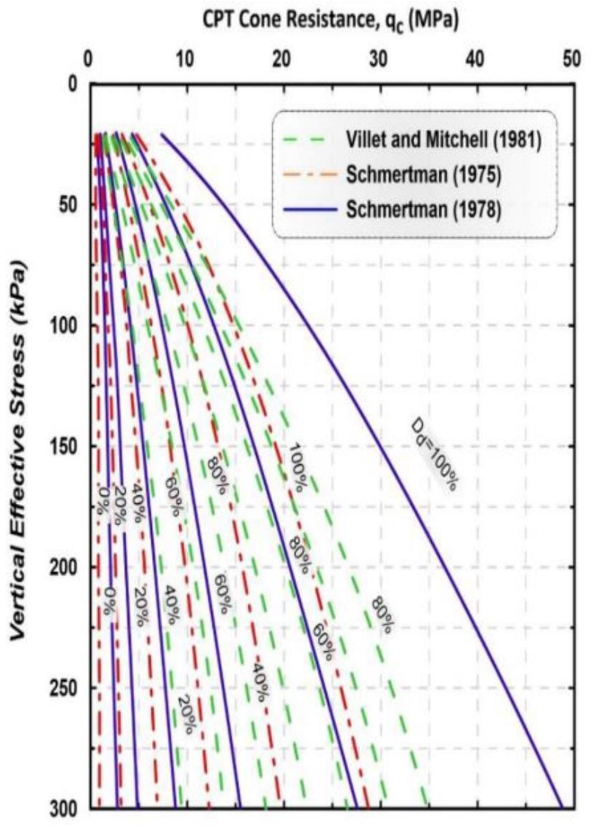
Soil type	Soil friction angle	Drilled Shafts		Driven Piles	
		β	N_t	β	N_t
Clay	25-30	0.25-0.32	3-10	0.25-0.32	3-10
Silt	28-34	0.2-0.3	10-30	0.3-0.5	20-40
Loose sand		0.2-0.4	20-30	0.3-0.8	30-80
Medium sand	32-42	0.3-0.5	30-60	0.6-1	50-120
Dense sand		0.4-0.6	50-100	0.8-1.2	100-120
Gravel	35-45	0.4-0.7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

Direct & Indirect Approach

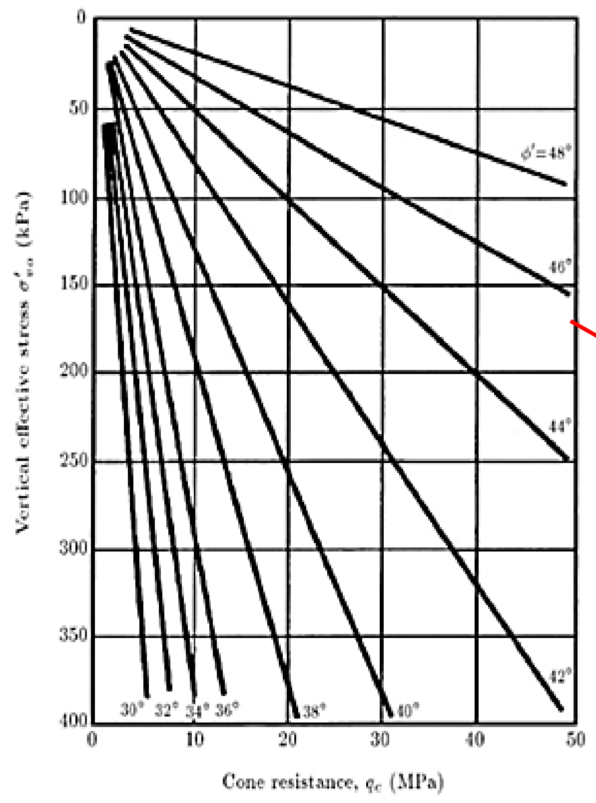


Estimating Soil Engineering Parameters

Relative Density (D_r)



Friction Angle (φ)



(Mitchell & Durgunoglu, 1983)

$q_u = \sigma' N_q$ Eq. 1

$q_u = q_c$ or q_t Eq. 2

$N_q = f(\varphi)$ Eq. 3

Eq. 1, Eq. 2 & Eq. 3 →

$\varphi = f\ (\sigma' \ \& \ q_c)$

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	$E_s = F.q_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$
OCR Sand	$E_s = (6-30)q_c$
Clay Sand	$E_s = (3-6)q_c$
Silty Sand	$E_s = (1-2)q_c$ $q_c < 2500kPa \quad E_s' = 2.5q_c$ $2500 < q_c < 5000kPa \quad E_s' = 4q_c + 5000$
Soft Clay	$E_s = (3-8)q_c$

Undrained Shear Strength (S_u)

Correlations for undrained shear strength of the cohesion of soils

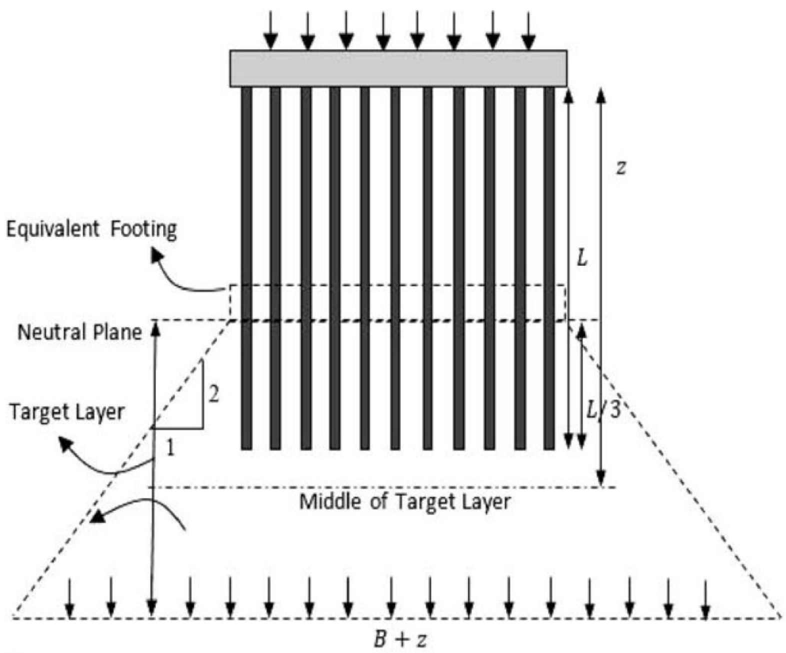
Reference	Correlations	Remarks
Lunne et al. (1997)	$S_u = {(q_c - \sigma_v)}/{N_c}$	N_c: cone factor
Risery (1974)	$S_u = {q_c}/{23}$	-
Kulhawy and Mayne (1990)	$S_u = \frac{\Delta u}{N_{\Delta u}}$	Δu = excess pore pressure measured at u_2 position = $u_2 - u_0$ $N_{\Delta u}$ = Pore pressure cone factor $N_{\Delta u} = N_{kt}$ 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

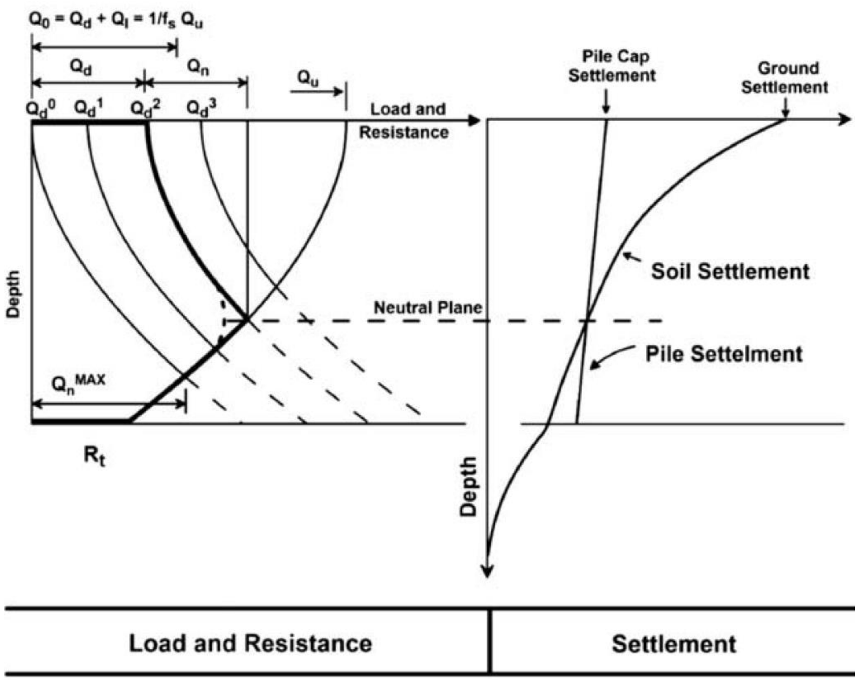
$$q_u = S_u N_C + \sigma \qquad \text{Eq. 1}$$

$$q_u = q_t \qquad \text{Eq. 2}$$

$$\text{Eq. 1 \& Eq. 2} \rightarrow S_u = \frac{q_t - \sigma}{N_C}$$

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

Schmertmann (1978)

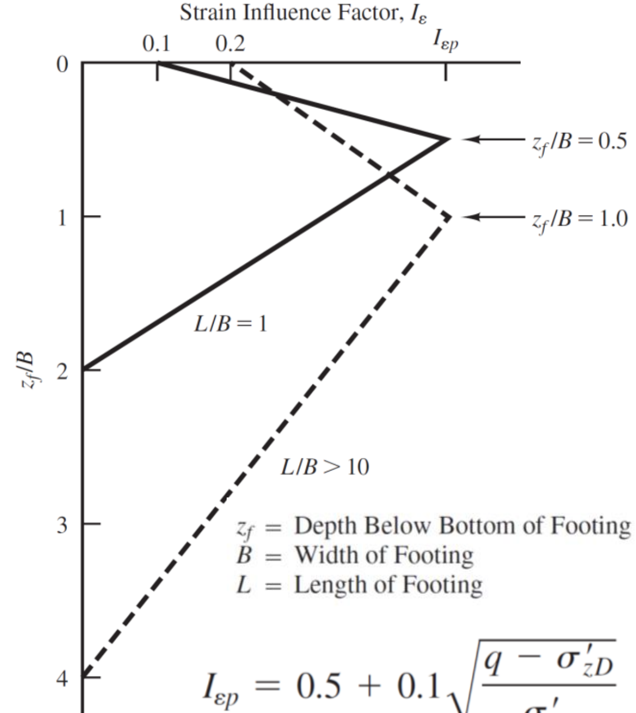
$$\delta = C_1 C_2 C_3 (q - \sigma'_{zD}) \sum \frac{I_\varepsilon H}{E_s}$$

$$C_1 = 1 - 0.5\left(\frac{\sigma'_{zD}}{q - \sigma'_{zD}}\right)$$

$$C_2 = 1 + 0.2 \log\left(\frac{t}{0.1}\right)$$

$$C_3 = \max\begin{bmatrix} 1.03 - 0.03L/B \\ 0.73 \end{bmatrix}$$

δ = settlement of footing
C_1 = depth factor
C_2 = secondary creep factor
C_3 = shape factor
q = bearing stress
σ'_{zD} = effective vertical stress at a depth D below the ground
I_ε = influence factor at midpoint of soil layer
H = thickness of soil layer
E_s = equivalent modulus of elasticity of soil layer
t = time since application of load (year) ($t \geq 0.1$ year)
B = footing width
L = footing length

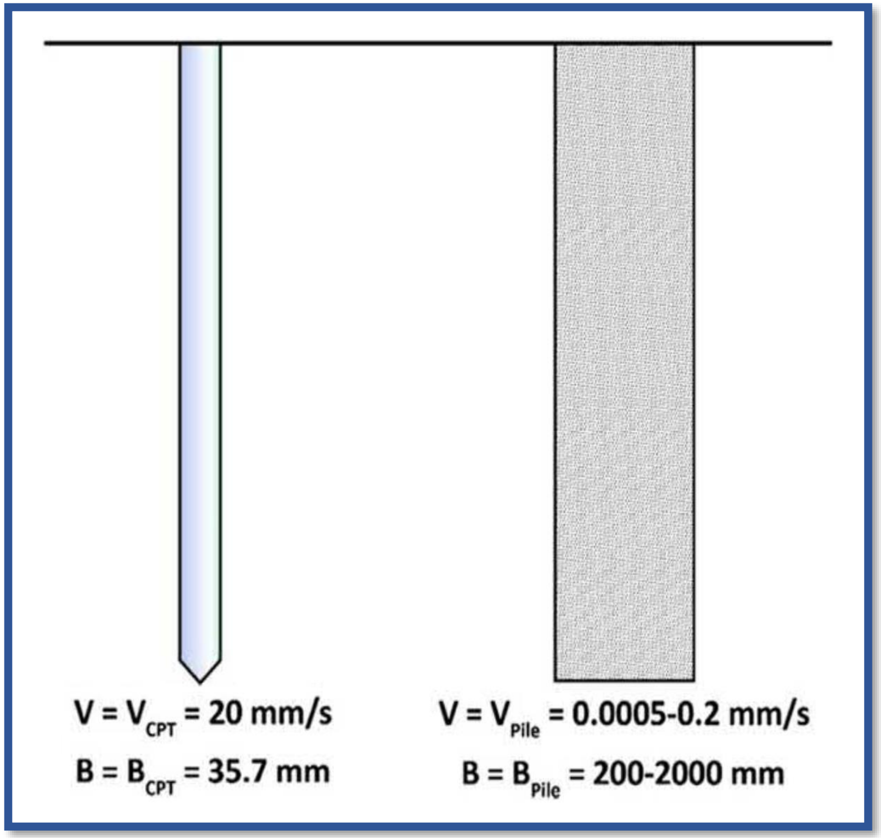


$$I_{\varepsilon p} = 0.5 + 0.1\sqrt{\frac{q - \sigma'_{zD}}{\sigma'_{zp}}}$$

$I_{\varepsilon p}$ = peak strain influence factor
q = bearing stress
σ'_{zD} = initial vertical effective stress at a depth D below the ground surface
σ'_{zp} = initial vertical effective stress at depth of peak strain influence factor (for square and circular footings ($L/B = 1$), compute σ'_{zp} at a depth of $D + B/2$ below the ground surface; for continuous footings ($L/B \geq 10$), compute it at a depth of $D + B$)

Scale Effect Correlations: Determinant Factors

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

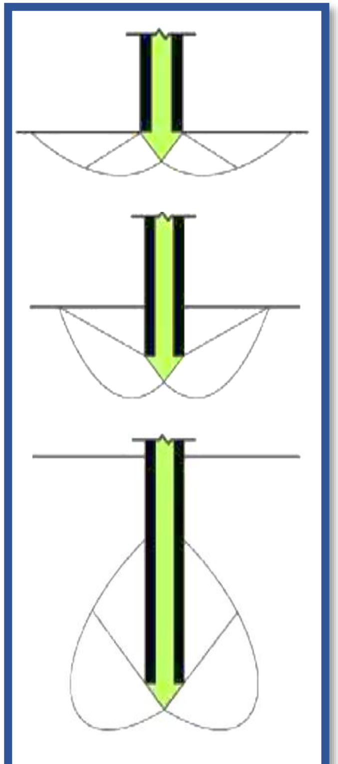


Schematic view of pile and cone penetration test differences in material, penetration rate, and dimensions (Eslami et al., 2020)

Penetrometers can be realized as a *model pile*

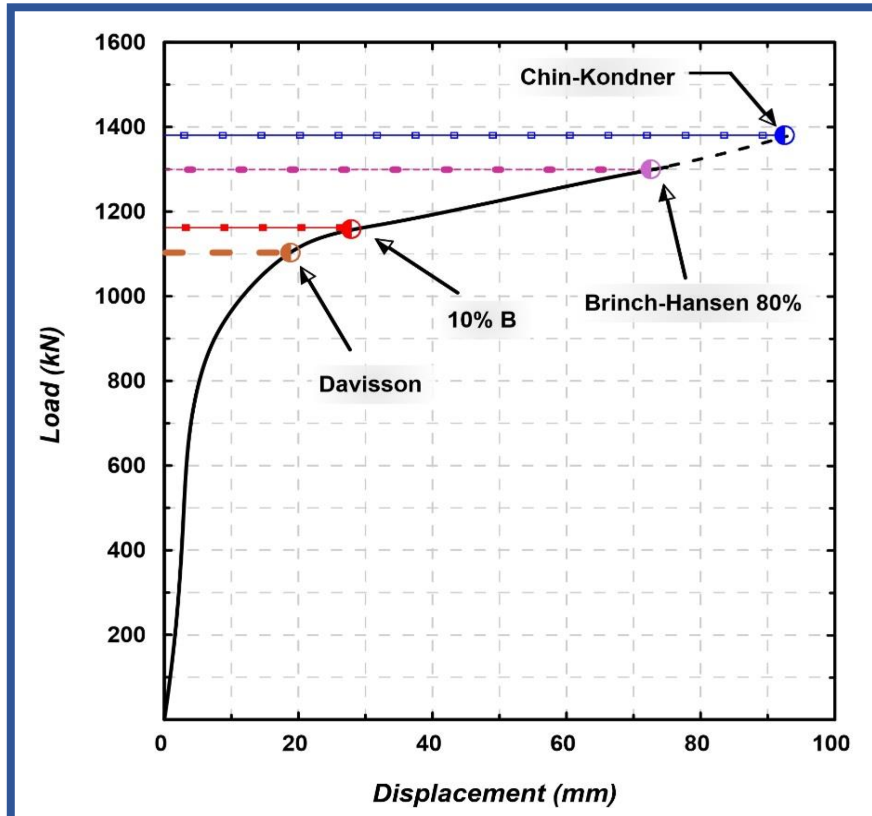
Scale Effect Correlations: Load-Displacement

❖ Embedment Depth



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

❖ Ultimate Capacity Condition

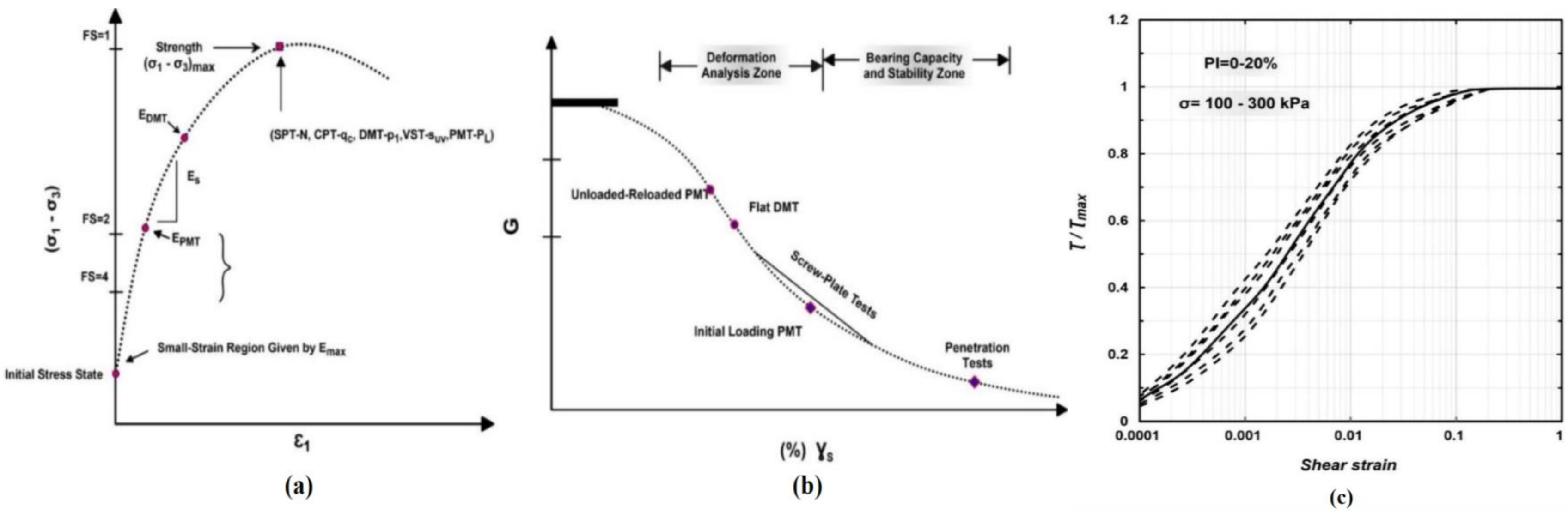


Interpretation of load displacement diagram for Case 001-L&D31 (Moshfeghi & Eslami, 2016)

Scale Effect Correlations: Shaft Resistance

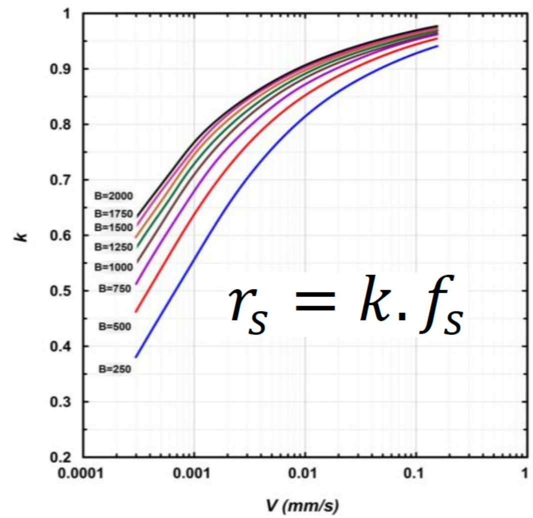
$$\frac{\gamma_{pile}}{\gamma_{CPT}} = \left(\frac{V_{pile}}{V_{CPT}}\right)^{0.61} . \left(\frac{D_{pile}}{D_{CPT}}\right)^{0.5}$$

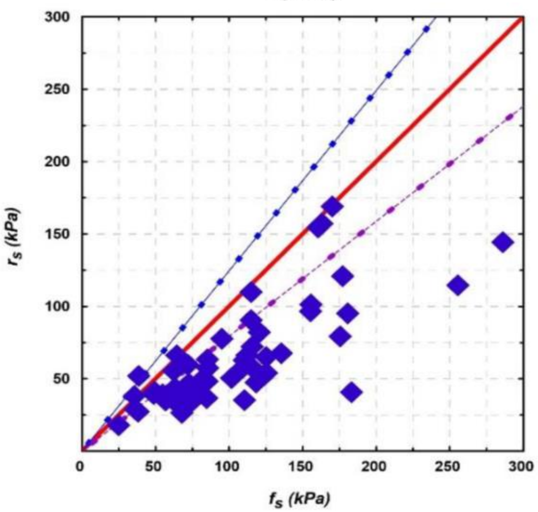
$$\gamma_{pile} = 0.0255\,(V_{pile})^{0.61} . (D_{pile})^{0.5}$$



