State Key Laboratory for Geomechanics & Deep Underground Engineering
(China University of Mining and Technology)

Piezocone and Cone Penetration Tests (CPTu & CPT) Applications in Geotechnical & Foundation Engineering

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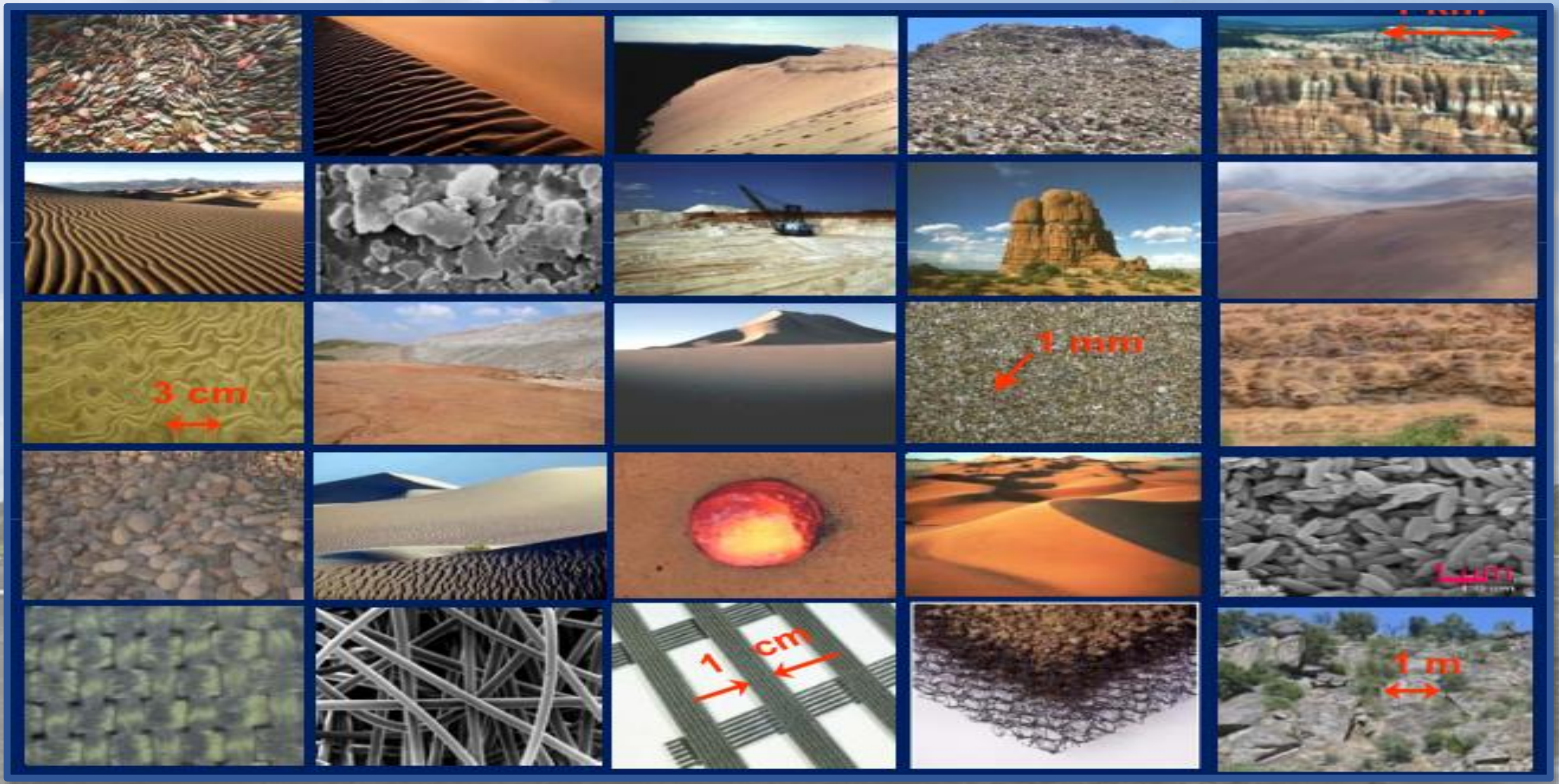
Outline

1 Geotechnical Engineering (GE)
2 Geotechnical Site Investigations
3 Background to Foundation Engineering (FE)
4 Cone and Piezocone Penetration Tests (CPT & CPTu)
5 Application of CPT & CPTu in GE
6 CPT and Foundation Engineering: Scale Effect
7 Geotechnical Design: Bearing Capacity & Settlement
8 Case Studies
9 Summary and Conclusion
10 Major References

1. Geotechnical Engineering

1.1. Geotechnical Engineering World

- ❖ Geomaterials: Soil, Rock, Ground Water
- ❖ Geosynthetics: Geotextile, Geogrid, Geomembrane, …



1.2. Major Topics in Geotechnical Engineering (GE)

(1) Sample recovery

(2) Subsurface profiling

(3) Groundwater conditions

(4) Consequences of human activities

(5) Site response to geohazards

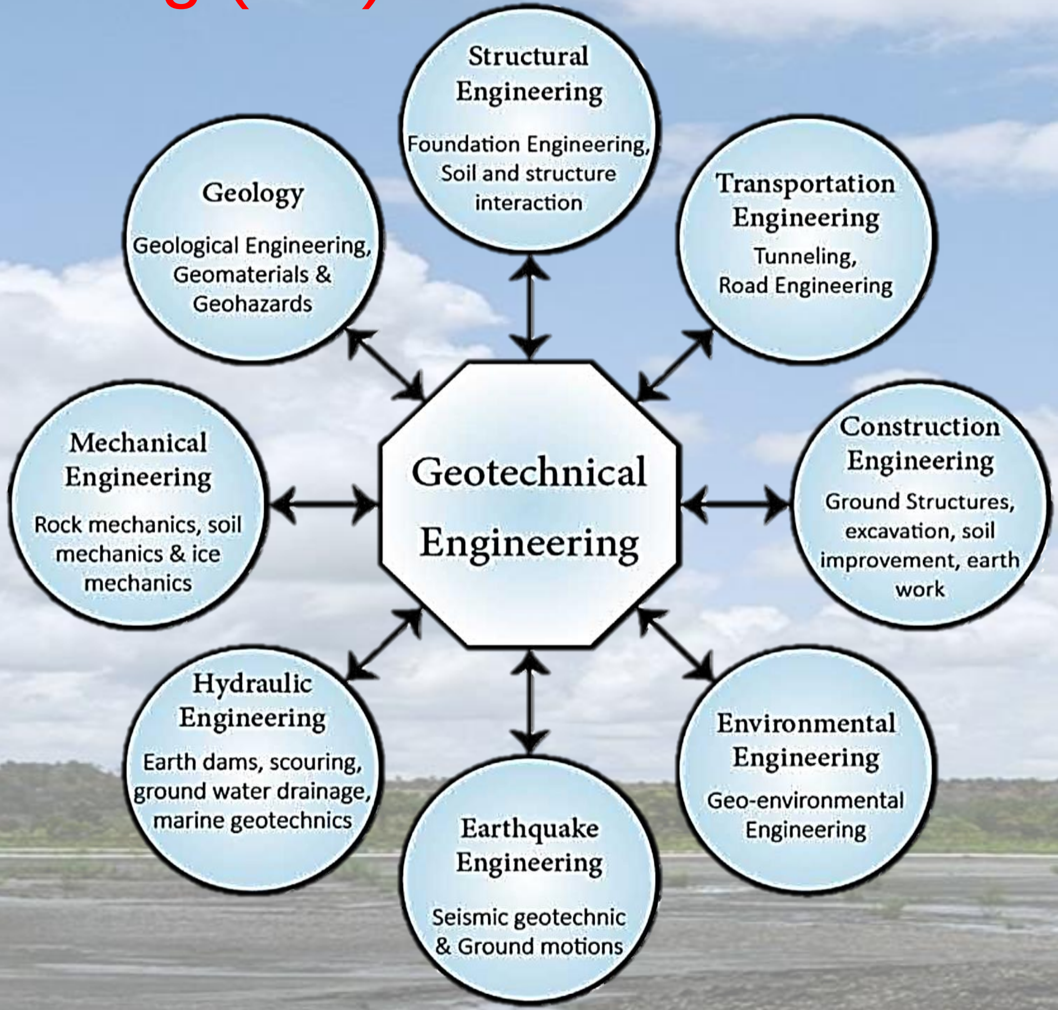
(6) Selecting and design of foundation systems

(7) Sufficiency of geomaterials for borrowing

(8) Health, safety and strategy management

(9) Recognition of underground structures behavior

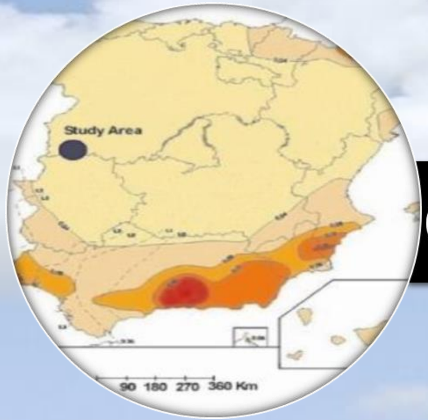
(10) Support and stabilization of deposits and slopes



Overlap of Geotechnical Engineering with Other Disciplines

2. Geotechnical Site Investigations

2.2. Data Sources

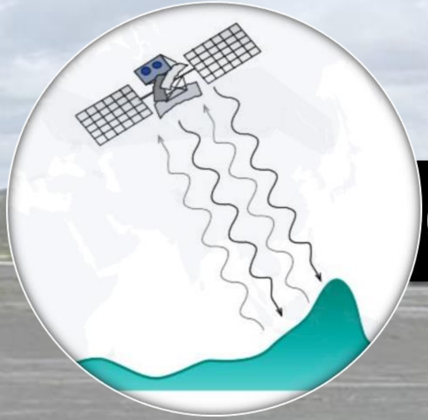


01 Maps

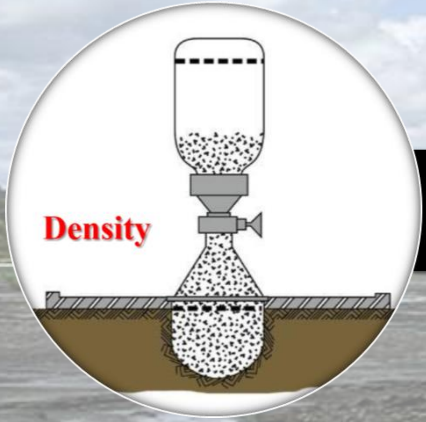
02 Aerial Photos

03 Site Visit

04 Non Destructive Tests



05 Remote Sensing



06 On Situ Testing

2. Geotechnical Site Investigations

2.2. Data Sources

07 Boring and Sampling

08 In-situ Penetration Testing

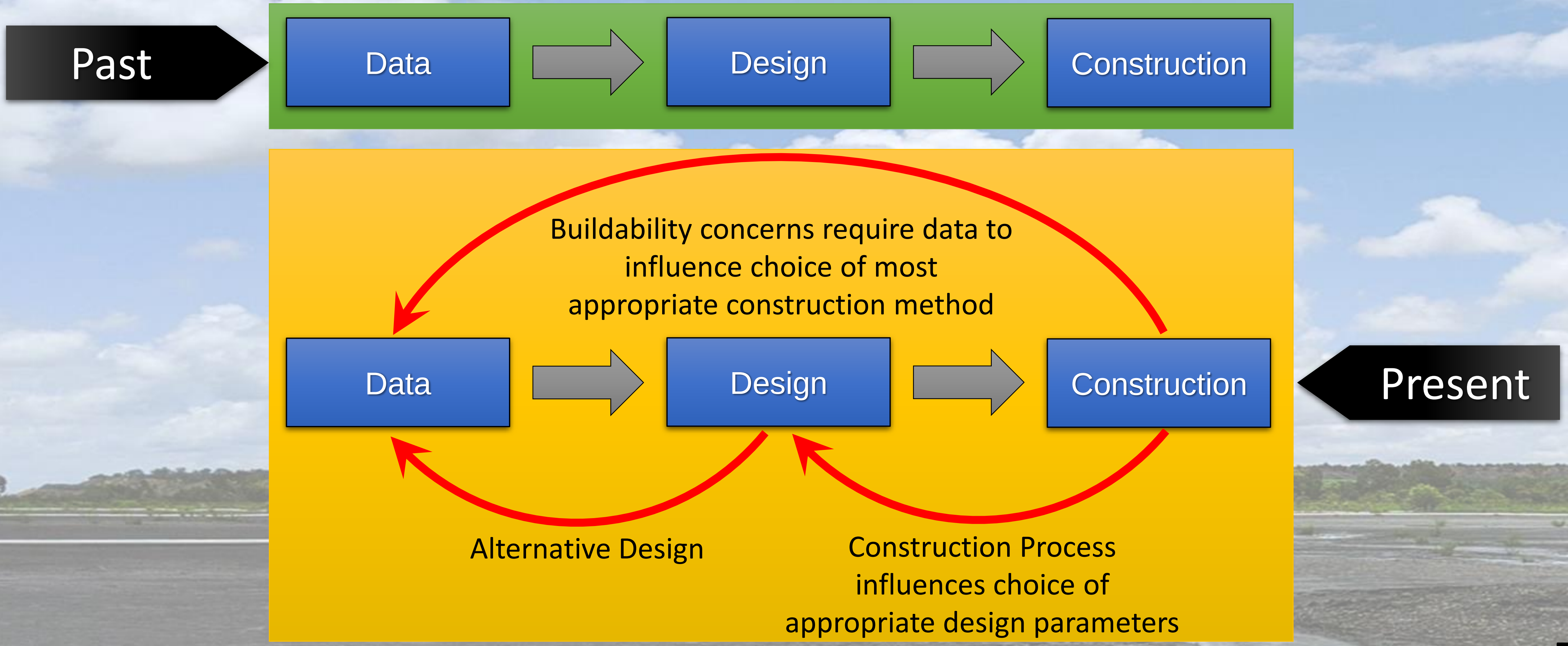
09 Laboratory Element Testing

10 Physical Modeling

11 Full-scale Tests

12 Instrumentation and Monitoring

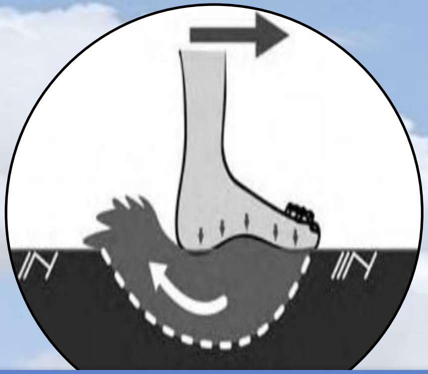
2.3. Cycle of Data, Design, and Performance



(ICE, Manual Geotechnics, 2012)

2.4. Major Approaches: On-Situ Testing

Visual & Manual



Surface Strength

Soil Density

Pocket Vane

Pocket Penetrometer

Field Moisture

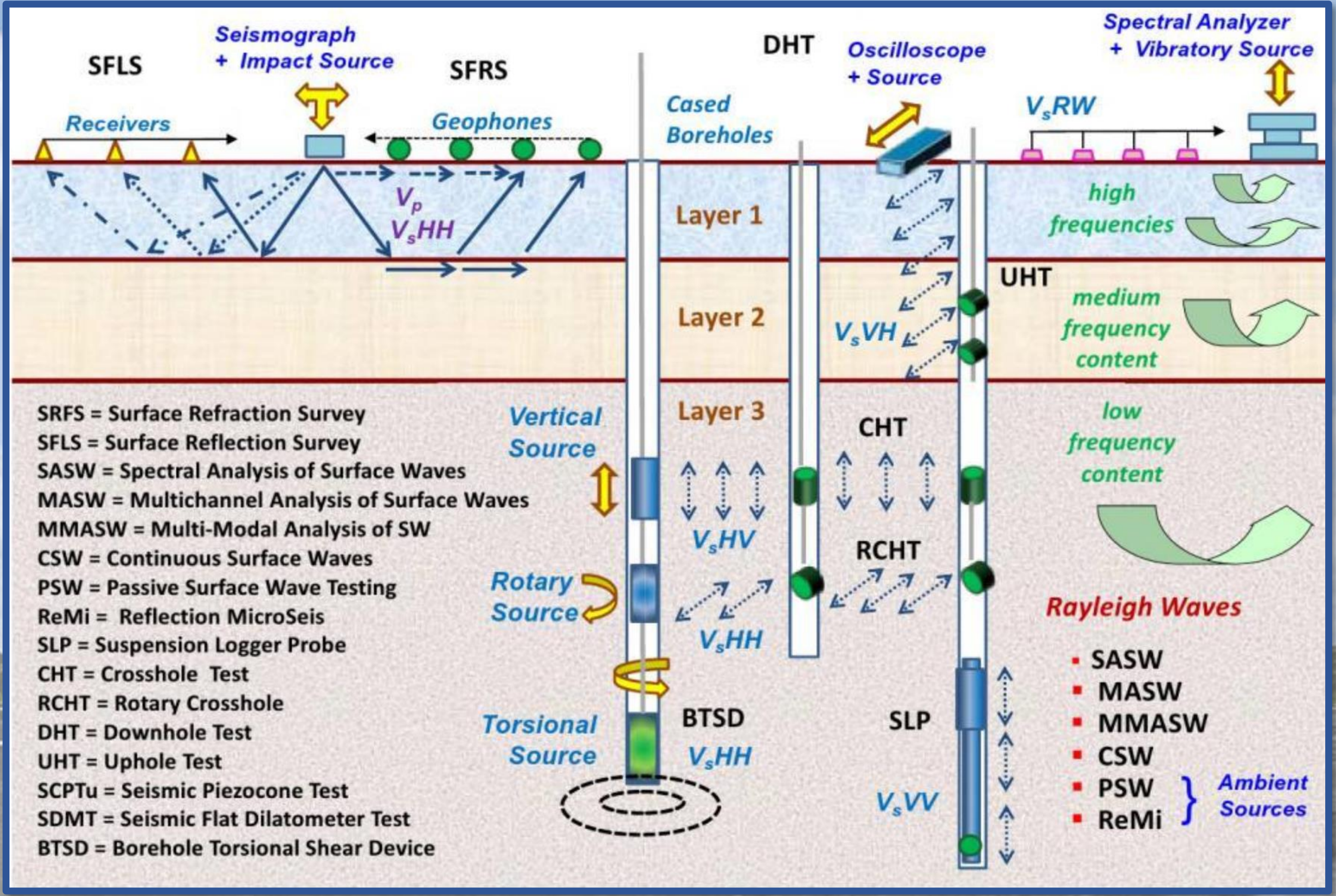
DCPT

PLT

CBR

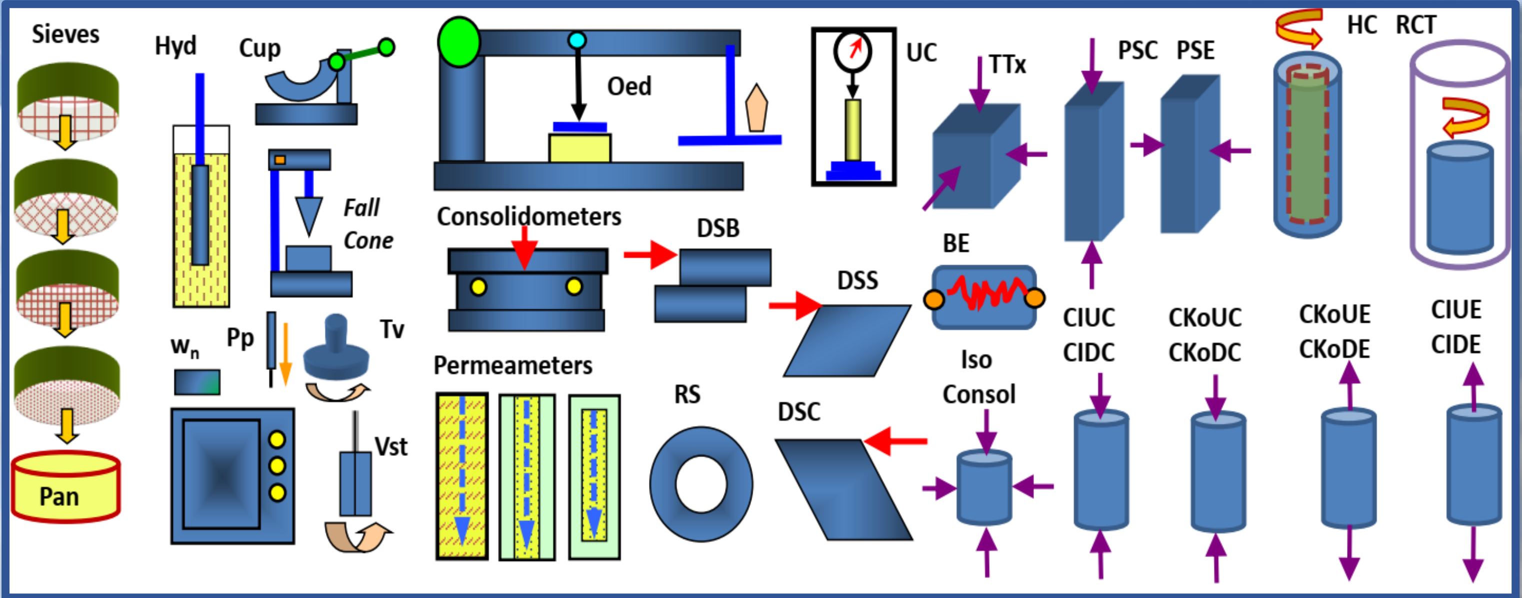
In Situ Direct Shear Test

2.5. Major Approaches: Geophysical Testing Methods



(Mayne, 2016)

2.6. Major Approaches: Boring, Sampling & Laboratory Testing

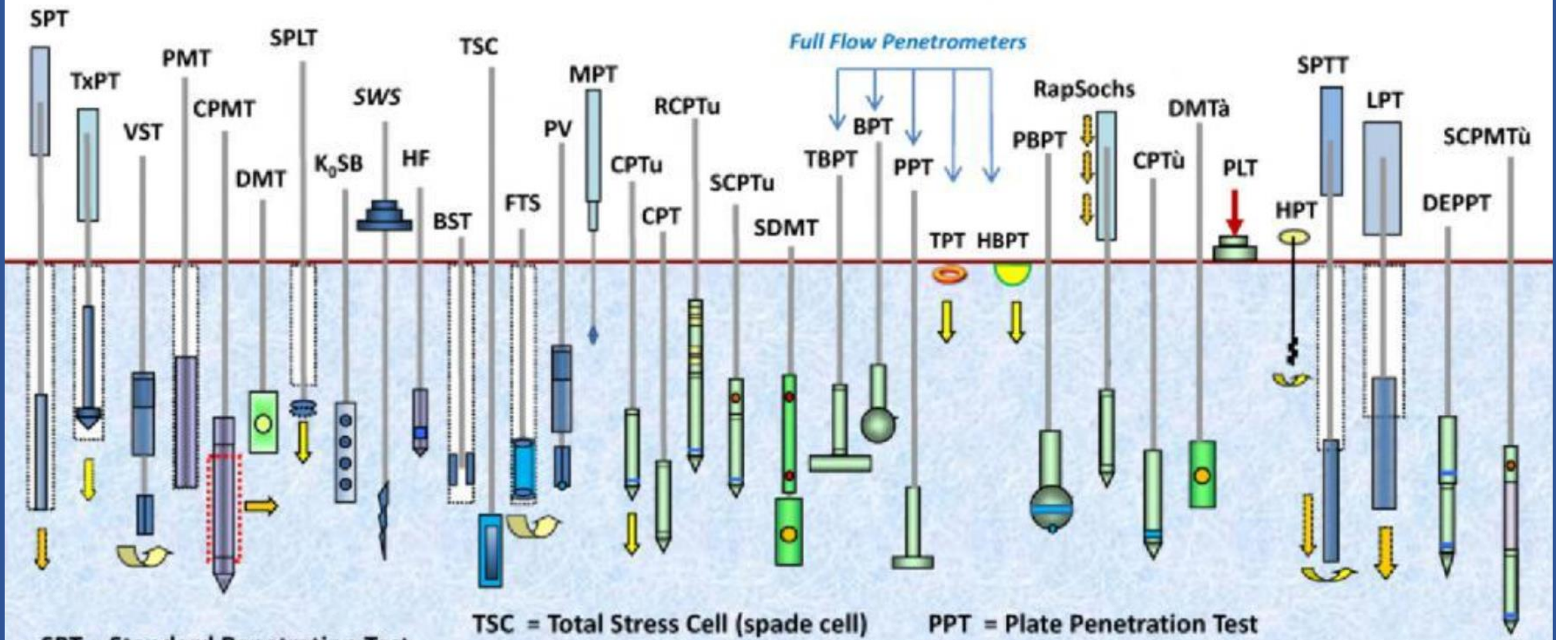


Grain size analyses
Hydrometer
Water content by oven
Liquid limit cup
Plastic limit thread
Fall cone device
Pocket penetrometer
Torvane
Unconfined compression
Miniature vane
Digital image analysis

Mechanical oedometer
Consolidometer
Constant rate of shear (CRS)
Falling-head permeameter
Constant-head permeameter
Flow permeameter
Direct shear box
Ring shear
Unconsolidated undrained Tx
Simple shear
Directional shear cell

Triaxial apparatus (iso-consols, CIUC, CKoUC, CAUC, CIUE, CAUE, CKoUE, stress path, CIDC, CKoDC, CIDE, CKoDE, constant P')
Plane strain apparatus (PSC, PSE)
True triaxial (cuboidal)
Hollow cylinder
Torsional Shear
Resonant Column Test device
Non-resonant column
Bender elements

2.7. Major Approaches: Field Testing Devices and Probes



SPT = Standard Penetration Test
TxPT = Texas Penetration Test
VST = Vane Shear Test
PMT = Pressurementer Test
CPMT = Cone Pressuremeter
DMT = Dilatometer Test
SPLT = Screw Plate Load Test
ISB = Iowa K_0 Stepped Blade
SWS = Swedish Weight Sounding
HF = Hydraulic Fracture
BST = Borehole Shear Test

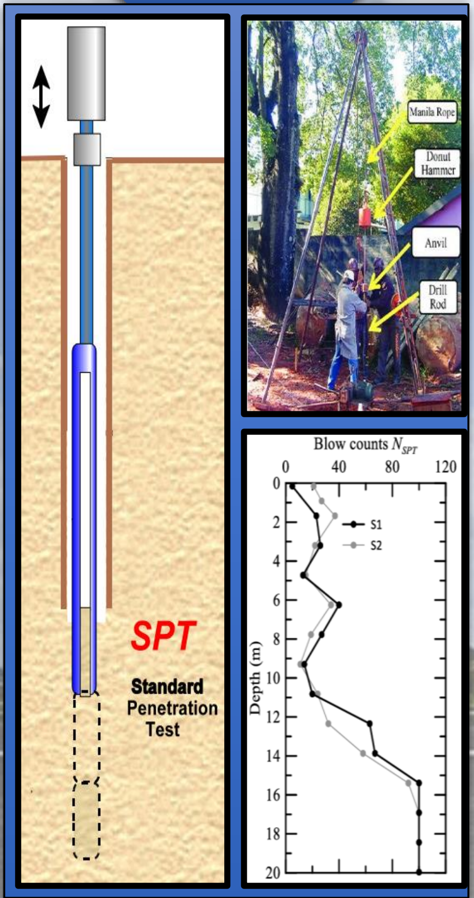
TSC = Total Stress Cell (spade cell)
FTS = Freestand Torsional Shear
PV = Piezovane
MPT = Macintosh Probe Test
CPT = Cone Penetration Test
CPTu = Piezocone Penetration
RCPTu = Resistivity Piezocone
SCPTu = Seismic Cone
SDMT = Seismic Flat Dilatometer
TBPT = T-Bar Penetrometer Test
BPT = Ball Penetrometer
TPT = Toroid Penetrometer Test

PPT = Plate Penetration Test
PLT = plate load test
HPT = Helical Probe Test
PBPT = piezoball penetration test
RapSochs = Rapid soil characterization system
CPTù = piezodissipation test
DMTà = Dilatometer with A-reading dissipations
SPTT = Standard Penetration Test with Torque
LPT = Large Penetration Test
DEPPT = Dual Element PiezoProbe Test
HBPT = hemi-ball penetration test
SCPMTu = Seismic Piezocone Pressuremeter

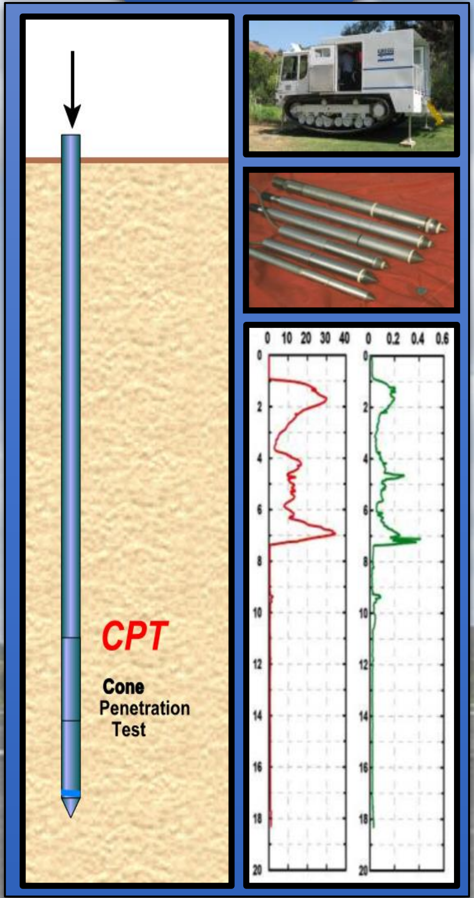
(Mayne, 2016)

2.8. Major Approaches: In Situ Penetration Tests

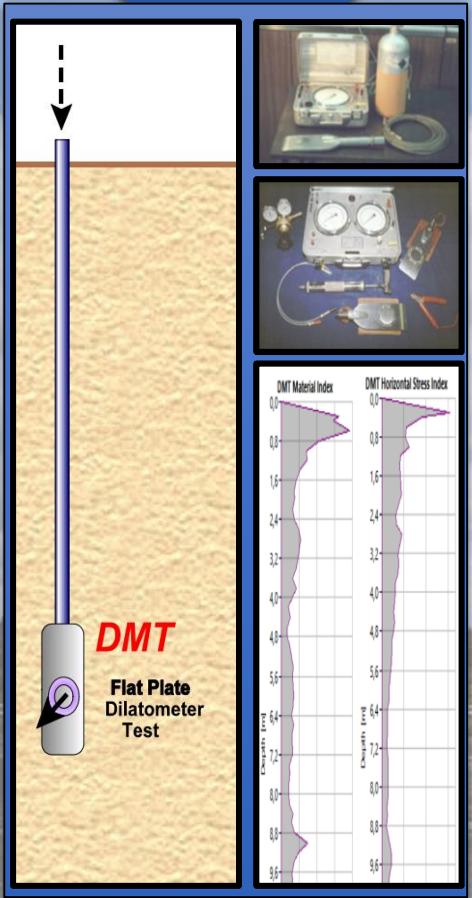
SPT



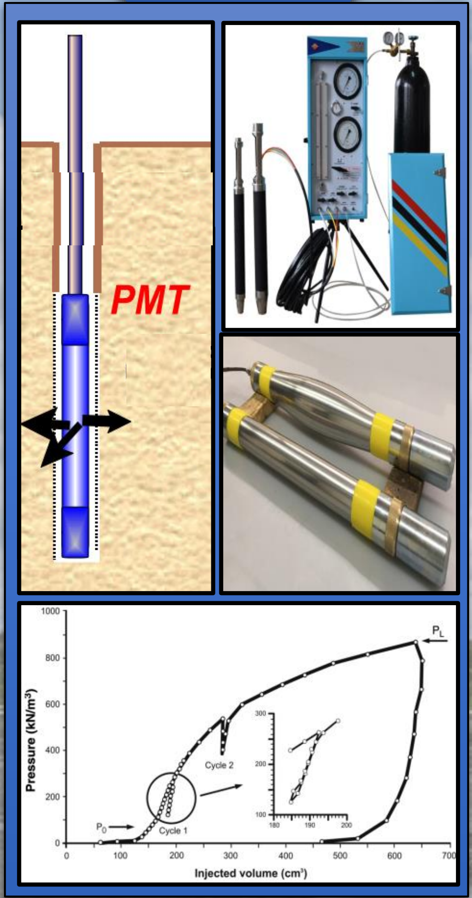
CPT



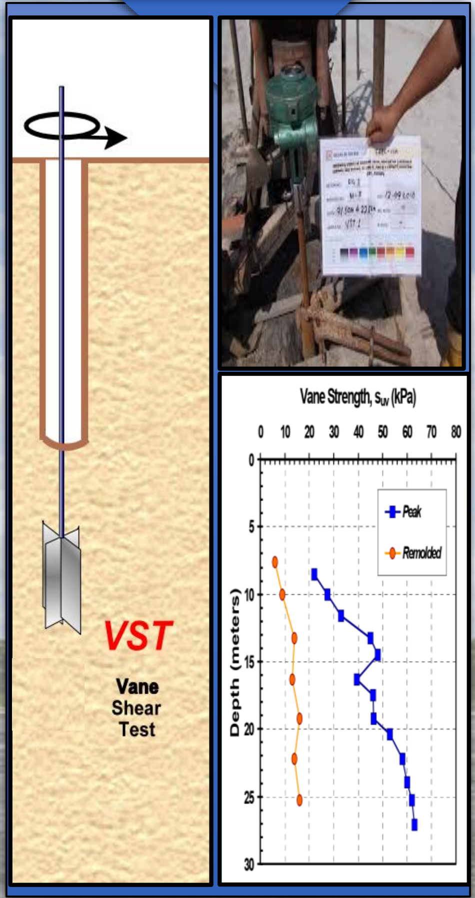
DMT



PMT



VST



2.9. In Situ Tests and Their Applicability

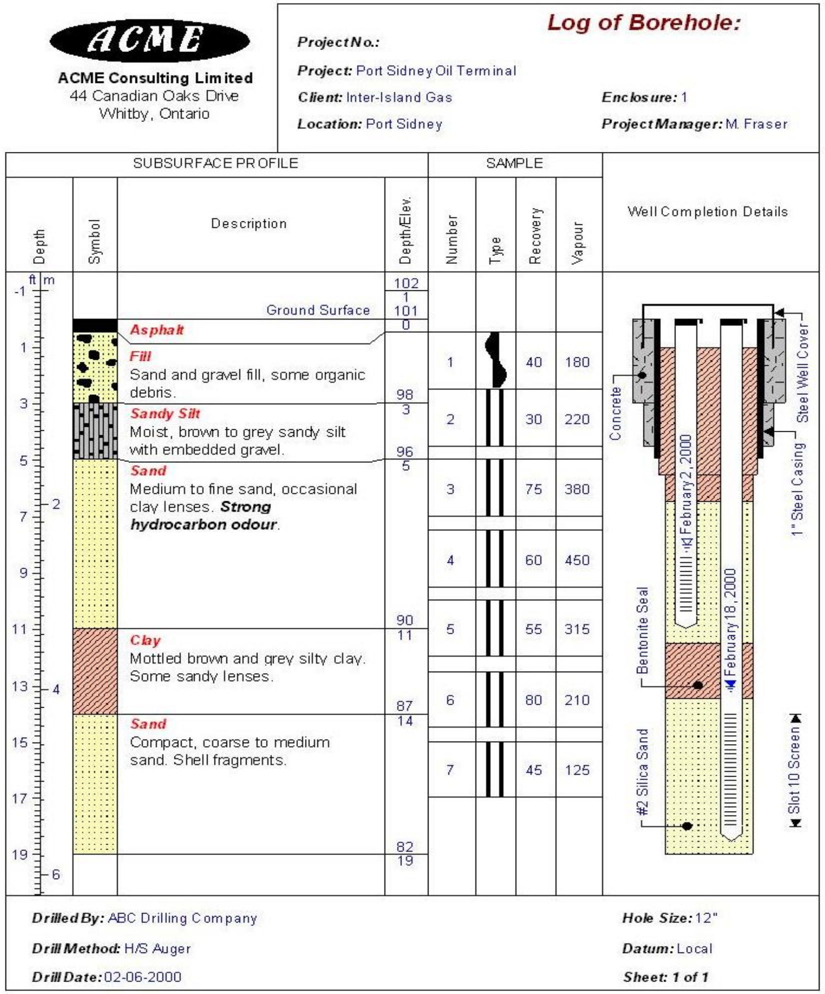
Group	Device	Soil Parameters													Ground Type						
		Soil type	Profile	u	*ϕ'	S_u	I_D	m_v	c_v	k	G_0	σ_h	OCR	σ-ε	Hard rock	Soft rock	Gravel	Sand	Silt	Clay	Peat
Penetrometers	Dynamic	C	B	-	C	C	C	-	-	-	C	-	C	-	-	C	B	A	B	B	B
	Mechanical	B	A/B	-	C	C	B	C	-	-	C	C	C	-	-	C	C	A	A	A	A
	Electric (CPT)	B	A	-	C	B	A/B	C	-	-	B	B/C	B	-	-	C	C	A	A	A	A
	Piezocone (CPTU)	A	A	A	B	B	A/B	B	A/B	B	B	B/C	B	C	-	C	-	A	A	A	A
	Seismic (SCPT/SCPTU)	A	A	A	B	A/B	A/B	B	A/B	B	A	B	B	B	-	C	-	A	A	A	A
	Flat dilatometer (DMT)	B	A	C	B	B	C	B	-	-	B	B	B	C	C	C	-	A	A	A	A
	Standard penetration test (SPT)	A	B	-	C	C	B	-	-	-	C	-	C	-	-	C	B	A	A	A	A
	Resistivity probe	B	B	-	B	C	A	C	-	-	-	-	-	-	-	C	-	A	A	A	A
Pressuremeters	Pre-bored (PBP)	B	B	-	C	B	C	B	C	-	B	C	C	C	A	A	B	B	B	A	B
	Self-boring (SBP)	B	B	A(1)	B	B	B	B	A(1)	B	A(2)	A/B	B	A/B(2)	-	B	-	B	B	A	B
	Full displacement (FDP)	B	B	-	C	B	C	C	C	-	A(2)	C	C	C	-	C	-	B	B	A	A
Others	Vane	B	C	-	-	A	-	-	-	-	-	-	B/C	B	-	-	-	-	-	A	B
	Plate load	C	-	-	C	B	B	B	C	C	A	C	B	B	B	A	B	B	B	A	A
	Screw plate	C	C	-	C	B	B	B	C	C	A	C	B	-	-	-	-	A	A	A	A
	Borehole permeability	C	-	A	-	-	-	-	B	A	-	-	-	-	A	A	A	A	A	A	B
	Hydraulic fracture	-	-	B	-	-	-	-	C	C	-	B	-	-	B	-	-	-	-	A	C
	Crosshole/downhole/surface seismic	C	C	-	-	-	-	-	-	-	A	-	B	-	A	A	A	A	A	A	A

Applicability: A = high, B = moderate, C =low, - = none

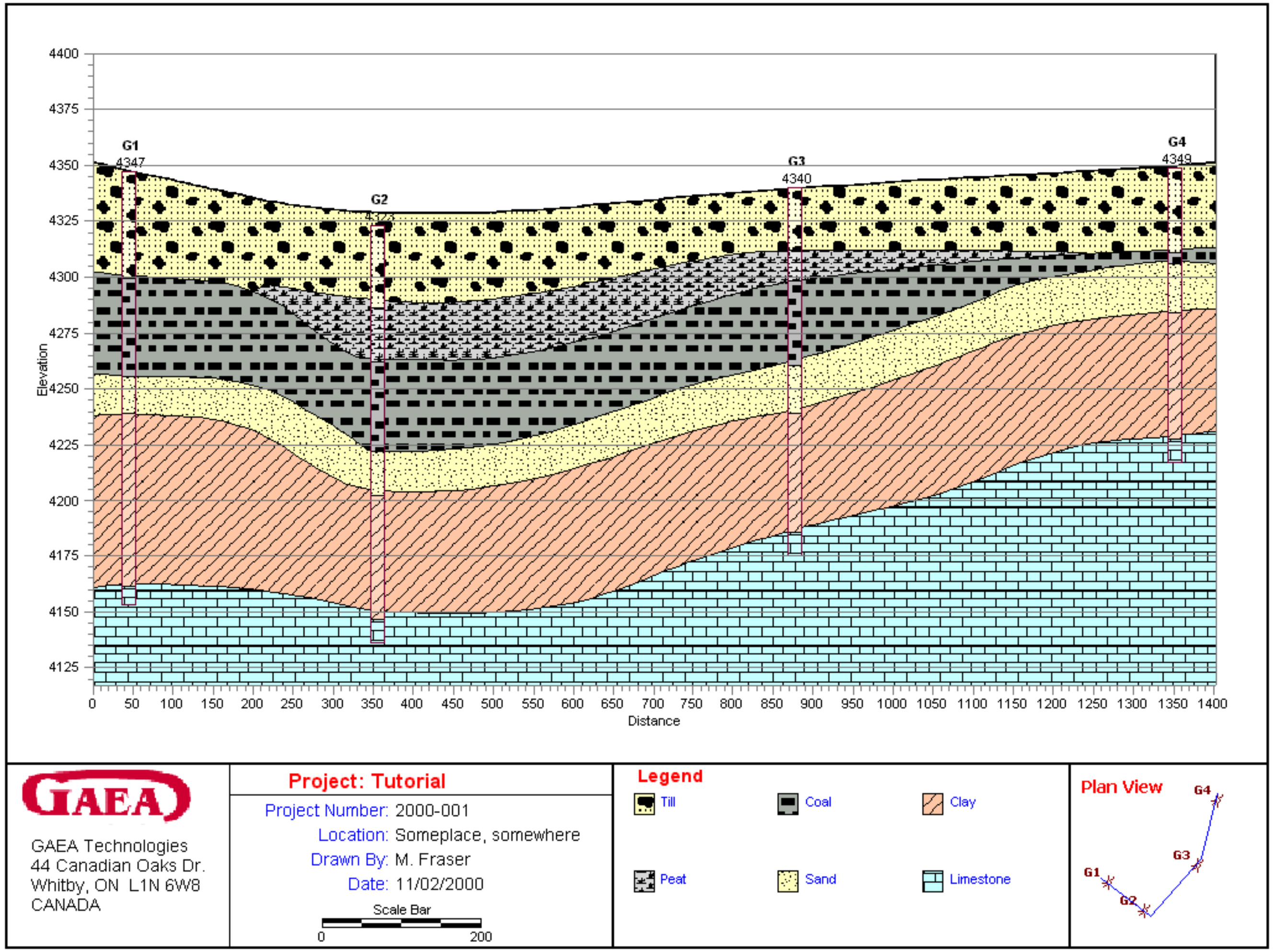
*ϕ' =Will depend on soil type, (1) = Only when pore pressure sensor fitted; (2) = Only when displacement sensor fitted

(Lunne et al., 1997)

2.10. Typical Subsurface Log & Profile



Number	Recovery	Vapour
1	40	180
2	30	220
3	75	380
4	60	450
5	55	315
6	80	210
7	45	125



2.11. Major Approaches: Instrumentation and Monitoring

1 Tilt Meter

2 Displacement Meter

3 Inclinometer

4 Borehole Extensometer

5 Piezometer

6 Load Cell

7 Strain Gauge

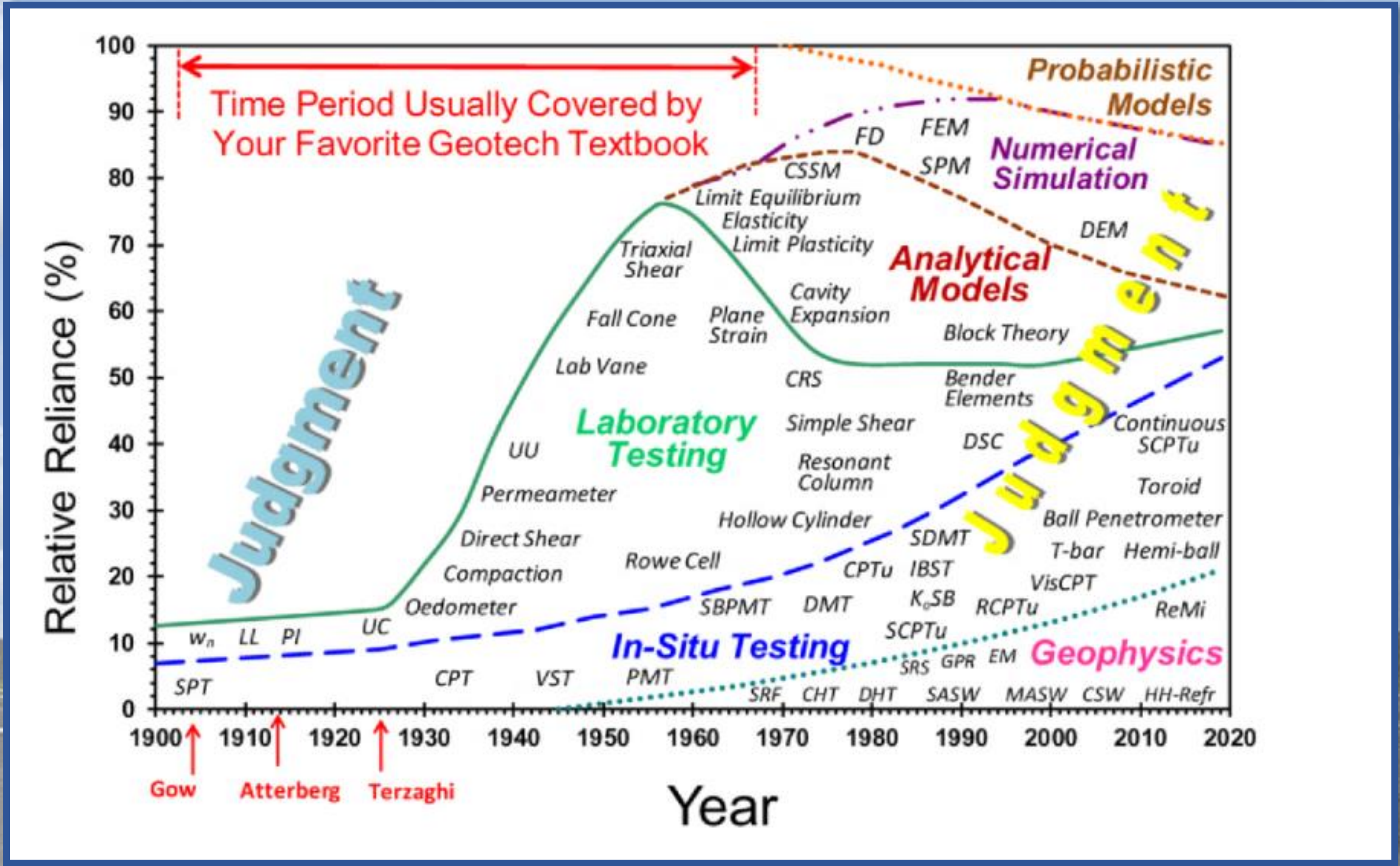
8 Laser Scanning

9 Seismograph & Accelerometer

2.12. In-Situ Testing vs. Laboratory Testing

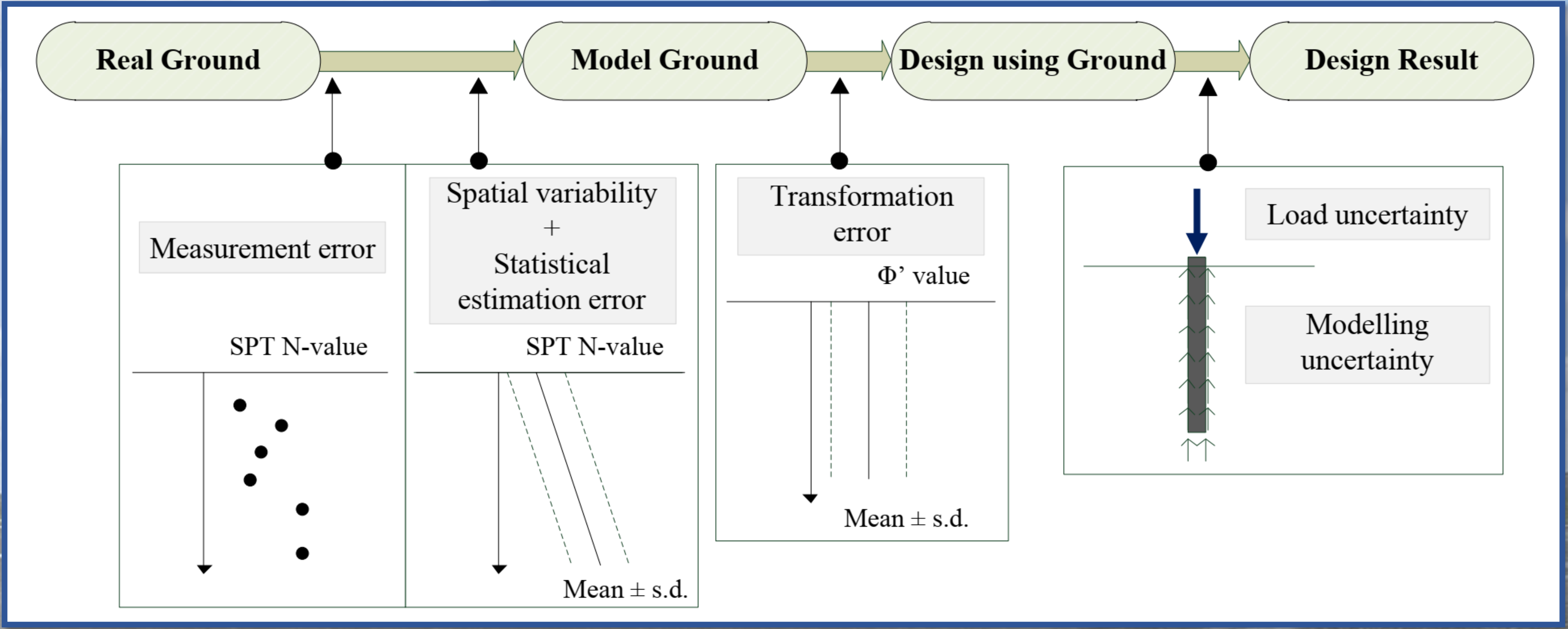
Laboratory Tests Limitations	Field Tests Advantages
Difficulties for undisturbed sampling	Overcome sampling difficulties
Soil disturbance & maintenance	No change in stress state
Soil volume change	Simple and fast
Omitting confinement pressure	Economical
Size effect and boundaries	Dominant applications in FE

2.13. Evolution of Geotechnical Design Basis



(Mayne, 2016, adapted from Lacasse 1985)

2.14. Uncertainty in GE



A procedure for geotechnical RBD (Honjo, 2011)

2.14. Uncertainty in GE

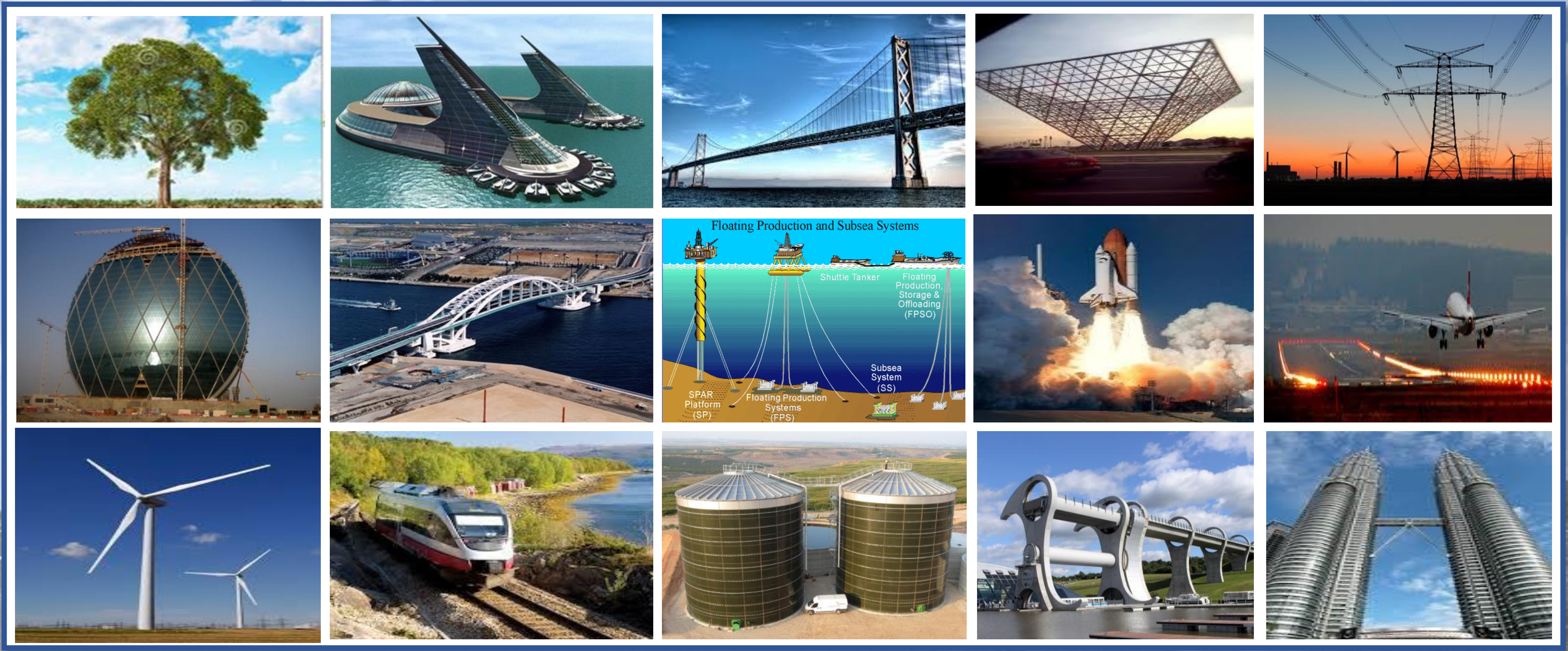
❖ **Variability of Laboratory & In-Situ Testing Data (Phoon & Kulhawy, 1999)**

Test	Property	Soil type	Coefficient of variation (%)	
			Range	Mean
Atterberg tests	Plasticity index	Fine grained	5 - 51	24
Triaxial compression	Effective angle of friction	Clay, silt	7 - 56	24
Direct shear	Shear strength	Clay, silt	19 - 20	20
Triaxial compression	Shear strength	Clay, silt	8 - 38	19
Direct shear	Effective angle of friction	Sand	13 - 14	14
Direct shear	Effective angle of friction	Clay	6 - 22	14
Direct shear	Effective angle of friction	Clay, silt	3 - 29	13
Atterberg tests	Plastic limit	Fine grained	7 - 18	10
Triaxial compression	Effective angle of friction	Sand, silt	2 - 22	8
Atterberg tests	Liquid limit	Fine grained	3 - 11	7
Unit weight	Density	Fine grained	1 - 2	1

Test	Equipment	Oper./proc.	Random	Total	Range
SPT	0.05 - 0.75	0.05 - 0.75	0.12 - 0.15	0.14 - 1.00	0.15 - 0.45
CPT	0.05	0.10 - 0.15	0.10 - 0.15	0.15 - 0.22	0.15 - 0.25
ECPT	0.03	0.05	0.05 - 0.10	0.07 - 0.12	0.05 - 0.15
VST	0.05	0.08	0.10	0.14	0.10 - 0.20
DMT	0.05	0.05	0.08	0.11	0.05 - 0.15
PMT	0.05	0.12	0.10	0.16	0.10 - 0.20
SBPMT	0.08	0.15	0.08	0.19	0.15 - 0.25

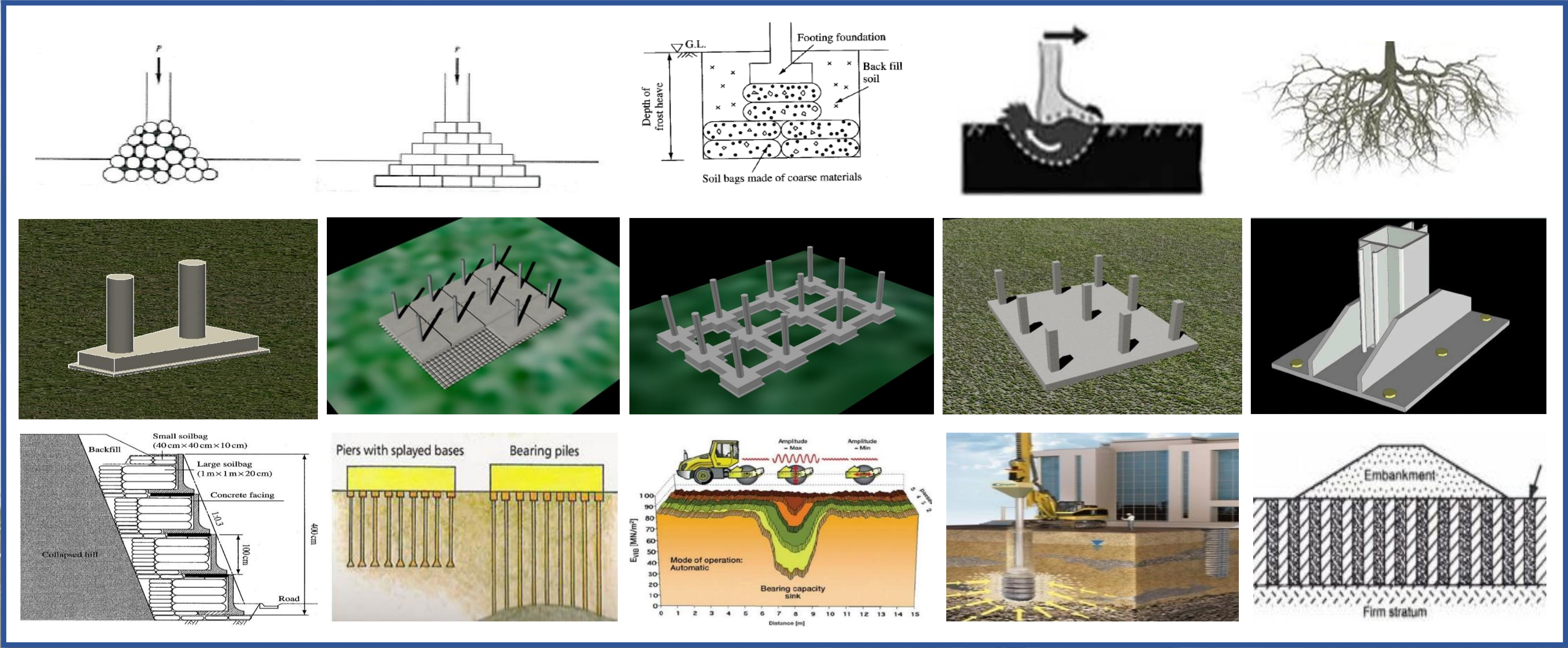
3. Background to Foundation Engineering

3.1. Typical Structures



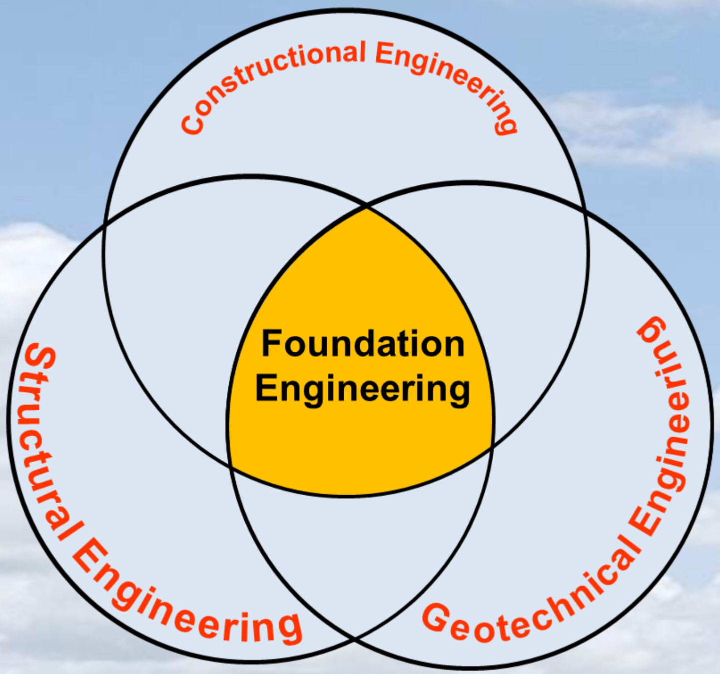
3. Background to Foundation Engineering

3.1. Various Foundations



3.2. Major Requirements: Analysis & Design

1. Bearing Capacity
2. Serviceability (Settlement and Torsion)
3. Structural Design
4. Stability Control
5. Full or Model Scale Testing
6. Constructional Aspects
7. Durability
8. Economic Requirements



Multidisciplinary: Structural, Geotechnical and Constructional

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.

3.3. Foundations Classification

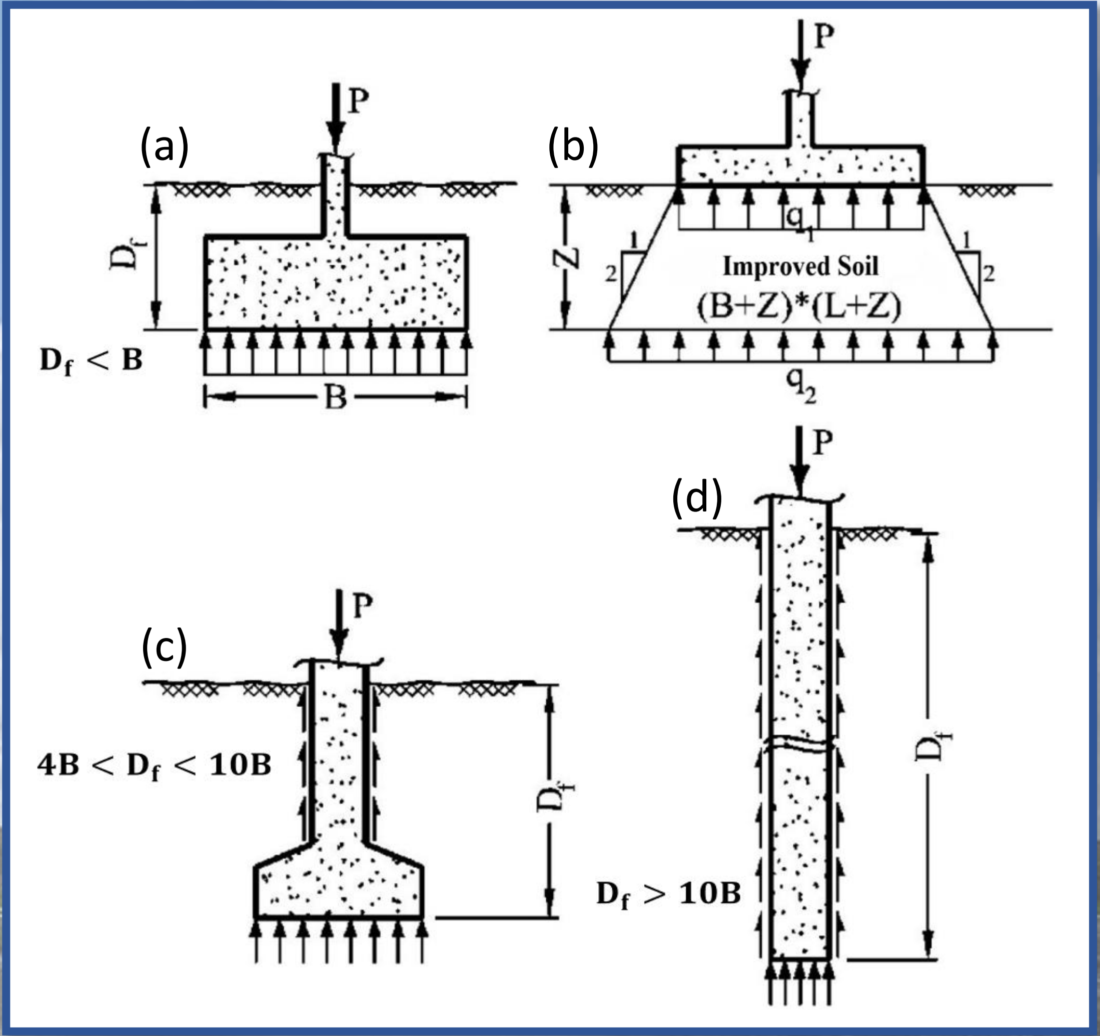
- **Embedment Depth**

✓ Shallow Foundations (a)

✓ Shallow Foundation + Soil Improvement (b)

✓ Semi-deep Foundations (c)

✓ Deep Foundations (d)