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: Shaft Resistance (Eslami et al., 2020)



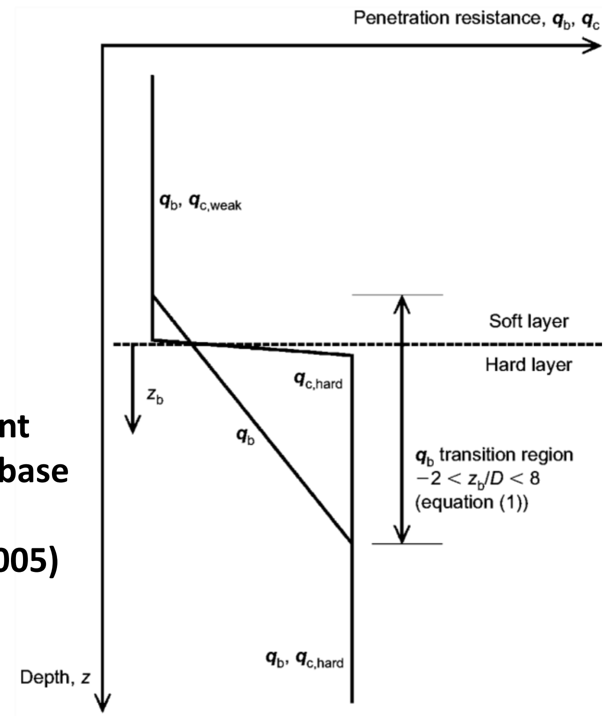


Scale Effect: Toe Resistance (White & Bolton, 2005)

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

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

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)

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

LCPC (French) Method, (Bustamante & Gianeselli, 1982)

$$r_t = k_c q_{ca}$$

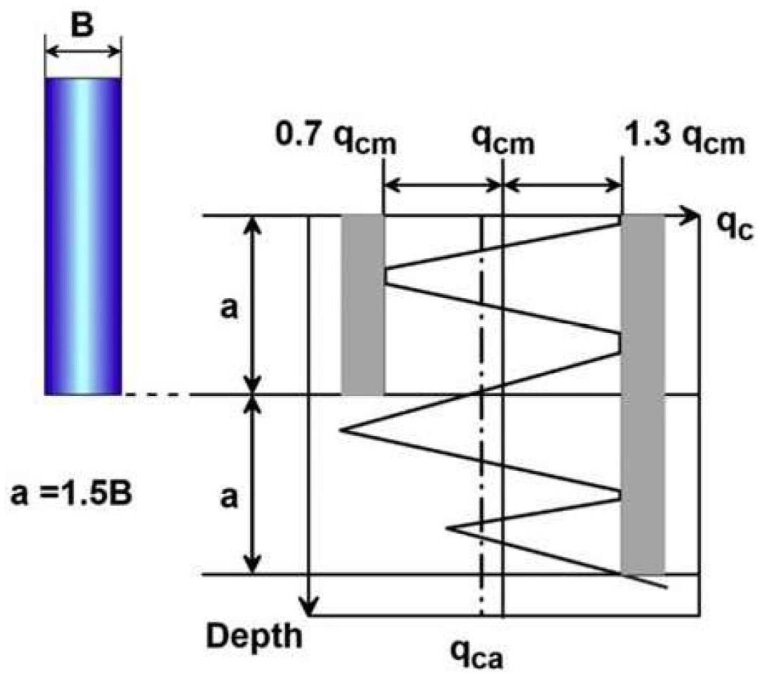
$$r_s = \frac{1}{\alpha} q_c$$

k_c : Toe resistance factor

q_{ca}: Average tip resistance for toe

α : Shaft resistance factor

q_c : Average tip resistance for shaft



Method for determining equivalent q_c

LCPC (French) Method, (Bustamante & Gianeselli, 1982)

Toe Resistance Factor; k_c

Soil type	q_c (MPa)	k_c Group I	k_c Group II
Soft clay and mud	<1	0.4	0.5
Moderately compact clay	1–5	0.35	0.45
Silt and loose sand	≤5	0.4	0.5
Compact to stiff clay and compact silt	>5	0.45	0.55
Soft chalk	≤	0.2	0.3
Moderately compact sand and gravel	5–12	0.4	0.5
Weathered to fragmented chalk	>5	0.2	0.4
Compact to very compact sand and gravel	>12	0.3	0.4

Note: Group I: Plain bored piles, mud bored piles, micropiles (grouted under low pressure), cased bored piles, hollow auger-bored piles, piers, and barrettes. Group II: Cast-in-place screwed piles, driven precast piles, prestressed tubular piles, driven cast piles, jacked metal piles, micropiles (grouted under high pressure with diameters < 250 mm).

LCPC (French) Method, (Bustamante & Gianeselli, 1982)

Shaft Resistance Factor; α

		Coefficient, α				Maximum limit of q_s (MPa)					
		Category									
		I		II		I		II		III	
Soil type	q_c (MPa)	A	B	A	B	A	B	A	B	A	B
Soft clay and mud	<1	30	90	90	30	0.015	0.015	0.015	0.015	0.035	—
Moderately compact clay	1–5	40	80	40	80	0.035 (0.08)	0.035 (0.08)	0.035 (0.08)	0.035	0.08	≥0.12
Silt and loose sand	≤5	60	150	60	120	0.035	0.035	0.035	0.035	0.08	—
Compact to stiff clay and compact silt	>5	60	120	60	120	0.035 (0.08)	0.035 (0.08)	0.035 (0.08)	0.035	0.08	≥0.20
Soft chalk	≤5	100	120	100	120	0.035	0.035	0.035	0.035	0.08	—
Moderately compact sand and gravel	5–12	100	200	100	200	0.08 (0.12)	0.035 (0.08)	0.08 (0.12)	0.08	0.12	≥0.2
Weathered to fragmented chalk	>5	60	80	60	80	0.12 (0.15)	0.08 (0.12)	0.12 (0.15)	0.12	0.15	≥0.2
Compact to very compact sand and gravel	>12	150	300	150	200	0.12 (0.15)	0.08 (0.12)	0.12 (0.15)	0.12	0.15	≥0.2

Note: Bracketed values of maximum limit unit skin friction, q_s, apply to careful execution and minimum disturbance of soil due to construction. Categories: IA, plain bored piles, mud bored piles, hollow auger bored piles, micropiles (grouted under low pressure), cast-in-place screwed piles, piers, and barrettes; IB, cased bored piles, driven piles; IIA, driven precast piles, prestressed tubular piles, jacked concrete piles; IIB, driven metal piles and jacked metal piles; IIIA, driven grouted piles and driven rammed piles; IIIB, high pressure grouted piles with diameters >250 mm and micropiles grouted under high pressure.

UniCone Method, Eslami & Fellenius (1997)

Toe Capacity

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

$$\mathbf{c_t = 1}$$

$$\mathbf{q_E = q_t - u_2}$$

$$\mathbf{q_t = q_c + (1 - a)u_2}$$

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

Fellenius (2002):

Adjustment factor:

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

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

UniCone Method, Eslami & Fellenius (1997)

Shaft Capacity

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

$$q_{Eg} = \text{Geometrical} - \text{mean within proposed layer}$$

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

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

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

$$\text{Eq. 1 \& Eq. 2} \rightarrow \frac{r_s}{r_t} = \frac{\beta}{N_t} \qquad \text{Eq. 4}$$

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

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

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%

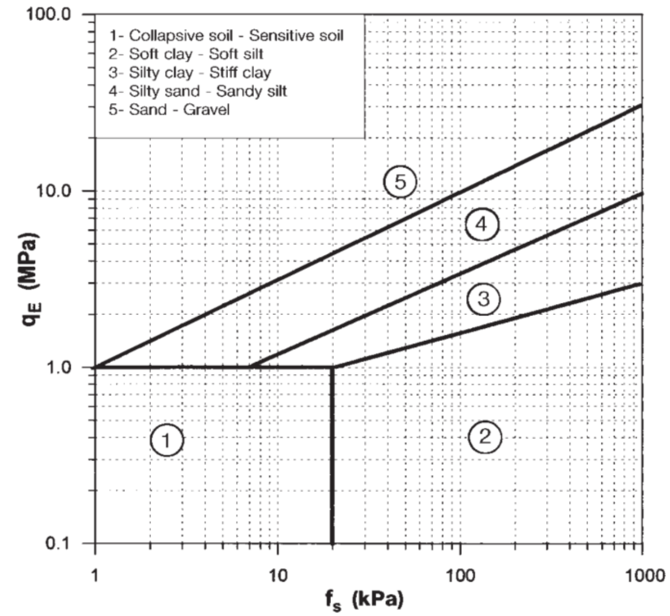
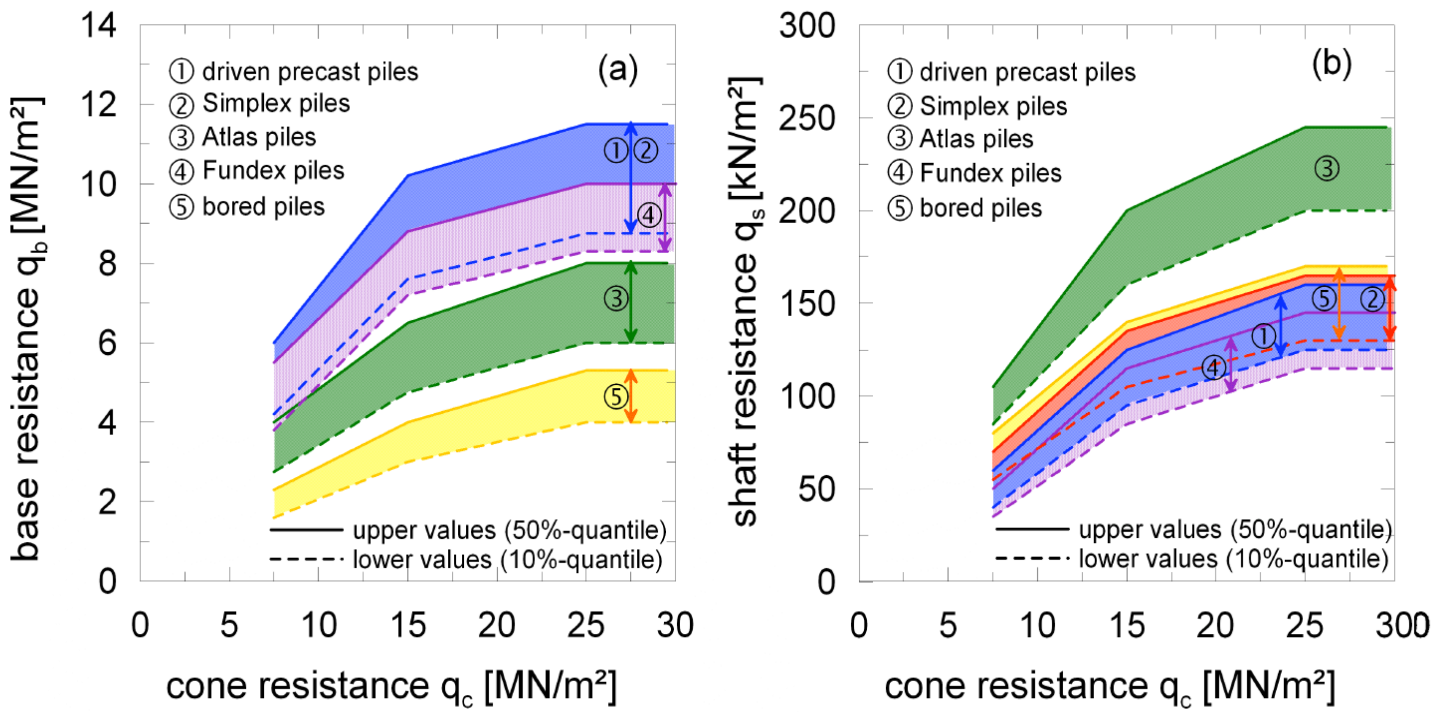


Chart for soil classification (Eslami & Fellenius, 1997)

German Method (Kempfert & Becker, 2010)



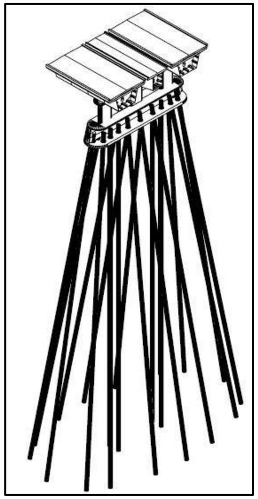
1000 case records

Case No. 4: Urmia Lake Causeway: Long Piles & Super Soft Deposits (Eslami et al., 2011, 2019 & 2024)

- Decreasing the existing route: 300 to 120 km
- Total length of 1260 m
- 19 spans
- 100 m in length for the main span
- 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 for static & dynamic tests

(a)

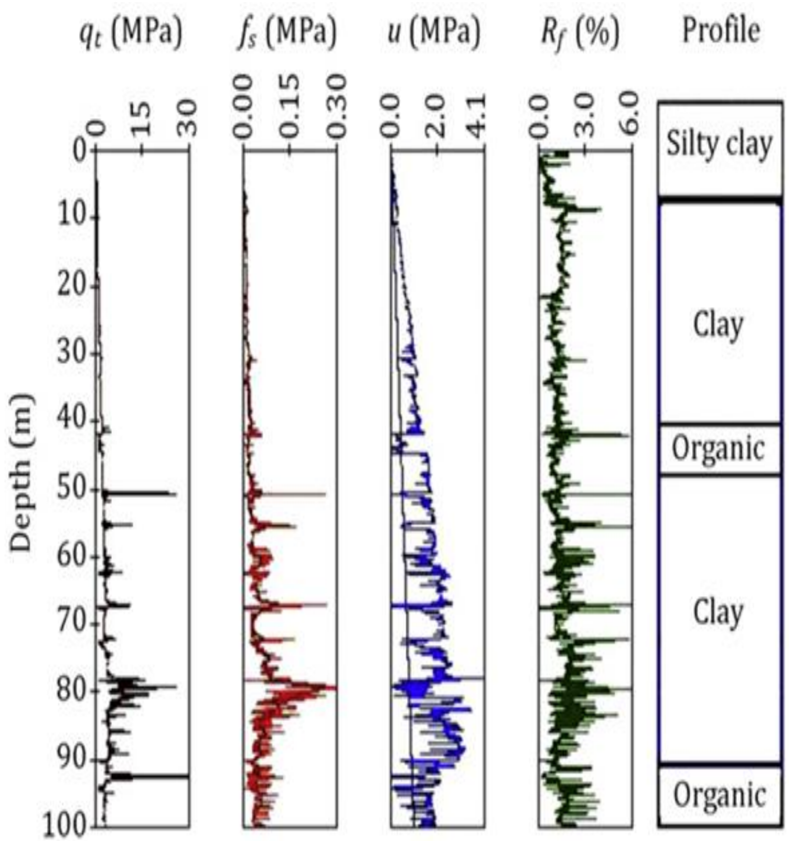
(b)



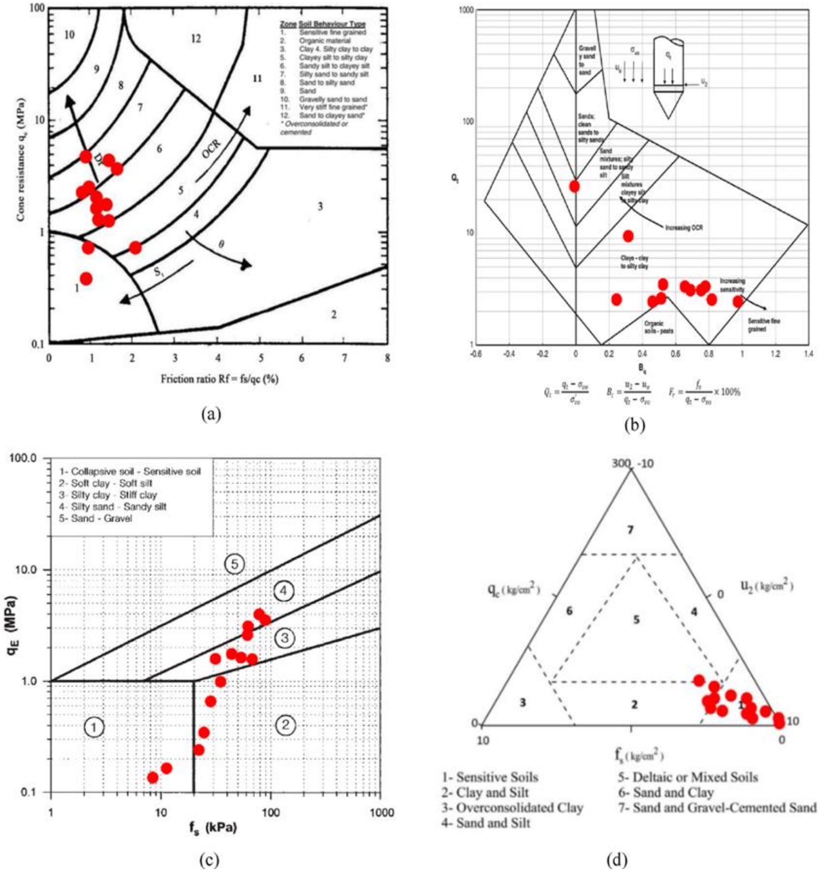
(c)

a) Location, b) Longitudinal view of the causeway, c) Configuration of installed piles

Case No. 4: Continued – Profiling & SBC

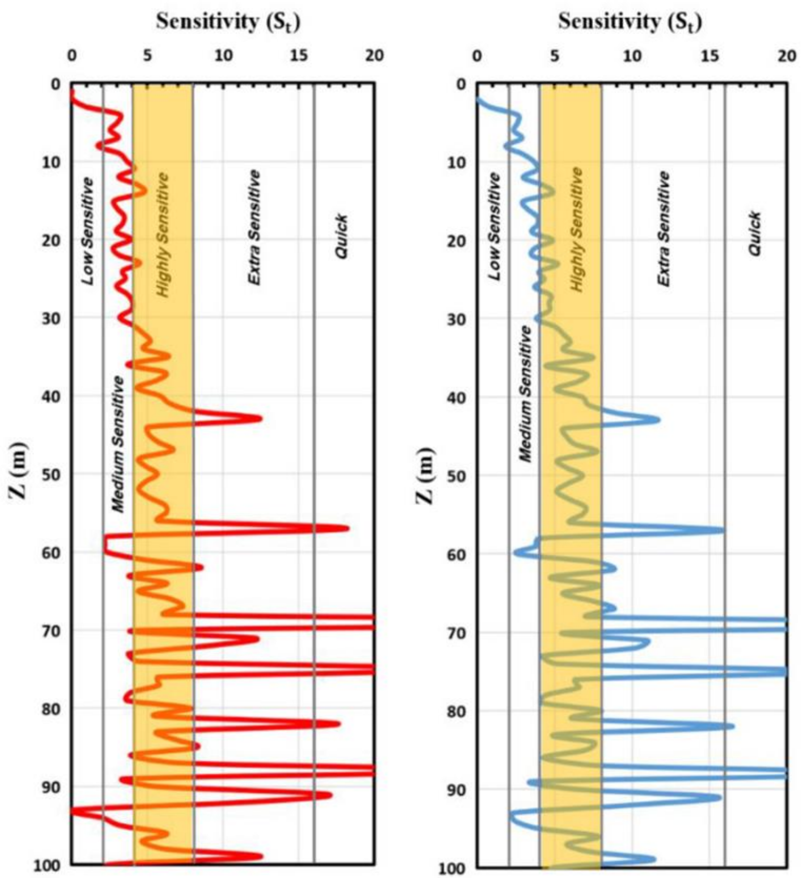


Typical CPT logs



SBC charts via a) Robertson et al. (1986), b) Robertson (1990), c) Eslami & Fellenius (1997), d) triangular chart (Eslami et al., 2023)

Case No. 4: Continued - Sensitivity



$$S_t = \frac{S_{u-undisturbed}}{S_{u-remolded}}$$

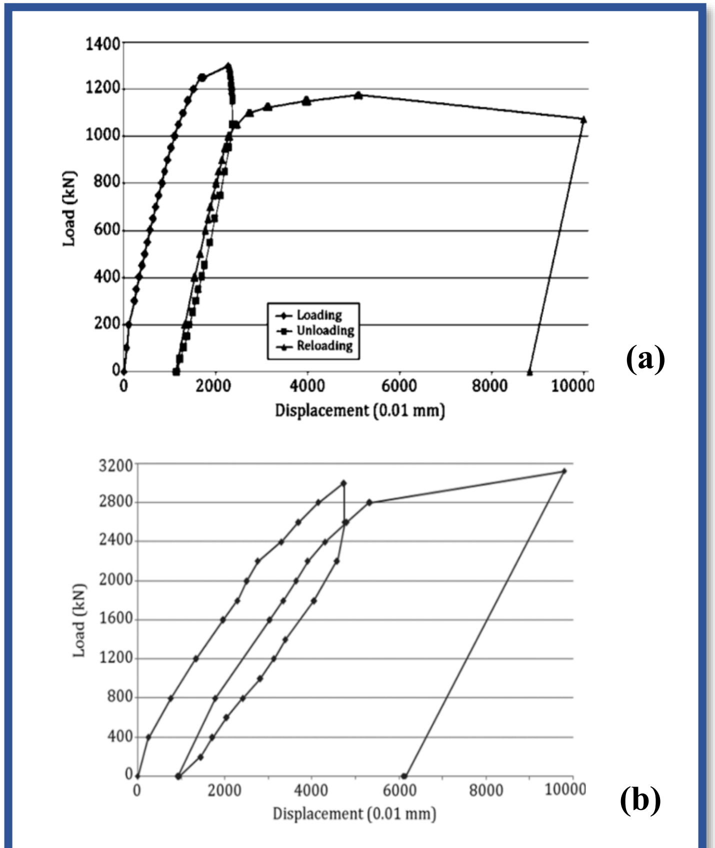
$$S_u = \frac{q_t - \sigma_{v0}}{N_k}$$ *(Mayne & Peuchen, 2018)*

$$S_{u-remolded} = f_s$$ *(Eslami et al., 2020)*

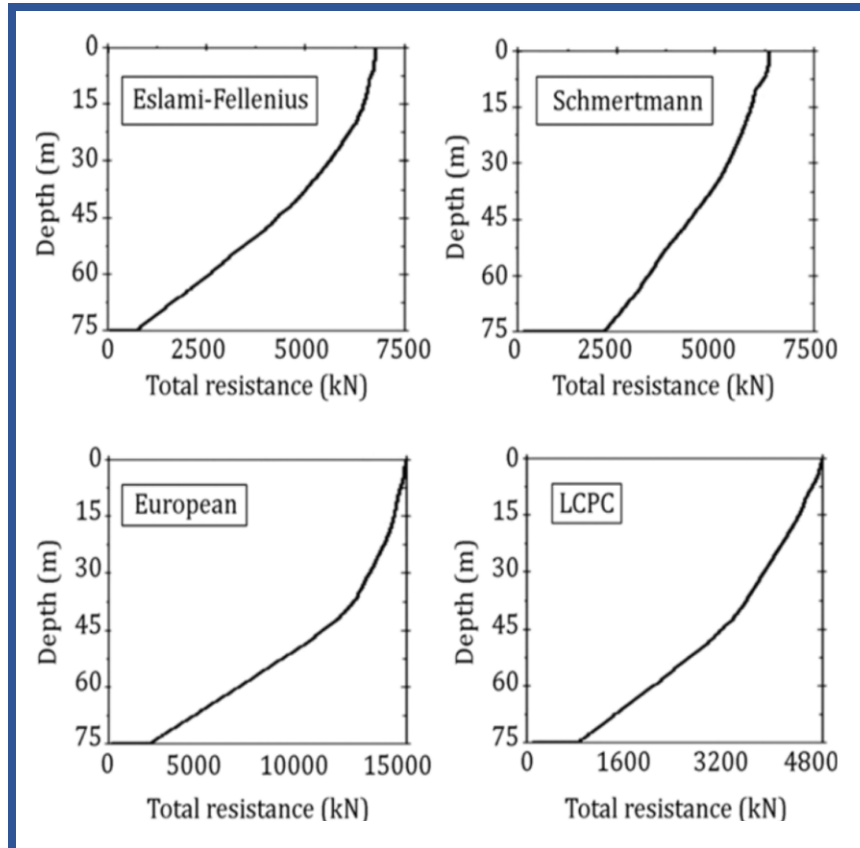
$$S_t = \frac{N_s}{R_f}$$ *(Schmertmann, 1978)*