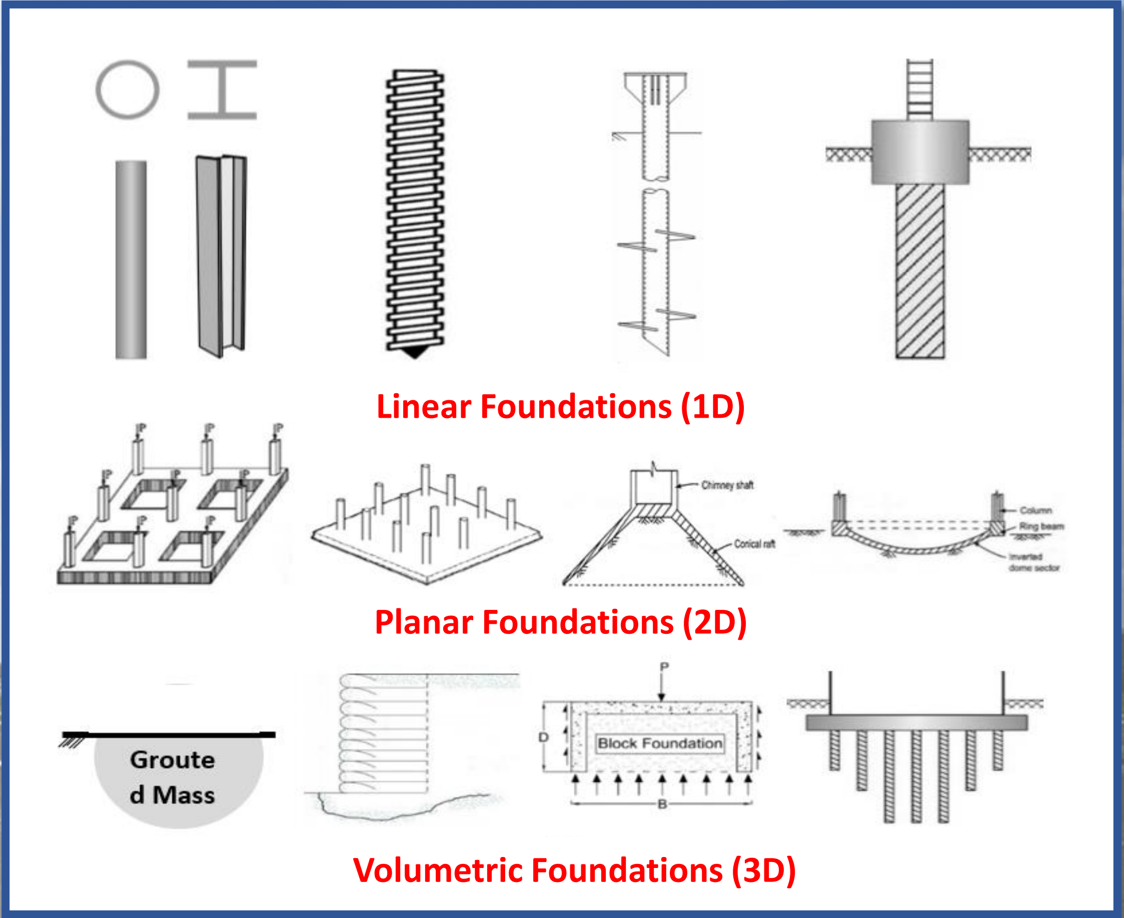


Current categories of foundations

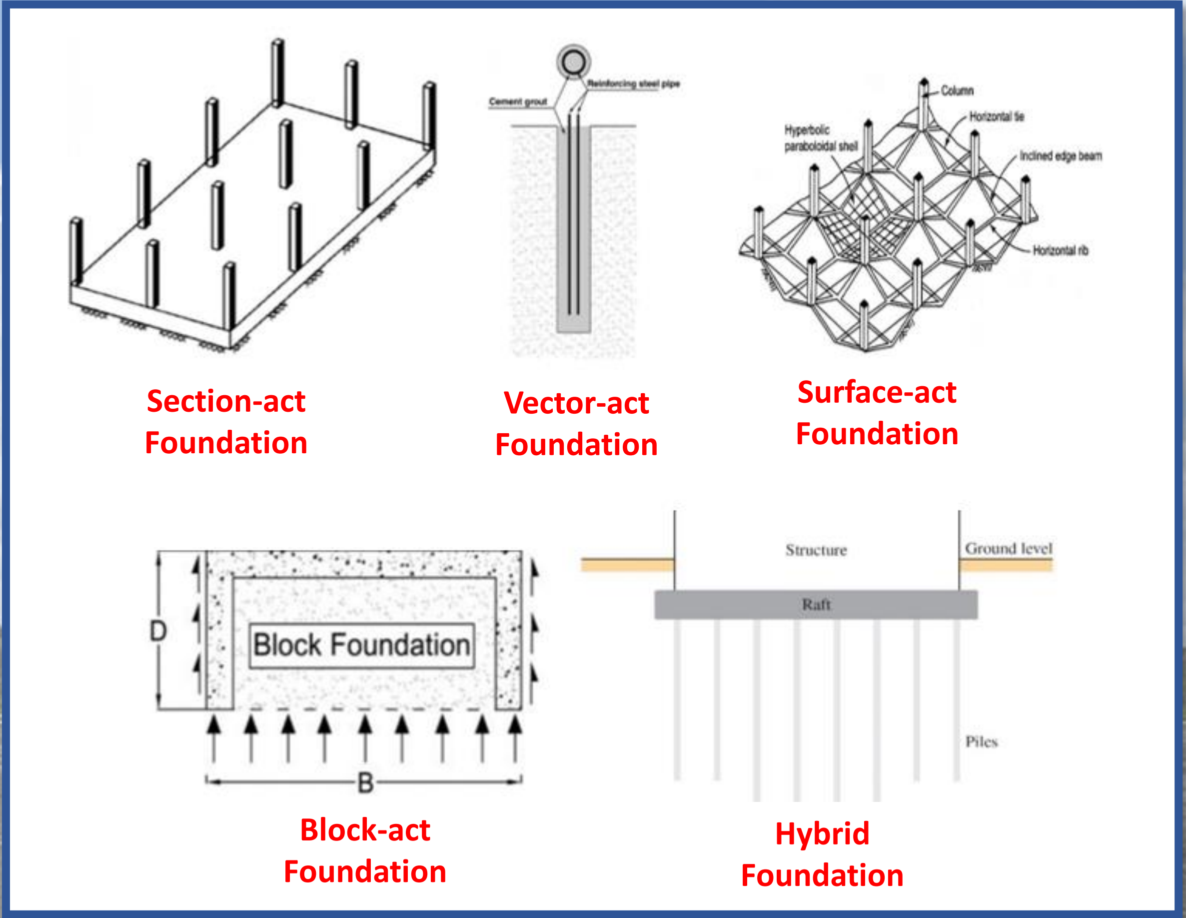
(Eslami et al., 2019)

3.3. Foundations Classification

- **Geometry**



- **Load Transition System**

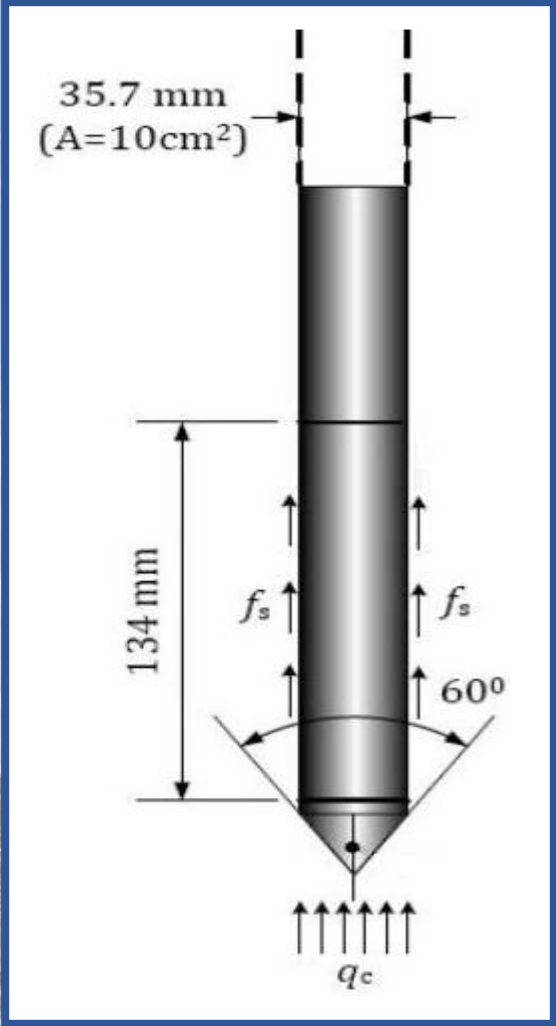


(Eslami & Ebrahimipour, 2022)

4.1. Background

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

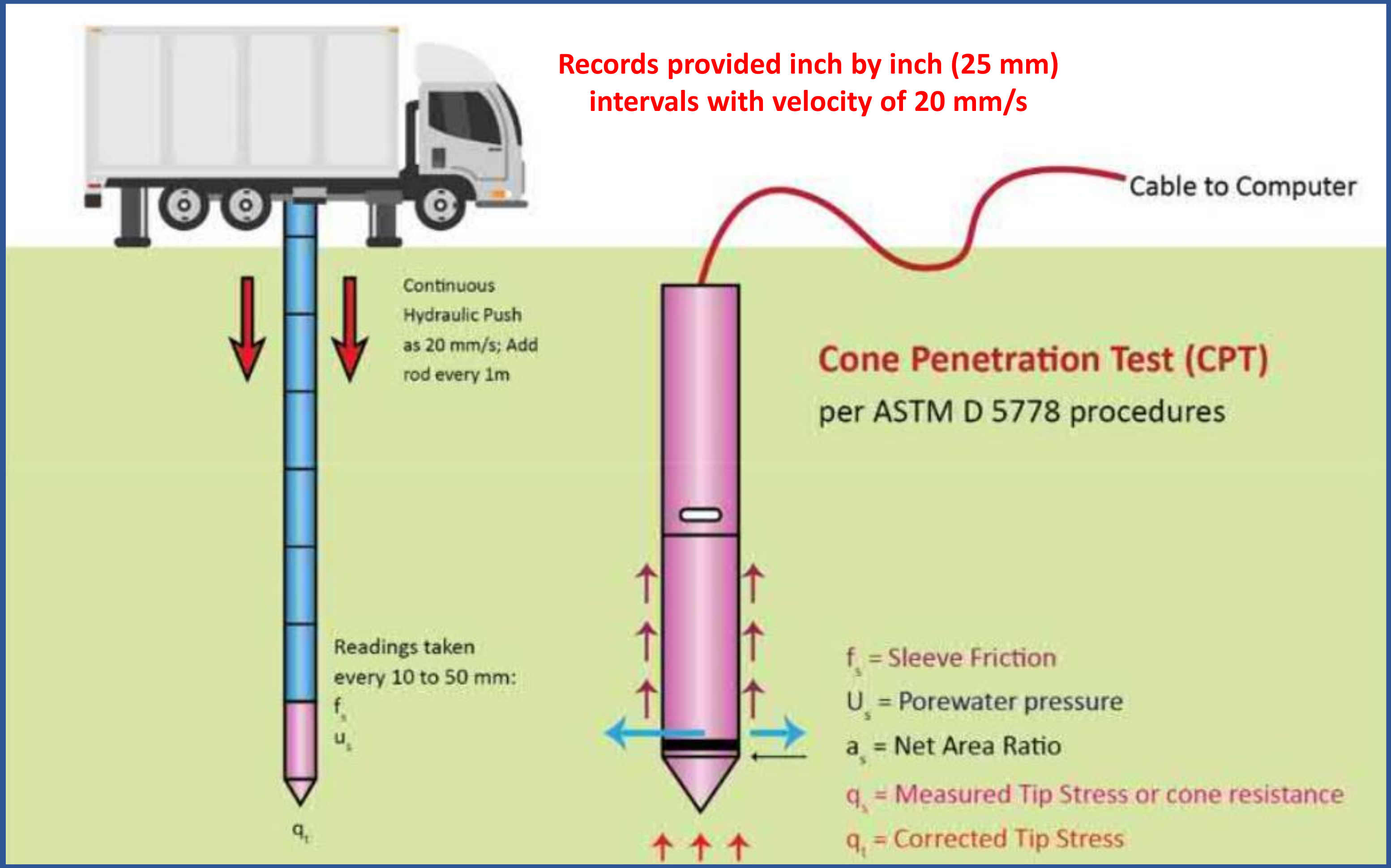
- ❖ Simple and relatively economical.
- ❖ Continuous records with depth.
- ❖ Interpretable on both empirical and analytical bases.
- ❖ Sensors can be incorporated with penetrometer.
- ❖ A large experience-based knowledge is now available.



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

4. Cone & Piezocone Penetration Tests (CPT & CPTu)

4.2. Equipment & Procedure



(Eslami et al., 2019)

4.3. Piezocone Penetration Test (CPTu)

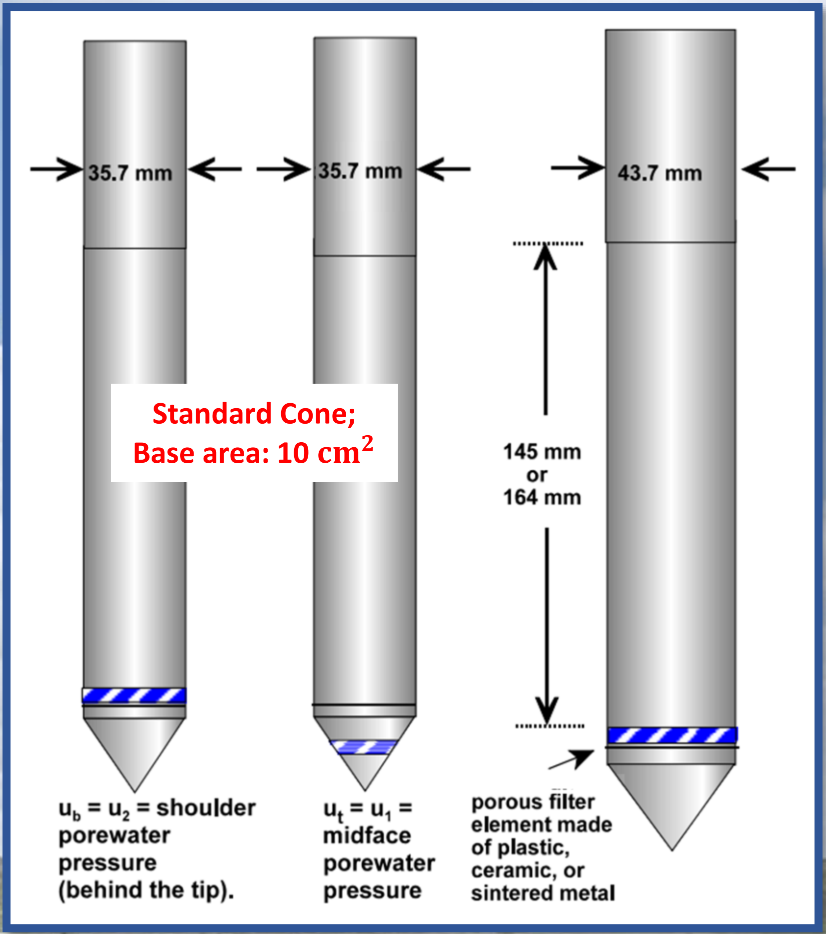
- Pore pressure measurement (u_1, u_2 & u_3)
- The main advantages of the CPTu over CPT are:

✓ Improved

- ➢ Soil profiling and interpretation
- ➢ Evaluation of geotechnical parameters

✓ Ability to

- ➢ Evaluate consolidation characteristics
- ➢ Assess pore pressure gradients
- ➢ Distinguish between drained, partially drained, and undrained parameters
- ➢ Correct measured cone data to account for unbalanced water forces



4.4. Data & Graphical Presentation

1. Measured Parameters

q_c, f_s, u

2. Corrected Parameters

- Corrected tip resistance:

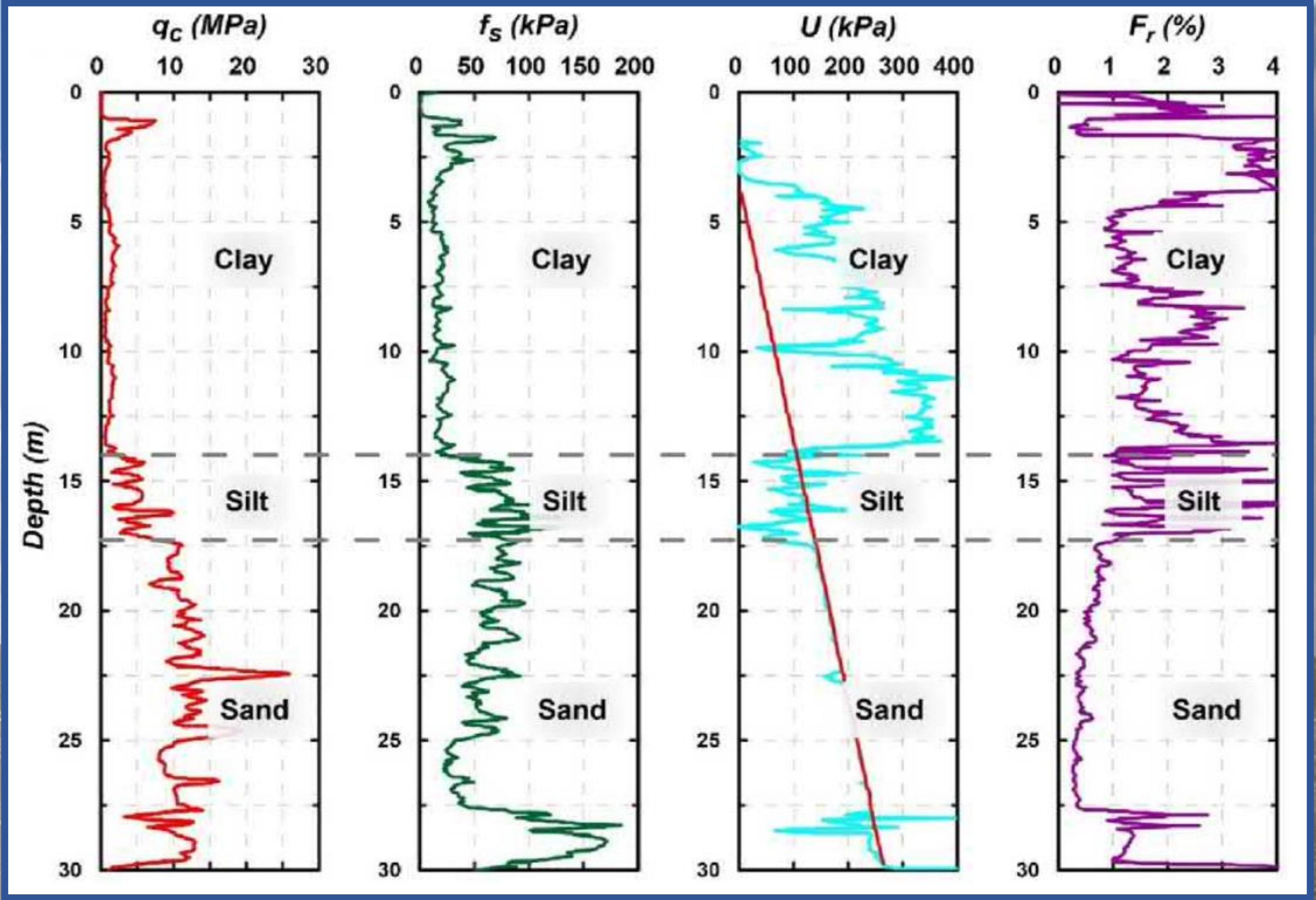
$$q_t = q_c + u_2(1 - a)$$

- Friction ratio:

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

- Pore pressure coefficient:

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



Example of soil profiling by Eslami & Fellenius (1997)

4. Cone & Piezocone Penetration Tests (CPT & CPTu)

Case Study No.1: Eslami et al. (2019)

Geotechnical site characterization of the Lake Urmia super-soft sediments using laboratory and CPTu records

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ABSTRACT

Comprehensive study has been carried out in the clay deposits in the middle part of the Lake Urmia. A causeway over this lake was constructed to connect the eastern area of the lake to the western area and Europe in the middle part of the lake. Geotechnical properties were obtained from laboratory (classification, unit weight of the soil and undrained shear strength) tests. Moreover, using CPTu test results, the classification of sediments, together with unit weight of the soil and undrained shear strength were determined. According to the research results, the sediments of the Lake Urmia are super soft and sensitive. Geotechnical parameters obtained from laboratory and CPTu approaches were compared and evaluated in this study. According to the research findings, although the extracted geotechnical parameters of both methods follow a similar trend by increasing depth in most cases, there were slight differences in values. CPTu correlations are fundamental for a proper geotechnical site characterization and in such cases because of the difficulties for obtaining high-quality undisturbed samples in super-soft and sensitive sediments, it is very important and suggested to compare and join the geotechnical parameters of these two approaches for engineering judgment and engineering design.

ARTICLE HISTORY
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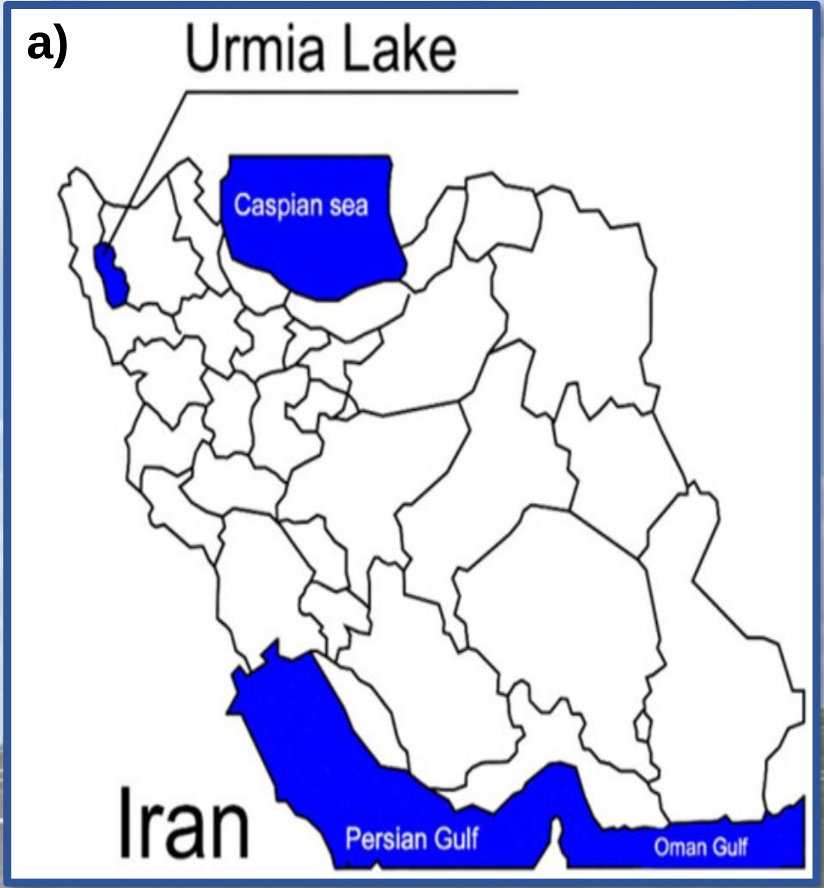
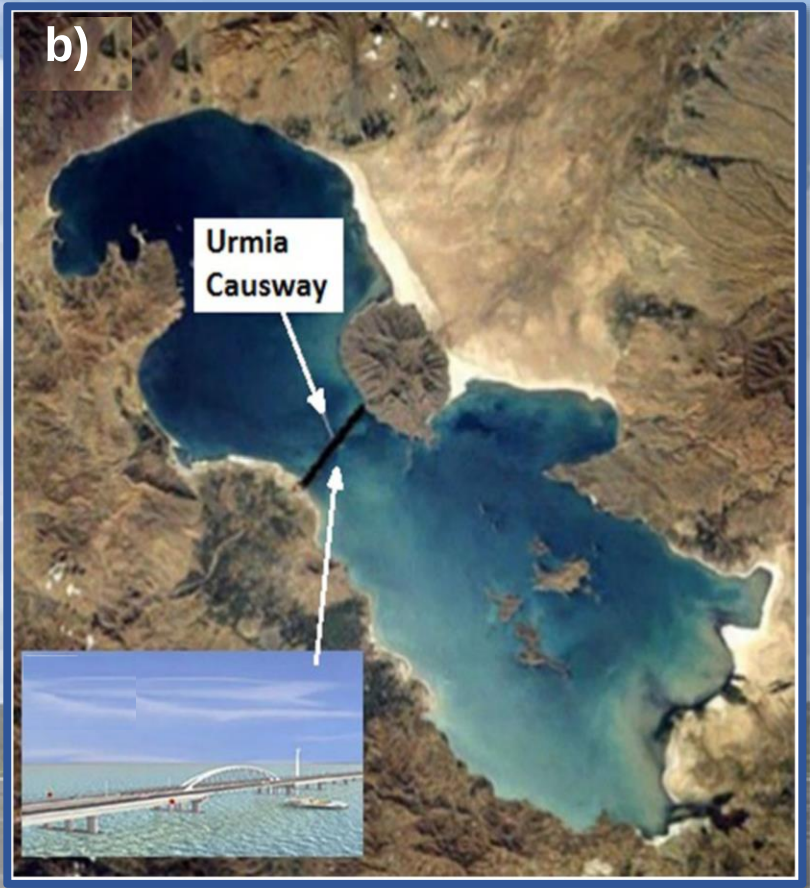
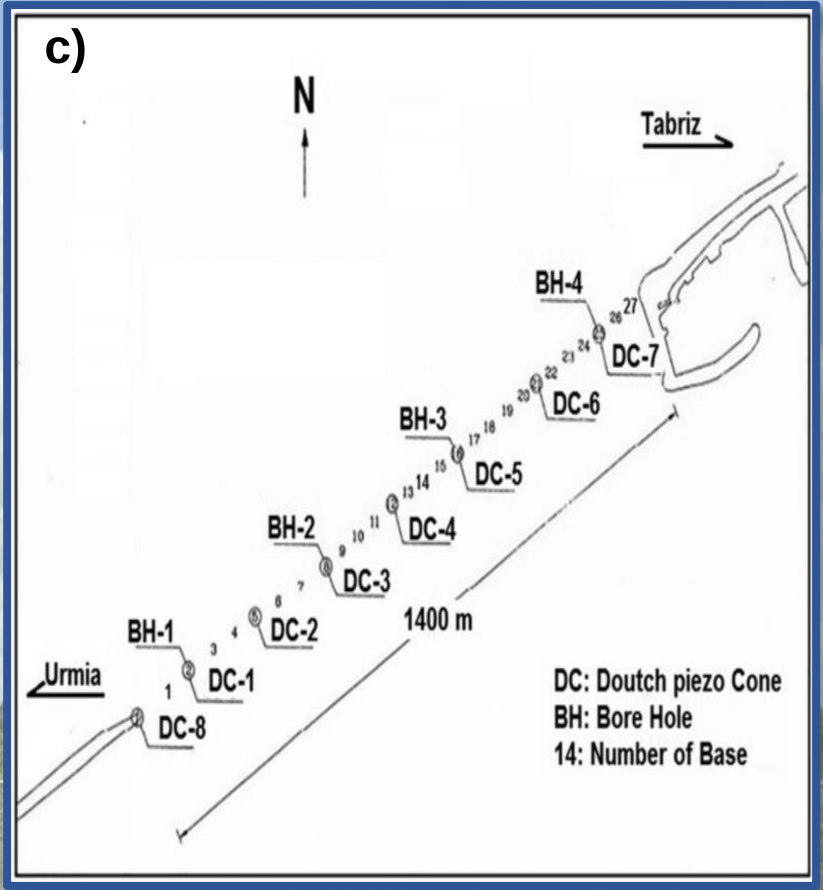
KEYWORDS
Lake Urmia; super-soft sediments; CPTu; laboratory tests; geotechnical characteristics

4. Cone & Piezocone Penetration Tests (CPT & CPTu)

Case Study No.1: Eslami et al. (2019)



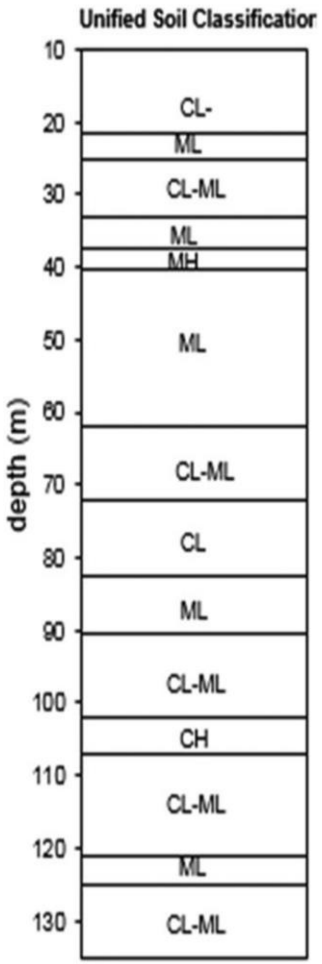


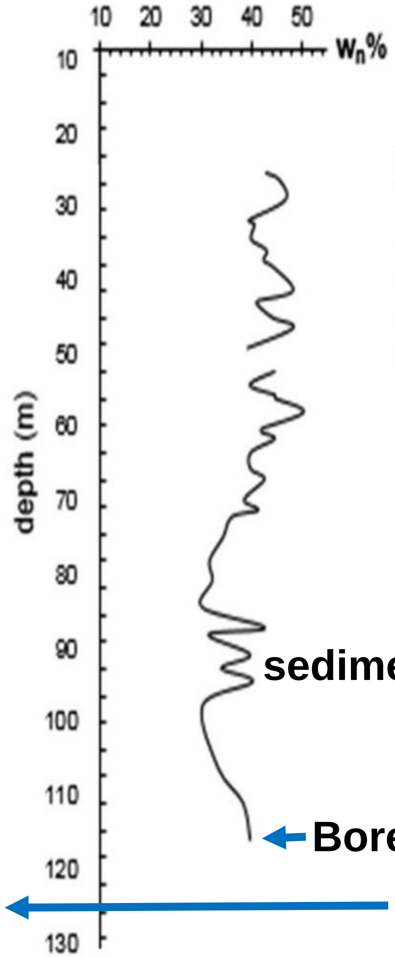


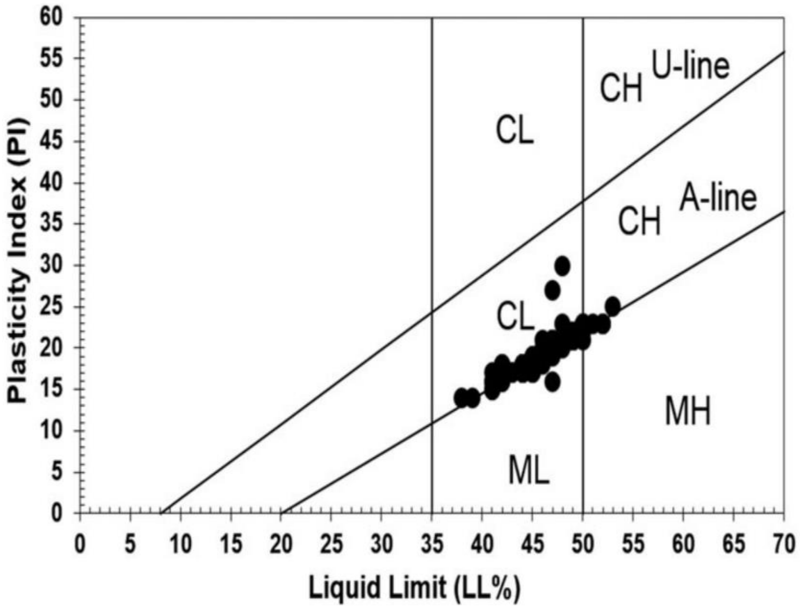
a) The Lake Urmia, b) Causeway route and causeway bridge, c) Locations of Urmia causeway bridge bore holes and CPTu tests

4. Cone & Piezocone Penetration Tests (CPT & CPTu)

Case Study No.1: Eslami et al. (2019)







sediments classification using Casagrande chart

Site characterization from lab tests

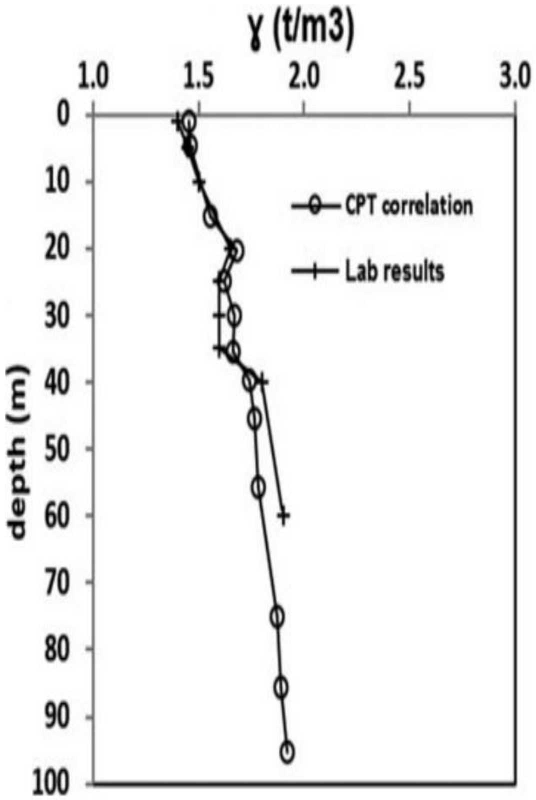
Boreholes sediment moisture content by depth

Boreholes sediment classification

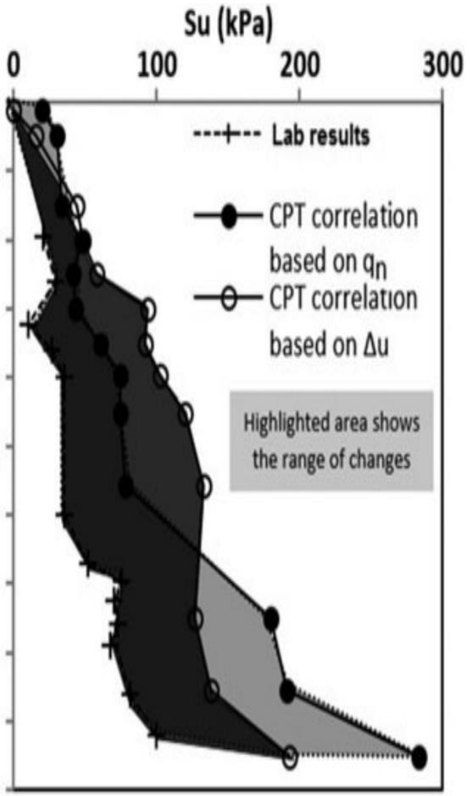
Depth (m)	S_u (Kpa) (from triaxial test)	γ (t/m^3) (from thin-wall sampler samples)
1	–	1.4
5	–	1.45
7	–	–
9	–	–
10	–	1.5
11	–	
13	–	
17	–	1.65
19.5	20.5	
26.0	29.5	1.6
30		1.6
32.5	10.5	–
36.0	26.5	1.6
40.0	35.5	1.8
60.0	35	1.9
67.0	52	–
69.5	75.5	–
72.5	70.5	–
76.0	72.5	–
79.0	67.5	–
86.0	81	–
92.0	100	–
97.5	106	–
107.0	110	–

4. Cone & Piezocone Penetration Tests (CPT & CPTu)

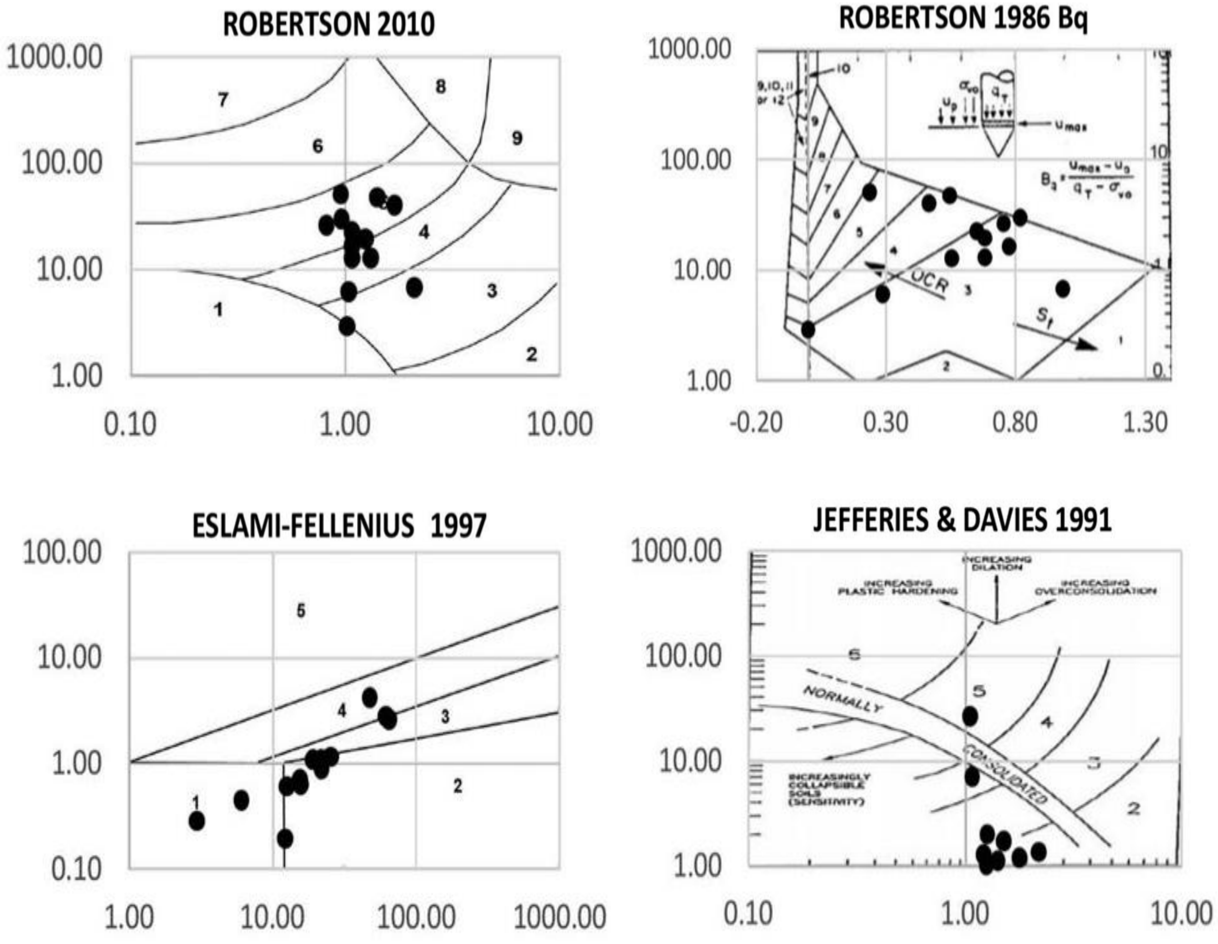
Case Study No.1: Eslami et al. (2019)



Comparison of unit weight of the soil by sampling and CPT correlation



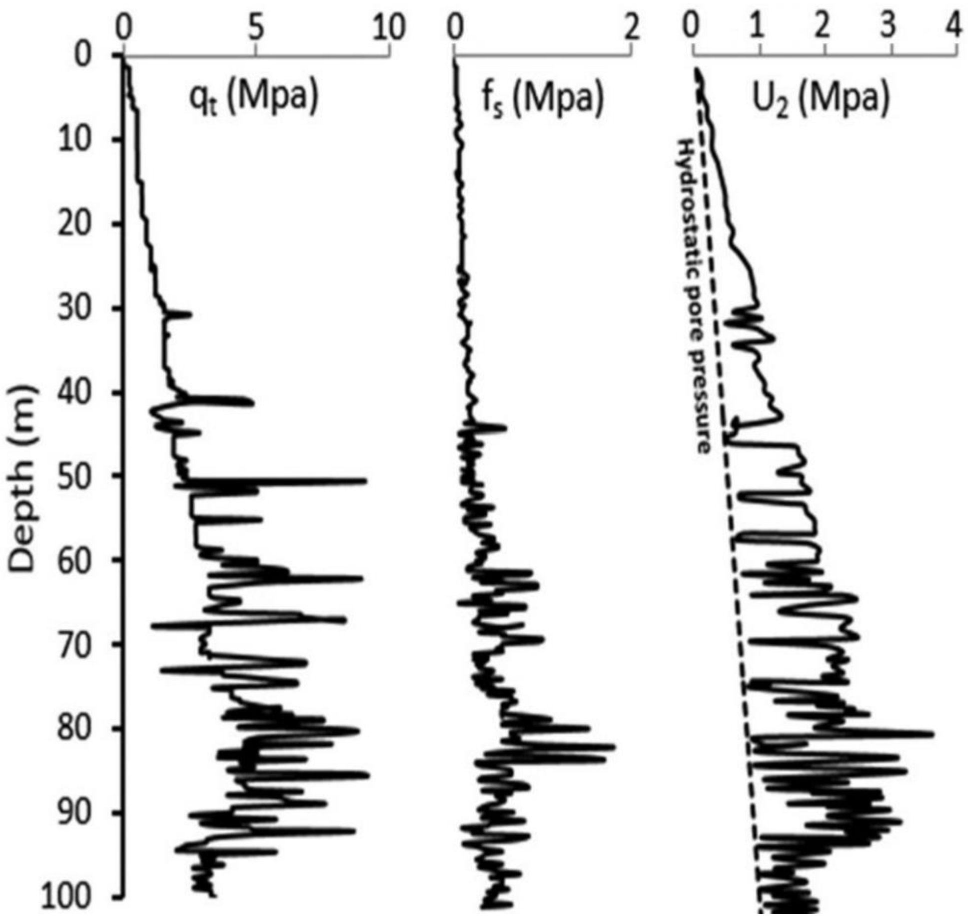
Comparison of S_u by triaxial tests and CPT correlation



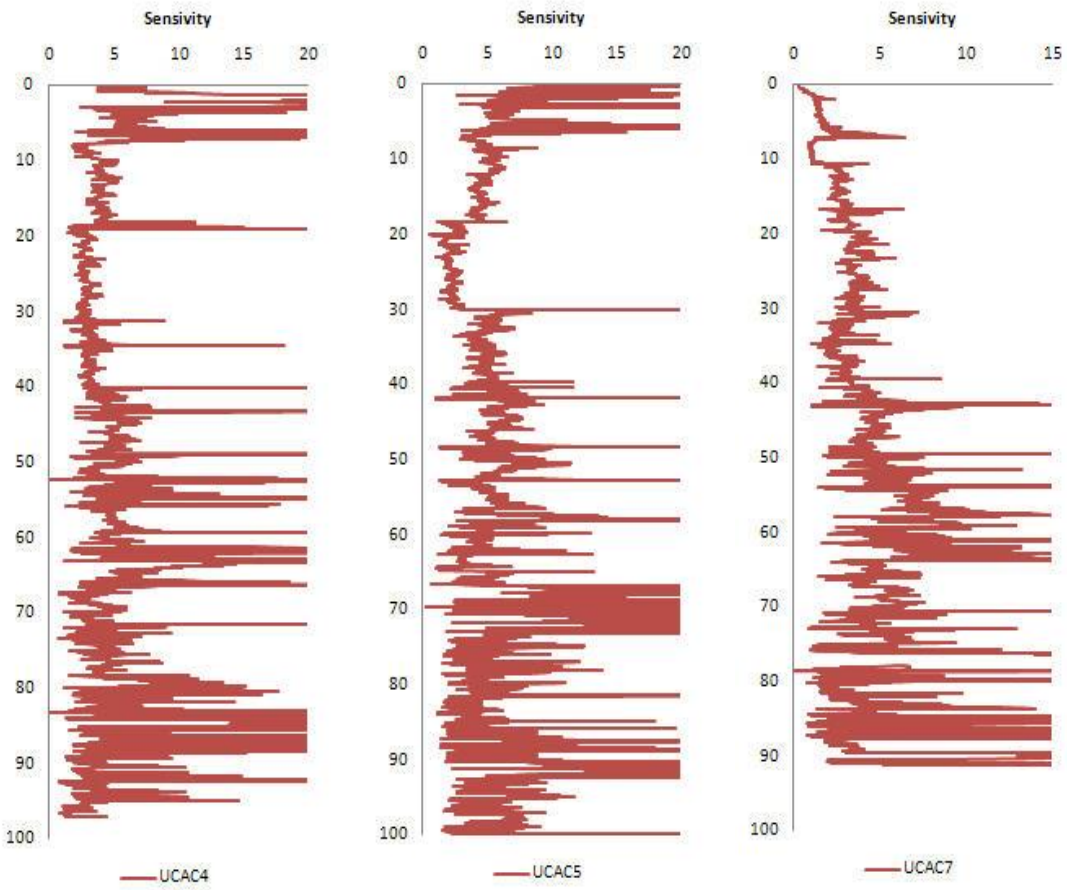
CPT test soil profiling

4. Cone & Piezocone Penetration Tests (CPT & CPTu)

Case Study No.1: Eslami et al. (2019)



Typical CPT profile



Sensitivity profile (Eslami et al., 2011)

4.5. Factors Influencing CPT Measurements and Interpretation

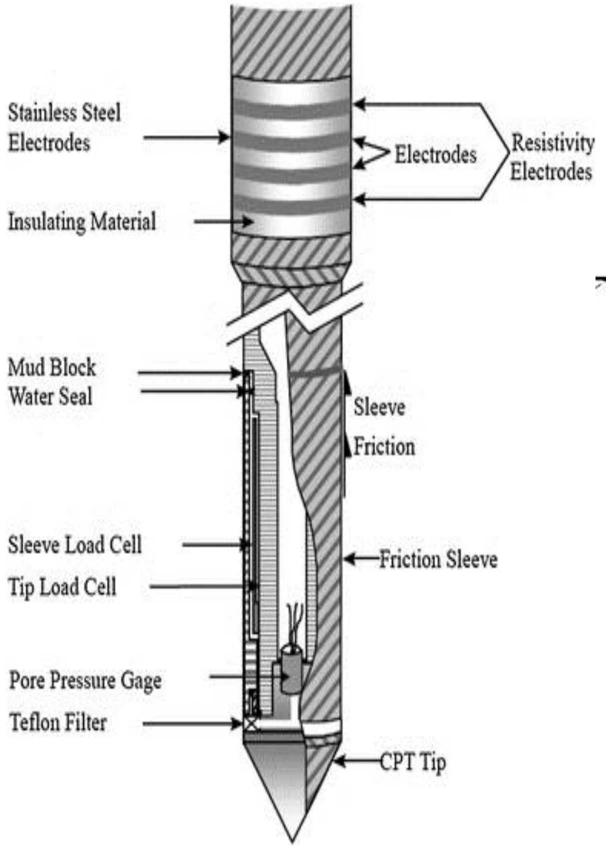
- The factors affecting CPT measurement and interpretation:

1. Equipment design and appropriate selection for a specific soil
2. Lack of qualified operator and wrong use of methods
3. Rate of penetration
4. In situ stress
5. Compressibility
6. Temperature
7. Porous filter calibration & maintenance
8. Penetrometer geometry

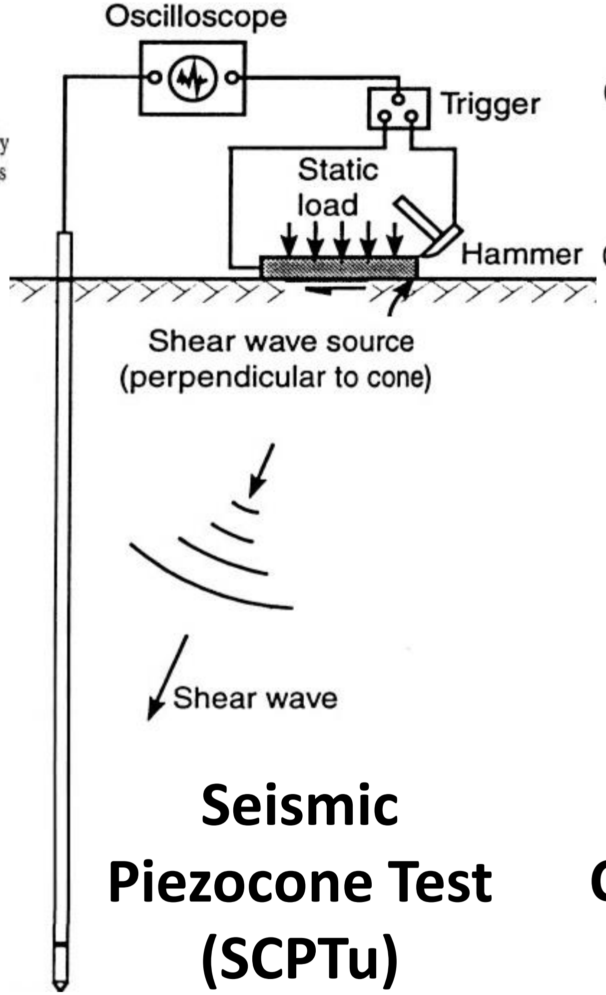
A few CPT Limitations:

1. High capital investment
2. Requires skilled operator
3. No soil sample obtained
4. Difficulties in hard deposits

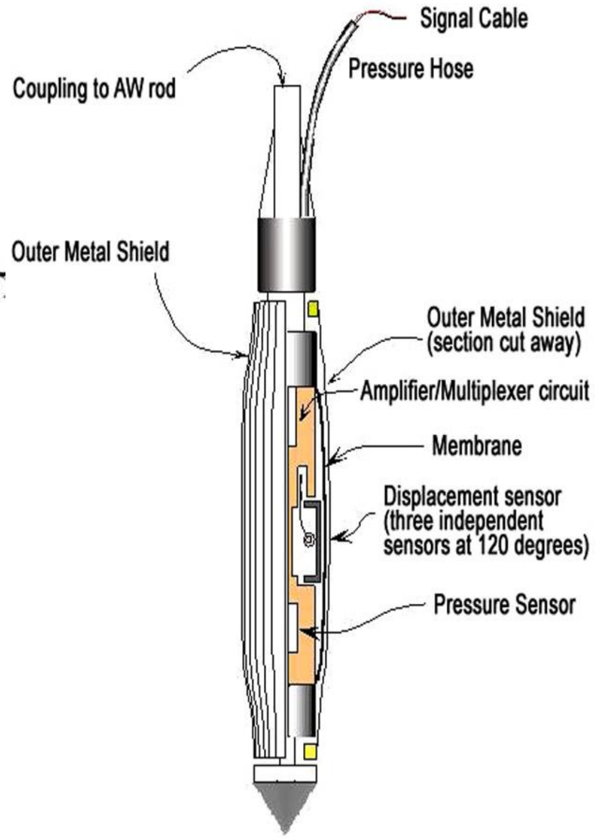
4.6. Special Cones



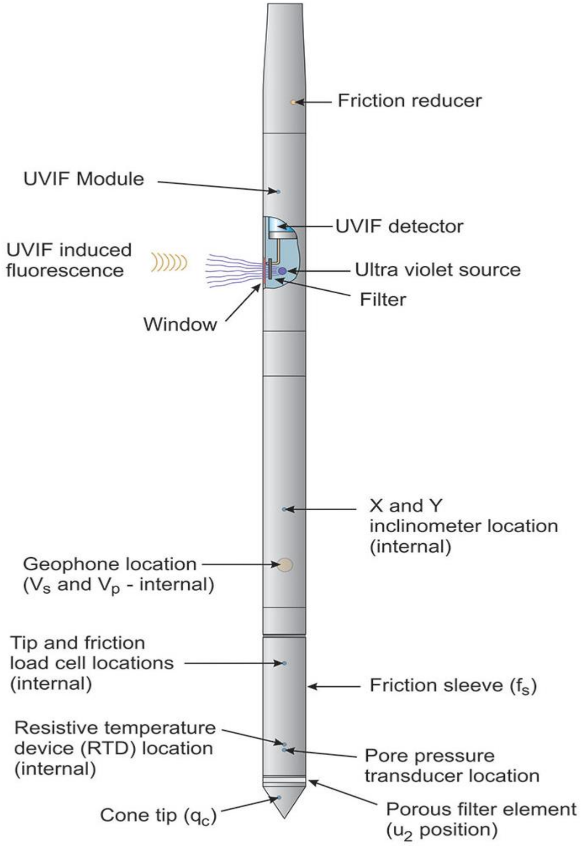
Resistivity Piezocone Penetration (RCPTu)



Seismic Piezocone Test (SCPTu)



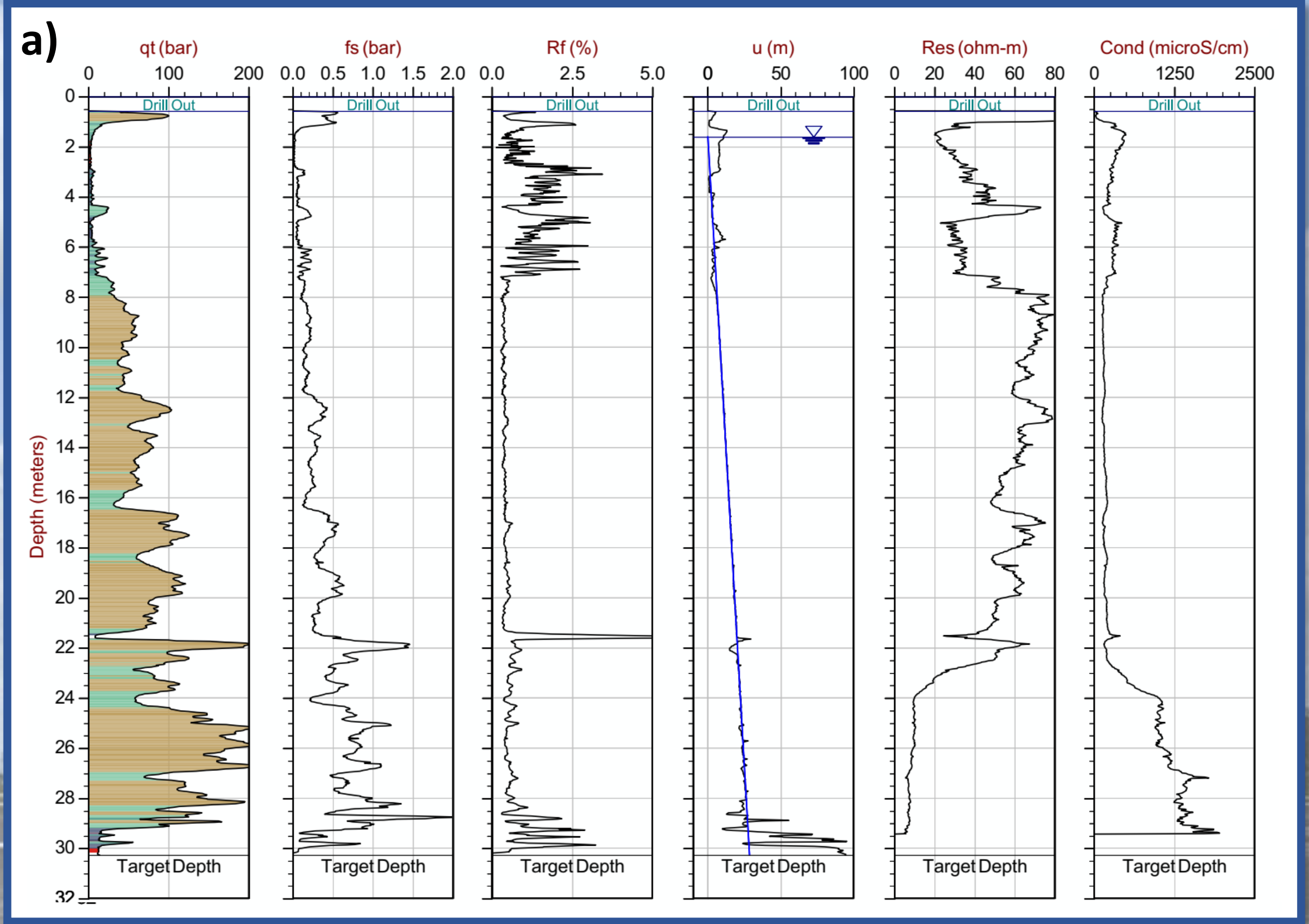
Cone Pressuremeter (CPMT)

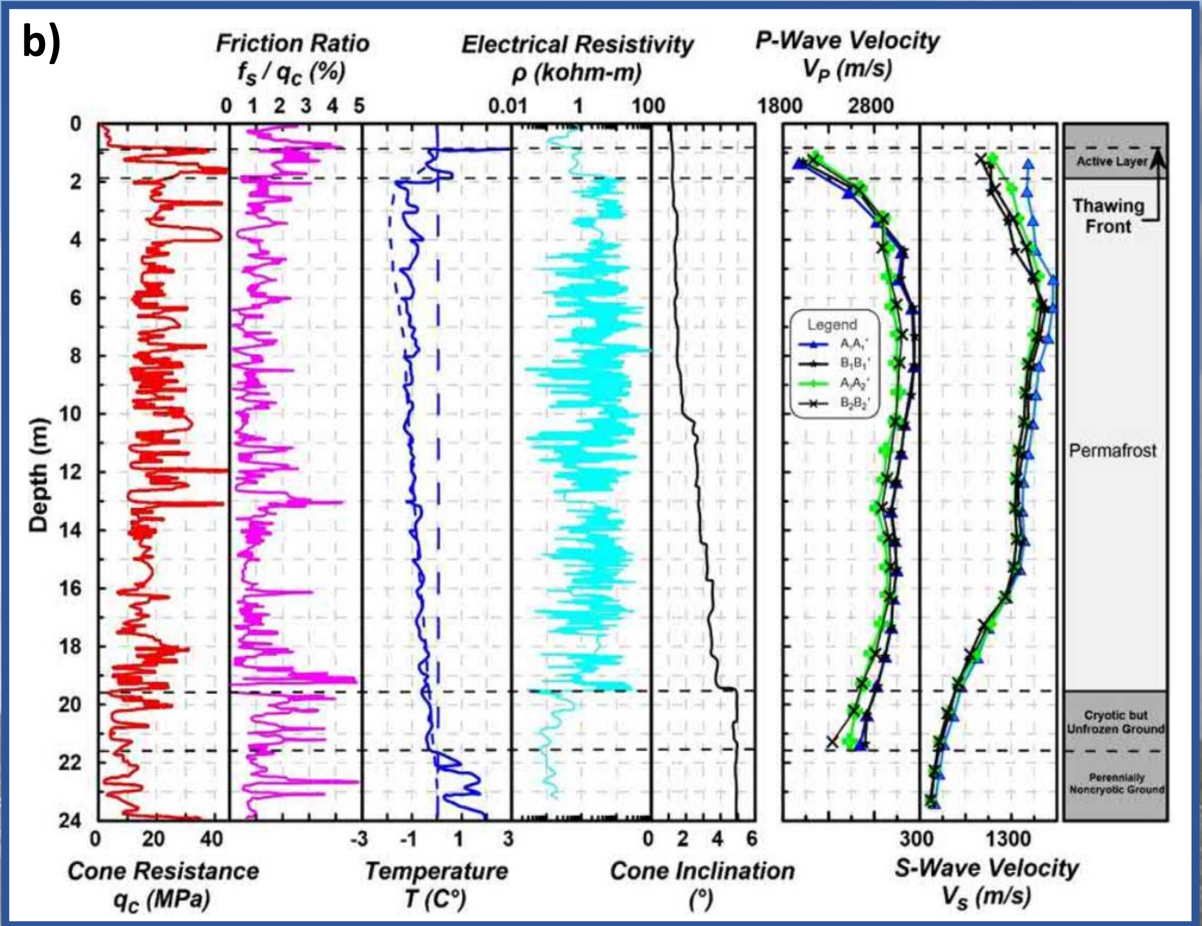


Ultraviolet Induced Florescence CPT(UVIF-CPT)

4. Cone & Piezocone Penetration Tests (CPT & CPTu)

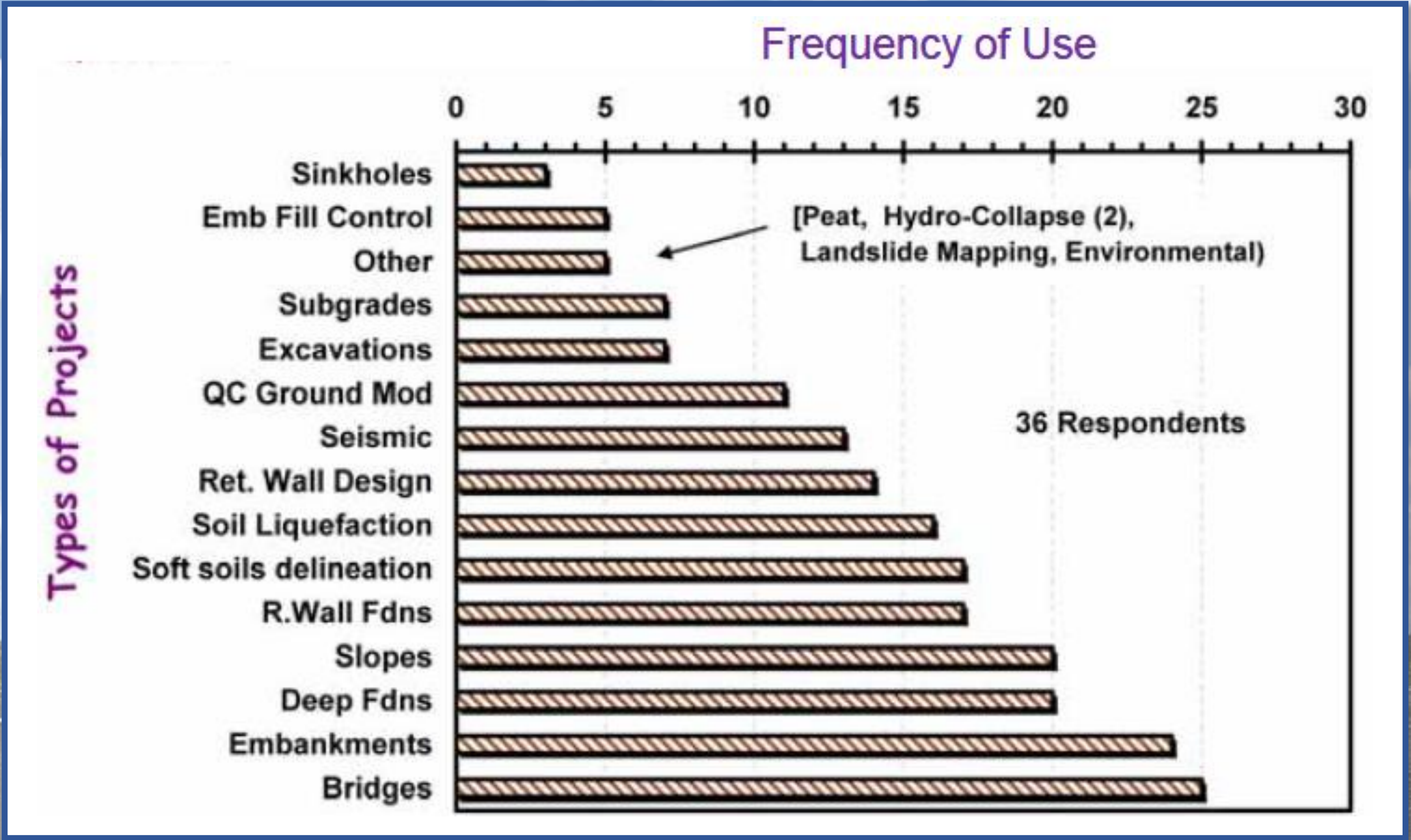
4.6. Special Cones





a) Example of resistivity piezocone profiles (ConeTec, 2019) & b) Example of seismic cone records and soil profiling (Eslami, 2019)

5. Application of CPT & CPTu in GE



(NCHRP 368, 2007)

5. Application of CPT & CPTu in GE

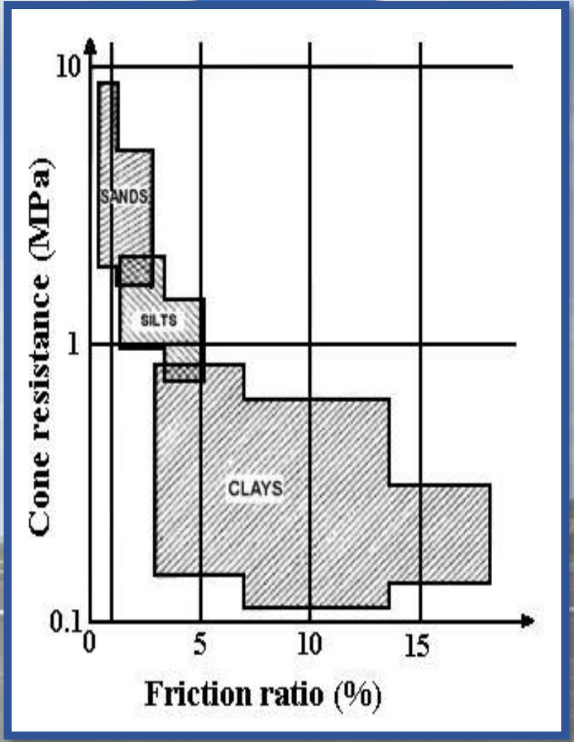
Major Application of CPT in GE

- Soil Behavior Classification and Profiling
- Estimating Soil Engineering Parameters
- Identify Problematic Deposits and Ground Improvement
- Foundation Engineering: Design & Construction

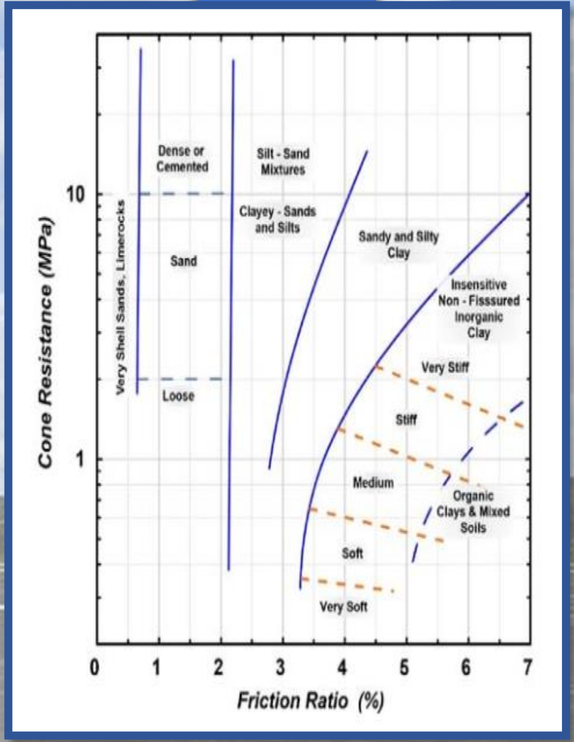
5. Application of CPT & CPTu in GE

5.1. Soil Behavior Classification and Profiling

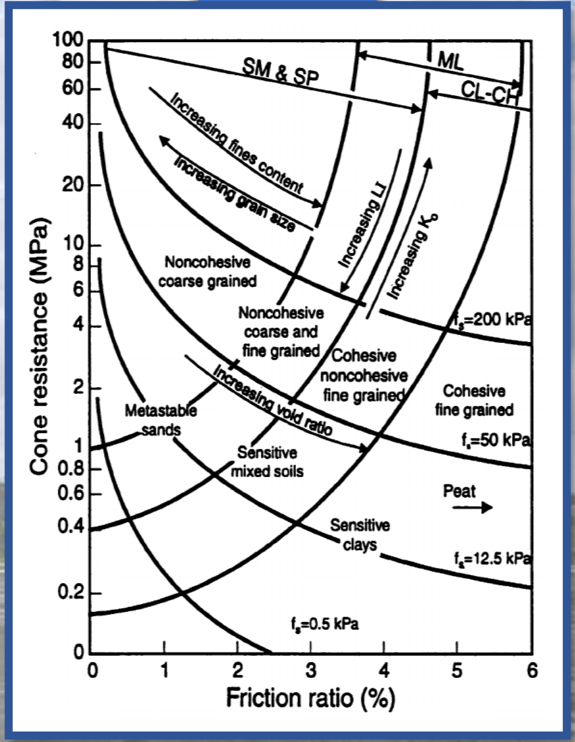
Begemann (1965)



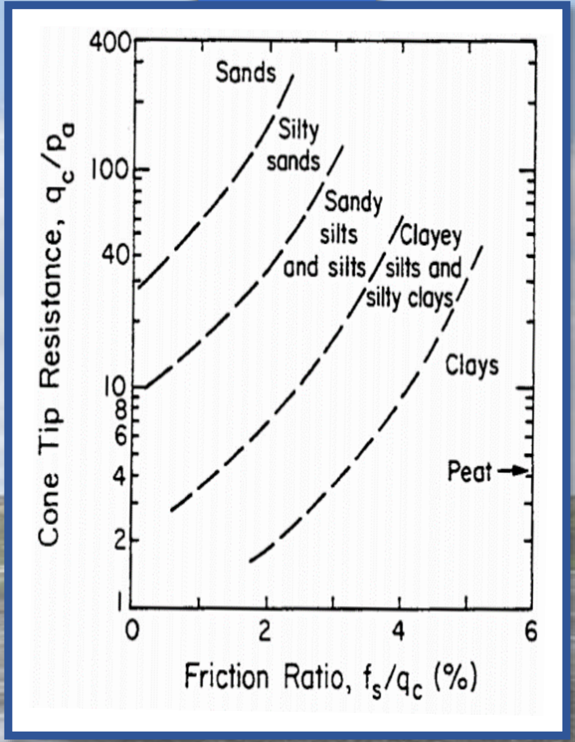
Schmertmann (1978)



Douglas and Olsen (1981)