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

Case No. 4: Continued – Load Tests & Capacity Distribution



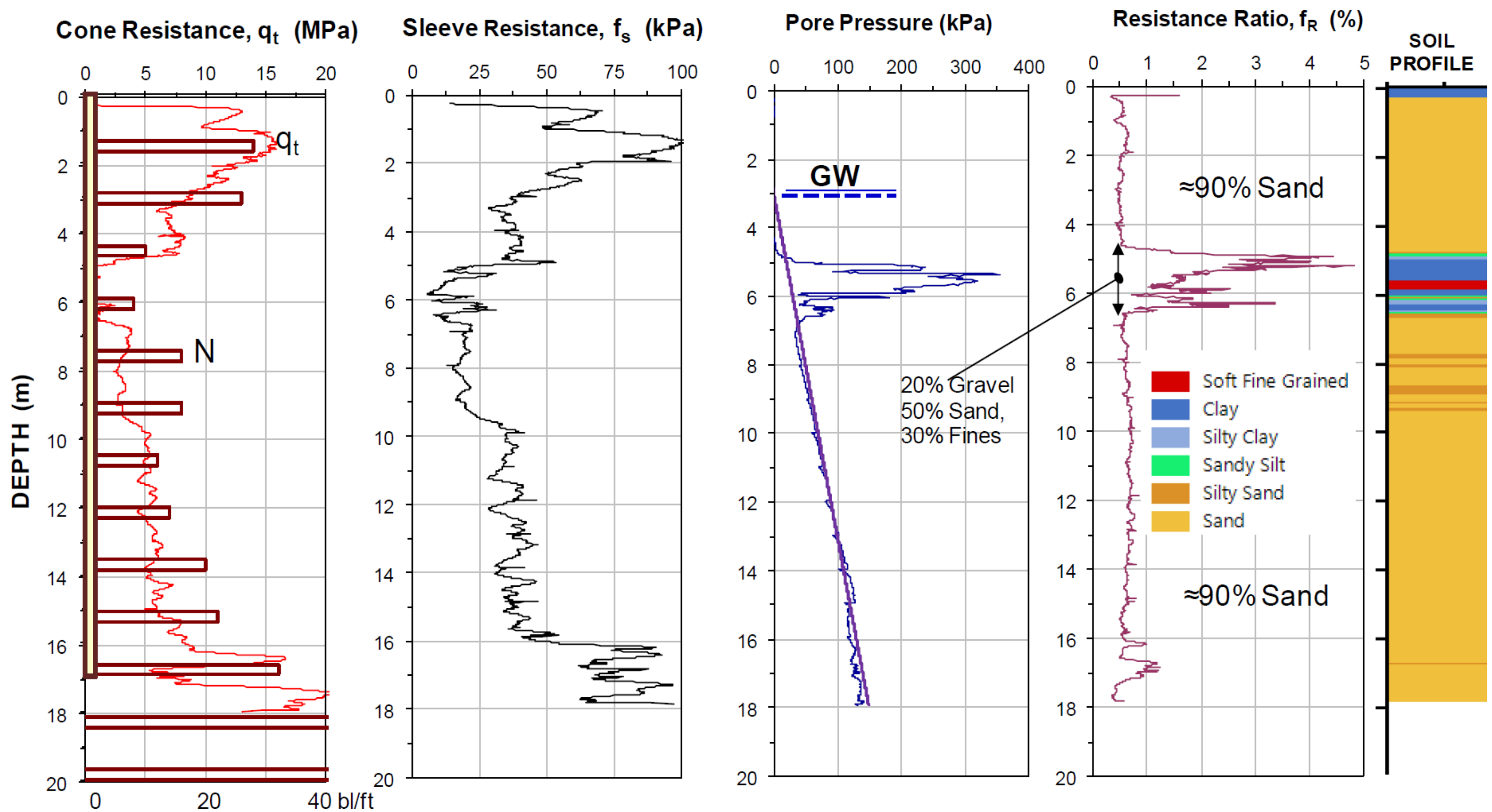
Static load test result; a) Compressive (length of 30 m, diameter of 356 mm), b) Tension (length of 70 m, diameter of 305 mm (Eslami et al., 2011)



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

Case No. 5: Pile Prediction Event, Alabama (Fellenius, 2025)

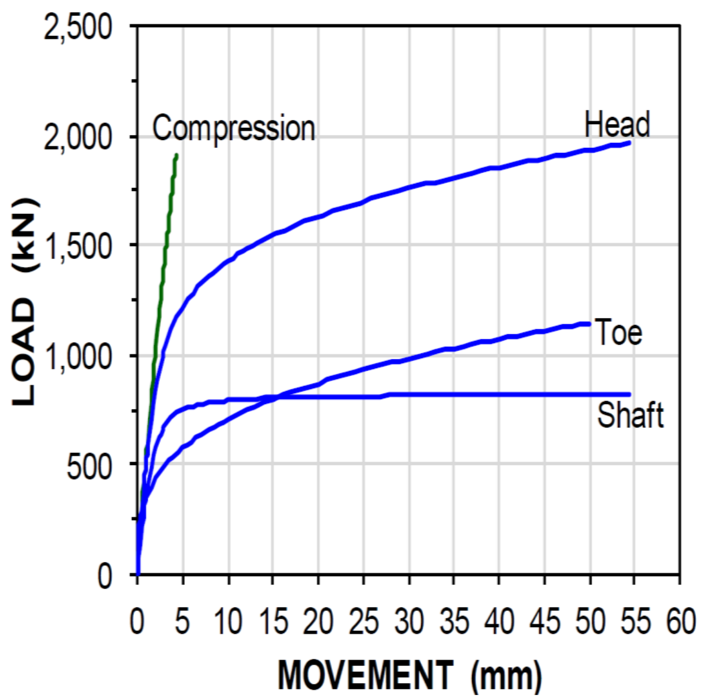
The test comprises a 457 mm diameter, 9.5 mm wall (18-inch, 0.375 inch), closed-toe, steel-pipe pile to be driven to 17 m (56 ft) depth (stick-up is about 3 ft) and grouted after driving.



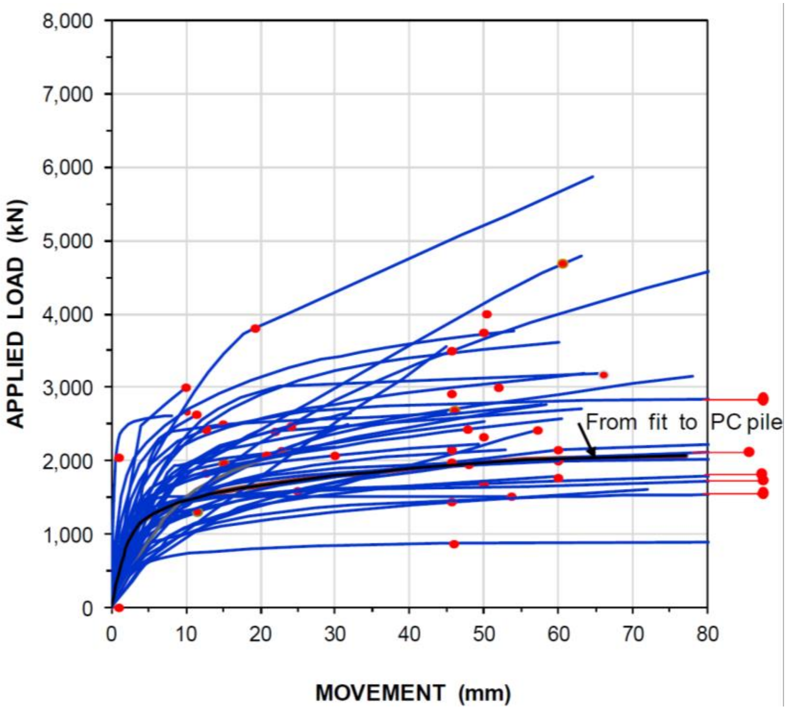
Soil profiles with CPTu data and N-indices

Case No. 5: Continued – Predicted Load-Displacements

- Attracted invitation response: total of 94 individuals from 26 countries.
- Separate received predictions :74, mostly (52) from USA, Canada, and Brazil

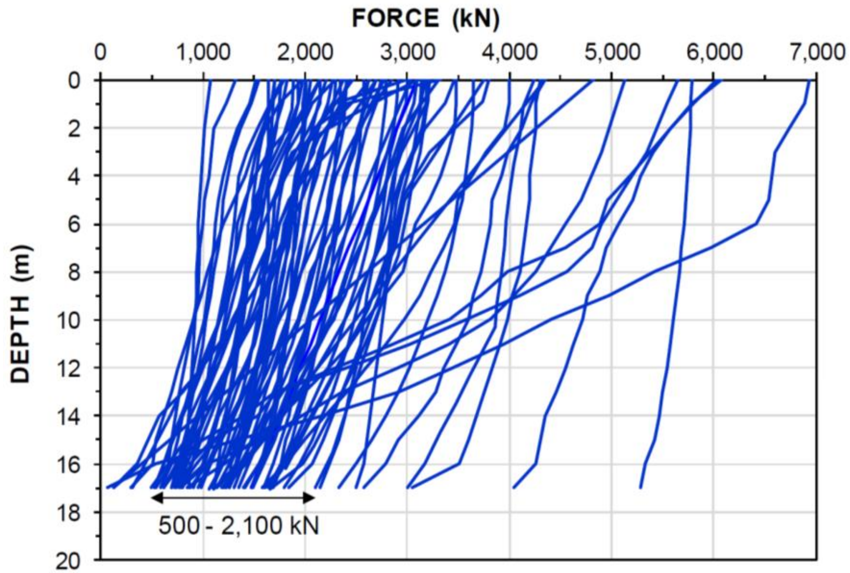


Load-movement curves for the prediction pile calculated for unchanged input by Prof. Fellenius



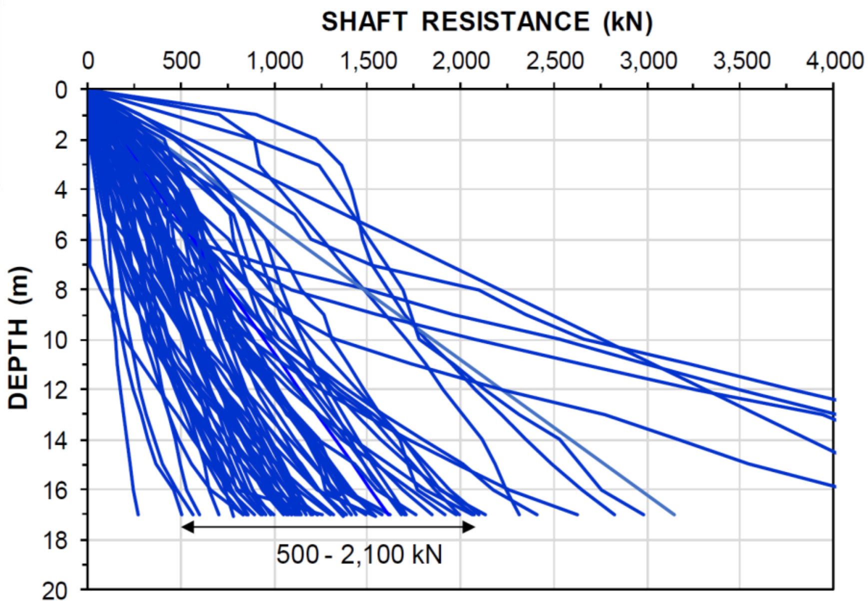
Submitted revised load-movement curves and associated "capacities" by predictors

Case No. 5: Continued – Predicted Load Distributions



Submitted force distributions at an applied load equal to "capacity"

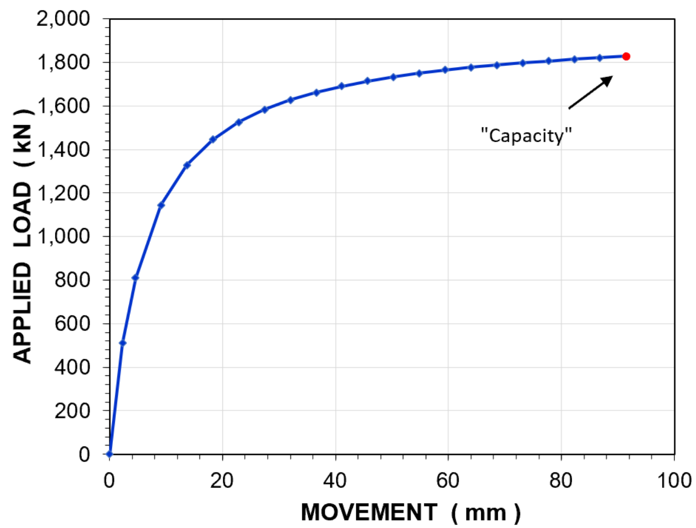
Distributions of shaft resistance extracted from the records

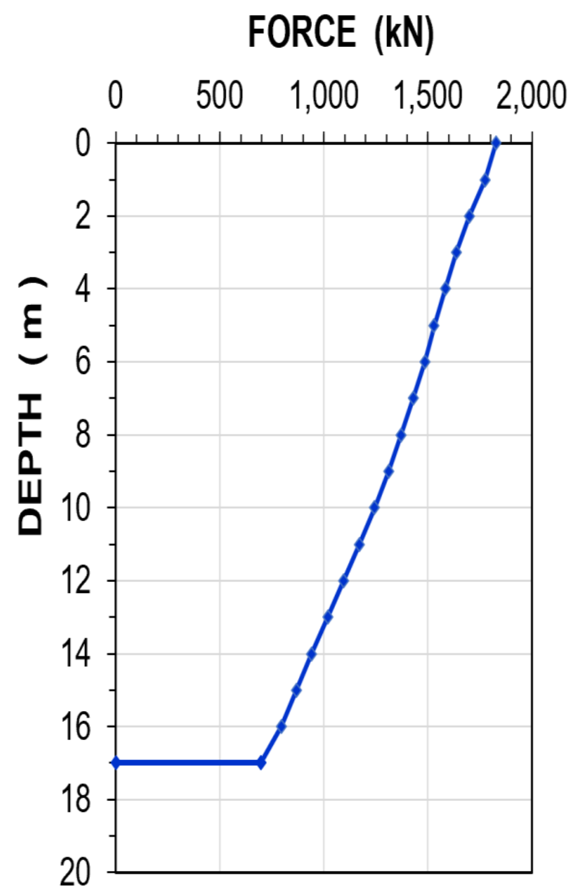


Case No. 5: Continued – Sample Prediction

Prediction through CPT-Based Methods (Heidarie Golafzani & Ebrahimipour, 2025)

#	Methods	Year	Qt (kN)	Qs (kN)	Qult (kN)
1	Modified Unicone	2016	434	1225	1659
2	German-Upper	2010	1064	663	1727
3	German-Lower	2010	745	486	1232
3	ICP	2005	1093	945	2038
4	UWA	2005	734	942	1676
5	UniCone	1997	964	1028	1992
6	LCPC	1983	1082	858	1940
7	Dutch	1979	734	573	1307
8	Schmertman	1978	1224	903	2127
9	Meyerhof	1976	1670	780	2450





Case No. 6: Probabilistic Comparative Assessment of Static Analysis, SPT and CPT-Based Methods (Heidari et al., 2020)

Geotech Geol Eng
https://doi.org/10.1007/s10706-020-01346-x

ORIGINAL PAPER

Probabilistic Assessment of Model Uncertainty for Prediction of Pile Foundation Bearing Capacity; Static Analysis, SPT and CPT-Based Methods

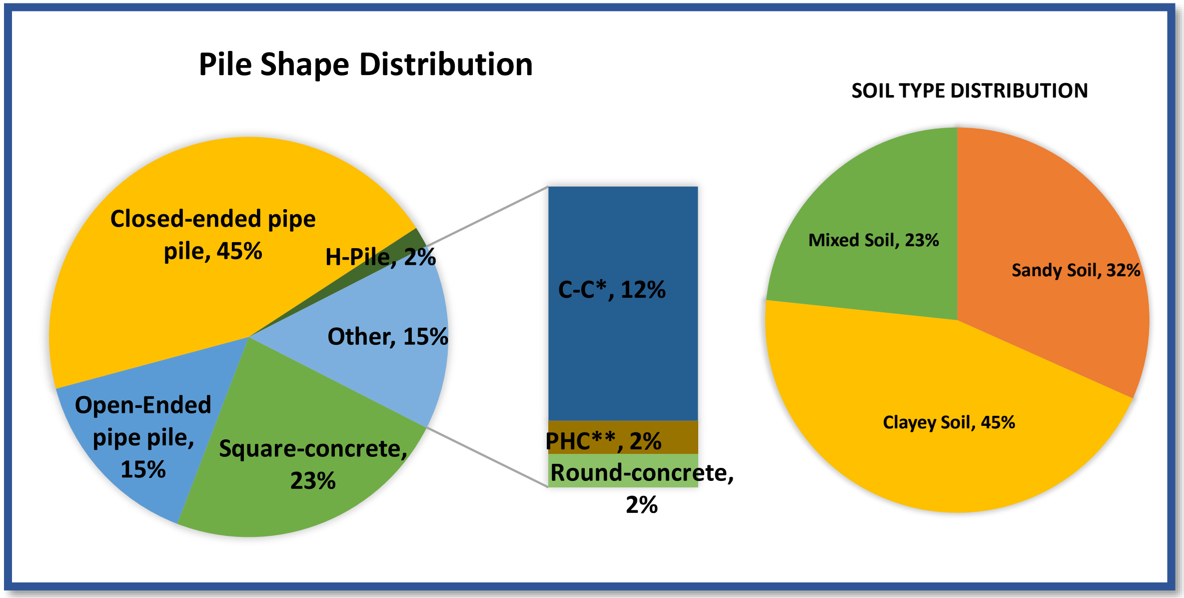
Sara Heidarie Golafzani · Abolfazl Eslami · Reza Jamshidi Chenari

Abstract Geotechnical designs, like other engineering disciplines, are always accompanied by uncertainties. Supplying continuous and reliable records and reducing the uncertainty associated with measurement errors, the cone penetration test (CPT) enhances the geotechnical designs to a more decent level. Deep foundation design, as an essential challenge of foundation engineering, is also involved with different sources of uncertainty. Moreover, the presence of various design methods, relying on different assumptions and requirements, introduces further complications to the selection of an appropriate method, which leads to the broad spectrum of the predictions. Hence, a database, including 60 driven pile load test results and CPT records in the vicinity of them, was compiled to investigate the model uncertainties embedded in various predictive approaches. Investigated approaches include two static analyses, five SPT, and five CPT-based methods, and were implemented to predict axial pile bearing capacity. The model parameters for these methods are investigated through seven statistical, probabilistic, and reliability-based criteria. Performance of the methods under these criteria is assessed by the use of radar charts. Moreover, the resistance factor, adopted in this study to estimate the efficiency ratio and actual factor of safety, is calibrated by four different prevailing methods. Eventually, among conventional available predictive methods, CPT-based methods perform better than others and result in cost-effective and optimized trends.

Keywords Uncertainty · Bearing capacity · CPT · LRFD · Reliability · Driven piles · SPT · Static analysis

Case No. 6: Continued – List of Methods

Assessment of different predictive methods for a database including 60 driven piles and their adjacent CPT records



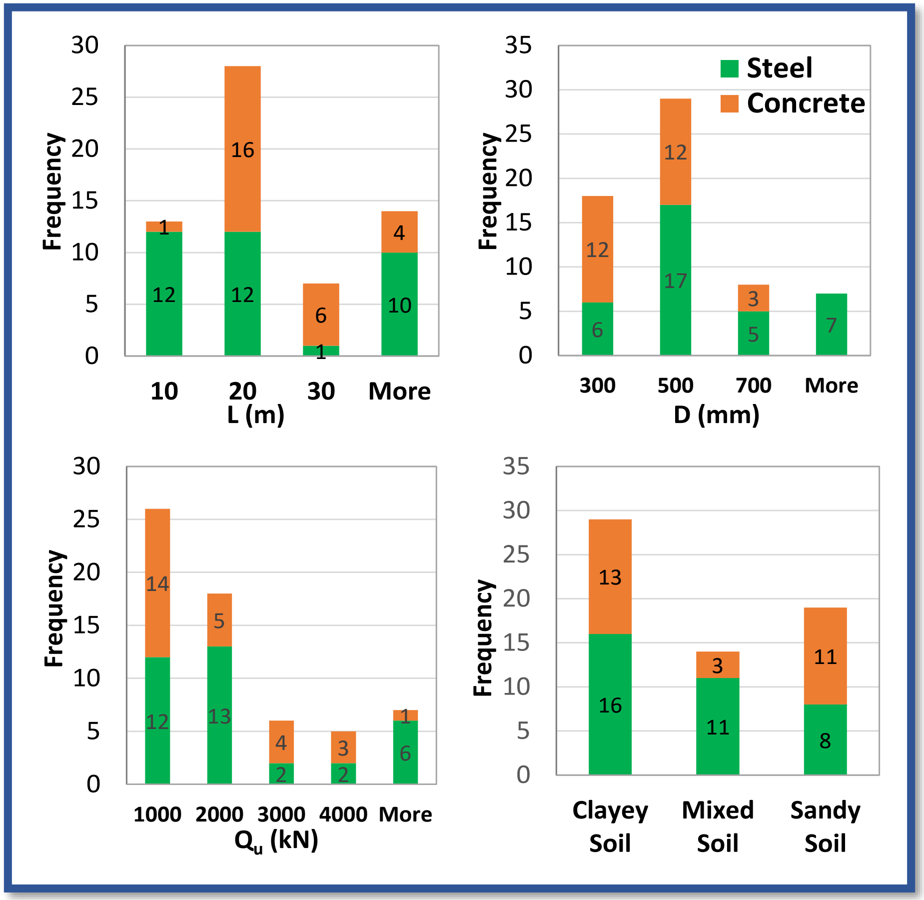
❖ **Static Analyses**
- **CFEM**
- **API**

❖ **SPT-based methods**
- **Meyerhof (1976)**
- **Shioi and Fukui (1982)**
- **Bazaara and Kurkur (1986)**
- **Briaud and Tucker (1988)**
- **Decourt (1995)**

❖ **CPT-based methods**
- **Schmertmann (1978)**
- **DeRuiter and Beringen (1979)**
- **Bustamante and Gianeselli (1982)**
- **Meyerhof (1956, 1976, 1983)**
- **UniCone (1997)**

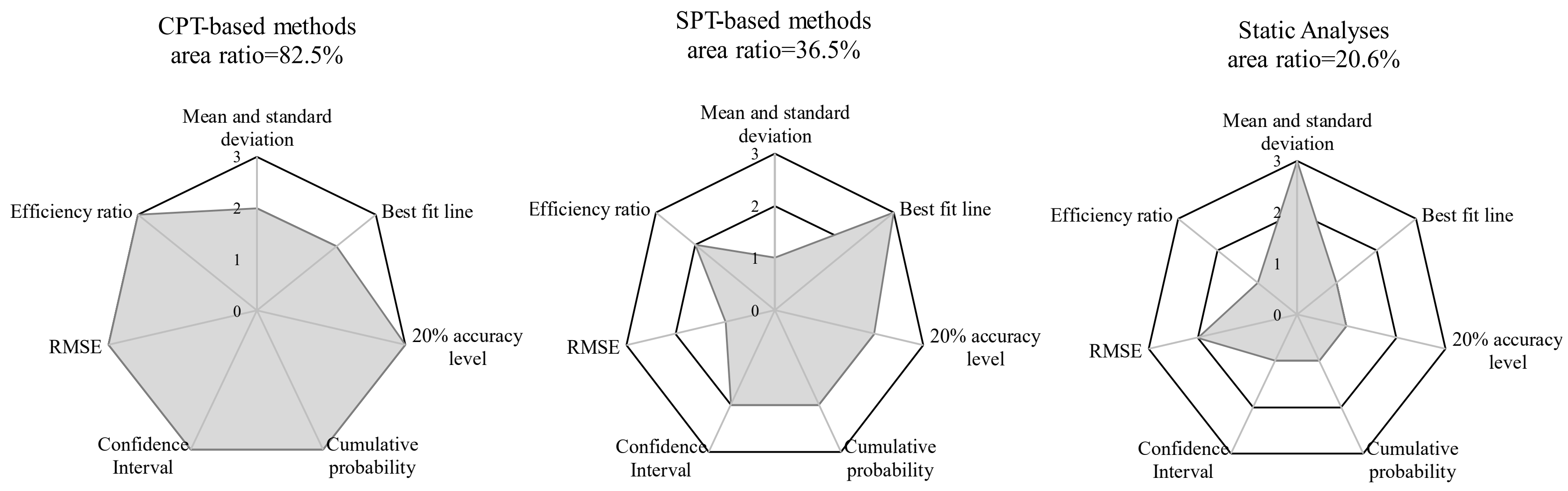
Case No. 6: Continued – Database Characteristics

- A database of 62 piles with their adjacent CPT records from AUT:GEO-CPT&Pile database
- From to different continents:
  - Asia (31%),
  - Europe (24%),
  - North America (42%), and
  - Australia (3%).
- About 47%, 23%, and 30% of site soils are clayey soil, mixed soil, and sandy soil, respectively.



Characteristics of studied piles according to their material; (a) Length, (b) dimension, (c) ultimate bearing capacity and (d) soil type

Case No. 6: Continued - Probabilistic Assessment of Model Uncertainty for Predictive Methods



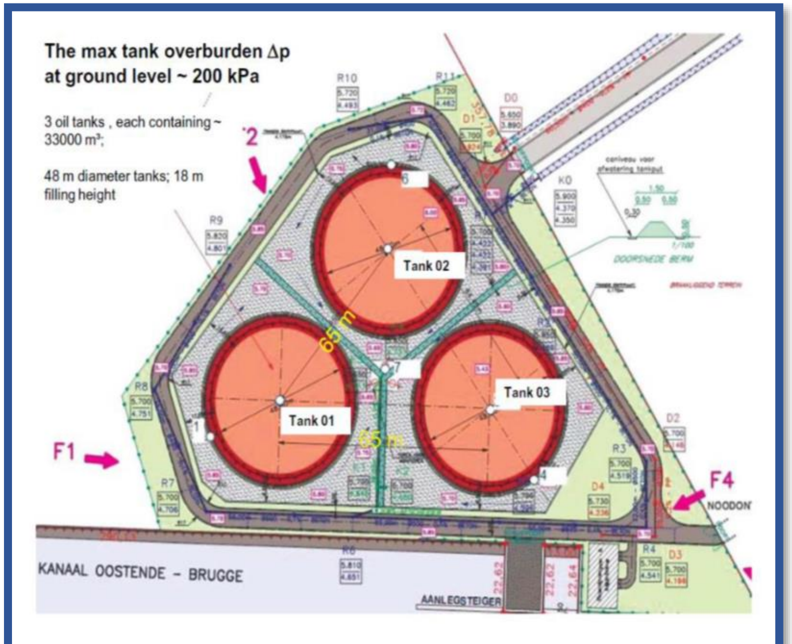
Case No. 7: Three oil tanks - Belgium (Van Impe et al., 2013 & 2015)

❖ Tanks Characteristics

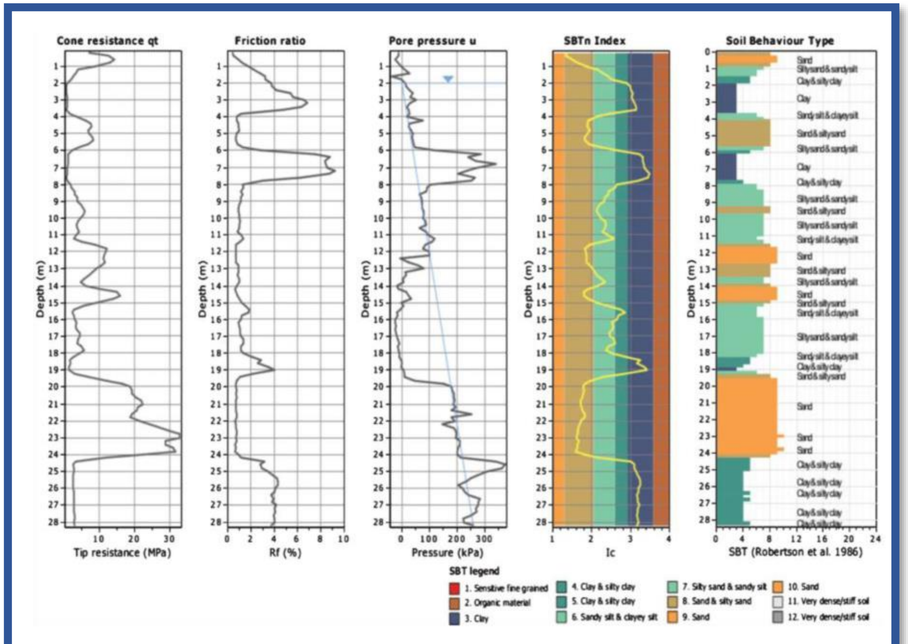
- 48 m in diameter and 19 m in height,
- Each holding 33000 m^3
- Triangle configuration - 65 m center to center

❖ 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



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



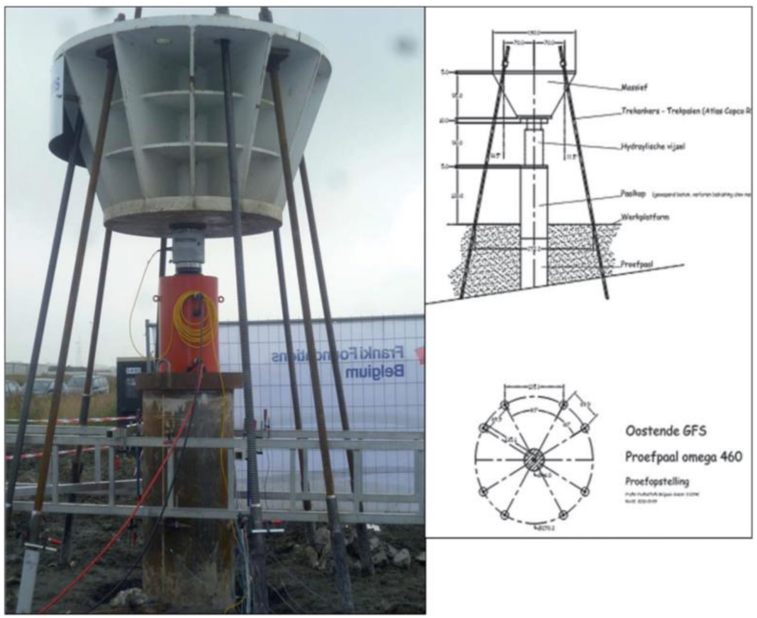
Relevant CPTu log in Ostend (Van Impe et al., 2013)

Case No. 7: Continued

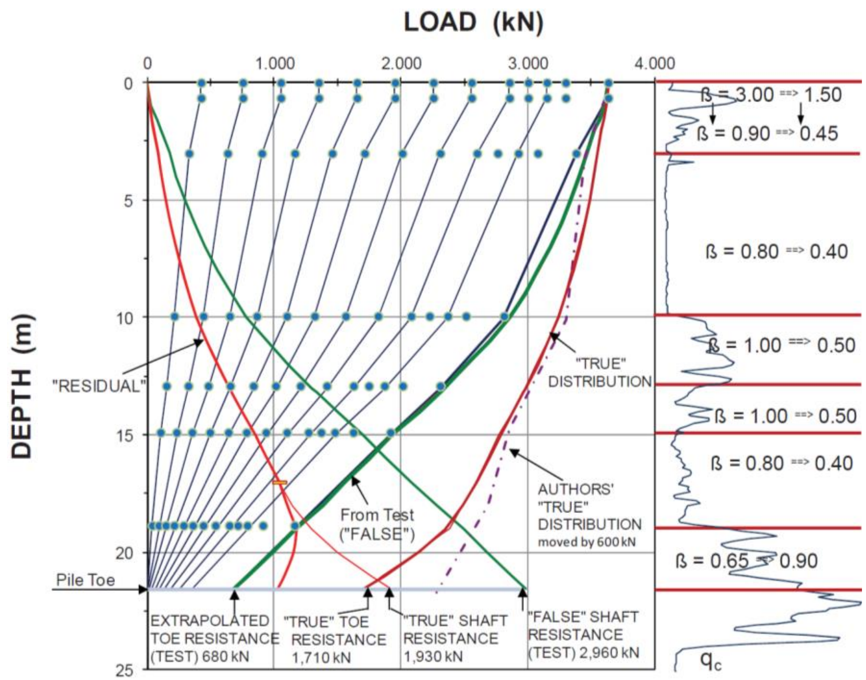
❖ Foundation System

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

❖ Loading Test & Resistance Distribution



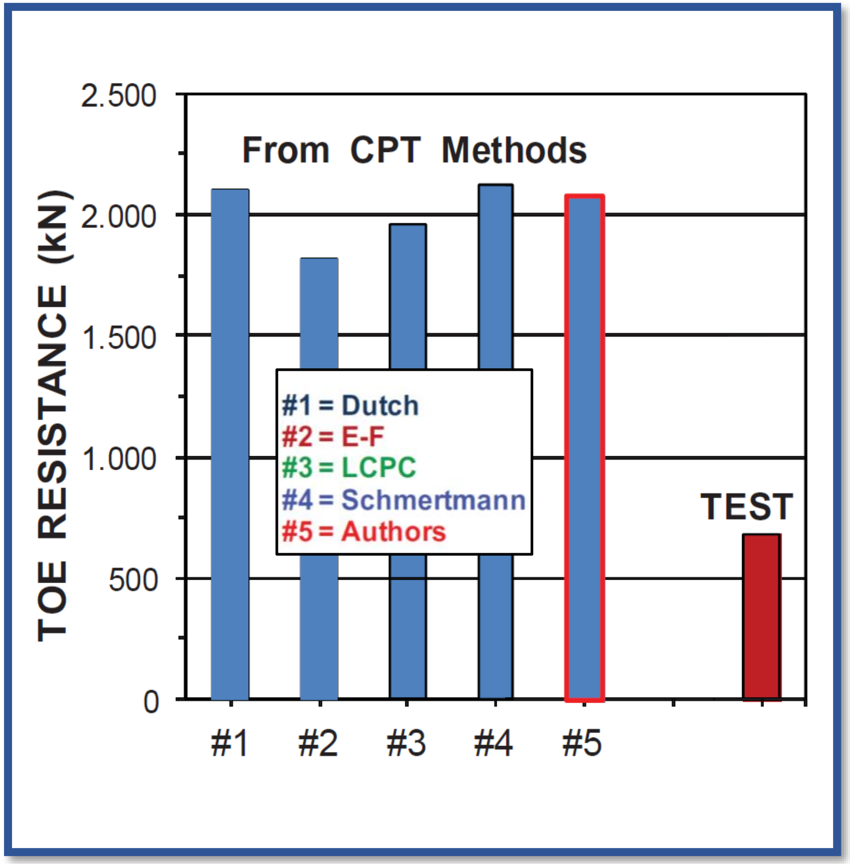
Test pile loading system, with 8 deep anchors inclined at 12◦, each enabling a 500 kN tensile force



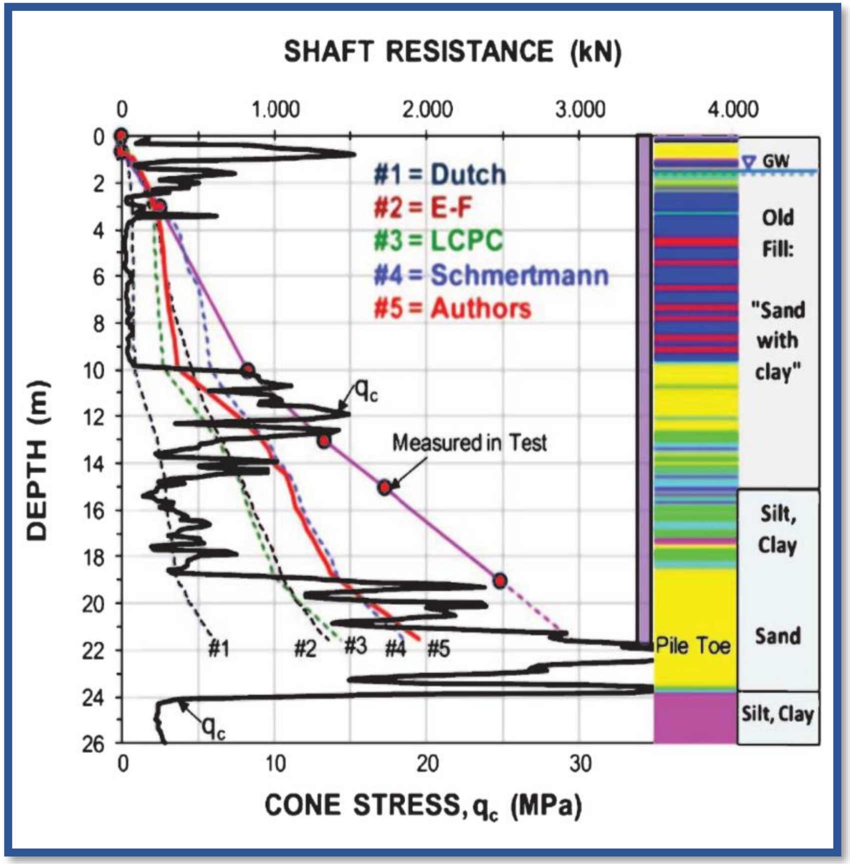
Load and resistance distribution (Fellenius, 2014)

Case No. 7: Continued

❖ Discussion on Toe & Shaft Resistances



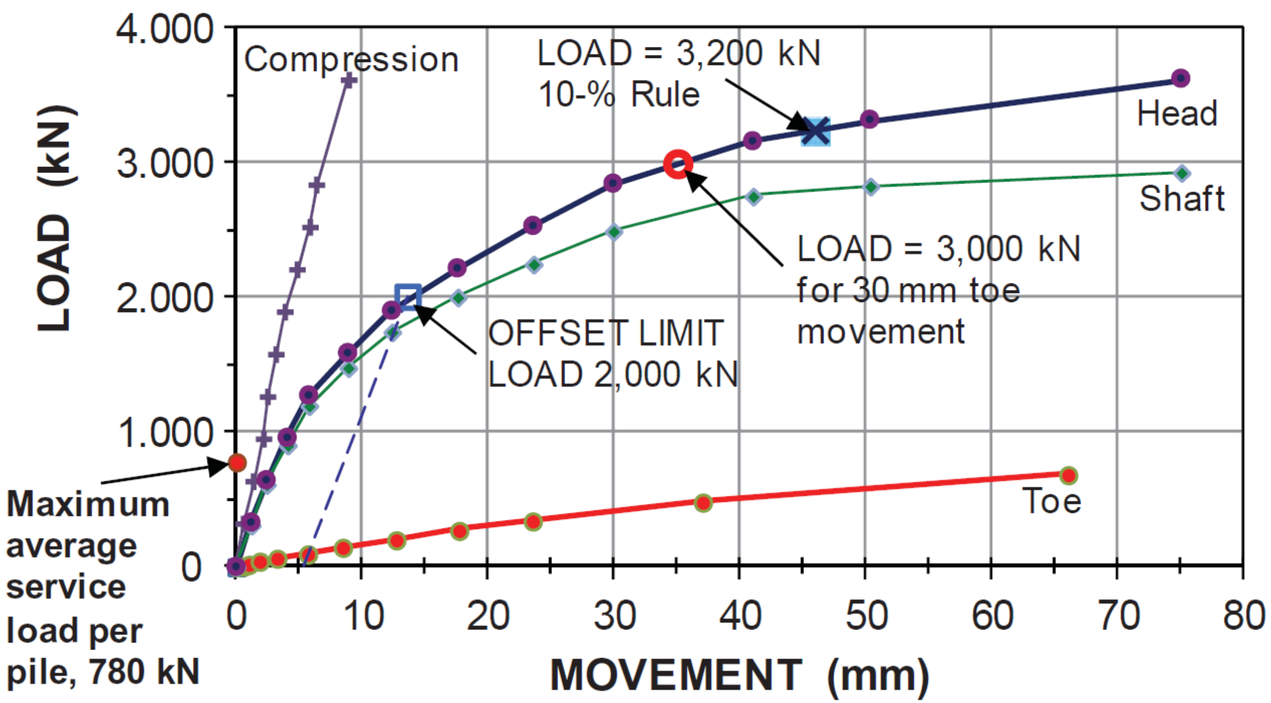
Pile ultimate toe resistance: calculated and measured (Fellenius, 2014)



Pile ultimate shaft resistance: calculated and measured (Fellenius, 2014)

Case No. 7: Continued

❖ Discussion on Load-Displacement and Capacity

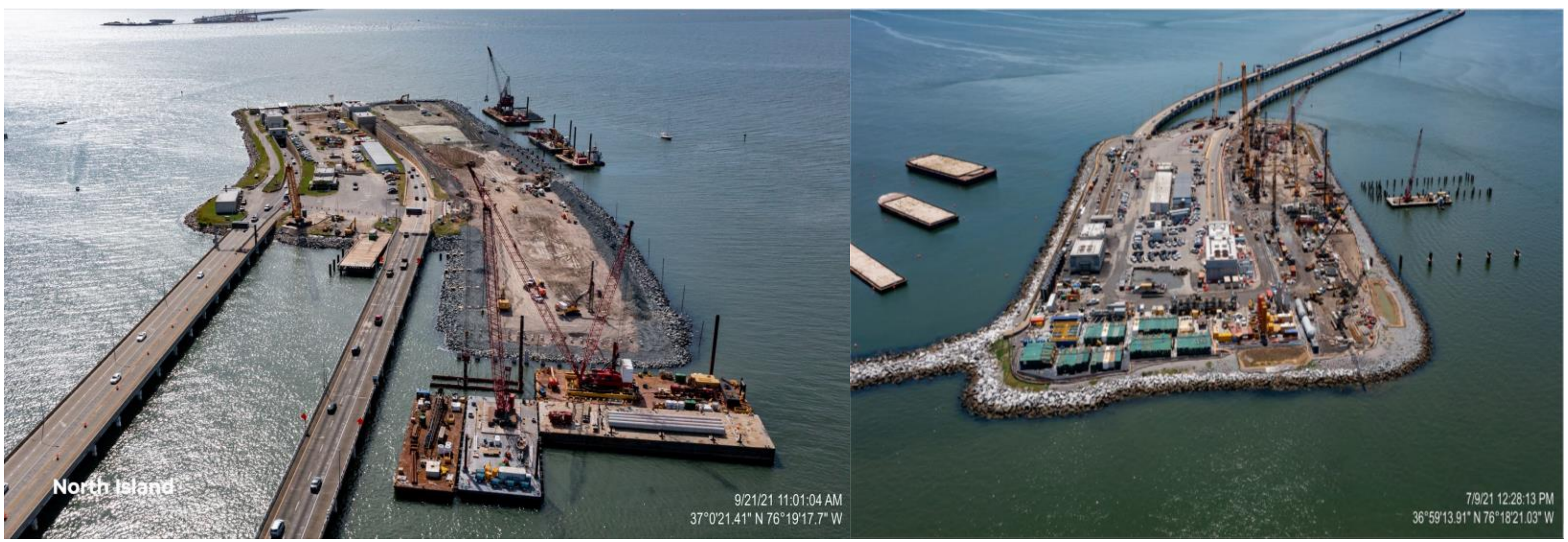


Load-movement Curve (Fellenius, 2014)