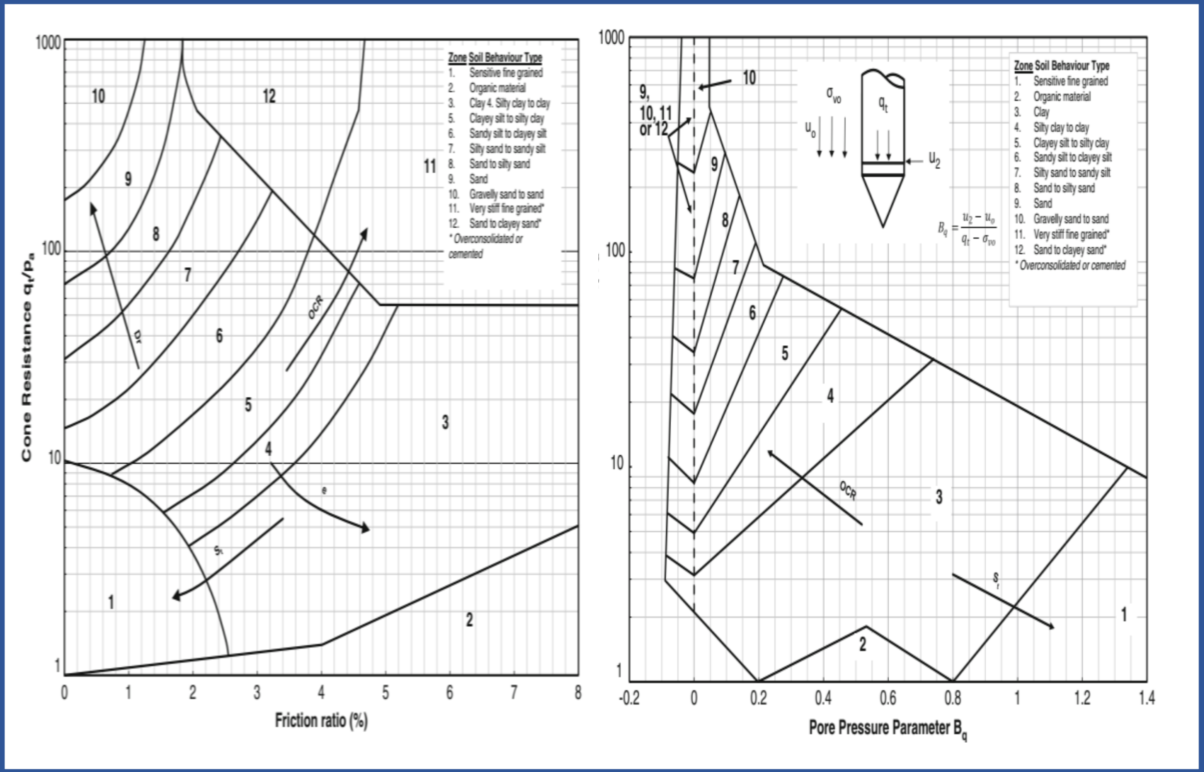


Robertson and Campanella (1983)

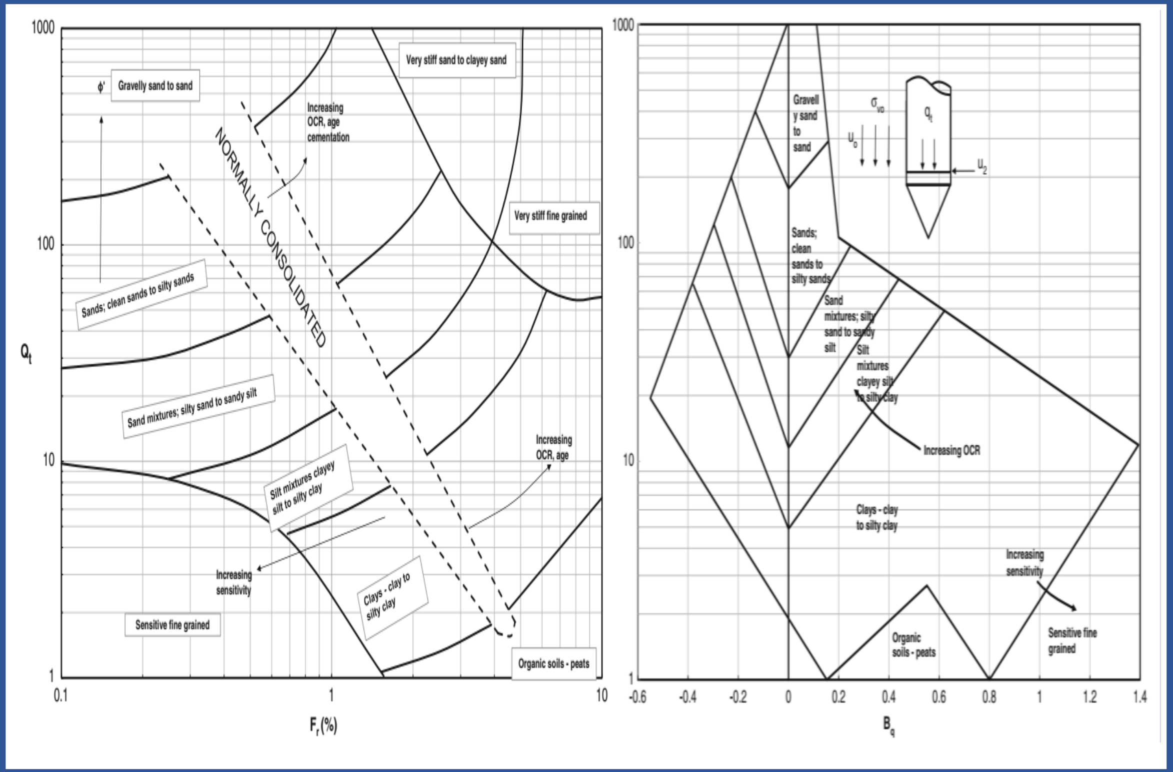


5.1. Soil Behavior Classification and Profiling

Robertson et al. (1986)

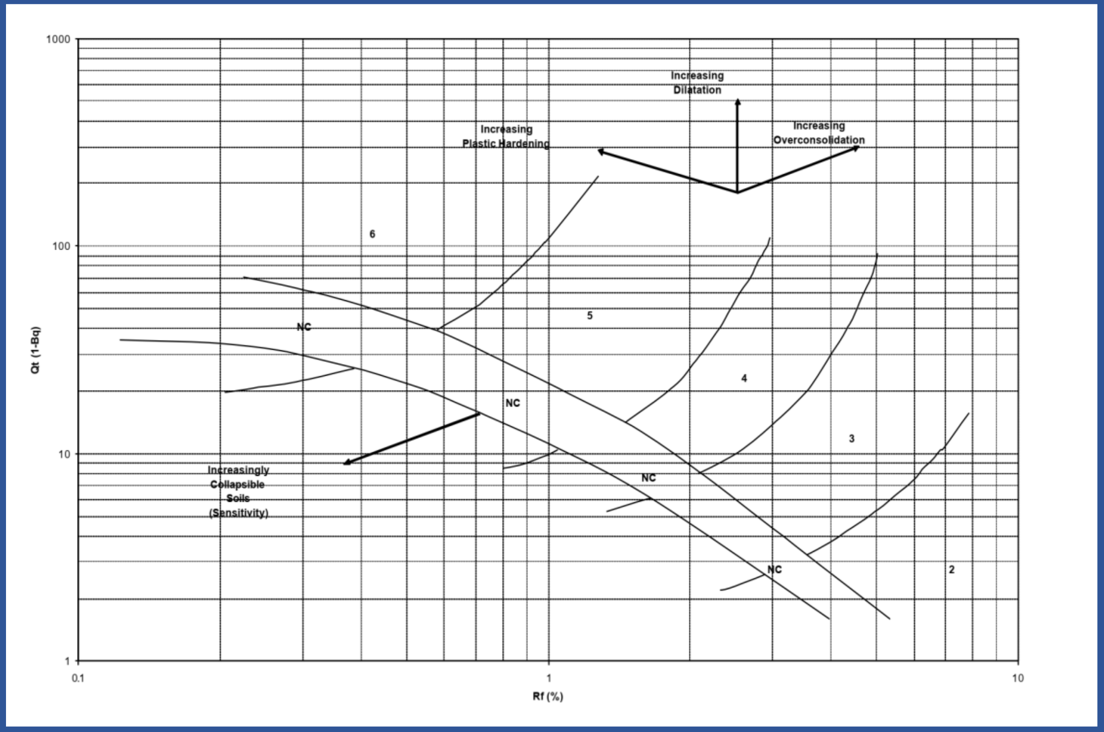


Robertson (1990)

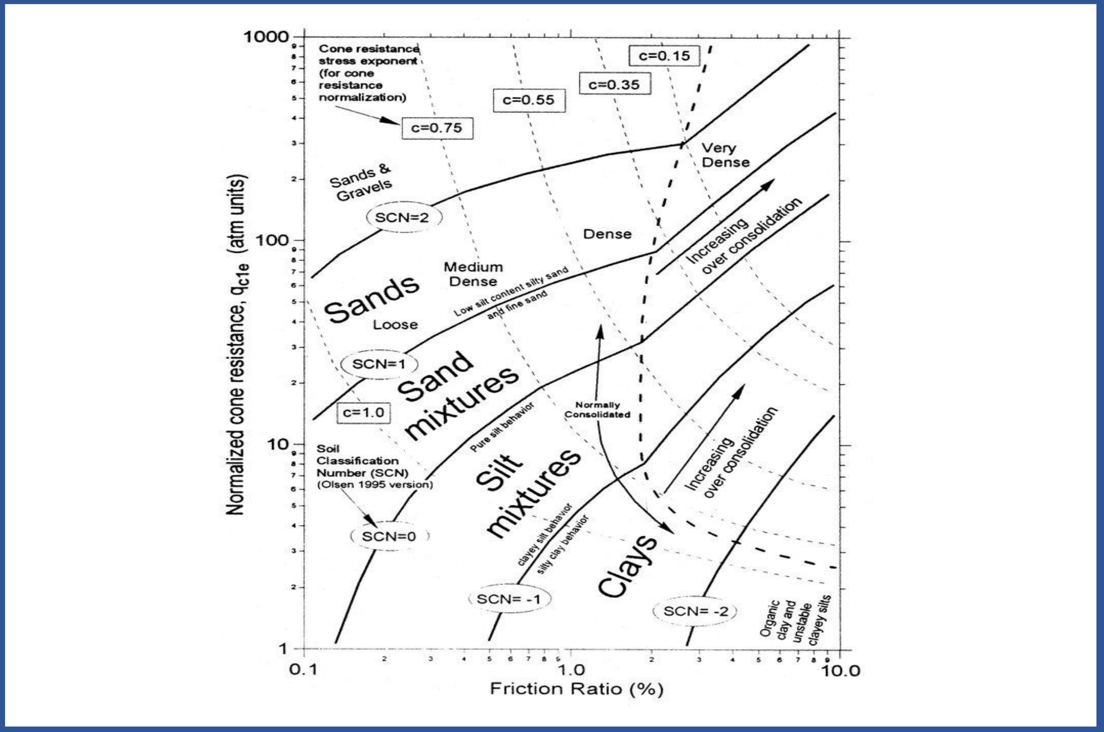


5.1. Soil Behavior Classification and Profiling

Jefferies and Davies (1993)



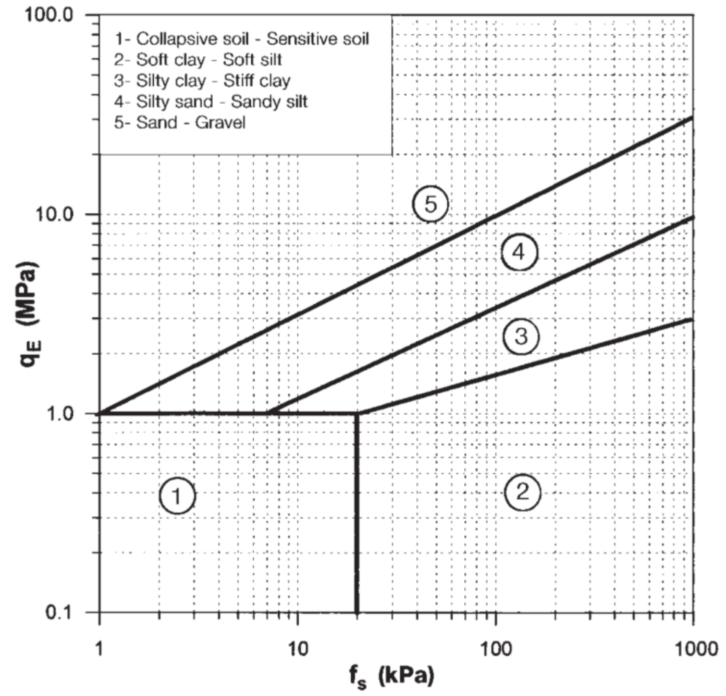
Olsen and Mitchel (1995)



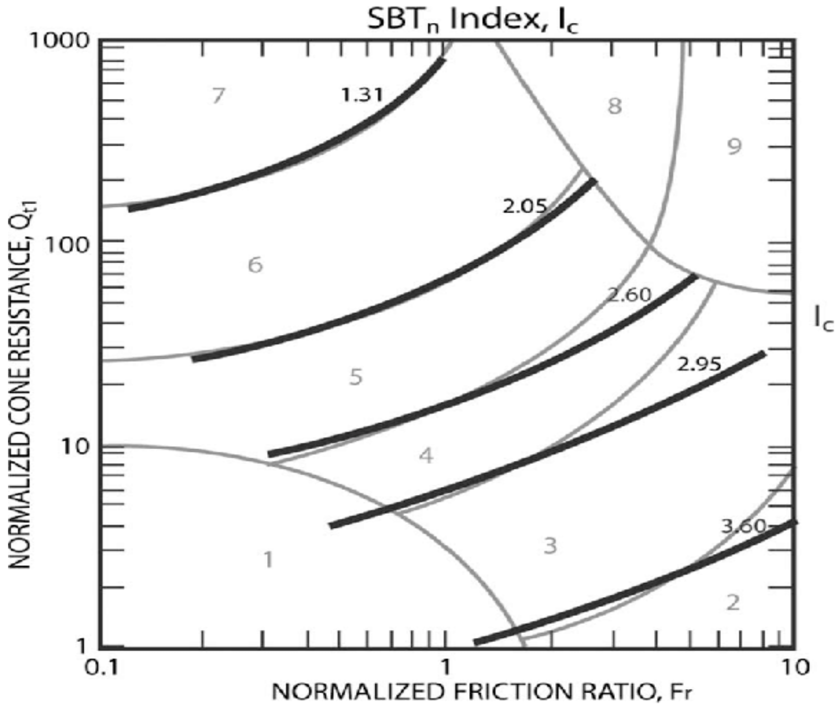
5. Application of CPT & CPTu in GE

5.1. Soil Behavior Classification and Profiling

Eslami and Fellenius (1997)



Robertson (2010)

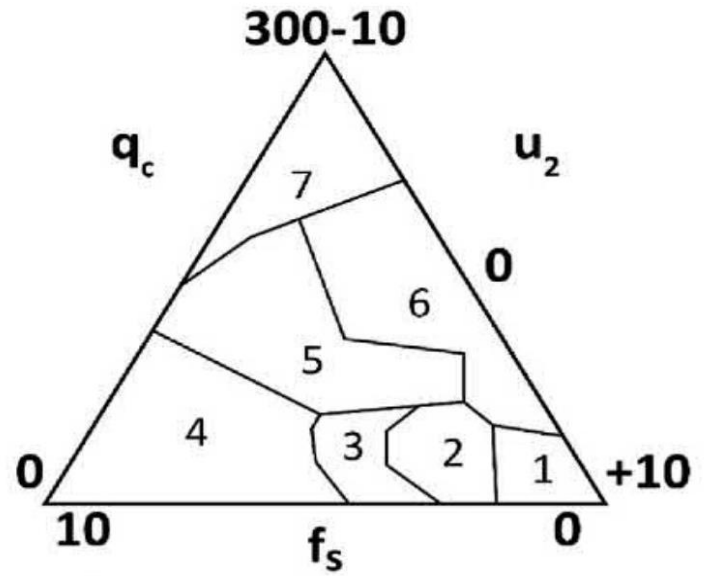


$$I_c = \left[(3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2\right]^{0.5}$$

5. Application of CPT & CPTu in GE

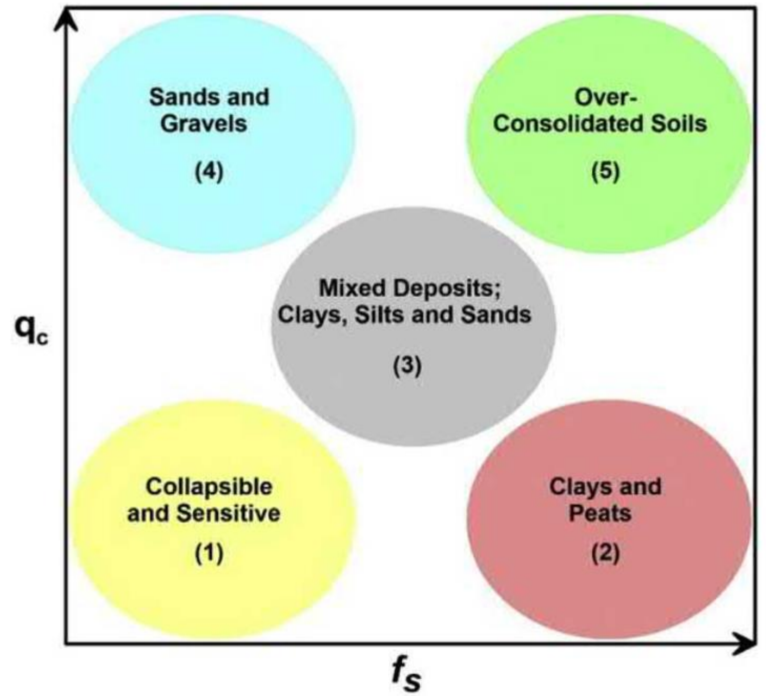
5.1. Soil Behavior Classification and Profiling

Eslami et al. (2016)



1	2	3	4	5	6	7
Sensitive Soil	Clays	Silts	Over Consolidated Clay	Mixed Soil (Clayey silt to sandy silt)	Sands	Very Dense Sand

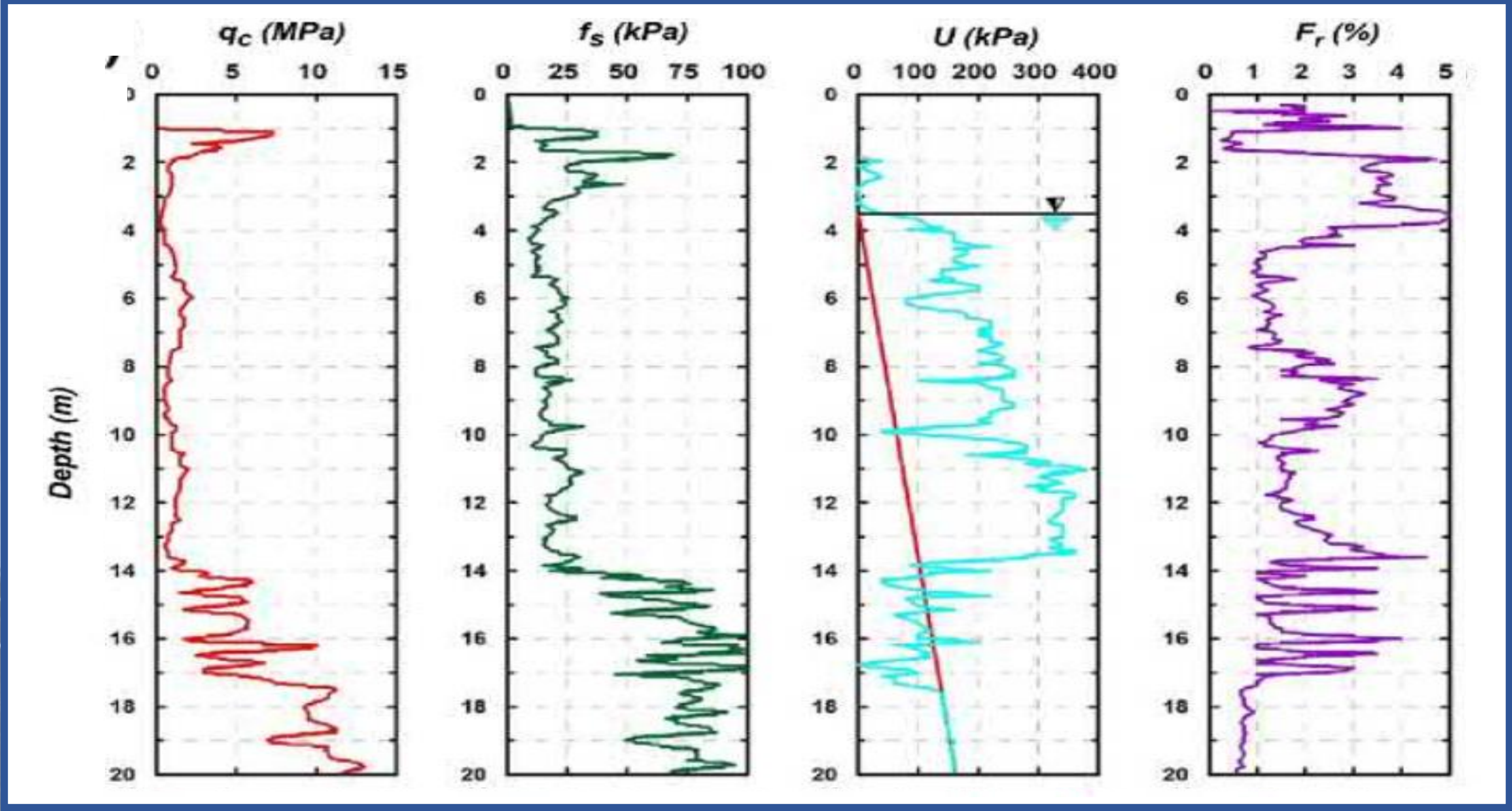
Eslami et al. (2018)

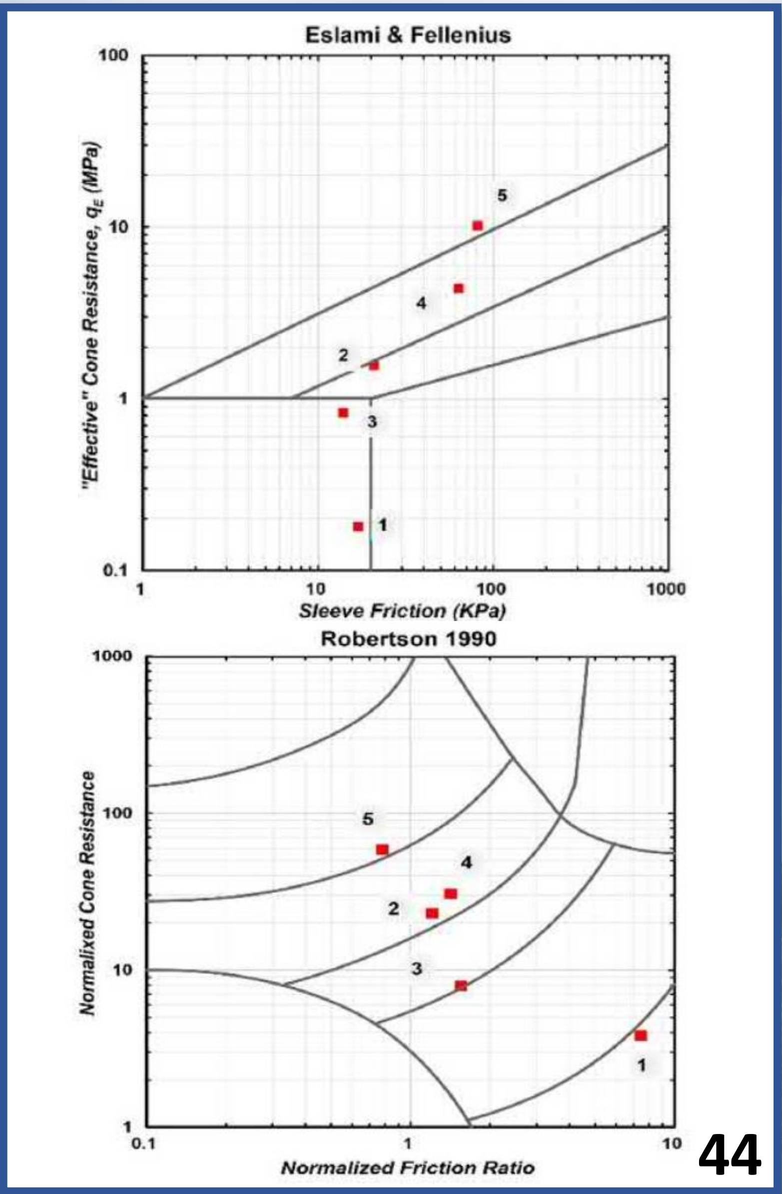


5. Application of CPT & CPTu in GE

5.1. Soil Behavior Classification and Profiling

- **Vancouver (Eslami & Fellenius, 2004)**





5. Application of CPT & CPTu in GE

Case Study No.2: Eslami et al. (2022)

Probabilistic Engineering Mechanics

Available online 9 November 2022, 103380

Developed Triangular Charts; Deltaic CPTu-Based Soil Behavior Classification Using AUT:CPTu-Geo-Marine Database

Abolfazl Eslami[1], Sara Heidarie Golafzani[2], Mohammad Hossein Naghibi [3]

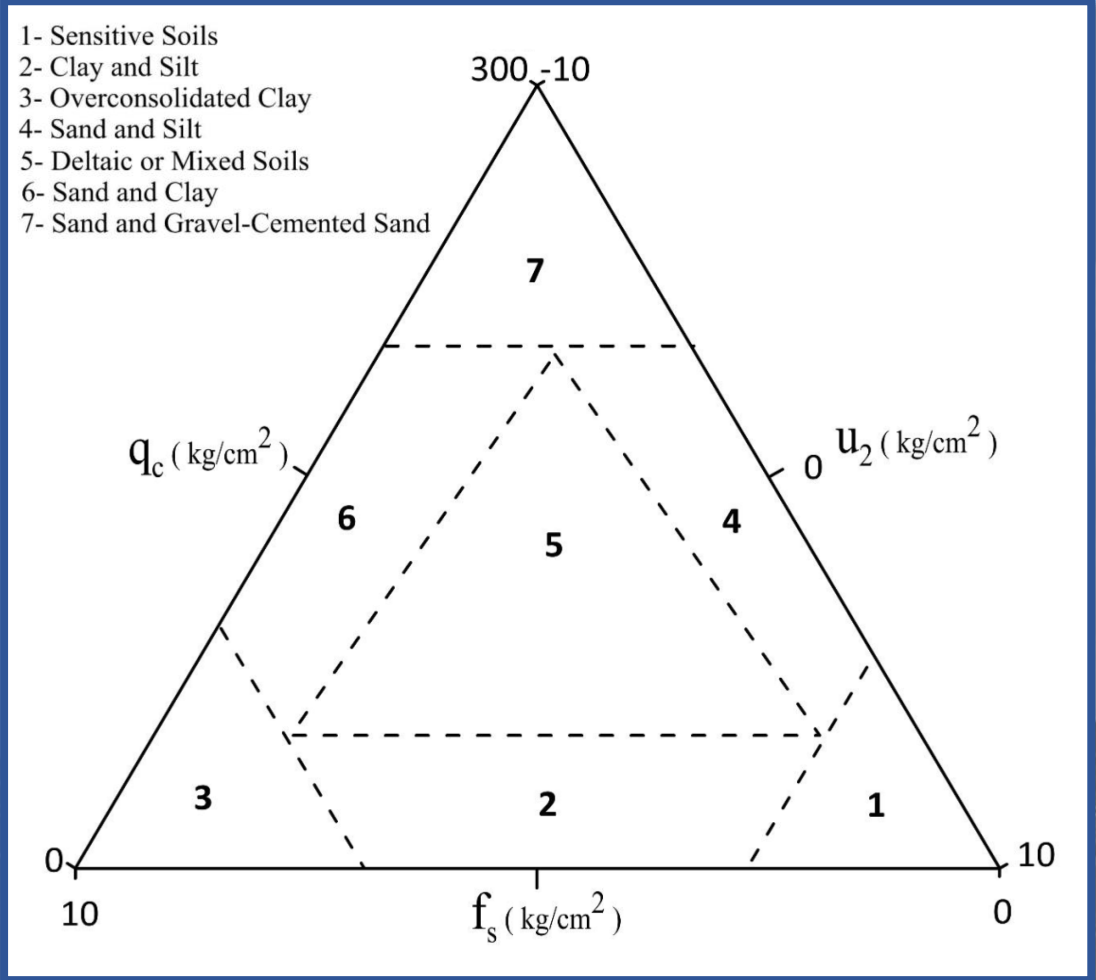
Abstract

One of the crucial issues in foundation engineering is subsurface reconnaissance, especially for offshore and onshore structures. The cone and piezocone penetration tests (CPT and CPTu) are one of the most remarkable geotechnical in-situ tests for soil behavior classification (SBC) due to their capability, rapid performance, accuracy, and providing continuous records in depth. Since classifying soils in marine environments is accompanied by more uncertainty sources, the procedures of validation and evaluation for subsoil classification are enhanced by applying CPT and CPTu databases. Regardingly, a marine database of the CPTu soundings and soil profiling in their vicinity from offshore, onshore, and riverine areas was compiled. About sixty cases were considered for performance assessment of eight common soil behavioral classification methods. These methods were evaluated via a parameter introduced as success rate. The probability of successfully classifying various marine deposits was demonstrated through radar charts for the investigated database. It was revealed that the input parameters and the implemented correlations and assumptions have had a key role in soil deposit identification and reducing the embedded uncertainties. Eventually, the recently developed triangular chart was introduced, which implements CPTu soundings and more accurately divides soil deposits into seven zones compared to the investigated SBC methods.

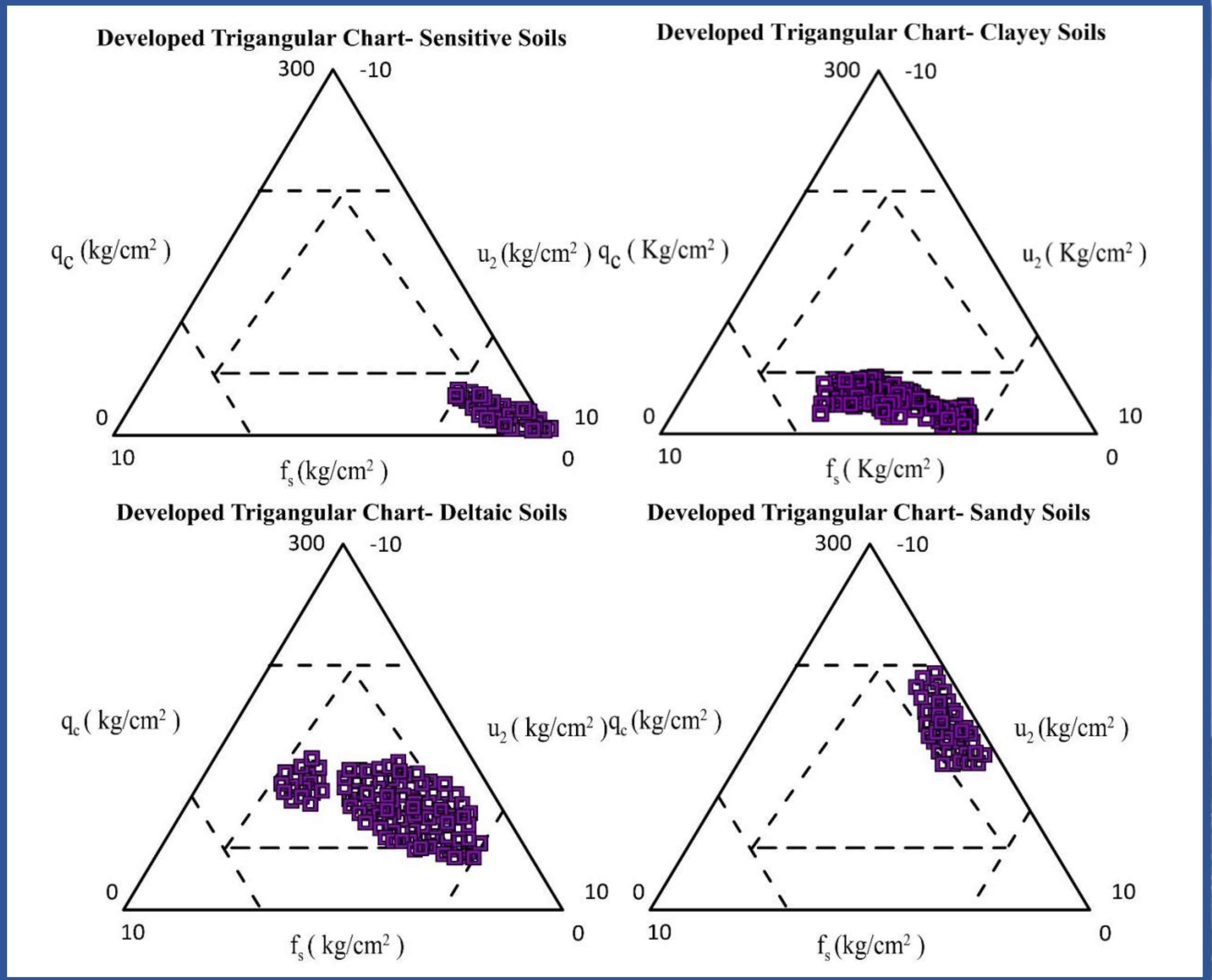
Keywords: Soil behavior classification (SBC), Deltaic deposits, CPTu, Triangular chart, Database, Probability.