Case No. 8: James River bridge, Virginia (Batten & Keaney, 2021)

❖ Project Overview

- $3.8 Billion Design-Build project, awarded by VDOT 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 No. 8: 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*

❖ Pile Types

- North & South Trestles:

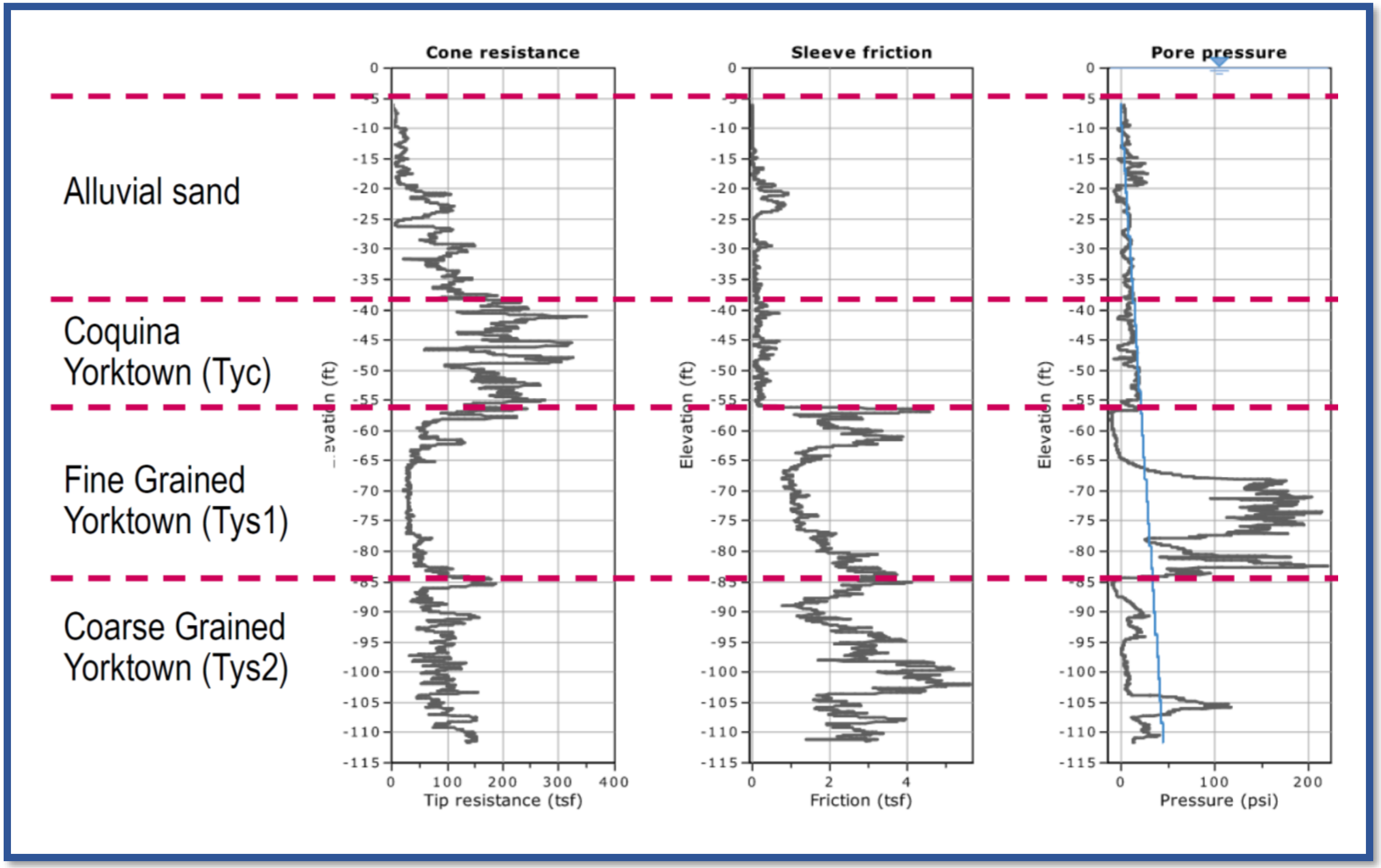
54-inch precast prestressed concrete cylinder piles, 6.75-inch wall thickness, bed cast, NOT spun cast

- Willoughby Bay Trestle:

24-inch precast prestressed concrete square piles

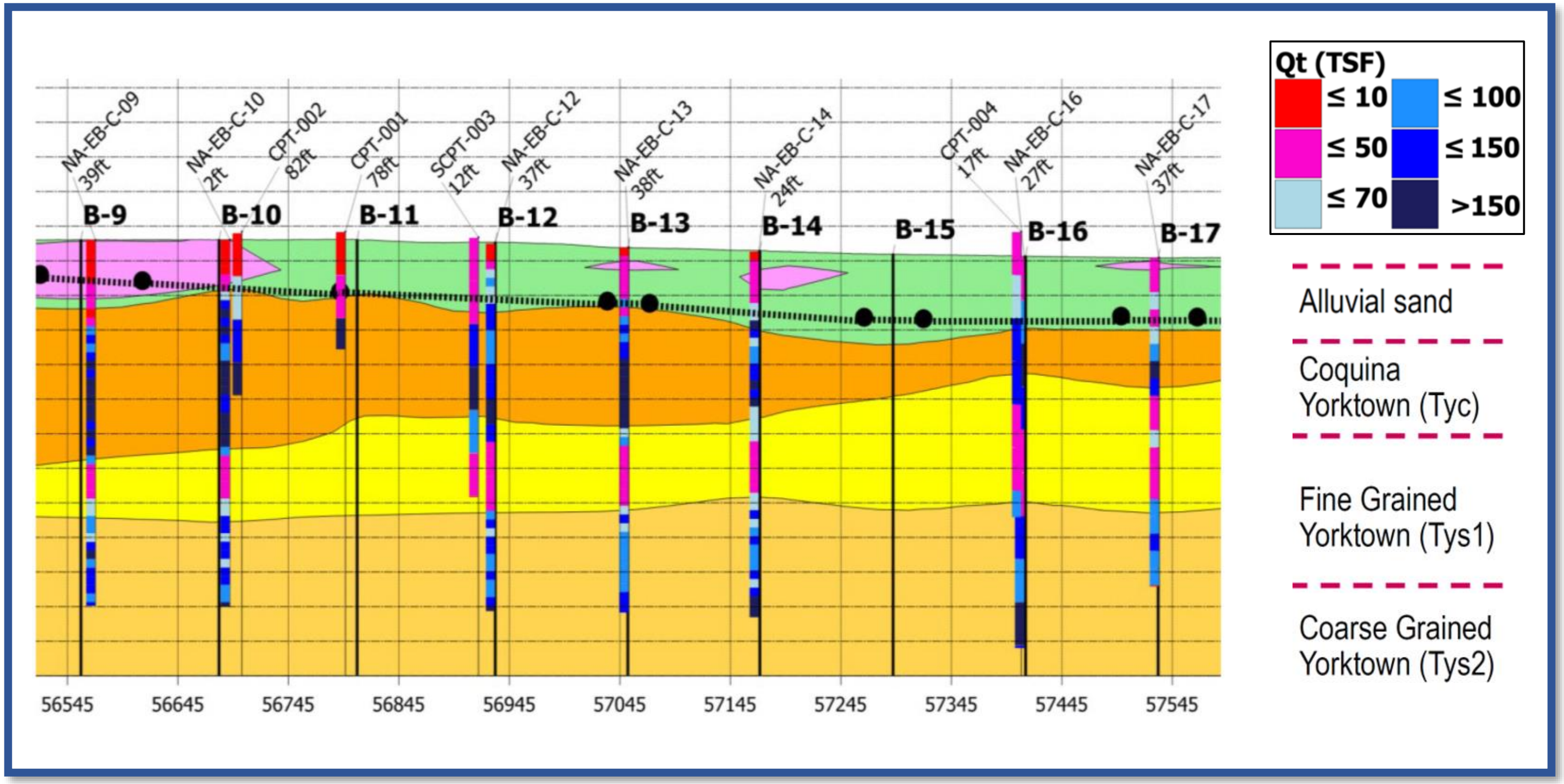
Case No. 8: Continued

❖ Typical CPT Sounding



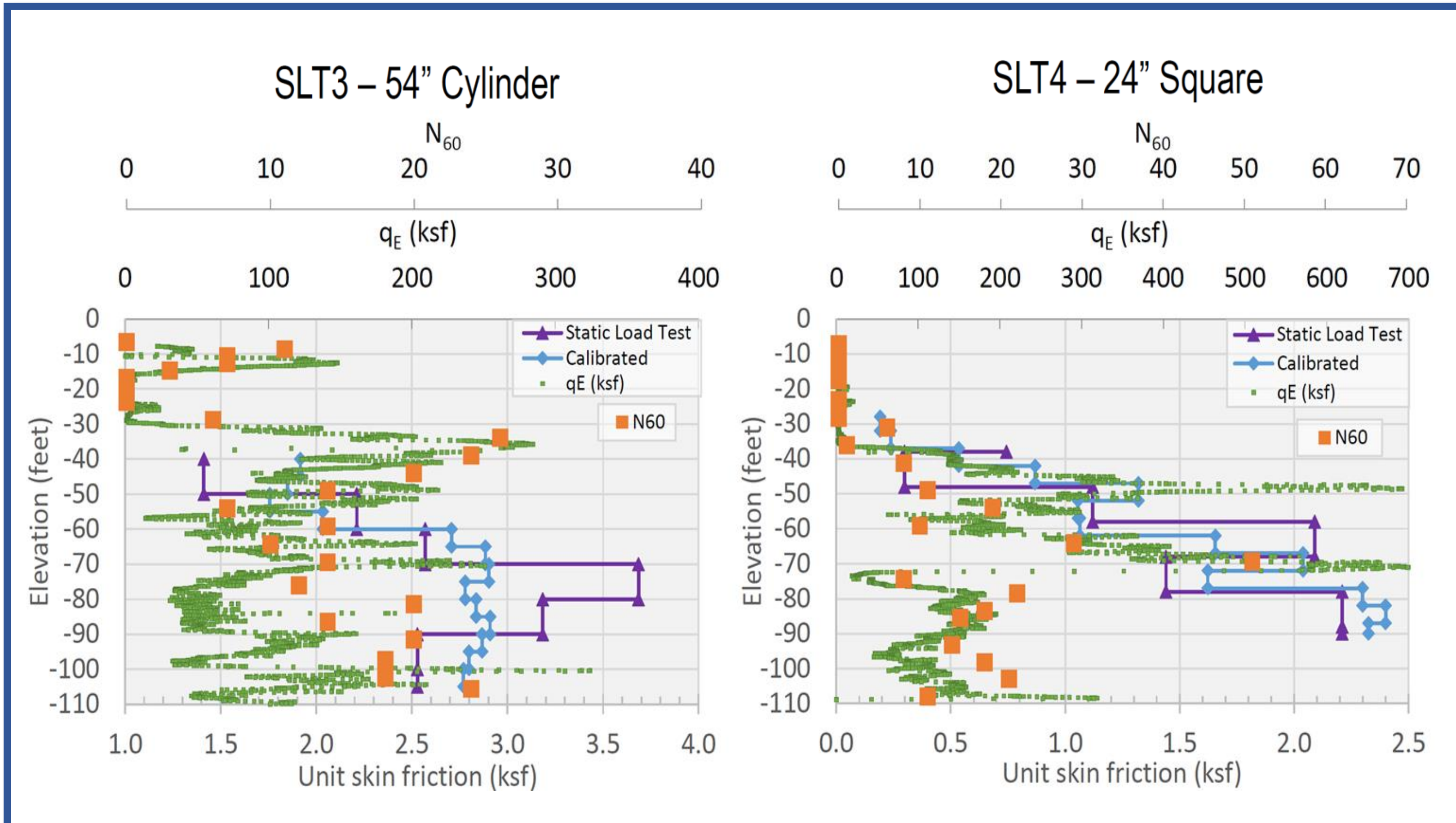
Case No. 8: Continued

❖ Subsurface Profile North Trestle



Case No. 8: Continued

❖ Calibration to SLTs



❖ Project-Based Calibration of Parameters

Case No. 8: Continued

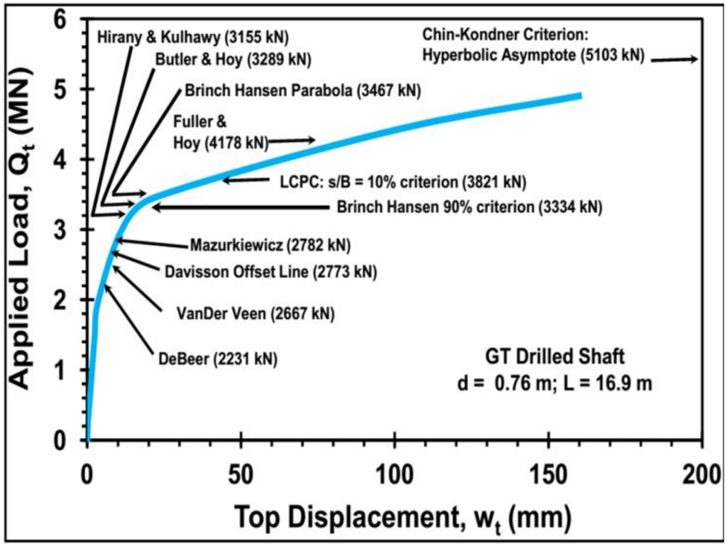
Soil Type	Soil Description	Original C_s (Fellenius 2021)	Calibrated Against HRBT Static Load Tests C_s	Calibrated Against HRBT Static Load Tests Limiting q_{skin} (ksf)
1	Soft sensitive soils	0.08	0.150	None
2	Clay	0.05	0.066	1.5
3	Silty clay, stiff clay and silt	0.025	0.066	3.0
4a	Sandy silt and silt	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

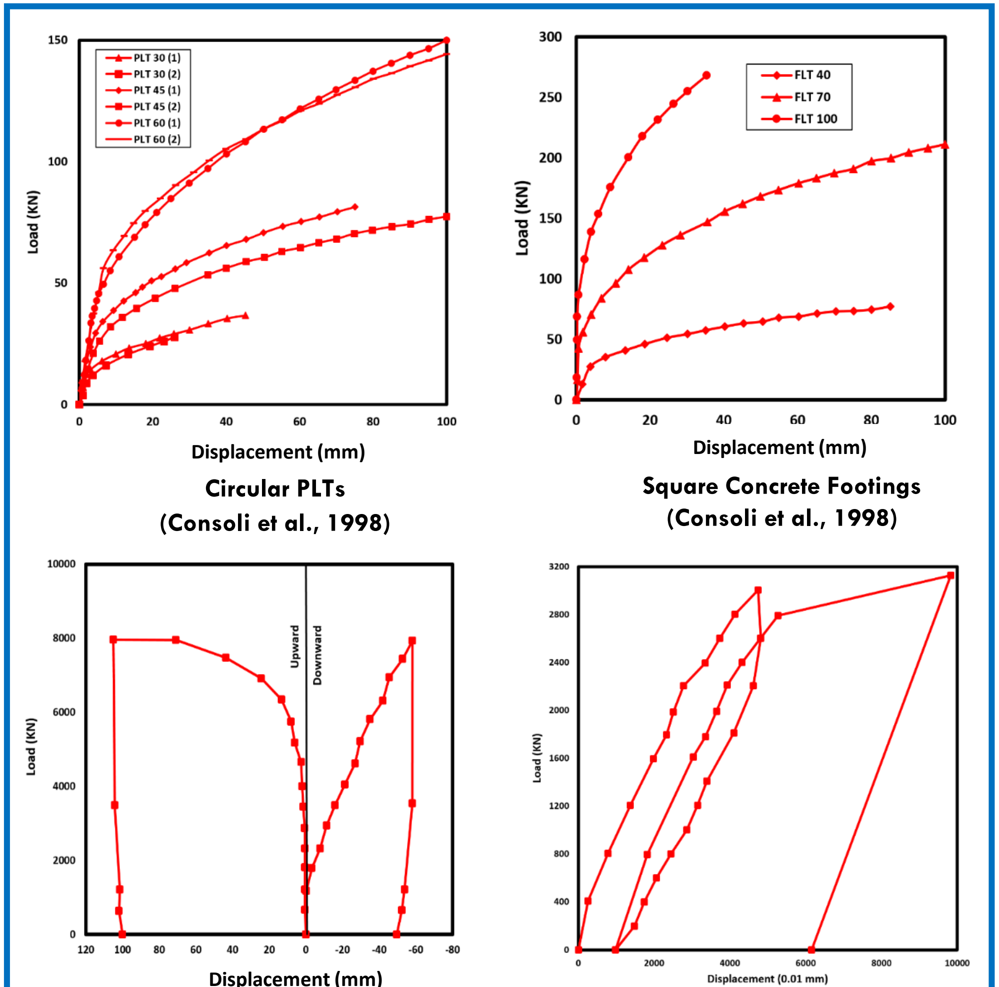
1 All axial resistance methods assumed an unplugged toe area for the open ended 54-inch cylinder piles.
2 The Martin et al (1987) method was only applied to pile embedment within the Yorktown strata. Conventional effective stress beta analysis was used to calculate unit skin friction for the Quaternary 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.

Load – Displacement; Significancy

- Accurate Capacity Determination
- Facilitates Settlement Check
- Provides Stiffness Parameters



Interpretation of load–displacement diagram (Fellenius, 1990)



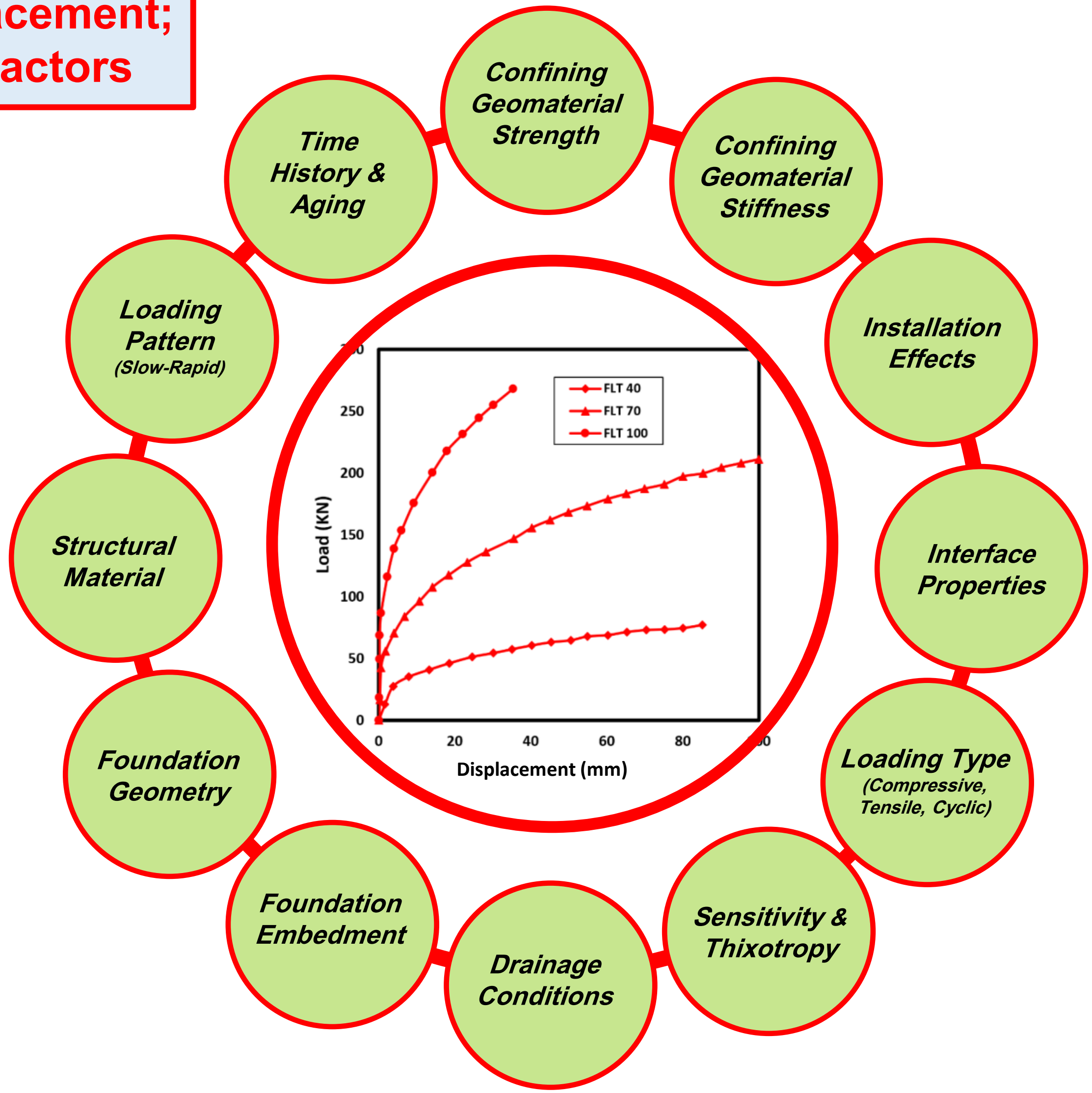
Circular PLTs (Consoli et al., 1998)

Square Concrete Footings (Consoli et al., 1998)

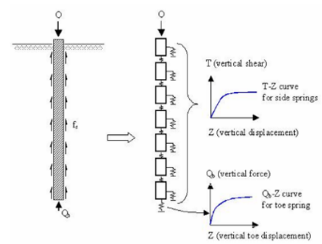
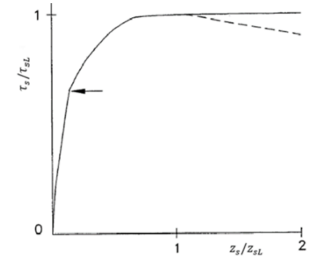
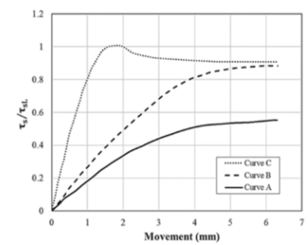
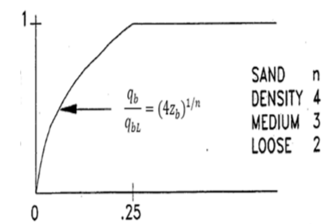
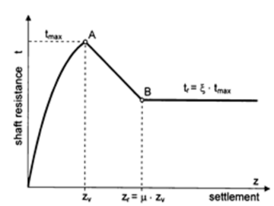
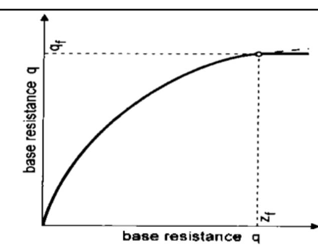
Bidirectional test on a bored pile (Fellenius, 2023)

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

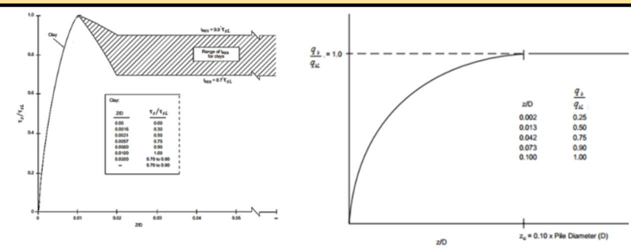
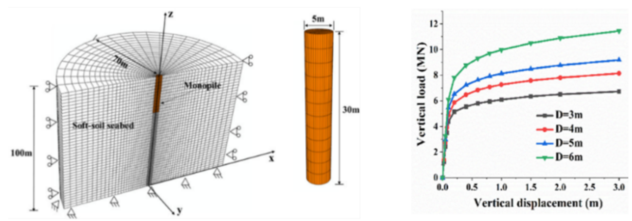
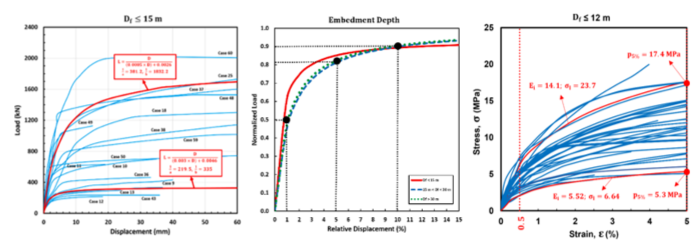
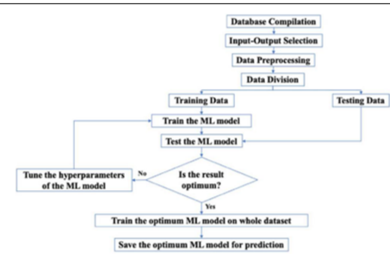
Load – Displacement; Dominant Factors



Methods for Load-Displacement Appraisal

No	Method	Reference	Details
1	Empirical functions of t-z and q-z	Seed & Reese (1957); Vijayvergiya (1977); Kraft et al. (1981)	 
2	Experimental Studies on t-z and q-z	Coyle & Reese (1966); Coyle & Sulaiman (1967); Coyle & Castello (1981); Mosher (1984)	 
3	Single Pile Settlement	Vesic (1977)	Elastic shortening of the pile $s_{e1} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$ Settlement of pile at the toe $s_{e2} = \frac{q_{wp} D}{E_s}(1-\mu_s^2) I_{wp}$ Settlement of pile along the shaft $s_{e3} = \left(\frac{Q_{ws}}{pL}\right)\frac{D}{E_s}(1-\mu_s^2) I_{ws}$
4	CPT-based functions of t-z and q-z	Gwizdala (1996); Valikhah et al. (2019)	 

Methods for Load-Displacement Appraisal – Cont.