5. Application of CPT & CPTu in GE

Case Study No.2: Eslami et al. (2022)



The delineated zones for developed triangular chart using deltaic deposit



Various soil types on developed triangular chart

5.2. Estimating Soil Engineering Parameters

CPT – based methods for prediction of geomaterial engineering properties:

- Case – based empirical methods
- Simplified analytical methods
- Numerical analyses
- Soft computing in data handing

CONDUCTIVITY
- Hydraulic: k_v, k_h
- Thermal: k_e
- Electrical: Ω, ζ
- Chemical: D_f
- Transmissivity, T_m
- Permittivity, P_m

COMPRESSIBILITY
- Recompression index, C_r
- Yield Stress, σ_y' (and YSR)
- Preconsolidation, σ_p' (and OCR)
- Coefficient of Consolidation, c_v
- Virgin Compression index, C_c
- Swelling index, C_s

RHEOLOGICAL
- Strain rate, $\delta\varepsilon/\delta t$
- Time since consolidation (T)
- Secondary compression, $C_{\alpha\varepsilon}$
- Creep rate, α_R
- Time to failure, t_f

STIFFNESS
- Stiffness: $G_0 = G_{max}$
- Shear Modulus, G' and G_u
- Elastic Modulus, E' and E_u
- Bulk Modulus, K'
- Constrained Modulus, D'
- Tensile Stiffness, K_T
- Poisson's Ratio, ν
- Effects of Anisotropy (G_{vh}/G_{hh})
- Nonlinearity (G/G_{max} vs γ_s)
- Subgrade Modulus, k_s
- Spring Constants, k_z, k_x, k_ψ, k_θ

STRENGTH
- Drained and Undrained, τ_{max}
- Peak (s_u, c', ϕ')
- Post-peak, τ'
- Remolded strength
- Softened or critical state, $s_{u\,(rem)}$
- Residual (c_r', ϕ_r')
- Cyclic Behavior (τ_{cyc}/σ_{vo}')

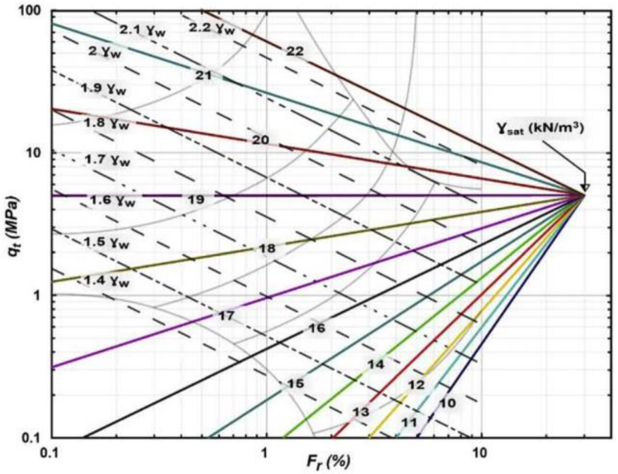
5. Application of CPT & CPTu in GE

5.2. Estimating Soil Engineering Parameters

Unit Weight

Soil unit weight (γ) and cone penetration test parameters relationships

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)	$\gamma = 26 - \frac{14}{1 + (0.5\log(f_s + 1))^2}$	f_s (kPa)
Baginska (2016)	$\gamma = 11 + 24\ln(f_s + 0.7)$	f_s (MPa)
Lengkeek et al. (2018)	$\gamma_{sat} = 19 - 4.12\frac{\log(5/q_t)}{\log(30/R_f)}$	R_f(%) and q_t (MPa)

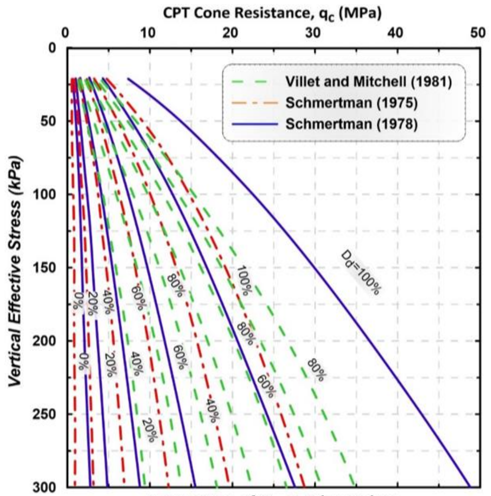


Normalized γ and Rf (%) by Robertson and Cabal (2010) (dotted line) and by Lengkeek et al. (2018) (continuous line).

Relative Density

Correlations predicting D_r from CPT records

Reference	Proposed correlation	Remarks
Baldi et al. (1986)	$D_r = \frac{1}{C_2}\ln\left(\frac{q_c}{C_0(\sigma'_v)^{0.55}}\right)$	C_0 and C_2: soil constants, C_0=157 and C_2=2.41 normally consolidated sand). q_c and σ_v' are in kPa unit
Jamiolkowski et al. (2001)	$D_r = 26.8\ln\frac{q_c/p_a}{(\sigma'_v/p_a)^{0.5}} - b_x$	$b_x = 52.5$ for *high* compressibility sands $b_x = 67.5$ for *medium* compressibility sands $b_x = 82.5$ for*low* compressibility sands
Kulhawy and Mayne (1990)	$D_r = \frac{Q_{cn}}{305 Q_c Q_{OCR}}$	$Q_{cn} = q_t/p_a/(\sigma'_v/p_a)^{0.5}$ Q_c =Compressibility factor (0.91 for high, 1.0 for medium, and 1.09 for low). Q_{OCR} = Over consolidation factor, $OCR^{0.18}$
Mayne (2007)	$D_r = 100\left(0.268\ln\left(\frac{q_t/p_a}{\sqrt{\sigma'_v/p_a}}\right) - 0.675\right)$	q_c and σ_v' are in kPa unit



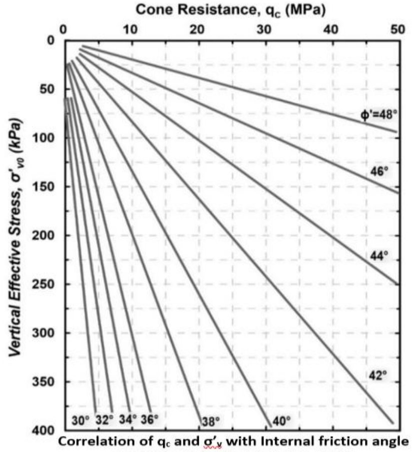
Comparison of D_r - q_c relationships

5.2. Estimating Soil Engineering Parameters

Friction Angle

Proposed correlations for friction angle based on CPT result

Reference	Correlations	Soil type and Remarks
Meyerhof (1974)	$\varphi = tan^{-1}(q_c/0.5N_q)$	Sand q_c (MPa)
Robertson et al. (1986)	$\varphi = tan^{-1}[0.1 + 0.38log(q_c/\sigma'_v)]$	Sand
Kulhawy and Mayne (1990)	$\varphi = 17.6 + 11Log\left(q_c/\sqrt{100\sigma'_v}\right)$	Sand σ_v' and q_c are in kPa unit
Uzielli et al. (2013)	$\varphi = 25\left(q_c/\sqrt{100\sigma'_v}\right)^{0.1}$	Sand σ_v' and q_c are in kPa unit
Mayne (2007)	$\varphi' = 17.6 + 11log(q_{t1})$	Sand $q_{t1} = {(q_{ct}/Pa)}/{\sqrt{(\sigma'_v/Pa)}}$
Robertson and Cabal (2012)	$tan\varphi' = \frac{1}{2.68}(log(q_c/\sigma'_v) + 0.29)$	Uncemented, unaged, moderately compressible quartz sands
Mayne (2014)	$\varphi = 29.5B_q^{0.121}[0.256 + 0.33B_q + \log Q_t]$	Cohesive Soils $Q_t = \frac{q_t-\sigma_v}{\sigma'_v}$ $B_q = \frac{u_2-u_0}{q_t-\sigma_v}$

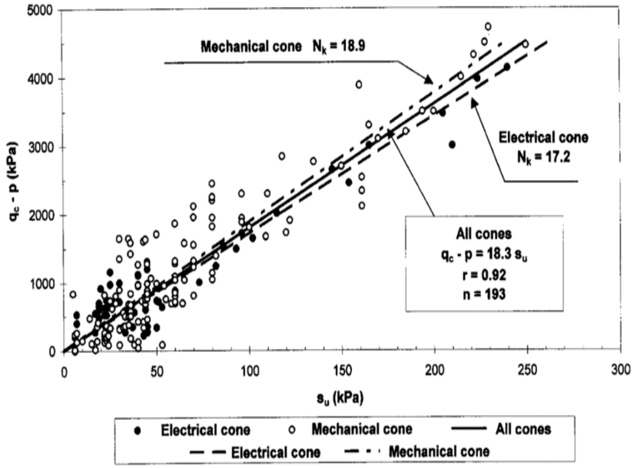


Correlation of q_c and σ'_v with Internal friction angle (Robertson and Campanella, 1983)

Undrained Shear Strength

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



Relationship between cone resistance and undrained shear strength for *cohesive soils* (Anagnostopolous et al, 2003)

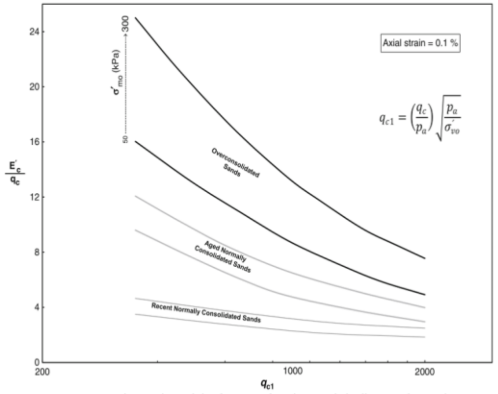
5. Application of CPT & CPTu in GE

5.2. Estimating Soil Engineering Parameters

Stiffness

Elastic modulus parameter of in-situ tests (Bowles, 1997)

Soil Type	CPT	SPT
Sand	$E_s = (2-4)q_u$ $= 8000q_u$ - - - - - - - - - - - - - - - - $E_s = 1.2(3D_r^2 + 2).q_c$ $E_s = (1 + D_r^2).q_c$	$E_s = 500(N + 15)$ $= 7000\sqrt{N}$ $= 6000N$ - - - - - - - - - - - - - - - - $E_s = (15000 - 22000).\ln N$
Saturated Sand	$E_s = F.q_c$ $e = 1.0 \quad F = 3.5$ $e = 0.6 \quad F = 7.0$	$E_s = 250(N + 15)$
OCR Sand	$E_s = (6-30)q_c$	$E_s = 40000 + 1050.N$ $E_{s(OCR)} \approx E_{s(NC)}\sqrt{OCR}$
Clay Sand	$E_s = (3-6)q_c$	$E_s = 320(N + 15)$
Silty Sand	$E_s = (1-2)q_c$ $q_c < 2500kPa \quad E_s' = 2.5q_c$ $2500 < q_c < 5000kPa \quad E_s' = 4q_c + 5000$	$E_s = 300(N + 6)$
Soft Clay	$E_s = (3-8)q_c$	-

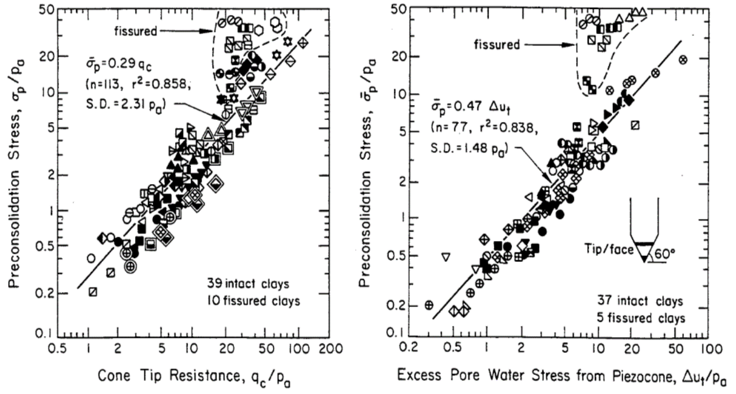


Drained Young's modulus from CPT for silica sands (Bellotti et al., 1989)

Over Consolidation Ratio

Proposed correlations for OCR

Reference	Correlations	Remarks
Mayne and Kemper (1988)	$OCR = 0.37\left(\frac{(q_c - \sigma_v)}{\sigma'_v}\right)^{1.01}$	-
Trevor and Mayne (2004)	$OCR = 2(0.029 + 0.409M)\left[\frac{1}{1.95M + 1}\left(\frac{q_t - u_2}{\sigma'_v}\right)\right]^{\frac{1}{\theta}}$	$\theta = 0.8 - 0.9$ $M = \frac{6\sin\varphi'}{3 - \sin\varphi'}$
Mayne (2007)	$\sigma'_p = k(q_t - \sigma_v)$	k: Pre consolidation cone factor with an expected range of 0.2–0.5
Robertson (2009)	$OCR = 0.25Q_t^{1.25}$	-
Robertson (2012)	$OCR = (2.625 + 1.75logFr)^{-1.25}Q_t^{1.25}$	-
Chanmee et al. (2017)	$OCR = k\left(\frac{q_t - \sigma_v}{OCR\sigma'_v}\right)$	Under consolidated deposits k=0.14-0.4



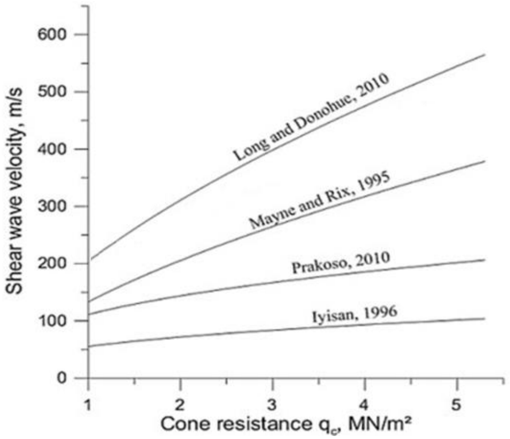
CPT correlation with OCR (Kulhawy and Mayne, 1990)

5.2. Estimating Soil Engineering Parameters

Shear Wave Velocity

Empirical correlations between V_s and CPT data (Ameratunga et al., 2016)

Reference	Proposed correlation (m/s)	Soil Type	Units of Parameters q_c	Units of Parameters f_s
Hegazy and Mayne (1995)	$V_s = 12.02(q_c)^{0.319}(f_s)^{-0.0466}$	Sand	kPa	kPa
	$V_s = 3.18(q_c)^{0.549}(f_s)^{0.025}$	Clay	kPa	kPa
Mayne and Rix (1995)	$V_s = 1.75(q_c)0.627$	Clay	kPa	kPa
Madiai and Simoni (2004)	(1) $V_s = 211(q_c)^{0.23}$ (2) $V_s = 155(q_c)^{0.29}(f_s)^{-0.10}$	All	MPa	MPa
Mayne (2006)	$V_s = 18.5+118.8\log(f_s)$	All	-	kPa
MolaAbasi et al. (2015)	$V_s=100[1.36-0.35f_s+0.15q_c-0.05f_s^2-0.018q_c^2+0.39f_s q_c]$	Clay	MPa	MPa
	$V_s=100[1.73+2.74f_s+0.03q_c-4.015f_s^2-0.00026q_c^2+0.007f_s q_c]$	Sand	MPa	MPa
	$V_s=100[1.47+2.07f_s+0.10q_c+9.50f_s^2-0.0023q_c^2-0.034f_s q_c]$	Mixed	MPa	MPa
	$V_s=100[1.40+1.59f_s+0.09q_c-1.33f_s^2-0.002q_c^2+0.05f_s q_c]$	All	MPa	MPa



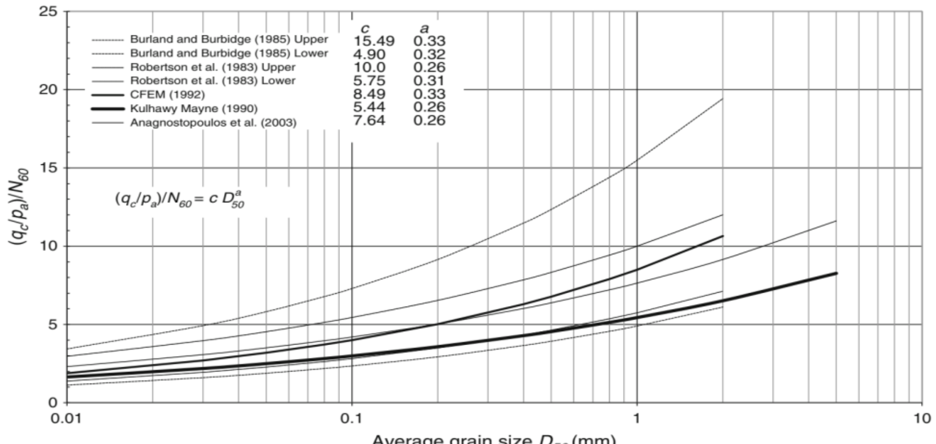
Shear Modulus at Small Strain

Empirical correlations for G_0 based on q_c

Reference	Correlations	Remarks
Mayne and Rix (1995)	$G_{max} = 99.5pa^{0.305}(qc)^{\frac{0.695}{e_0^{1.13}}}$	Cohesive Soils, e_0= initial void ratio
Eslaamizaad and Robertson (1997)	$\left(\frac{G_0}{q_c}\right) = 1634\left(\frac{q_c}{\sqrt{\sigma'_v}}\right)^{-0.75}$	Cohesionless Soils
Schnaid (2009)	$G_0 = b(q_c\sigma'_v Pa)^{0.3}$	Cohesionless Soils, b= 280 and 110 for an upper and lower bond.

Units (kPa)

CPT correlations with SPT



(Adopted from Robertson and Campanella, 1983)

5. Application of CPT & CPTu in GE

Case Study No.3: Eslami & Mohammadi (2016)

Ships and Offshore Structures, 2016
http://dx.doi.org/10.1080/17445302.2015.1131082

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Drained soil shear strength parameters from CPTu data for marine deposits by analytical model

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(*Received 11 January 2015; accepted 8 December 2015*)

Soil shear strength parameters, i.e. cohesion (C) and friction angle (φ) are typically determined using laboratory and *in situ* tests, although some limitations are involved in laboratory tests, such as the need for considering size effects and the use of undisturbed sampling. Cone penetration testing (CPT) has been recognised as a rapid and versatile procedure to provide continuous soil records, particularly in marine environment. In this study, an analytical approach is utilised to calculate drained soil strength parameters using piezocone penetration test (CPTu) records, i.e. q_t (corrected point resistance) and f_s (sleeve friction) and the results are compared with those obtained from laboratory tests. Current methods for obtaining shear strength parameters using CPT data are based on bearing capacity and cavity expansion theories and are able to estimate only φ in sands, and undrained shear strength (S_u) in cohesive soils. In this paper, by combining bearing capacity theories and direct shear modes of failure at CPTu tip and sleeve resistances, and considering the pore water pressure at the shoulder of the piezocone (u_2), a set of equations is derived. By inputting CPTu data including q_t, f_s and u_2 at a certain depth, soil shear strength parameters can be calculated simultaneously. Finally results obtained from this method are compared with measured soil shear strength parameters, using a data bank consisting of 50 sets of CPTu sounding carried out in marine deposits at various locations around the world. The comparison between predicted and measured C and φ values indicates good consistency and low scatter for the results obtain from the proposed method. This demonstrates that the proposed method is able to predict soil shear strength parameters in difficult marine environments with acceptable accuracy.

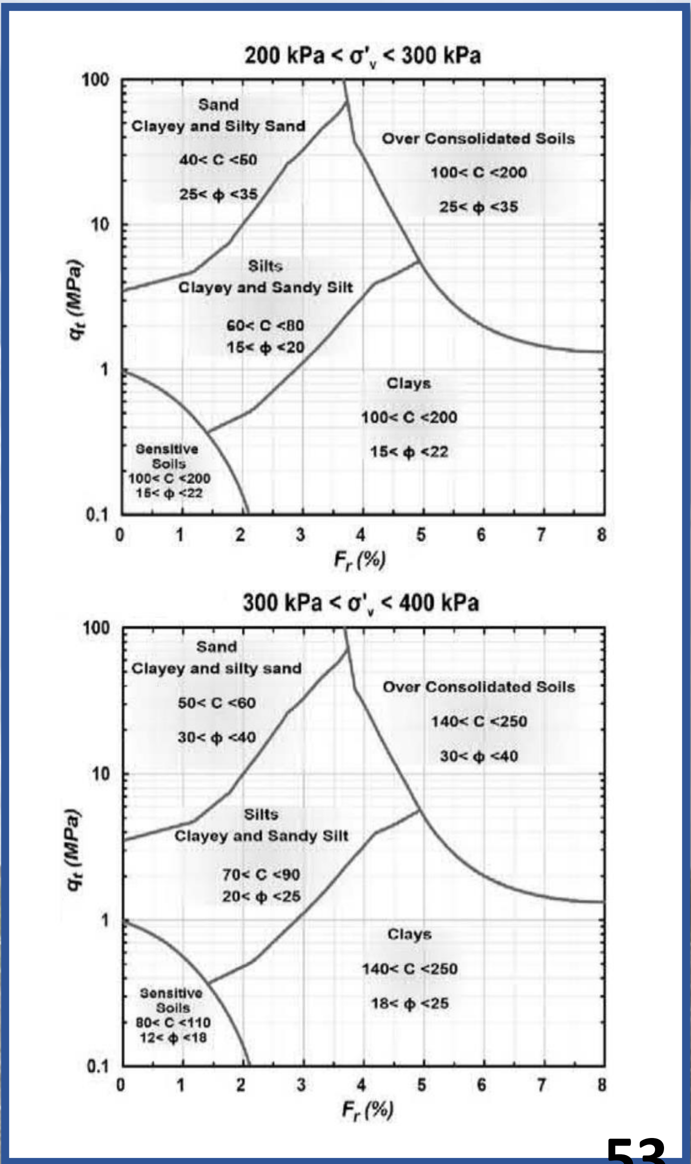
Keywords: shear strength parameters; CPTu; marine deposits; analytical approach

5. Application of CPT & CPTu in GE

Case Study No.3: Eslami & Mohammadi (2016)

Input Data	Output
q_c, f_s, u_2	C' , ϕ'

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



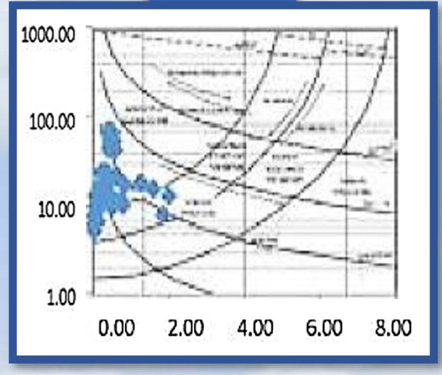
Variation range for C (kPa) and 4 (Degree)

5. Application of CPT & CPTu in GE

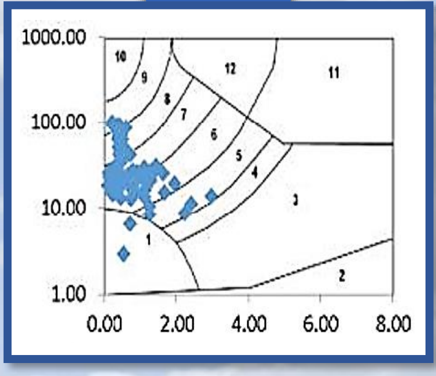
5.3. Identify Problematic Deposits

- ***Collapsible Soils*** **Boundaries in Different Charts (Eslami et al., 2016)**

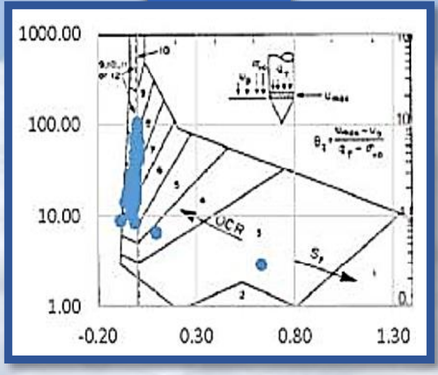
Douglas & Olsen (1981)



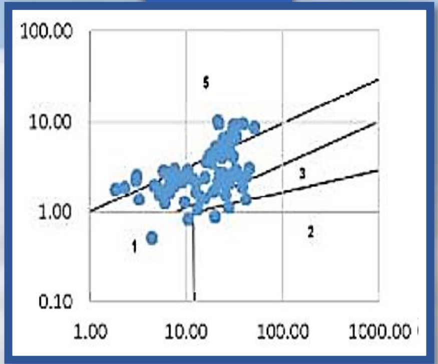
Robertson et al. (1986)



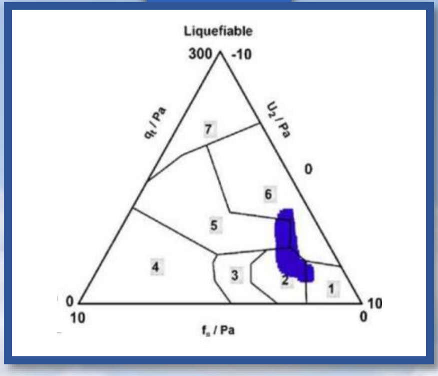
Robertson et al. (1986)



Eslami & Fellenius (1997)

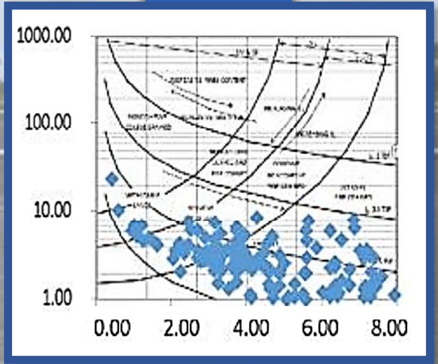


Eslami et al. (2016)

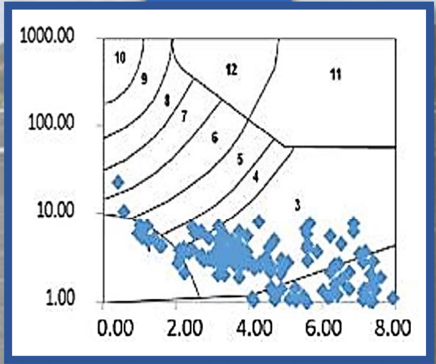


- ***Peaty Soils*** **Boundaries in Different Charts (Eslami et al., 2016)**

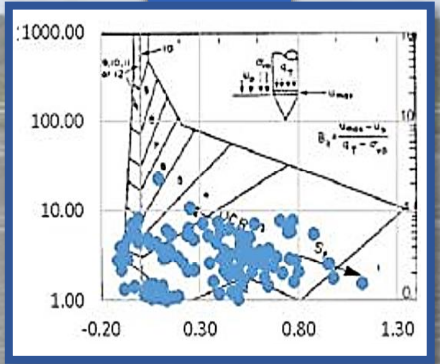
Douglas & Olsen (1981)



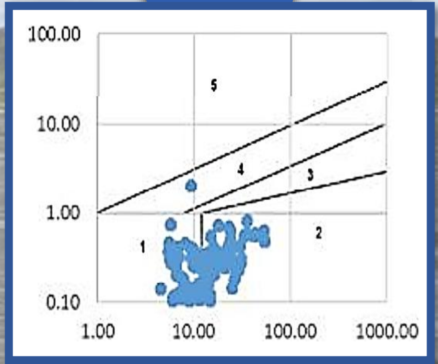
Robertson et al. (1986)



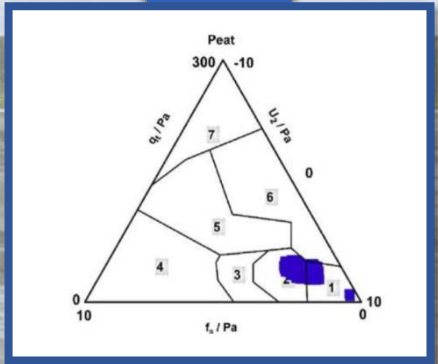
Robertson et al. (1986)



Eslami & Fellenius (1997)



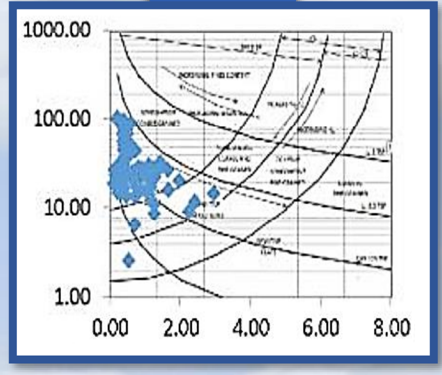
Eslami et al. (2016)



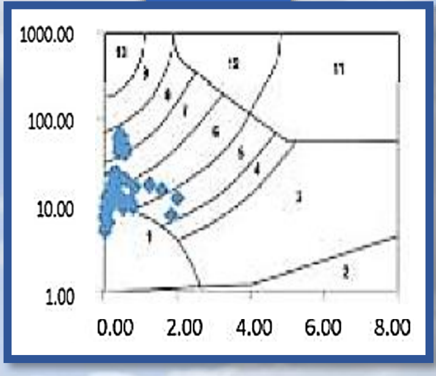
5.3. Identify Problematic Deposits

- ***Liquefiable Soils*** **Boundaries in Different Charts (Eslami et al., 2016)**

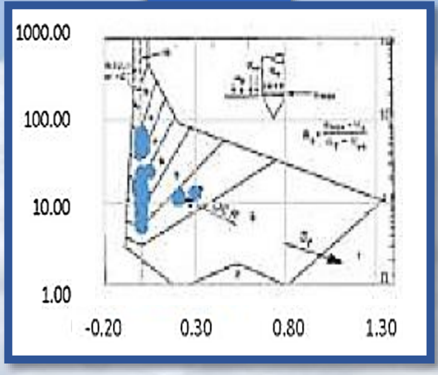
Douglas & Olsen (1981)



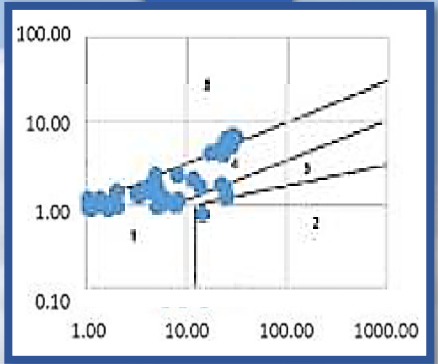
Robertson et al. (1986)



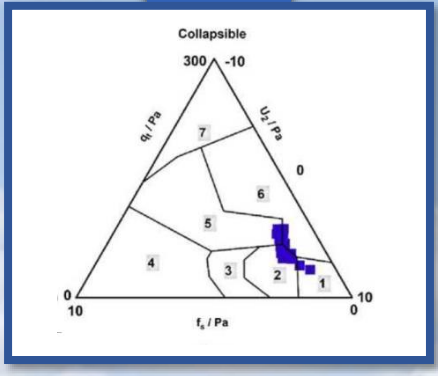
Robertson et al. (1986)



Eslami & Fellenius (1997)

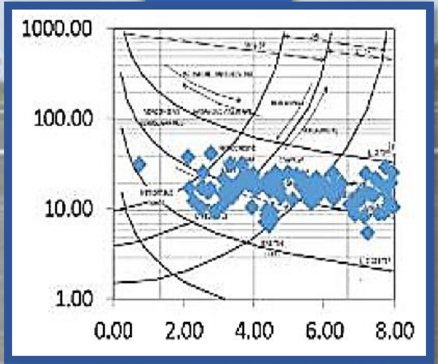


Eslami et al. (2016)

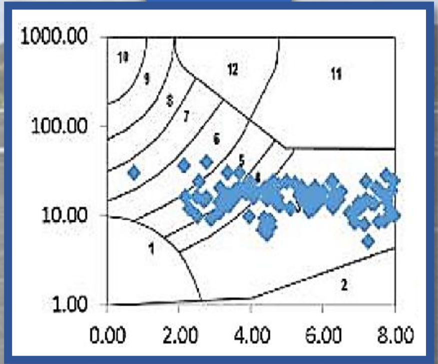


- ***Expansive Soils*** **Boundaries in Different Charts (Eslami et al., 2016)**

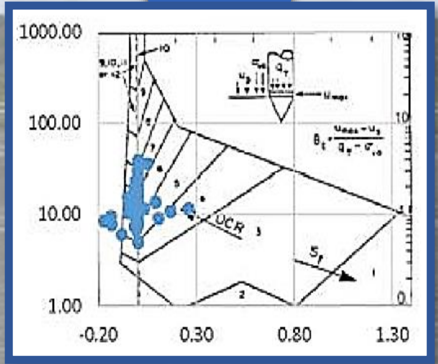
Douglas & Olsen (1981)



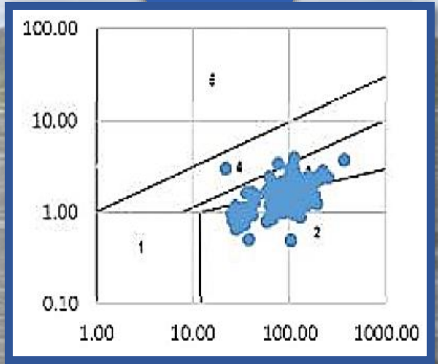
Robertson et al. (1986)



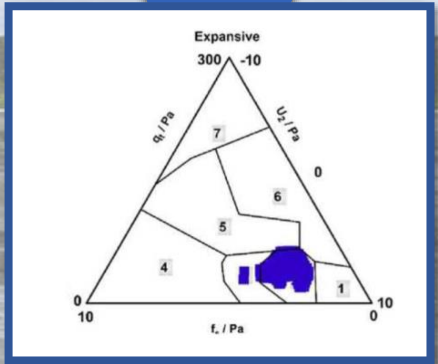
Robertson et al. (1986)



Eslami & Fellenius (1997)

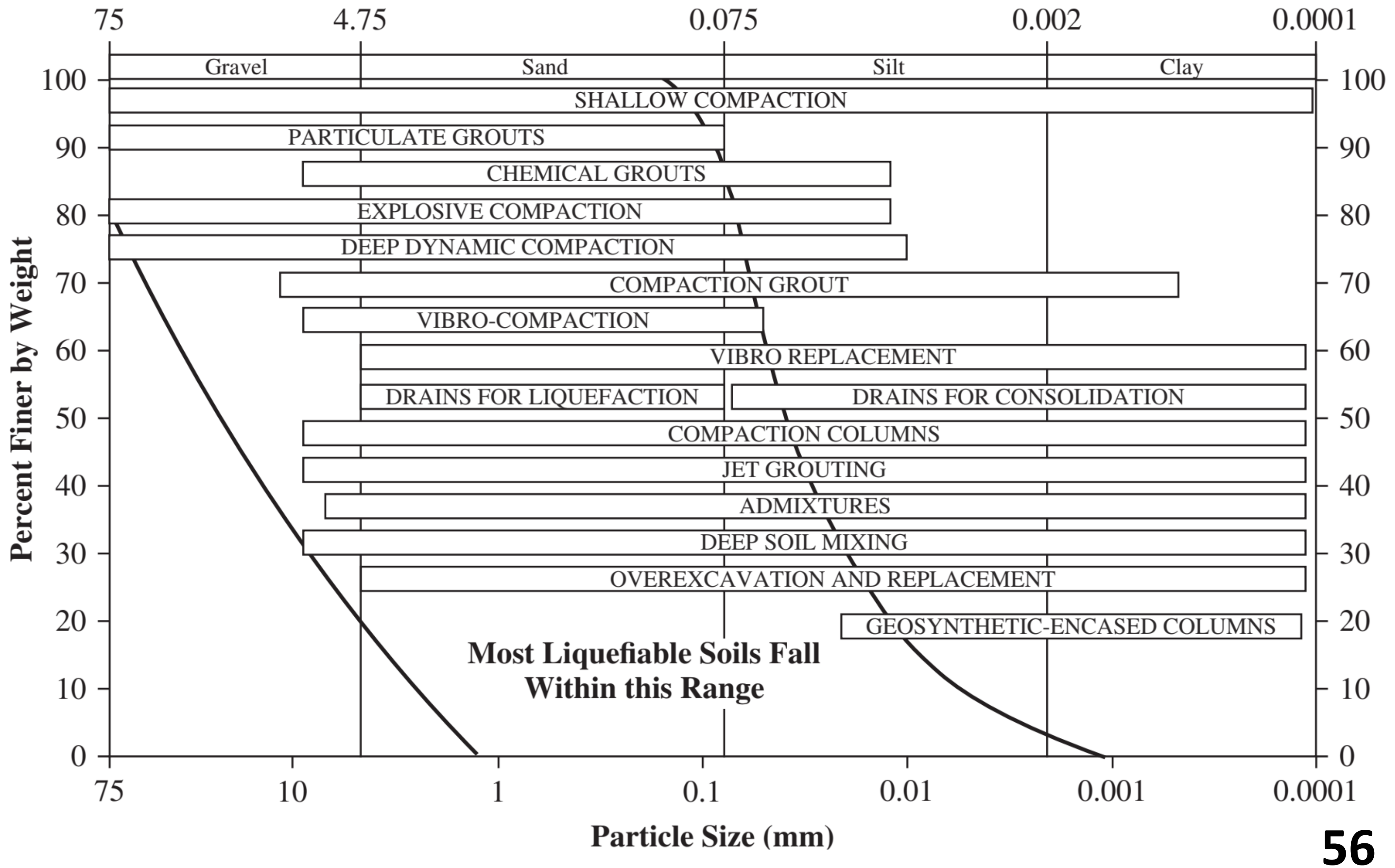


Eslami et al. (2016)



5.4. Role of CPT 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)

5. Application of CPT & CPTu in GE

Case Study No.4: Asadi, F., Eslami, A. & Valikhah, F. (2016)

MARINE GEORESOURCES & GEOTECHNOLOGY
http://dx.doi.org/10.1080/1064119X.2016.1213774

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Ground improvement and foundation practice for Persian Gulf Bridge (causeway); Bandar Abbas Harbor–Qeshm Island

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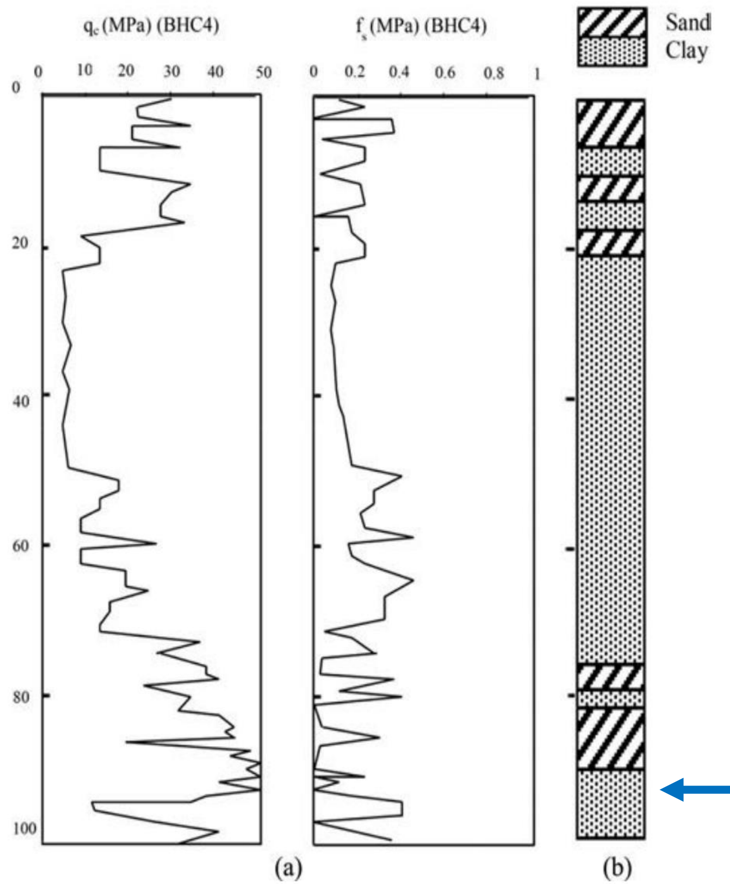
ABSTRACT

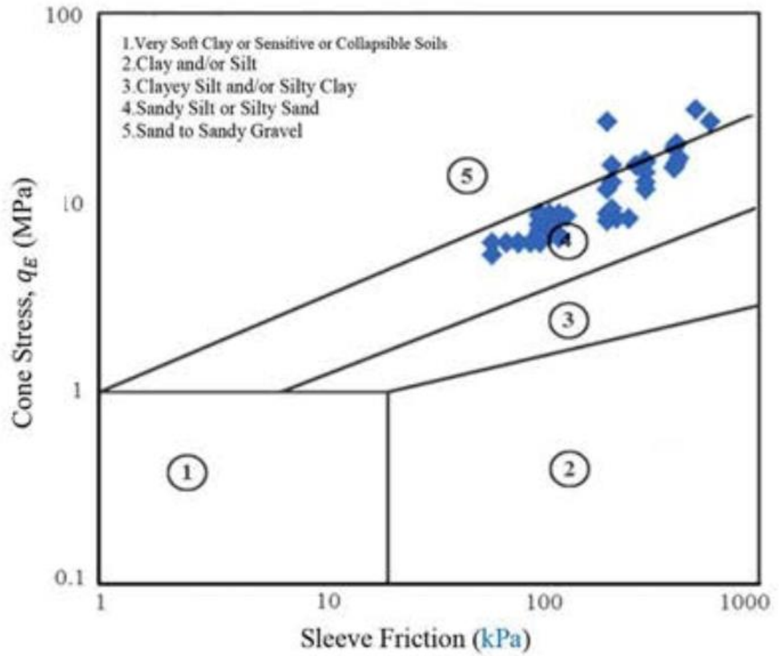
National Persian Gulf Bridge is a communication route between Bandar Abbas port and Qeshm Island located on the southern border of Iran. This causeway has important role for facilitating the transportation system between Qeshm Island and mainland, i.e., Bandar Abbas. Based on geotechnical and geological site investigation records, the bridge is located on the deposits with high seismic possibility and subsequently significant dynamic loading. Therefore, adequate substructure design of this bridge as an offshore project is realized as a major requirement. The geophysical and geotechnical investigations have been done to obtain the subsoil characteristics of the project site. For this purpose, 18 boreholes have been performed to do in situ tests and extract samples for laboratory testings. Data synthesis indicates that in the zones close to Qeshm Island and in the deeper parts of the sea, the strata is made of clay with loose sands and some depths, with silty sands. Hence, instability issues, including the low bearing capacity and the high differential settlement, are significant aspects in analysis and design of substructure for this project. Also, in this paper, the subsoil conditions have been studied from in situ tests such as standard penetration test (SPT) and cone penetration test (CPT) results in order to achieve an appropriate foundation system. Moreover, the necessity of the ground improvement of the site has been investigated to propose an efficient technique for safe and secure construction. Based on the analysis and methods screened, the vibro-replacement method is considered as a suitable and efficient ground improvement method for this project.

ARTICLE HISTORY
Received 5 January 2016
Accepted 10 June 2016

KEYWORDS
Foundation system; geotechnical site characterization; ground improvement; numerical analysis; Persian Gulf Bridge; subsoil instability

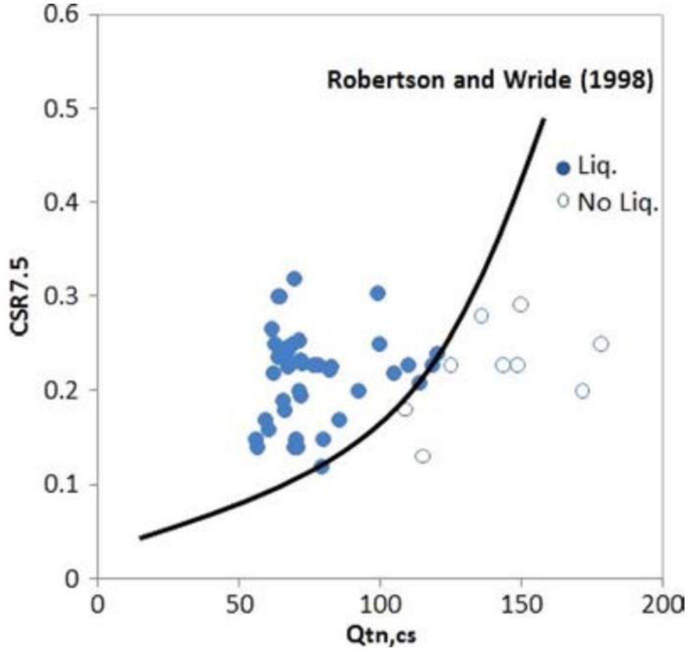
Case Study No.4: Asadi, F., Eslami, A. & Valikhah, F. (2016)





SBC in deep parts of the sea using charts of Eslami and Fellenius (2004)

(a) CPT results for BHC4 in deep parts of the sea and (b) The borehole log of BHC4.



Evaluating the soil liquefaction potential in deep parts of the sea using CPT records (Robertson and Wride 1998)

Alternatives: Vibro-Replacement, Explosive Compaction, Deep Soil Mixing & Compaction Pile

Proposed Method: *Vibro-Replacement*, environmental & economic aspects

5. Application of CPT & CPTu in GE

Case Study No.5: Eslami (2015) & Eslami & Shakeran (2016)

Bull Earthquake Eng
DOI 10.1007/s10518-015-9776-4

CrossMark

ORIGINAL RESEARCH PAPER

Investigation of explosive compaction (EC) for liquefaction mitigation using CPT records

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

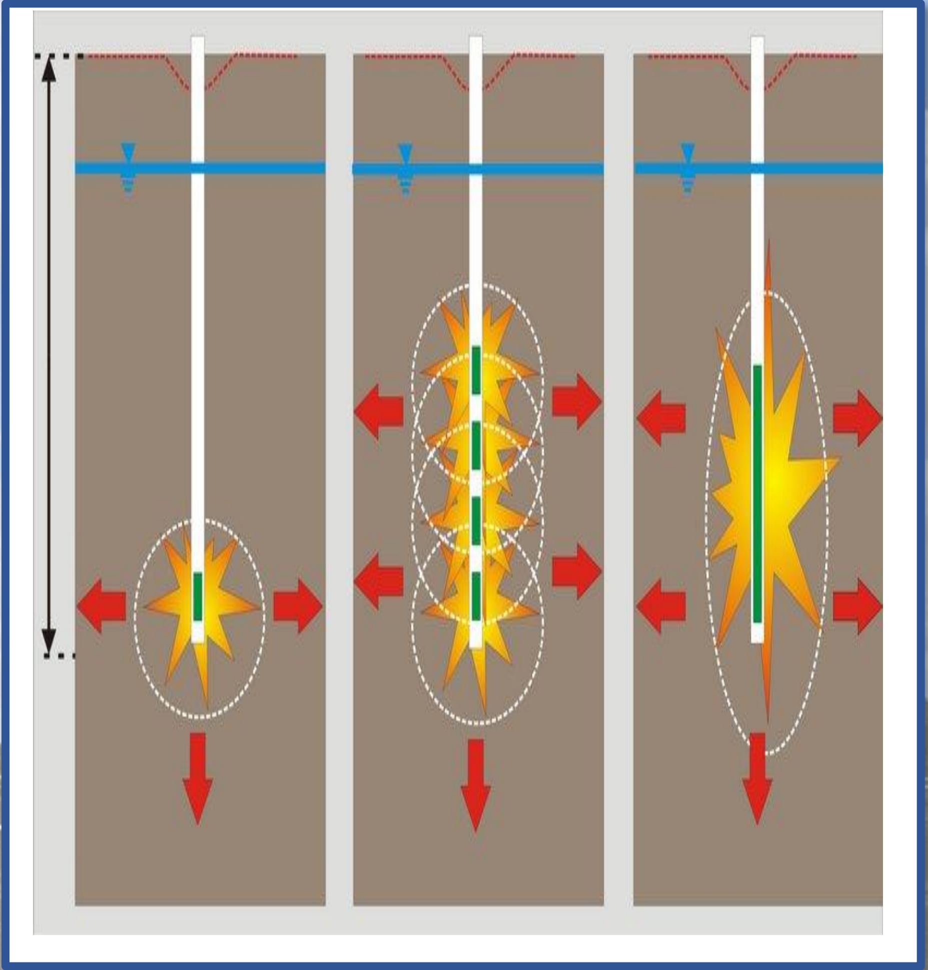
Received: 10 January 2015 / Accepted: 17 May 2015

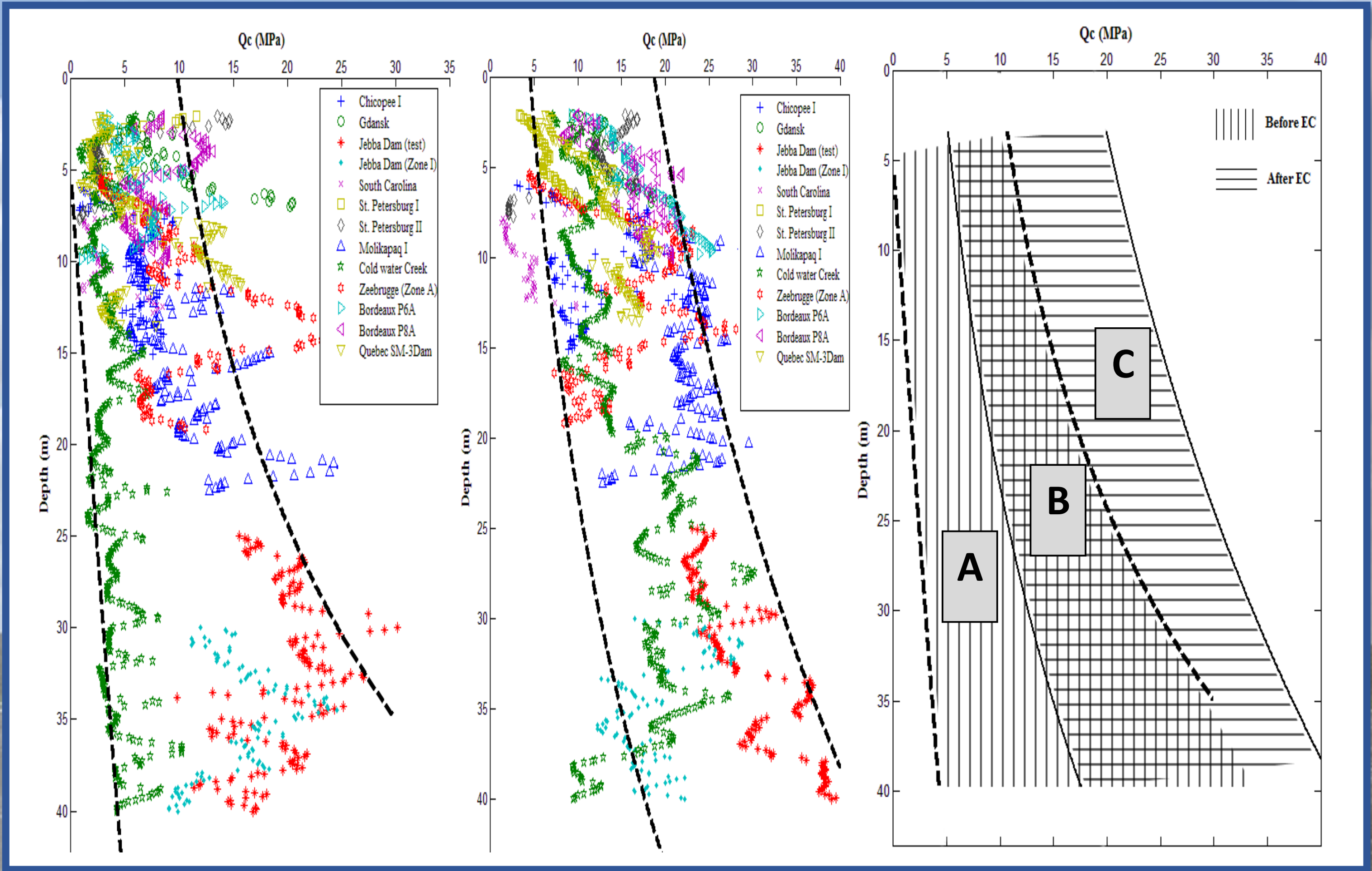
Abstract Explosive compaction (EC) or Blast densification (BD) has been realized as an efficient technique for soil improvement and mitigation of the liquefaction potential in loose saturated sands. Due to providing continuous and precise records, Piezocone (CPTu) is the most applicable in situ test in geotechnical practice for evaluation of liquefaction potential. In this research a data bank including eight case histories in different locations has been compiled for investigation of EC effects on mitigation of loose sands instability. The sites geomaterials are in the category of fine to medium sand, silty sand and mixture of sand and gravel with relative density between 30 and 60 % and thickness of 5–40 m. Four CPT-based criteria have been used including cyclic stress ratio approach, cone tip resistance (q_c) variations before and after modification, Q_{tn} and q_{c1N}, and soil behavior classification charts. Analyses have shown that due to EC the state of soil changes from loose to dense, the contractive behavior of sands changes to dilative, and the liquefaction potential diminishes. Also, by using soil behavior classification charts pre and post explosion, it can be observed that improved soils are not in the liquefiable zone, anymore. This improvement has a significant effect on layers where located in deeper zones, whereas in surface layers in some cases, liquefaction phenomenon has been observed. Moreover, by blasting in two stages between first and phases for boreholes, liquefaction potential decreases significantly.

Keywords Deep soil improvement · Explosive compaction (EC) · Loose deposits · Liquefaction · CPT records

5. Application of CPT & CPTu in GE

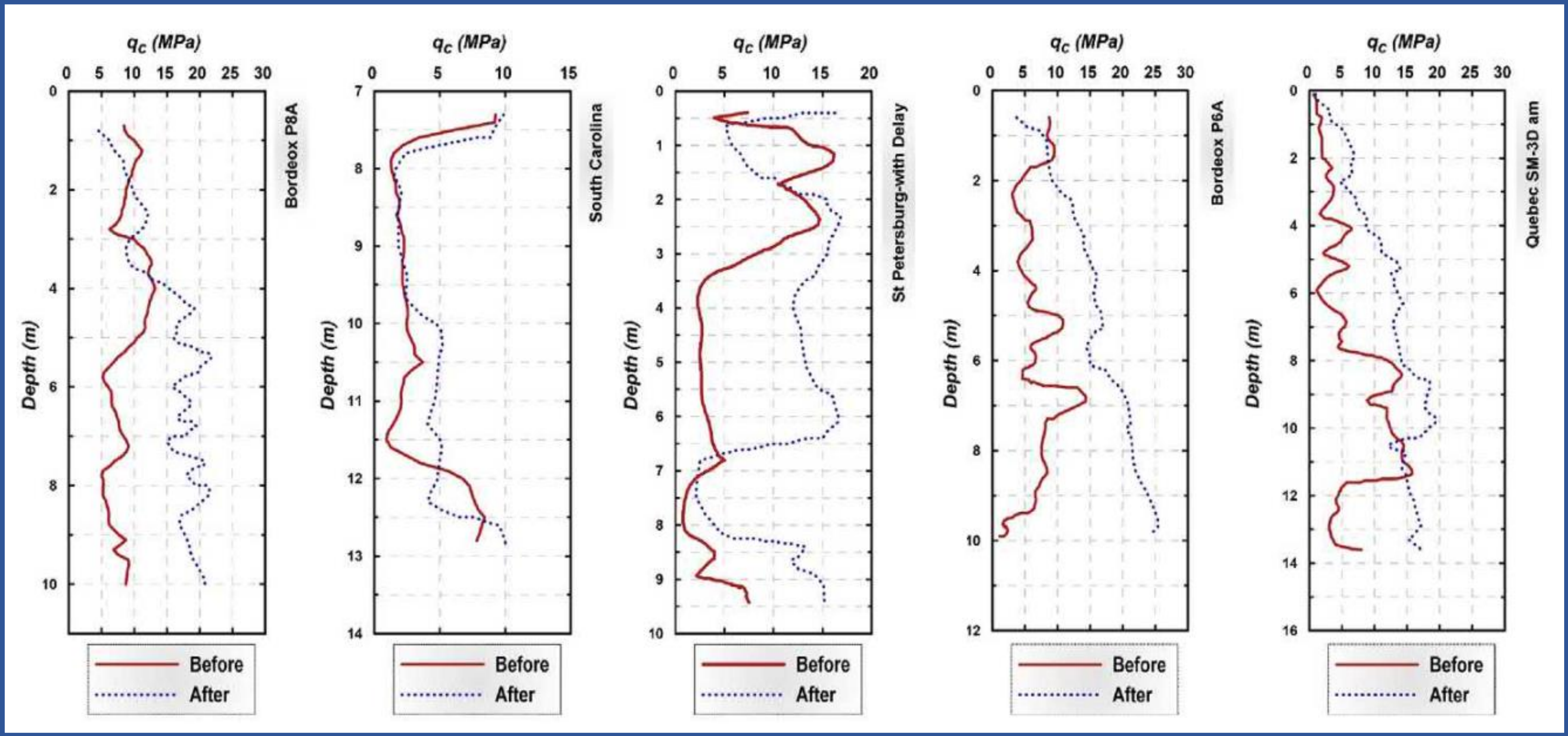
Case Study No.5: Eslami (2015) & Eslami & Shakeran (2016)





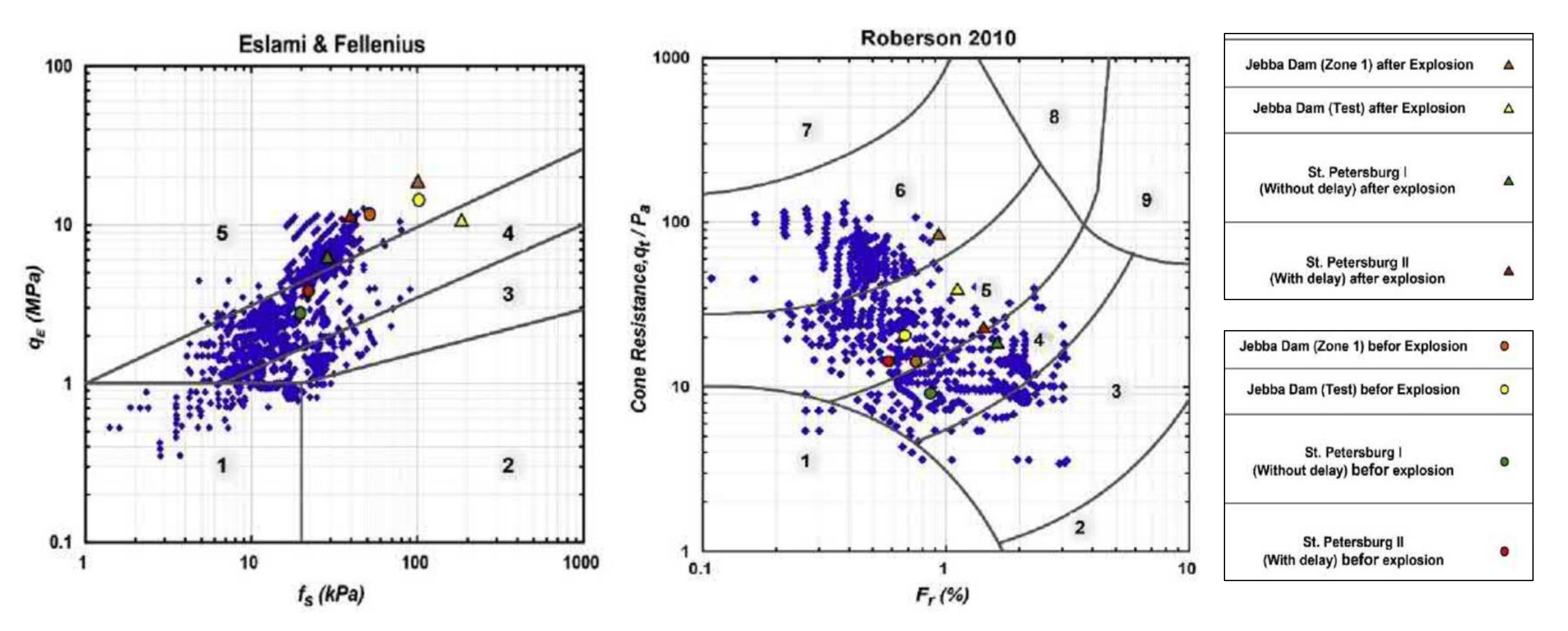
5. Application of CPT & CPTu in GE

Case Study No.5: Eslami (2015) & Eslami & Shakeran (2016)



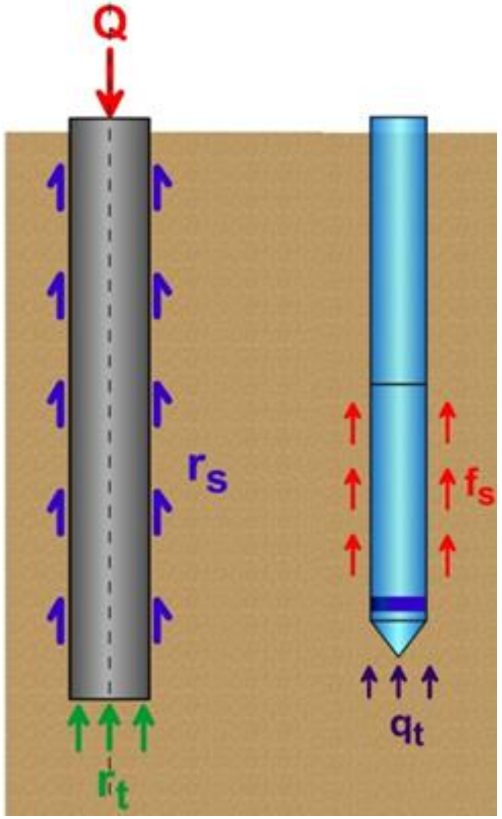
5. Application of CPT & CPTu in GE

Case Study No.5: Eslami (2015) & Eslami & Shakeran (2016)



SBC charts for soil behavior assessment before and after explosion

6.1. Background



Indirect Approach: q_c & $f_s \rightarrow C, \varnothing \rightarrow r_t, r_s$

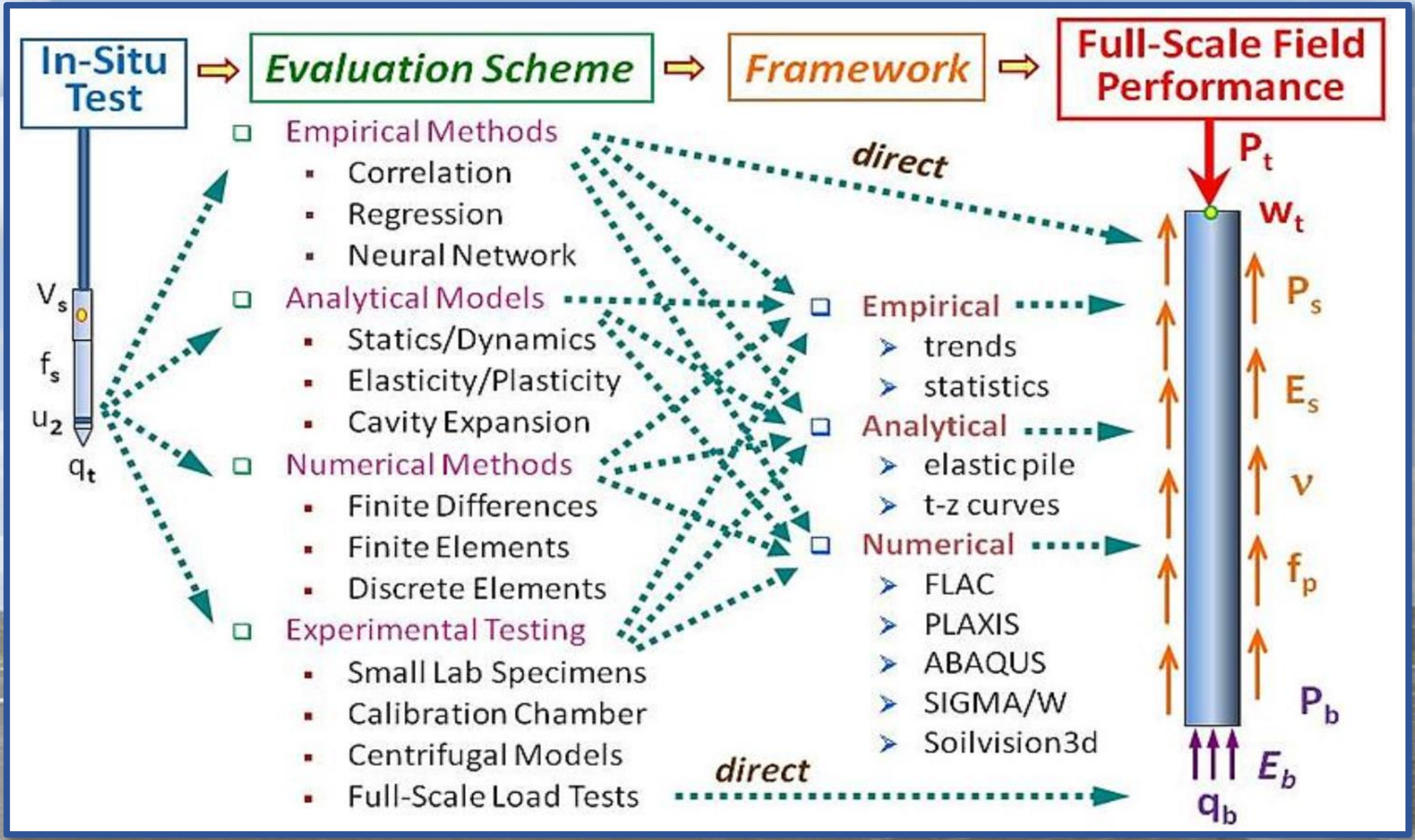
Direct Approach:

$q_c \rightarrow r_t$

$f_s \rightarrow r_s$

Penetrometers can be realized as a *model pile*

6.2. Methods to Calibrate & Interpret CPT Results



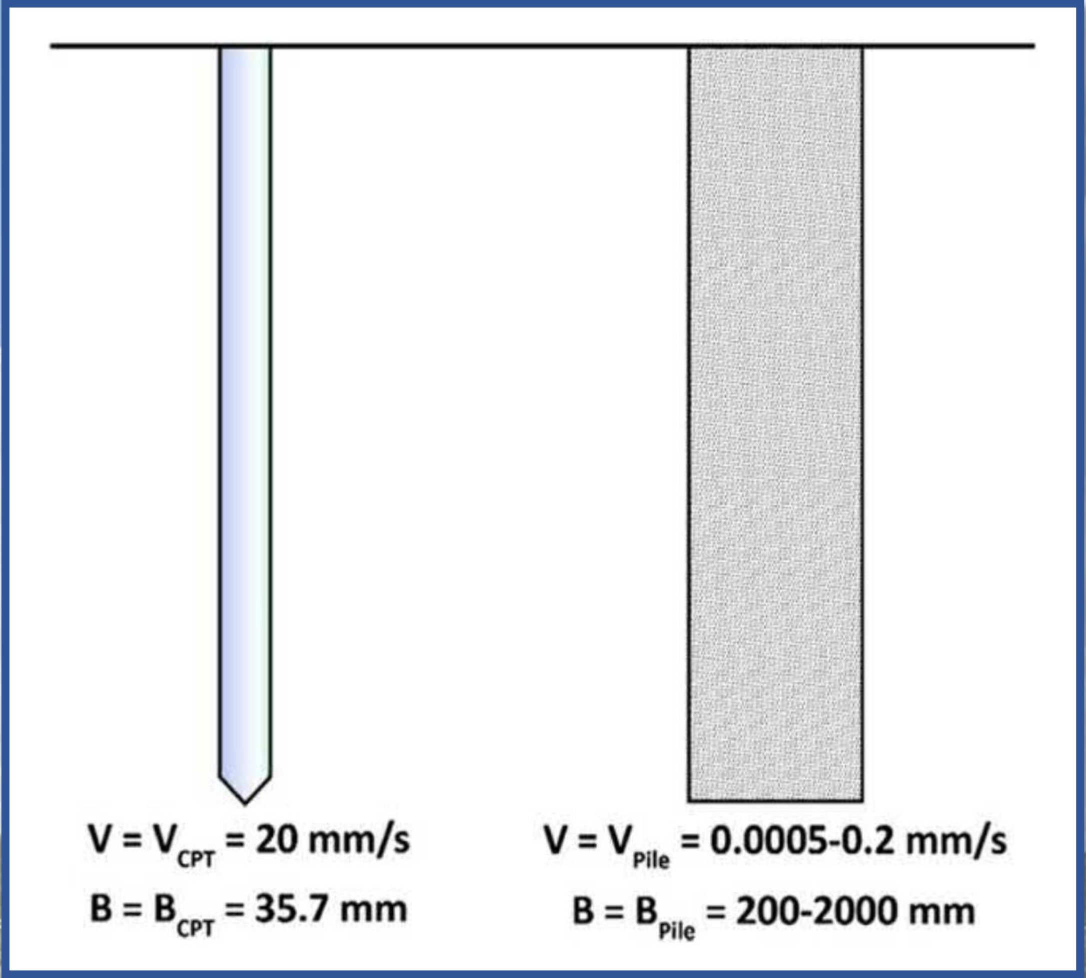
(Mayne, 2009)