No	Method	Reference	Details
5	Code-based functions of t-z and q-z	API (2011)	
6	Numerical Simulation (FEM)	Randolph & Wroth (1978); Zhang et al. (2010); Ellobody et al. (2013); Dai et al. (2014); Wang et al. (2021); Kourkoulis et al. (2022)	
7	Normalized Approach: Stress - Strain	Eslami & Ebrahimipour (2024); Eslami et al. (2026)	
8	ML, ANN and AI Approaches	Nejad and Jaksa (2017); Abu-Farsakh and Moontakim Shoaib (2024); Eslami et al. (2025)	$r=\frac{C_{y_j d_j}}{\sigma_{y_j}\sigma_{d_j}}$ $RMSE=\left\{\frac{1}{n}\sum_{j=1}^{n}(y_j-d_j)^2\right\}^{\frac{1}{2}}$ $\mathrm{MAE}=\frac{1}{n}\sum_{j=1}^{n}\lvert y_j-d_j\rvert$ 

Taylor & Francis

International Journal of Geotechnical Engineering

ISSN: 1938-6362 (Print) 1939-7879 (Online) Journal homepage: http://www.tandfonline.com/loi/yjge20

Study on pile ultimate capacity criteria and CPT-based direct methods

Sara Moshfeghi and Abolfazl Eslami*

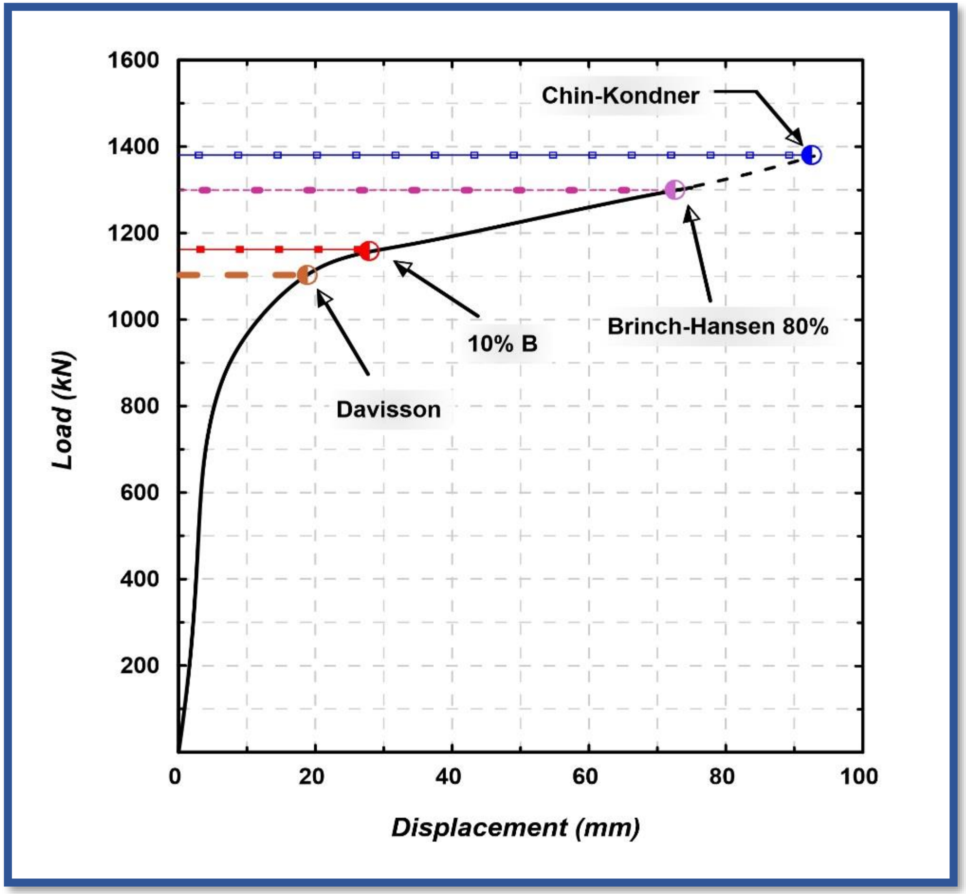
Due to the variety of current Cone Penetration Test (CPT)-based methods of estimating the pile bearing capacity, for optimum design, it is necessary to evaluate the performance of such methods in various geotechnical conditions. Geotechnical databases including piling and *in situ* testing records have been recognised as useful tools for analysis, design and economical construction. In order to evaluate current CPT-based pile bearing capacity methods, AUT-CPT and Pile database has been compiled including 450 full scale pile load tests and CPT sounding records. This database consists of different pile types with a relatively wide range of geometries and various soil conditions. Forty-three records of piles driven in sand deposits were then employed to evaluate effects of ultimate capacity interpretation criteria from load displacement diagrams. The Brinch Hansen 80% criterion and the load at the displacement of 10% of the pile diameter were compared to estimated capacities from 10 CPT-based design methods currently used in practice. The Brinch Hansen 80% criterion and the load at the displacement of 10% of the pile diameter lead to reasonable results, the Brinch Hansen 80% criterion showed less scatter. For evaluating the accuracy and the precision of CPT-based methods, the results were compared to estimated capacities. Methods with the best performance are introduced. Generally, comparisons indicate that the CPT-based methods mainly predict the pile capacity with reasonable accuracy.

Keywords: Ultimate pile bearing capacity, Direct CPT method, Load test, Failure criterion, Database

Summary of Selected Case Records

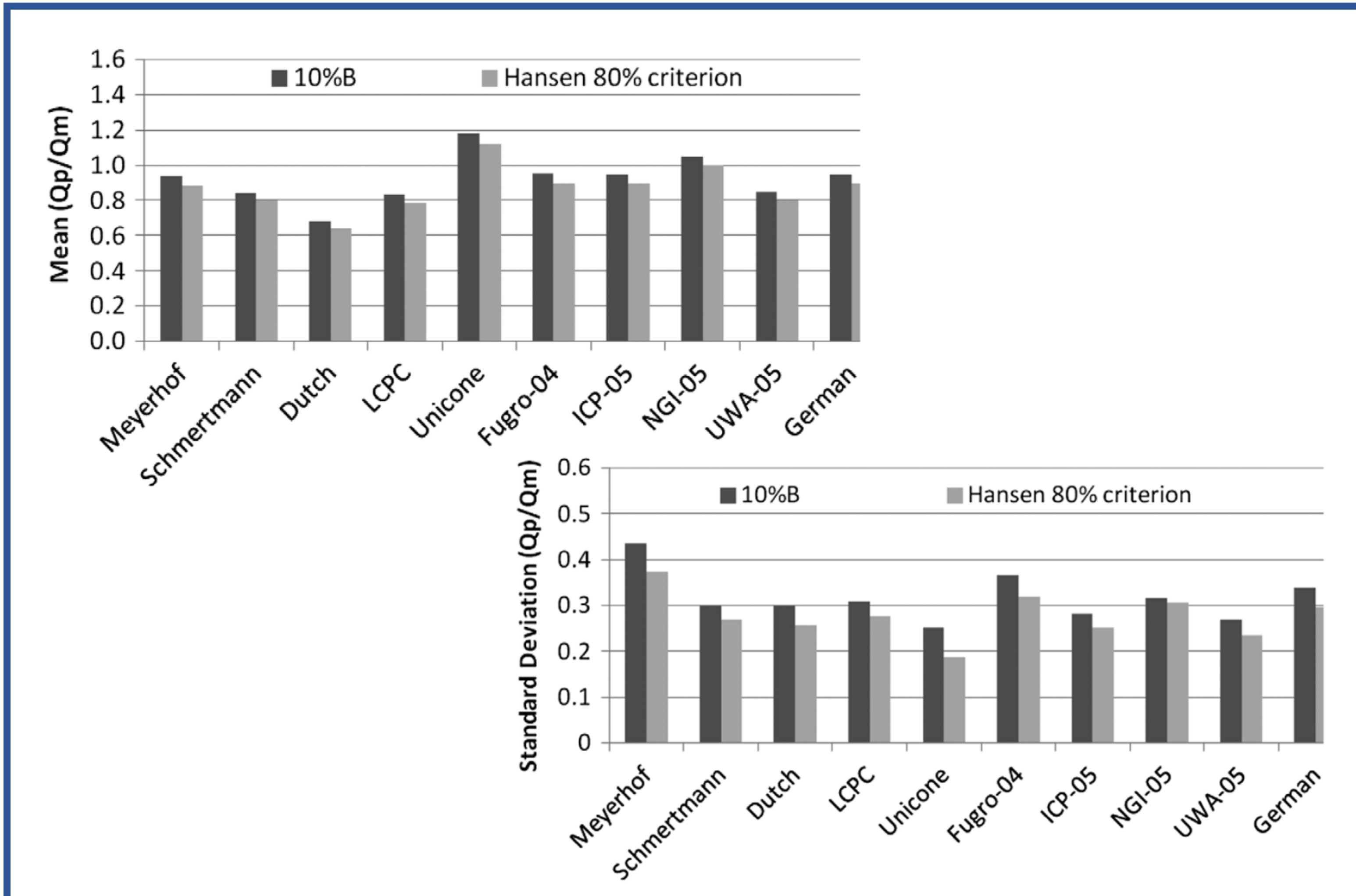
No.	Case ID	Reference	Shape	Material	D (m)	B (mm)	Q_u (kN)
1	001-FHWASF	O'Neill (1988)	Pipe (CE)	Steel	9.2	273	490
2	001-N&SBI42	Nottingham (1975)	Pipe (CE)	Steel	15.2	273	675
3	001-N&SBI43	Nottingham (1975)	Pipe (CE)	Steel	22.5	273	1631
4	001-OKLAST	Neveles and Donald (1994)	Pipe (CE)	Steel	18.2	660	3650
5	001-TWNTP4	Yen *et al*. (1989)	Pipe (CE)	Steel	34.25	609	4290
6	001-TWNTP6	Yen *et al*. (1989)	Pipe (CE)	Steel	34.25	609	4460
7	020-PIGEONCP	Paik *et al*. (2003)	Pipe (CE)	Steel	7	356	1726
8	126-HP1	Briaud *et al*. (1989)	Pipe (CE)	Steel	7.78	273	379
9	020-PIGEONOP	Paik *et al*. (2003)	Pipe (OE)	Steel	7	356	1275
10	176-DUNKIRK C1C	Jardine and Standing (2000)	Pipe (OE)	Steel	10.02	457	2819
11	176-DUNKIRK JP1	Jardine and Standing (2000)	Pipe (OE)	Steel	10	457	5175
12	001-BGHD1	Altaee *et al*. (1992)	Square	Concrete	11	285	1000
13	001-BGHD2	Altaee *et al*. (1992)	Square	Concrete	15	285	1600
14	001-KLO14B	Van Impe *et al*. (1988)	Round	Concrete	12	600	6100
15	001-LSUA1	Tumay and Fakhroo (1981)	Square	Concrete	9.5	350	900
16	001-N&SBI215	Nottingham (1975)	Square	Concrete	21.3	250	810
17	001-N&SBI316	Nottingham (1975)	Square	Concrete	15.85	350	1485
18	001-N&SBI348	Nottingham (1975)	Square	Concrete	14.9	450	1755
19	001-N&SJC1	Nottingham (1975)	Square	Concrete	9.15	450	1845
20	001-N&SWBP1	Nottingham (1975)	Square	Concrete	8	450	1140
21	001-N&SWBP2	Nottingham (1975)	Square	Concrete	11.3	450	830
22	001-OKLACO	Neveles and Donald (1994)	Round	Concrete	18.2	610	3600
23	001-POLA1	CH2M Hill (1987)	Octagonal	Concrete	25.8	600	5785
24	001-POLA2	Urkkada Technology Ltd (1995)	Octagonal	Concrete	32.6	600	3560
25	001-PTSER	Appendino (1981)	Round	Concrete	35.85	508	5500
26	001-UFL22	Avasarala *et al*. (1994)	Square	Concrete	16	350	1350
27	001-UFL52	Avasarala *et al*. (1994)	Square	Concrete	11	500	2070
28	066-ISC2C1	Fellenius *et al*. (2007)	Square	Concrete	6	350	1500
29	124-LAHW1-T233	Rauser (2008)	Square	Concrete	33.5	406	1899
30	124-LAHW1-T242	Rauser (2008)	Round	Concrete	41.8	1372	5760
31	124-LAHW1-T4	Rauser (2008)	Round	Concrete	49	610	3834
32	124-LAHW1-T544	Rauser (2008)	Square	Concrete	44.2	610	3287
33	124-LAHW1-T552	Rauser (2008)	Square	Concrete	51.8	610	3380
34	126-DRAMMEN A	Gregersen *et al*. (1975)	Round	Concrete	8	280	175
35	134-1-FITTJA A	Axelsson (1998)	Square	Concrete	19	235	560
36	134-1-FITTJA B	Axelsson (1998)	Square	Concrete	19	235	529
37	134-1-FITTJA C	Axelsson (1998)	Square	Concrete	19	235	736
38	134-DRAMMENAD	Fellenius (2002)	Round	Concrete	16	280	510
39	414-DUT1	Geerling (1996)	Square	Concrete	11.5	250	595
40	414-DUT2	Geerling (1996)	Square	Concrete	14.69	250	324
41	414-DUT3	Geerling (1996)	Square	Concrete	19.08	250	1117
42	414-DUT4	Geerling (1996)	Square	Concrete	18.98	250	1210
43	414-DUT5	Geerling (1996)	Square	Concrete	18.96	250	1215

Load-Displacement Interpretation

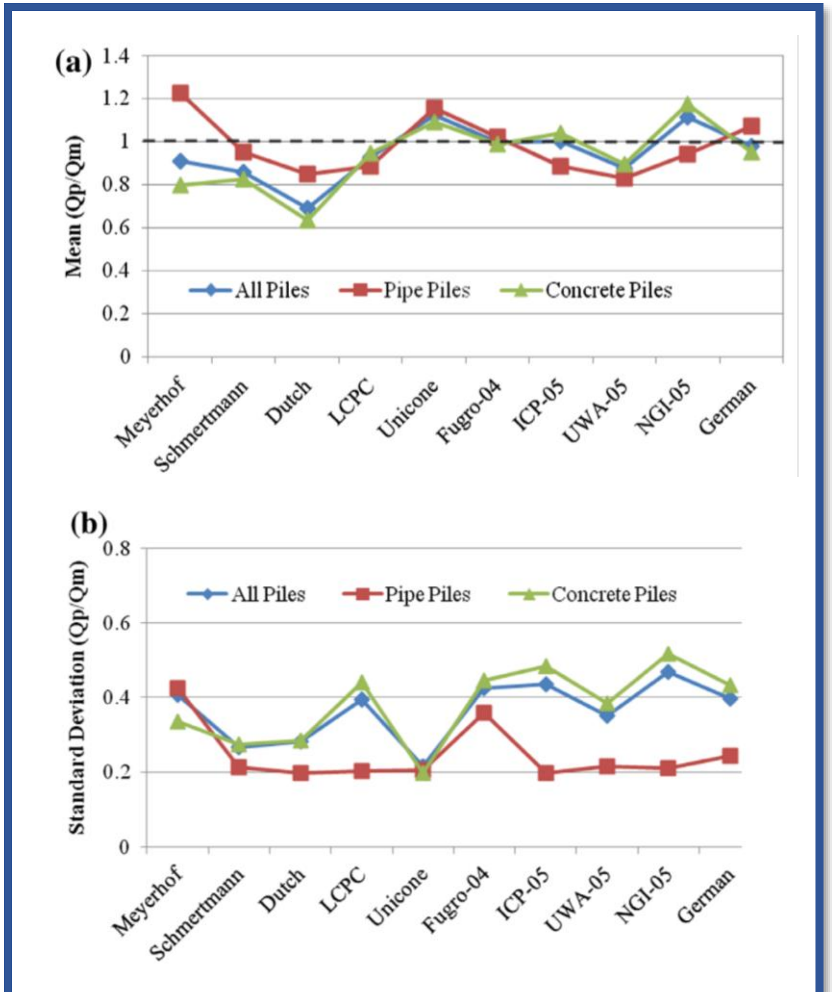


Interpretation of load displacement diagram for Case 001-L&D31 (Moshfeghi & Eslami, 2016)

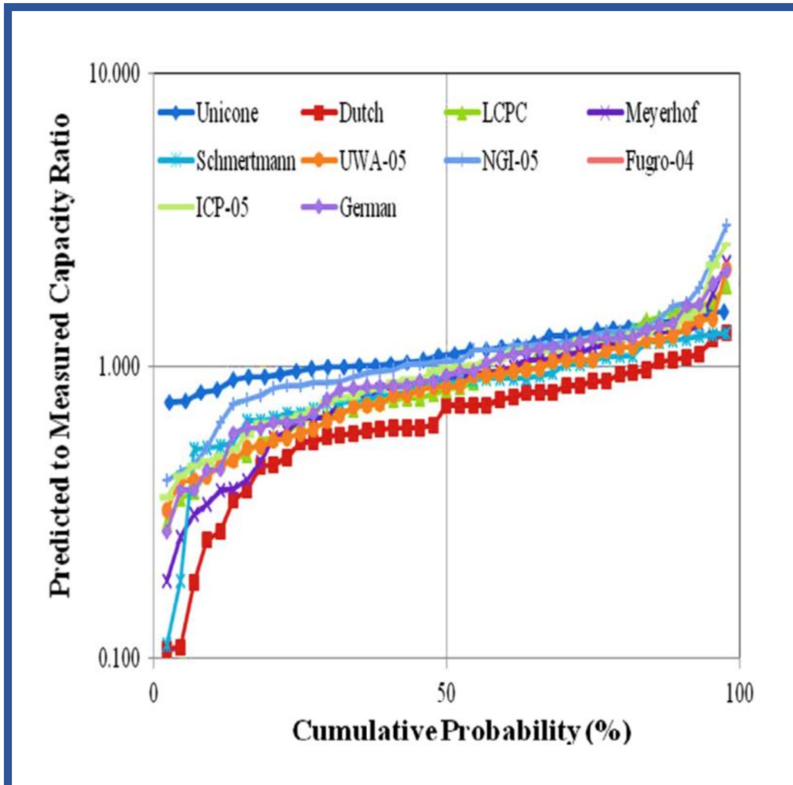
Comparison of Investigated Interpretation Criteria for The Ratio of Predicted to Measured Capacity

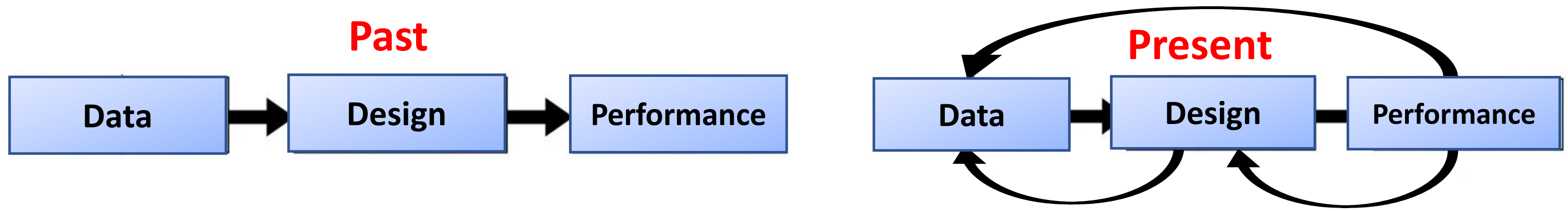


Comparison of The Estimated to Measured Capacity Ratios for Different Piles: a) Mean, b) Standard Deviation

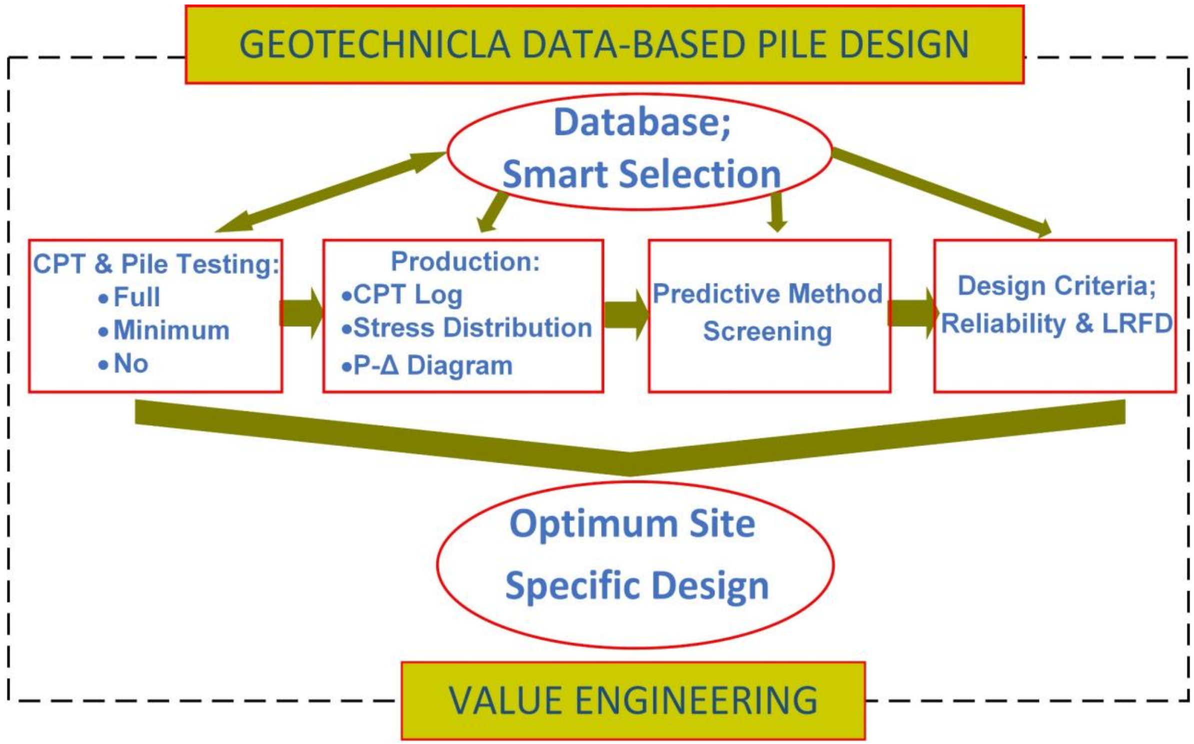


Cumulative Probabilities of Different CPT-Based Methods





Prospect: Data-Based Approach in Design

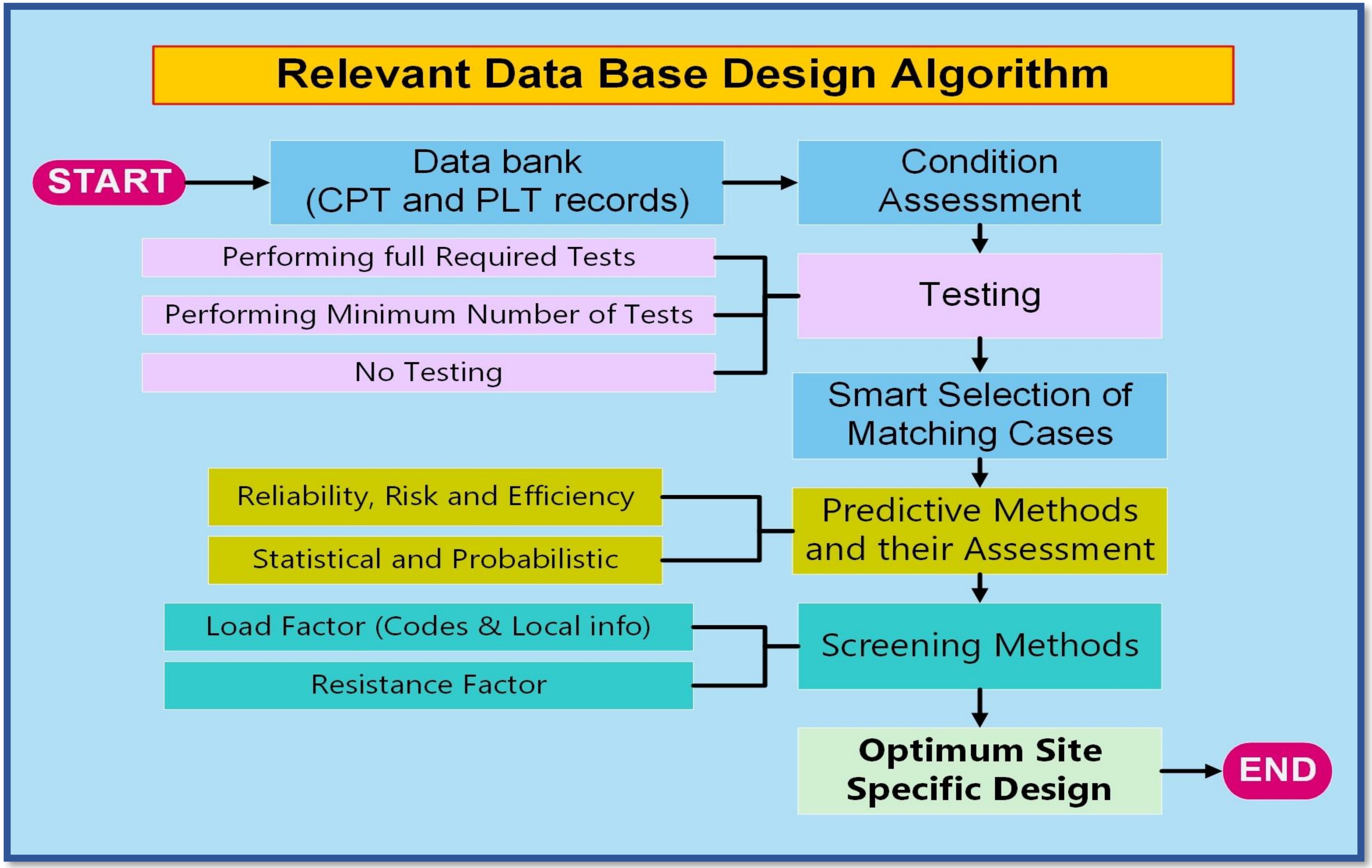


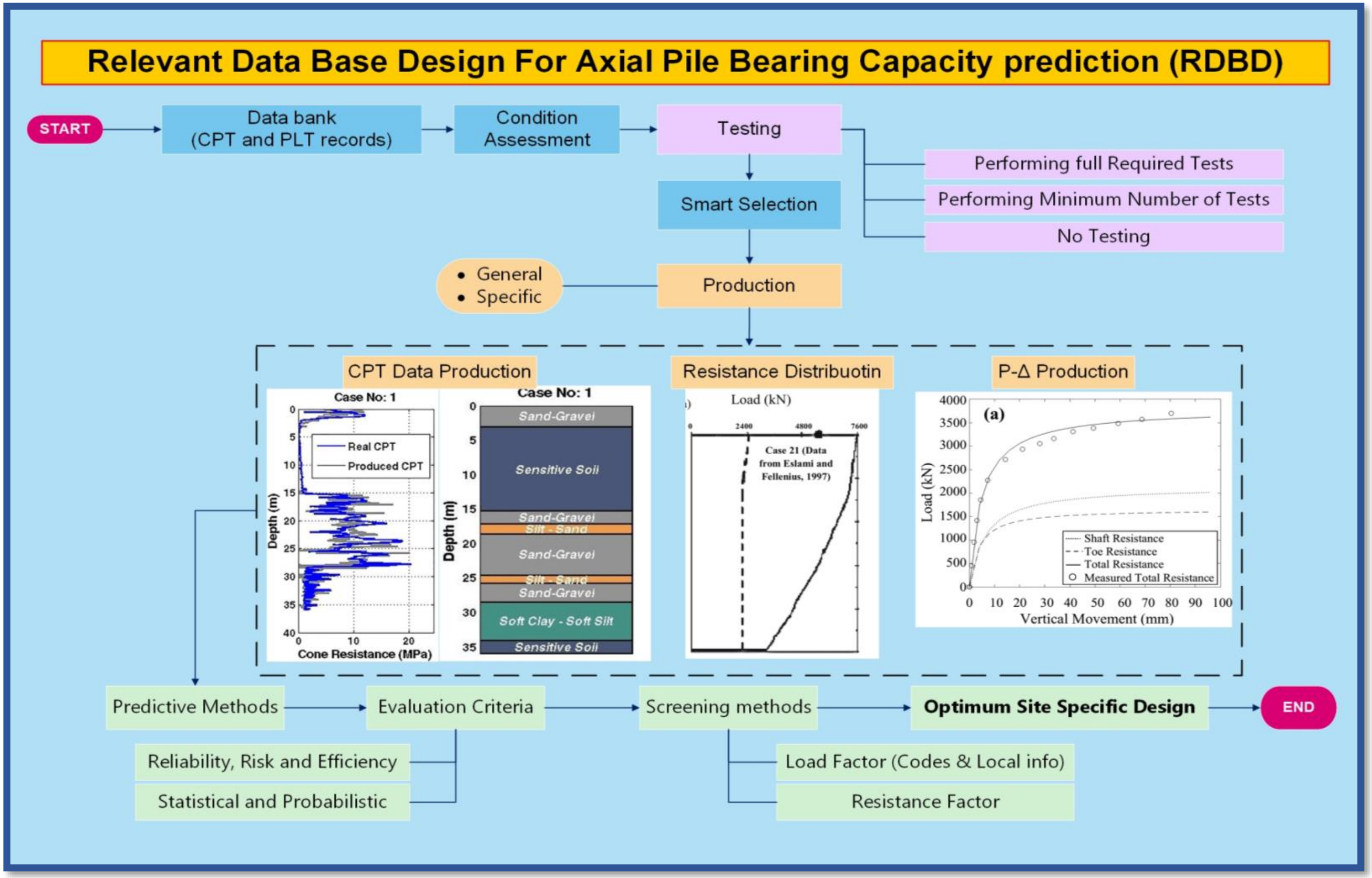
❖ Reliability-based criteria

- Input variables,
- Model assumptions,
- Soil behavior classification
- Loading direction and
- Influence zone and averaging technique
- Data processing
- Pile installation method
- Friction Fatigue
- Failure Criterion
- Data production-cone type

❖ The statistic and probabilistic criteria

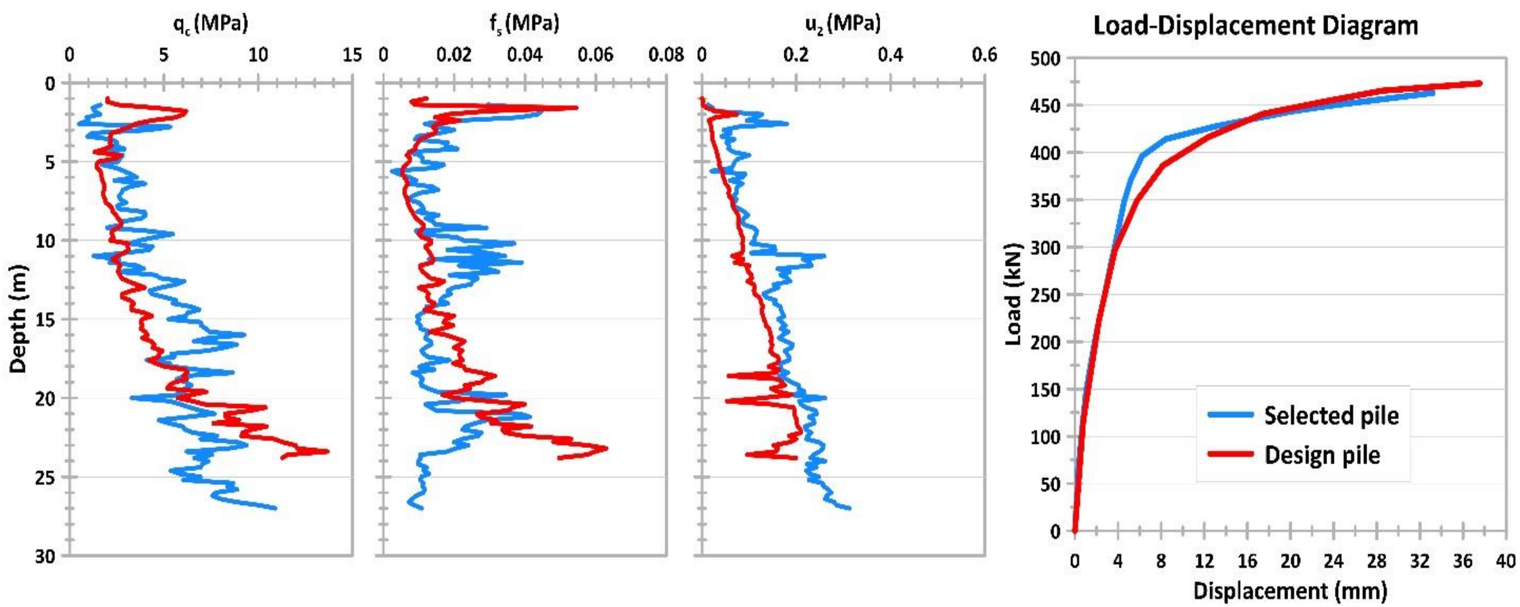
- Mean and coefficient of variation,
- Best-fitted line,
- Cumulative probability,
- 20% accuracy level, and
- RMSE
- Efficiency ratio


Relevant Data Base Design Algorithm
START
Data bank
(CPT and PLT records)
Condition
Assessment
Performing full Required Tests
Performing Minimum Number of Tests
No Testing
Testing
Smart Selection of
Matching Cases
Reliability, Risk and Efficiency
Statistical and Probabilistic
Predictive Methods
and their Assessment
Load Factor (Codes & Local info)
Resistance Factor
Screening Methods
Optimum Site
Specific Design
END


Relevant Data Base Design For Axial Pile Bearing Capacity prediction (RDBD)
START
Data bank (CPT and PLT records)
Condition Assessment
Testing
Performing full Required Tests
Performing Minimum Number of Tests
No Testing
Smart Selection
General
Specific
Production
CPT Data Production
Resistance Distribuotin
P-Δ Production
Case No: 1
Real CPT
Produced CPT
Depth (m)
Cone Resistance (MPa)
Sand-Gravel
Sensitive Soil
Silt - Sand
Soft Clay - Soft Silt
Load (kN)
Case 21 (Data from Eslami and Fellenius, 1997)
(a)
Shaft Resistance
Toe Resistance
Total Resistance
Measured Total Resistance
Vertical Movement (mm)
Predictive Methods
Evaluation Criteria
Screening methods
Optimum Site Specific Design
END
Reliability, Risk and Efficiency
Statistical and Probabilistic
Load Factor (Codes & Local info)
Resistance Factor

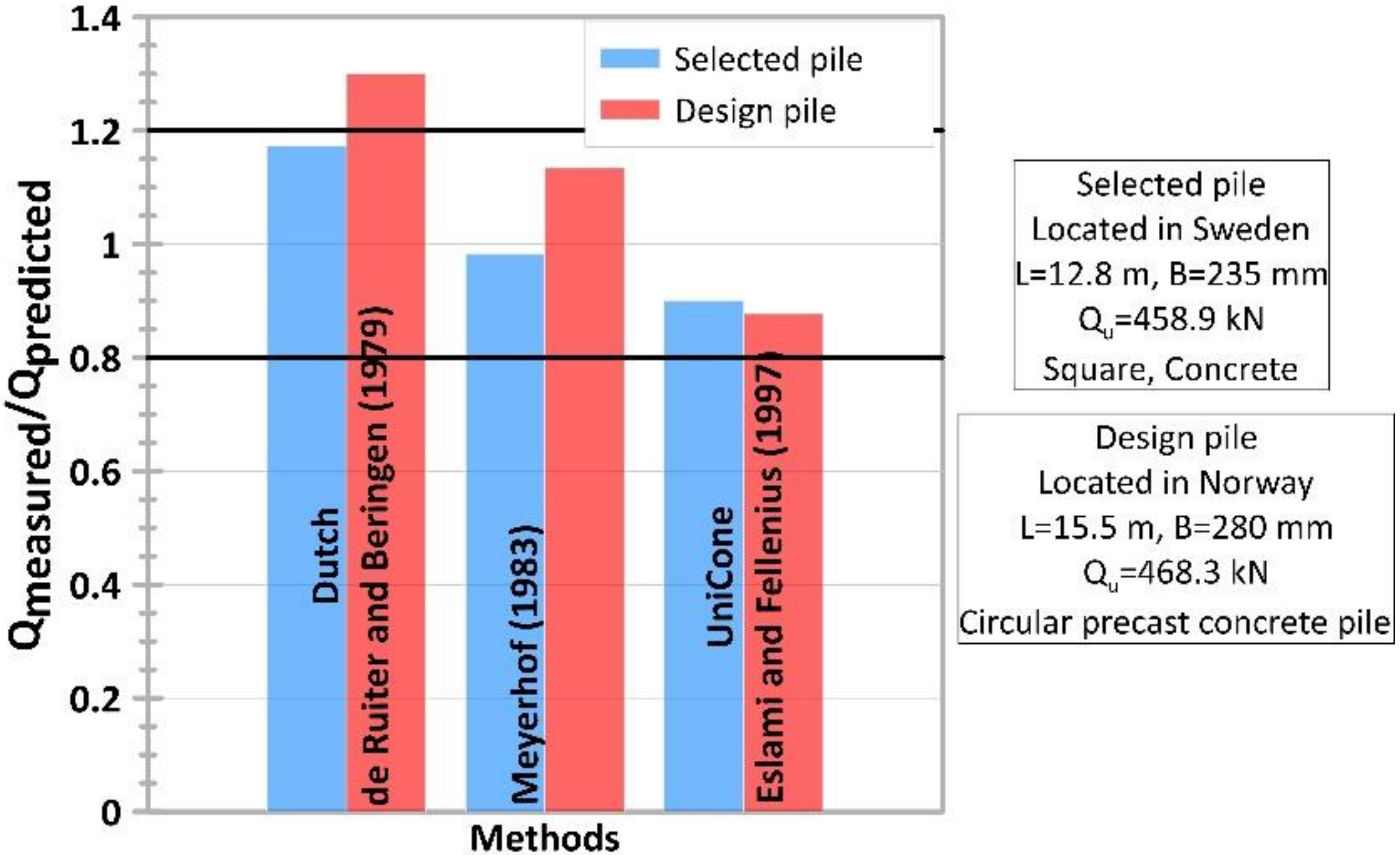
Typical Case 1

No.	Case ID	Location	Shape	Material	L (m)	B (mm)	Loading Mode	Qu (kN)
1	Selected Pile	Sweden	Square	Concrete	12.8	235	Compression	458.9
2	Design Pile	Norway	Circular	Concrete	15.5	280	Compression	468.3



CPT records and pile load test for selected and design piles (AUT:Geo-CPT&Pile database)

Typical Case 1



Variation of model parameter for selected and design piles

Database Summary

- 71 Cases of Driven Piles
- Driven in Sand, Clay and Mixed Deposits
- Embedment Depths between 6 to 56 m
- Diameter between 235 to 914 mm

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Research Paper

Load-displacement appraisal and analysis for driven piles; a data-centric approach

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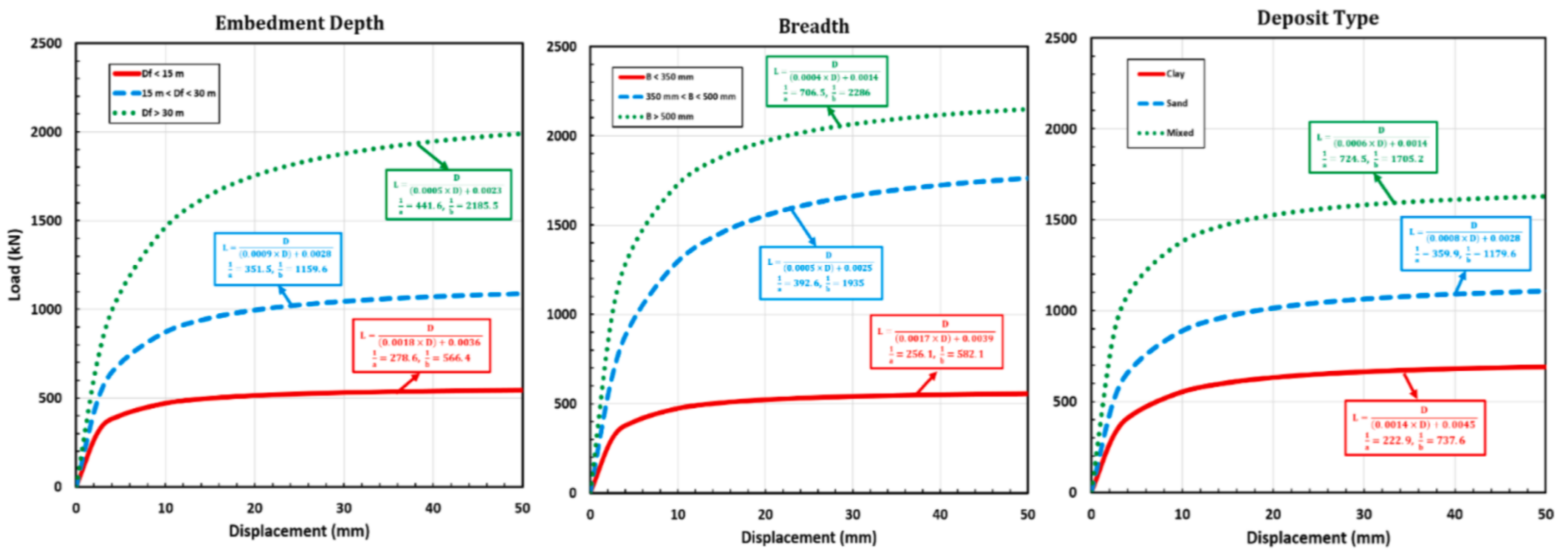
A R T I C L E I N F O

Keywords:
Data-centric
Load-displacement
Database
Driven piles
Normalization
Ultimate load
Linear stiffness

A B S T R A C T

Data-centric geotechnics is an ever-evolving field for facilitating digital transformation. The major issues for foundation design, bearing capacity, settlement and interactions of super and substructure are inherently emanated in load–displacement records. Considering the pivotal role of load–displacement behavior, it is believed to be a data center in foundation engineering. In this study, from the compiled FELADD database, including twelve foundation types load–displacement records, 71 driven piles have been gained. Aiming toward the data-centric approach, records have been processed, organized and filtered. For computing ultimate and limit load, three criteria of 10%B, Brinch-Hansen 80% and hyperbolic function are engaged. Through adopting promising criteria for normalizing load–displacement, dominant factors including embedment depth, breadth and surrounding soil type were appraised. The results indicated that higher stiffness and ultimate load are achieved for piles with higher embedment depth and larger breadth, in competent layers. Load-displacement normalization has revealed significant points. The relative displacement of 1% is recognized as the appropriate point in elastic stiffness calculation, somehow compatible with safety factor of 2. Moreover, the yielding trend is mobilized for relative displacement in the range of 5 to 10%. Overall, the load–displacement records and processing, as a data center proceeds value engineering in foundation design.

Trending of Load–displacement for Dominant Factors



(a) (b) (c)

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

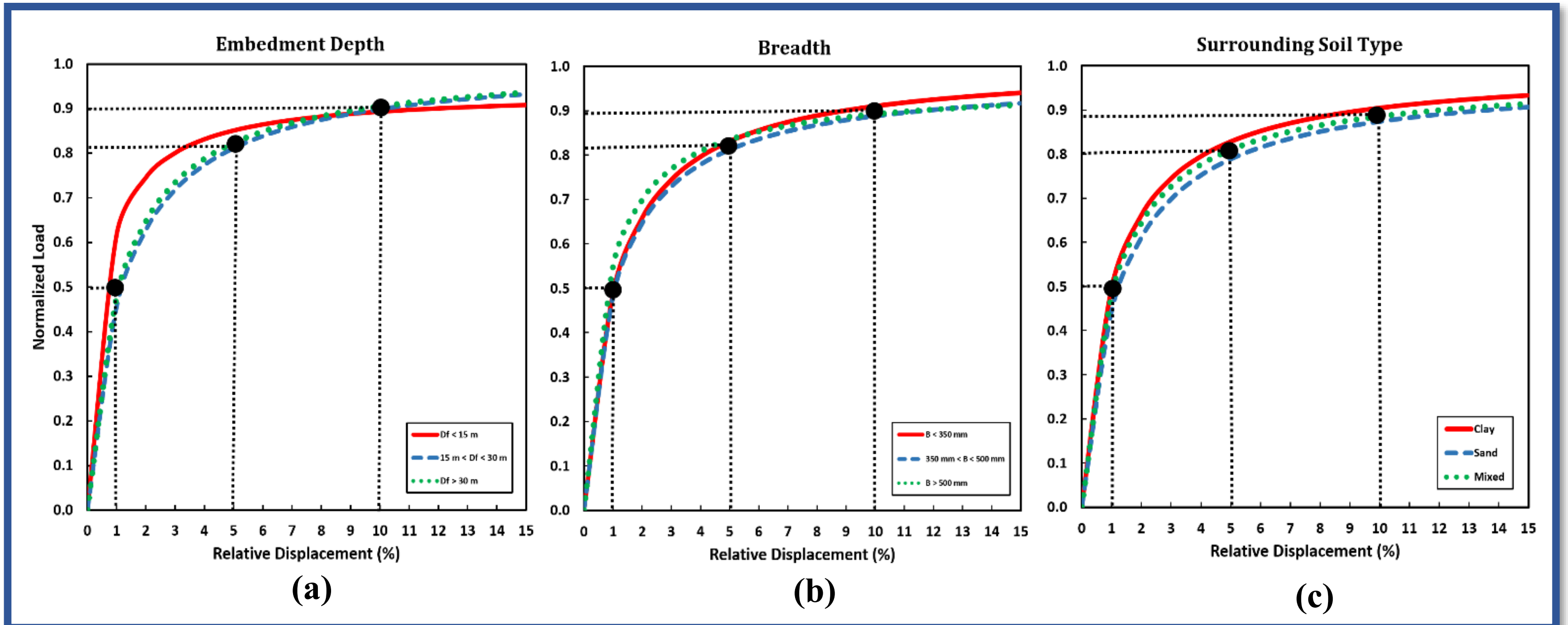
Normalized Trending

Normalization Approach:

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

Relative Displacement & Normalized Load:

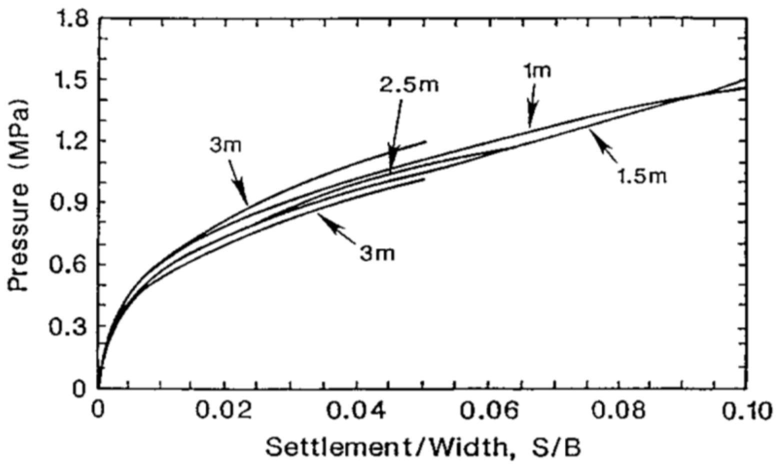
- 1 % → 0.5 Pu (FS=2)
- 5 % → 0.8 Pu
- 10 % → 0.9 Pu ➡ From CPT-Based Methods



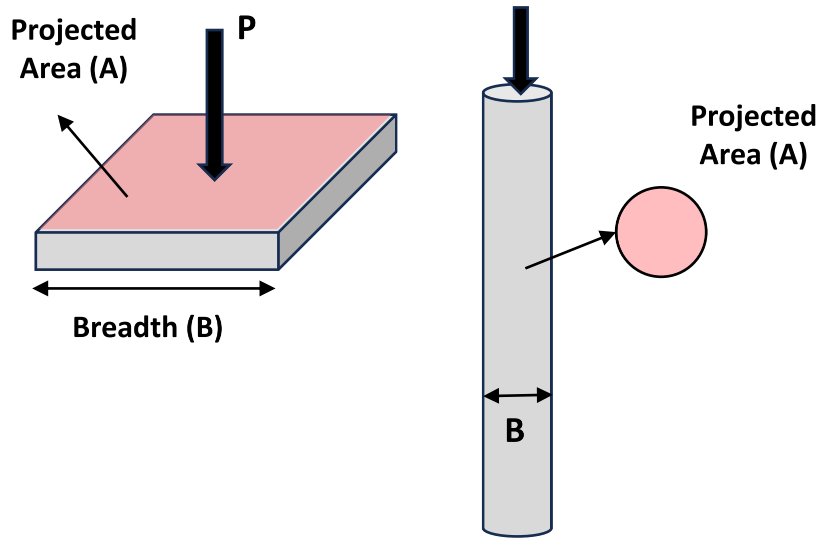
Normalized hyperbolic trending of load-displacement for dominant factors: a) embedment depth, b) breadth, c) surrounding soil type (Eslami & Ebrahimipour, 2024)

Stress-Strain Approach

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



Footings stress - strain (Briaud & Gibbens, 1999)

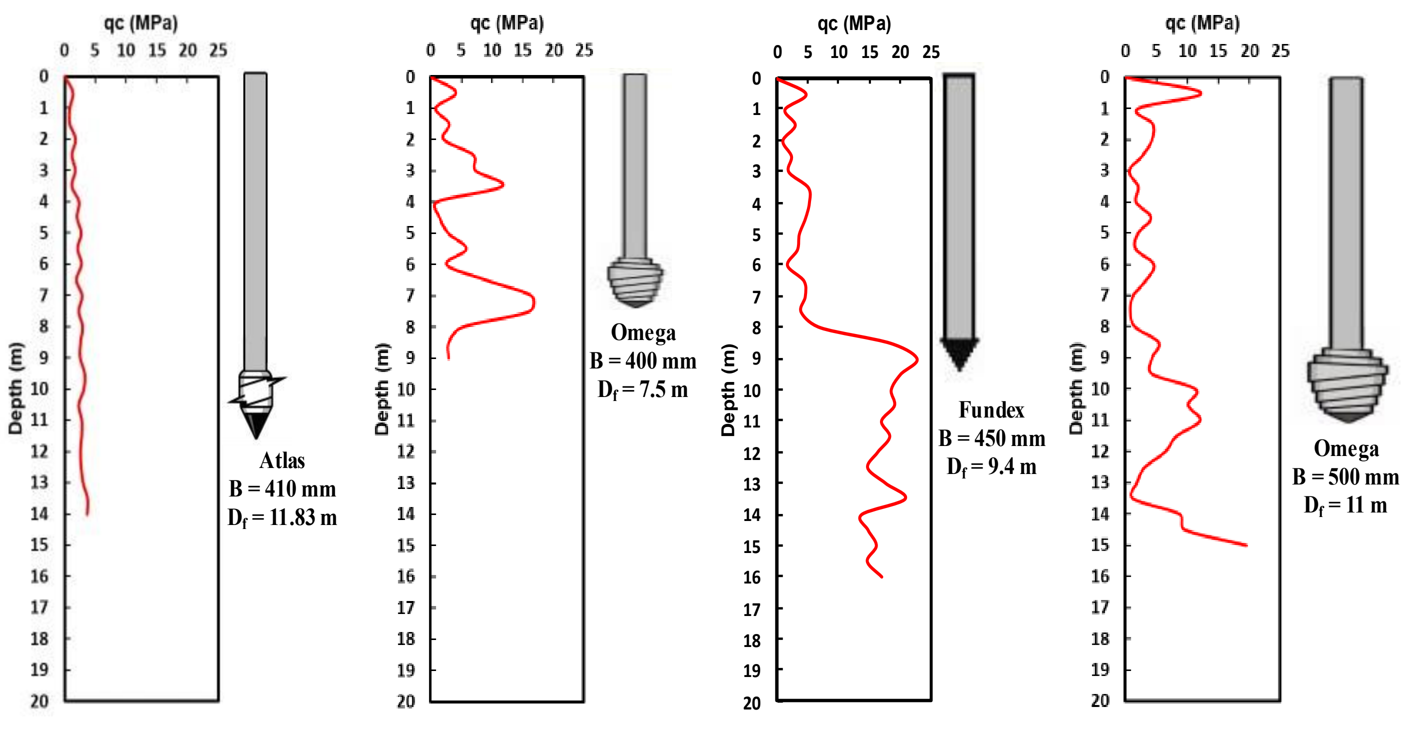


$$\text{Stress } (\sigma) = \frac{\text{Axial Load } (P)}{\text{Projected Area } (A)}$$

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

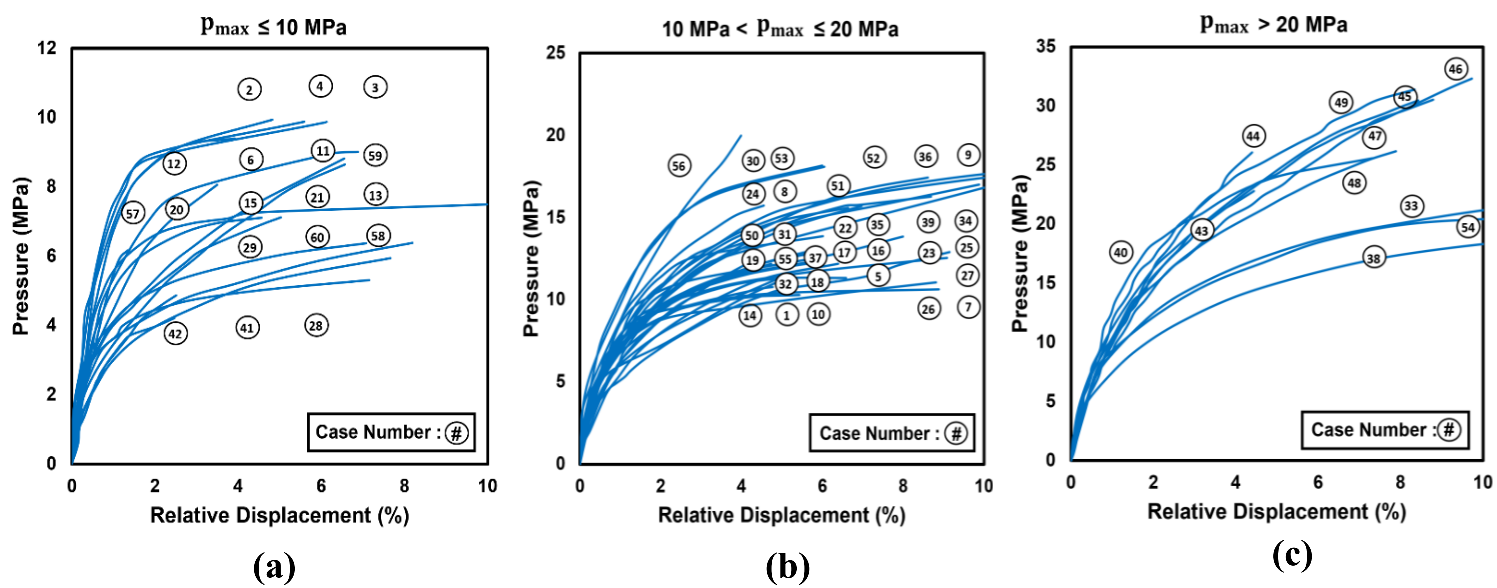
Database Summary

- 60 Cases of DD Piles: Fundex, De Waal, Olivier, Berkel D, APGD, Omega, and Atlas
- Installed in Sand, Clay and Mixed Deposits
- Embedment Depths between 6 to 21 m
- Diameter between 356 to 720 mm



Cone penetration test (CPT) q_c and schematic pile profile for selected cases

Stress-Strain Trending

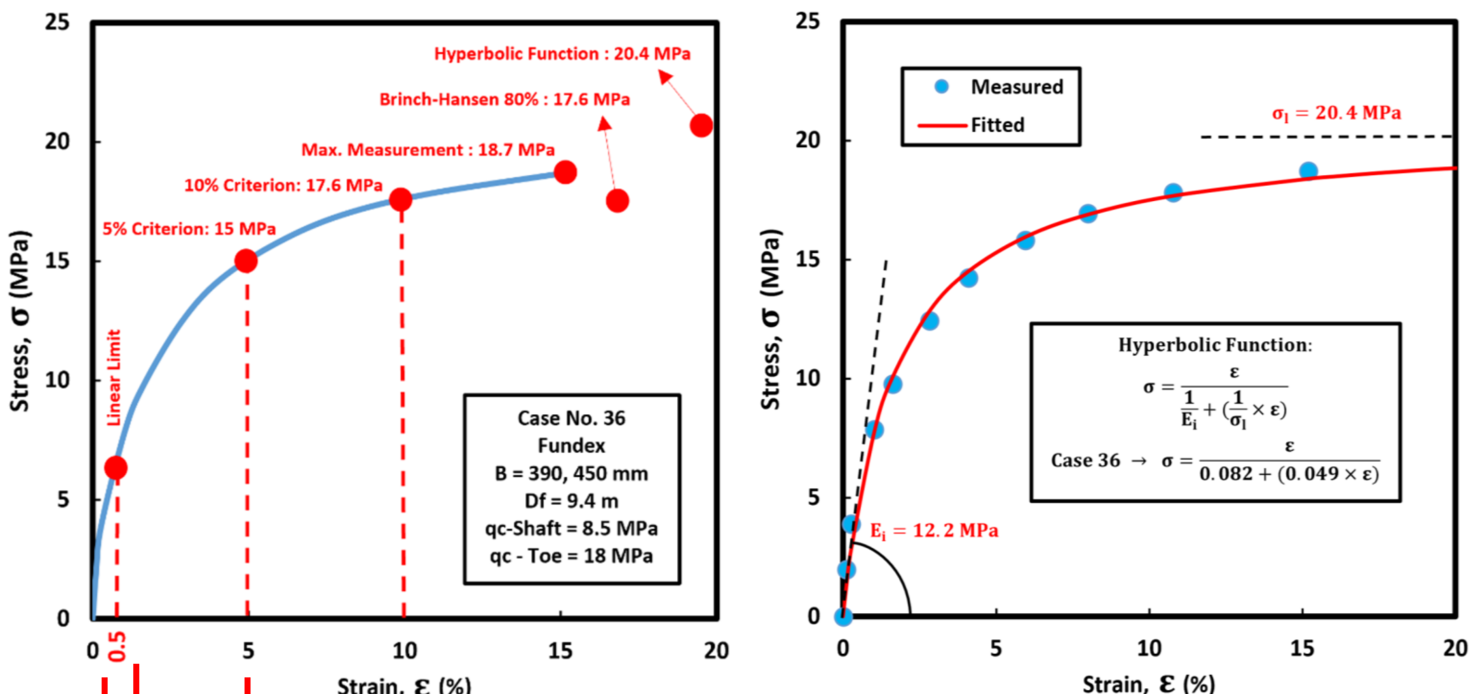


Pressure (stress)-relative displacement (strain) diagrams for all compiled cases: a) pmax ≤ 10 MPa, b) 10 < pmax ≤ 20 MPa, c) pmax > 20 MPa (Eslami et al., 2026)

Dominant Factors Appraisal

- Embedment Depth
- Pile Breadth
- Surrounding Deposit: (Shaft and Toe)

Dominant Criteria & Hyperbolic Fitting

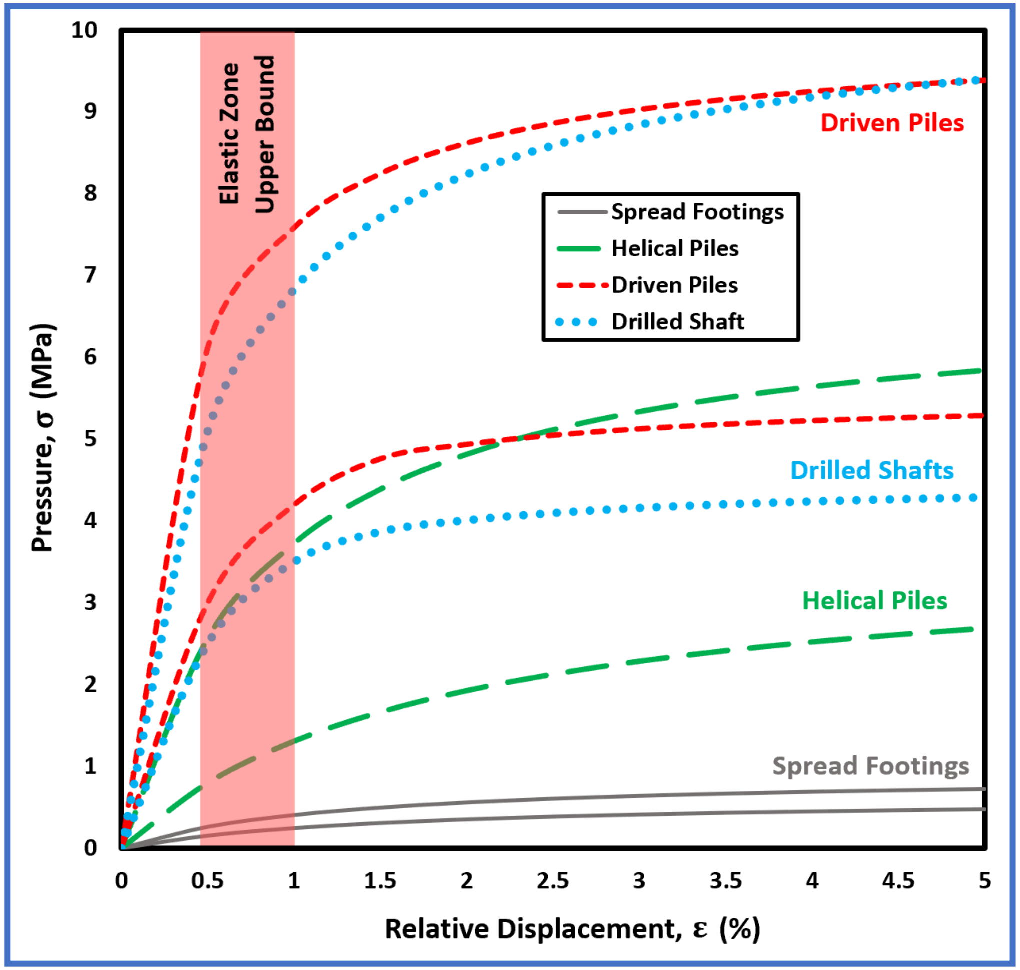


Ultimate Limit State (ULS), Plastic: 1 → 5 to 10 %

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

Operation State, Linear Elastic: 0.01 → 0.1 %

Stress – Strain Comparison for Various Foundations



Pressure (Stress, σ) – relative displacement (Strain, ε) diagrams for various foundations; Spread Footings: Heidari et al. (2025), Helical Piles: Rahimi et al. (2024), Driven piles: Eslami & Ebrahimipour (2024)

❖ Data Provision

- Soil behavior captured efficiently and comprehensively
- Deriving site-specific stiffness and strength parameters
- Reducing uncertainty, supplementing lab tests

❖ Geotechnical Design

- Determining optimum embedment depth
- Estimating axial capacity directly & indirectly
- Supporting reliable load-displacement predictions

❖ Load Transfer Mechanisms

- Integrating CPT records with measured Load–Displacement curves
- Resistance distribution: Separating shaft and toe resistances
- Capturing true mobilization at working loads, not only ultimate capacity

❖ Verification & Reliability

- Calibrating models with measured Load–Displacement records
- Validating CPT-based predictions against field performance
- Reducing uncertainty through performance-based verification

1. Digital Intelligence

From discrete snapshots to continuous subsurface intelligence, high-resolution CPTu data eliminates the blind spots of traditional sampling, ensuring design is built on the true stratigraphic record.

2. Interaction Mastery & Design Logic

The foundation is an integral part of a unified system, where soil–foundation interaction binds the substructure and superstructure into a single, harmonious behavioral response. Direct CPTu-based design bypasses empirical correlation, transforming cone resistance into precise load-transfer models through a reproducible, mathematical discipline.

3. The Reliability Handshake

When the measured load-displacement record aligns with the CPT-predicted trend, judgment is validated, uncertainty reduced, and professional confidence established.

4. Rational Implementation

Bearing capacity, settlement, and structural capability are not competing constraints, they are unified expressions of load-displacement behavior. The ground is a variable, the design adapts to it, but performance must be consistent.

Core Principle:

True performance-based foundation design is achieved by unifying;

"load, resistance, and deformation within a calibrated load–displacement framework"

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Special Thanks to:

Amirhossein Ebrahimipour; Research Fellow

For His Insightful Collaboration

Thank You;
Questions Welcome

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