6. CPT and Foundation Engineering: Scale Effect

6.3. Scale Effect Correlations

- **Determinant Factors for Toe Capacity**

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

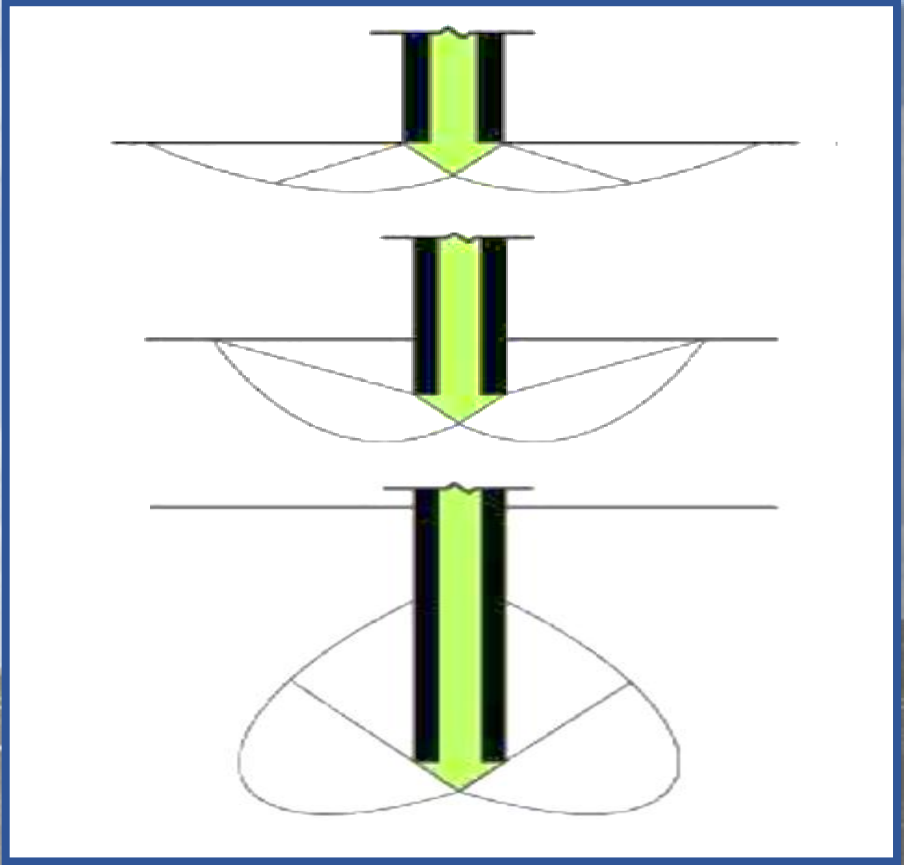


Schematic view of pile and cone penetration test differences in material, penetration rate, and dimensions (Eslami et al., 2019)

6. CPT and Foundation Engineering: Scale Effect

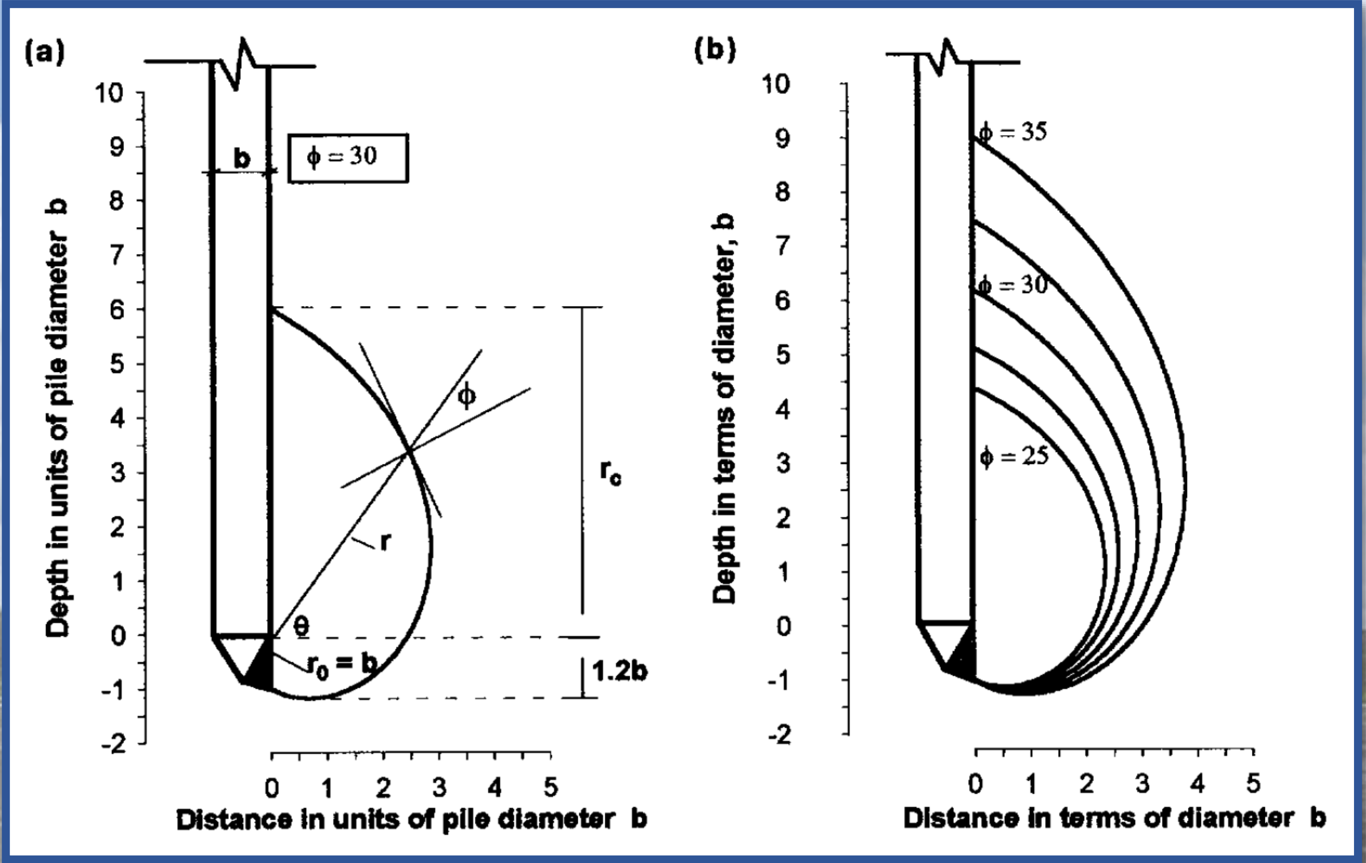
6.3. Scale Effect Correlations

Embedment Depth



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

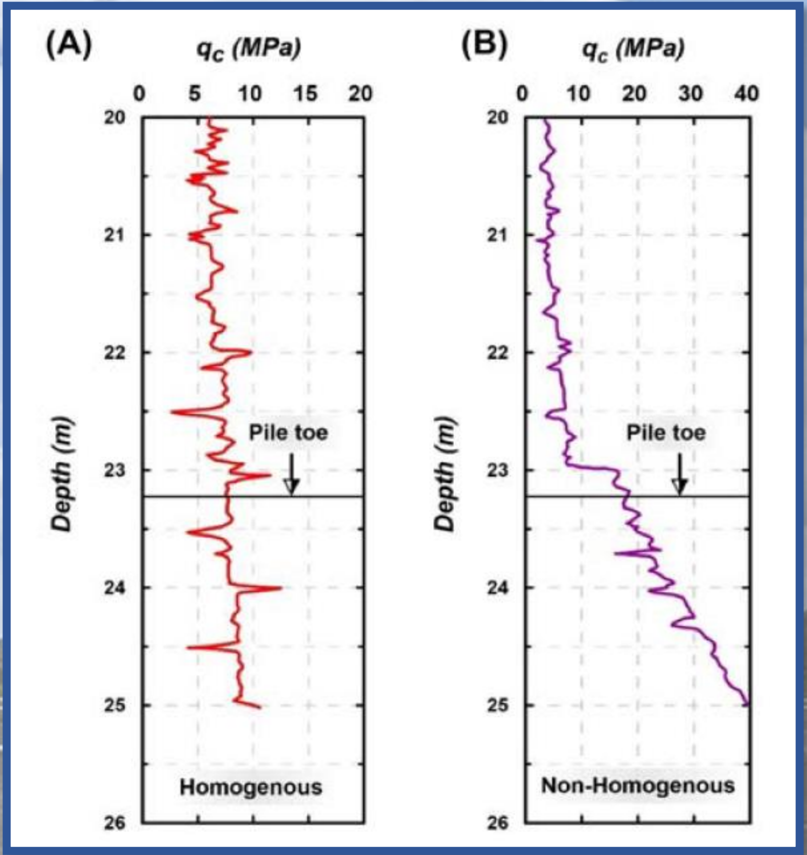
Influence Zone



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

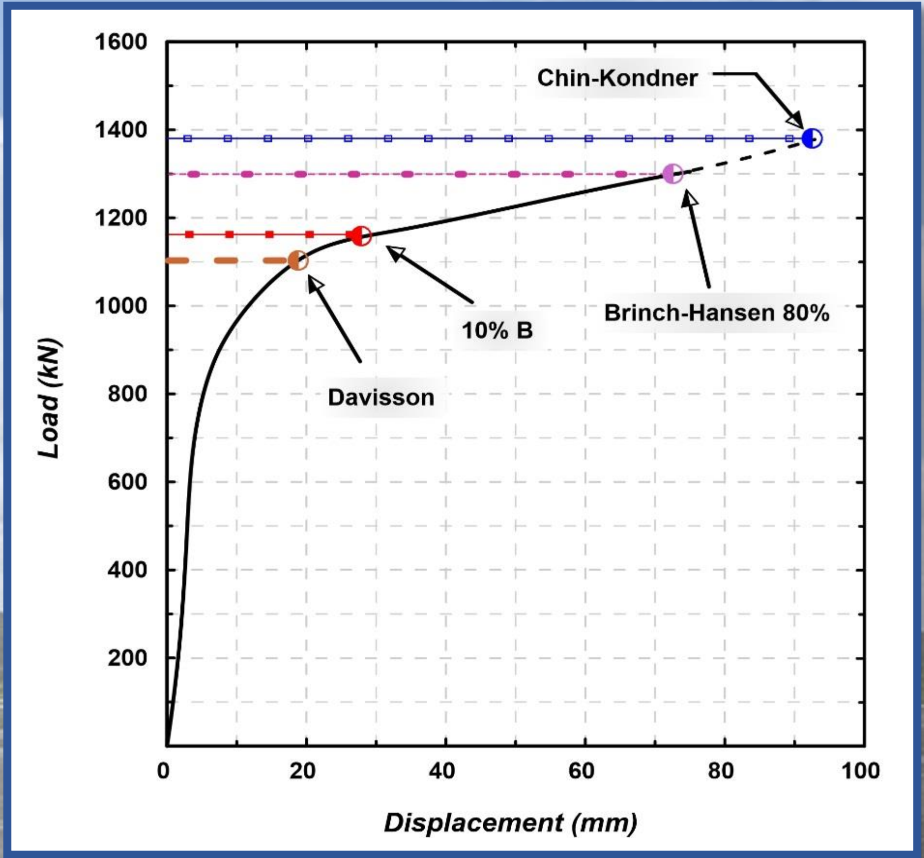
6.3. Scale Effect Correlations

Nonhomogeneous Condition



Comparison of pile unit toe resistance for different zones: (A) Homogeneous and (B) Nonhomogeneous (Eslami & Fellenius, 1997)

Ultimate Capacity Condition



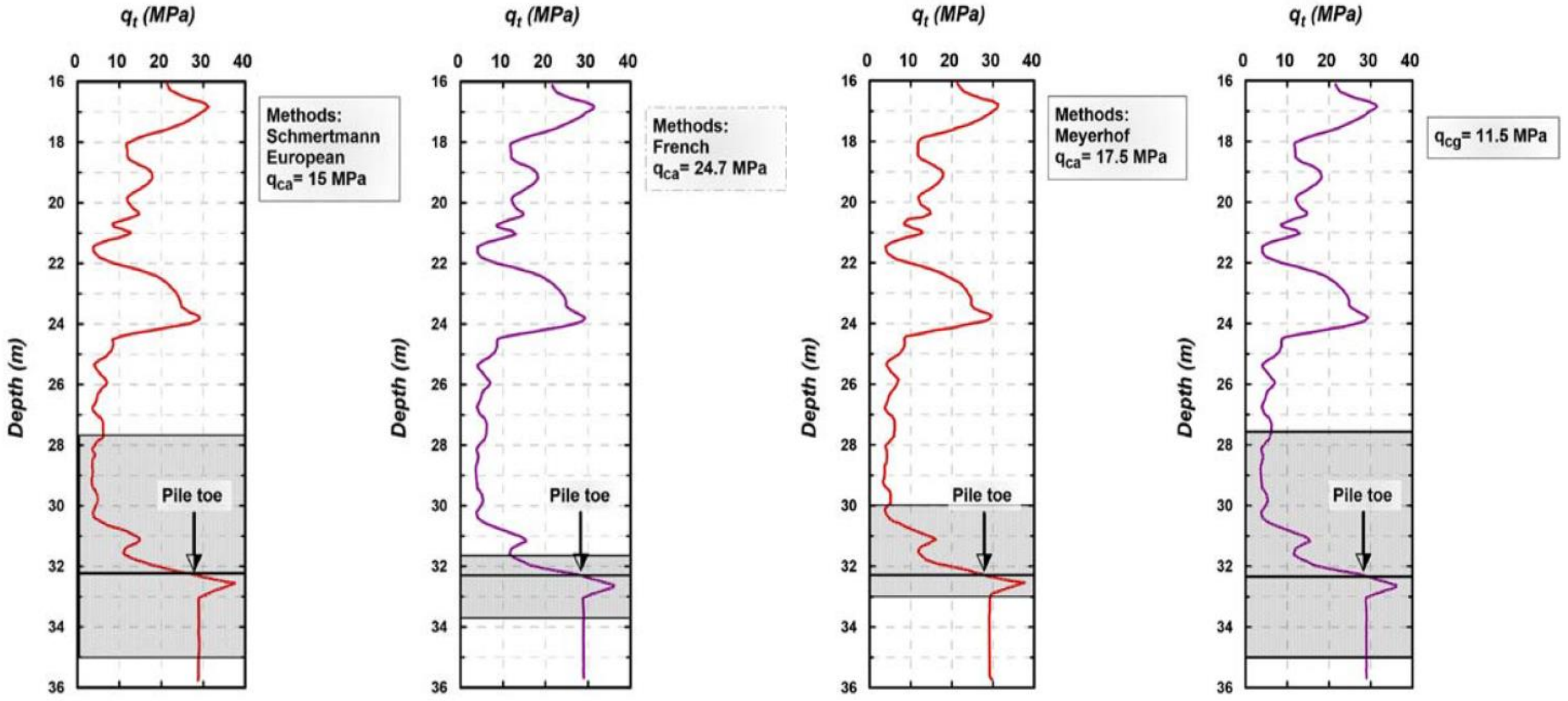
Interpretation of load displacement diagram for Case 001-L&D31 (Moshfeghi & Eslami, 2016)

6.3. Scale Effect Correlations

Data Processing, Averaging & Influence Zone

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

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



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

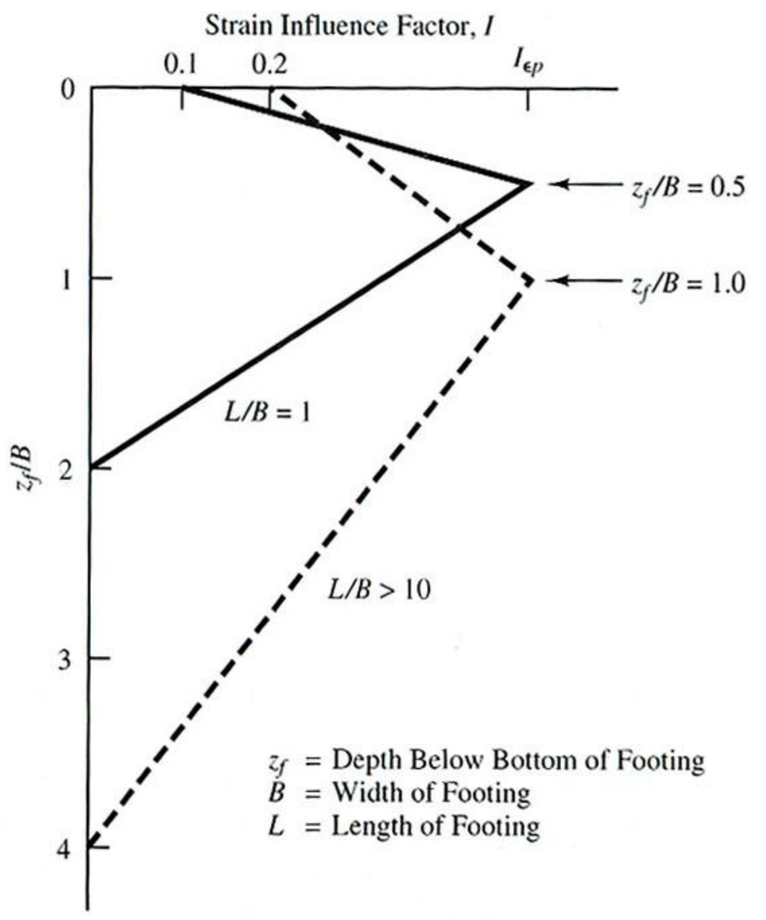
7.1. Direct Application of CPT Record for Settlement & Load-Displacement

- **Schmertmann (1978)**

$$S = C_1 . C_2 q_n \sum_{0}^{2B} \frac{I_z}{E} \Delta Z$$

$$C_1 = 1 - 0.5 \frac{\sigma'_{\circ}}{q_n}$$

$$C_2 = 1 + 0.2 \log \frac{t}{0.1}$$



7. Geotechnical Design: Bearing Capacity & Settlement

Case Study No.6: Valikhah & Eslami (2019)

Arabian Journal for Science and Engineering
https://doi.org/10.1007/s13369-019-04034-y

RESEARCH ARTICLE - CIVIL ENGINEERING

CPT-Based Nonlinear Stress–Strain Approach for Evaluating Foundation Settlement: Analytical and Numerical Analysis

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Abstract

Due to complexities in soil–foundation interaction and nonlinear behavior of subsoil, considerable uncertainty is involved in the foundation settlement evaluation. In the present paper, a new analytical approach is proposed to estimate the foundation settlement based on soil behavior classification charts developed from CPT records. The approach is founded on the Janbu nonlinear stress–strain method. However, instead of using fixed parameters in the Janbu method, the variable coefficients are used depending on geomaterial properties. Also, in the proposed approach, the scale effect is taken into account for foundation width considering soil stiffness. The proposed procedure is calibrated and verified by a data bank containing 46 case histories including 22 square, 17 circular and 7 rectangular foundations with widths varying between 0.3 and 2.4 m in conjunction with CPT data. Furthermore, the numerical finite difference analysis using a CPT-based stress characteristics method is carried out to validate the proposed approach for the prediction of foundation settlement. The accuracy of the calculations done by the proposed and some available common methods is investigated. Comparisons based on statistical and probabilistic methods apparently reveal that the proposed approach calculates the foundation settlement promisingly.

Keywords Nonlinear stress–strain · Settlement · CPT data · Stiffness modulus · Analytical and numerical analysis · Databank

Case Study No.6: Valikhah & Eslami (2019)

Proposed approach

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

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

b: penetration cone diameter

B: foundation width

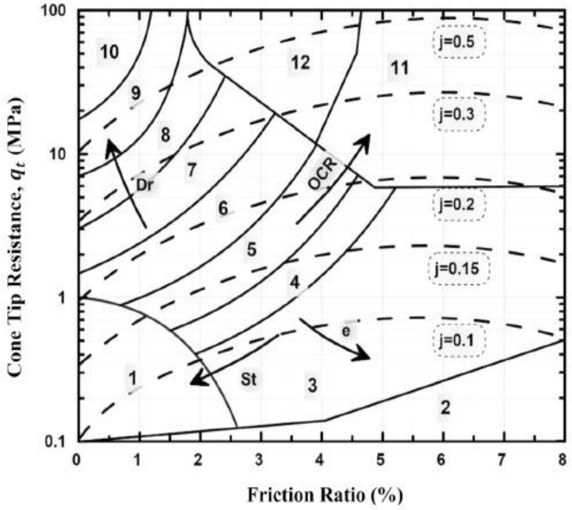
(b and B are in m and q_c is in kPa)

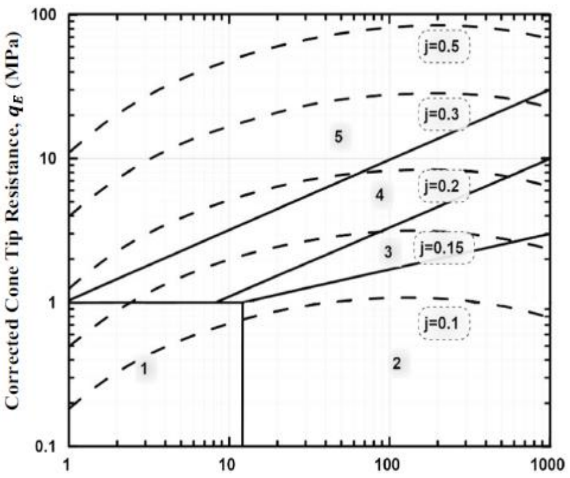
$$j = \frac{q_c}{x + yq_c}$$

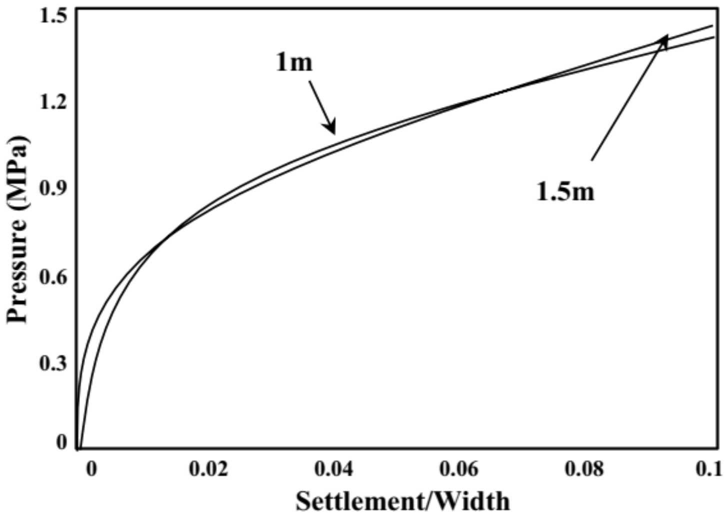
$$x = 0.02R_f + 0.5$$

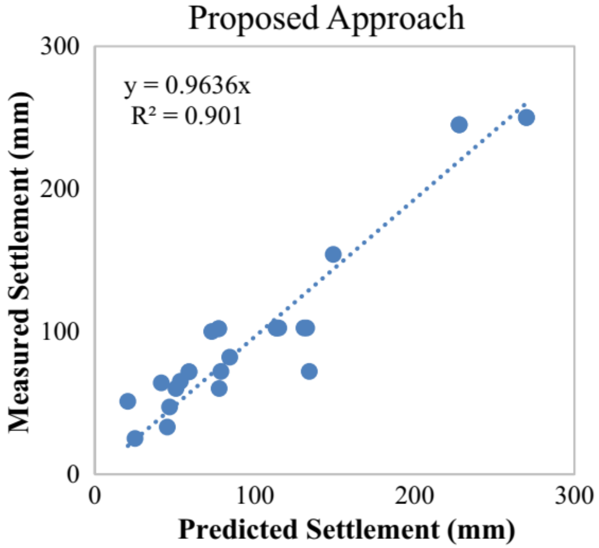
$$y = 7.53(\sigma_0')^{-0.25}$$

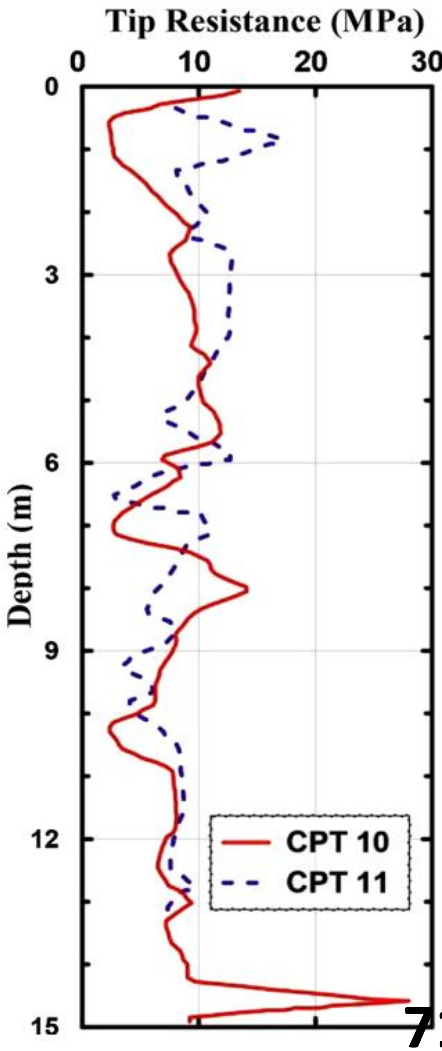
(q_c and σ_0' are in kPa)







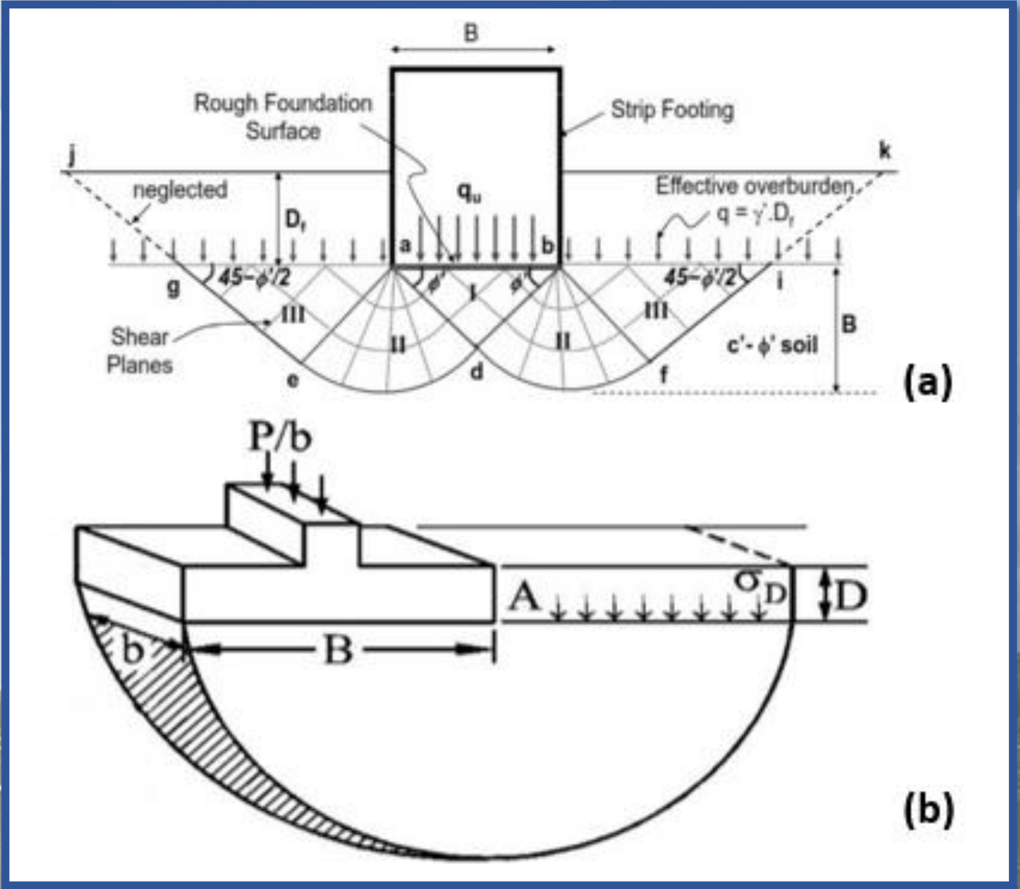




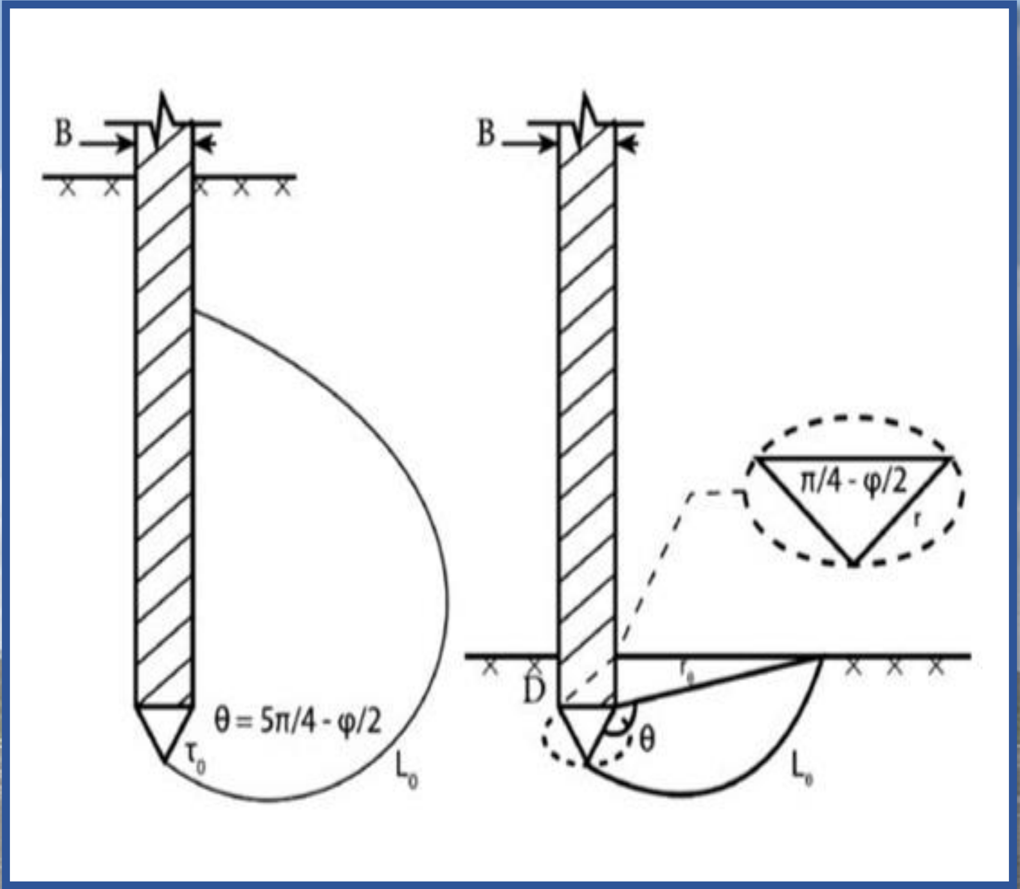
7. Geotechnical Design: Bearing Capacity & Settlement

7.2. Direct Application of CPT Records for Bearing Capacity

- **Shallow Foundations**



Schematic of shear failure zone, a) drained condition, b) undrained (Terzaghi, 1943)



Comparison of rupture surface length for shallow and deep conditions (Eslami & Gholami, 2006)

7. Geotechnical Design: Bearing Capacity & Settlement

7.2. Direct Application of CPT Records for Bearing Capacity

- **Shallow Foundations**

Reference	Equations	Remarks
Schmertmann (1978)	$q_{ult} = \bar{q}N_q + 0.5\gamma B N_\gamma$ $N_q = N_\gamma = 1.25\sqrt{q_{c1} \times q_{c2}}$	q_{c1}= arithmetic average of q_c values in an interval between footing base and 0.5B beneath footing base. q_{c2}= arithmetic average of q_c values in an interval between 0.5B to 1.5B beneath footing base.
Meyerhof (1976)	$q_{ult} = \bar{q}_c \left(\frac{B}{12.2}\right)\left(1 + \frac{D_f}{B}\right)$	$\bar{q}_c$= arithmetic average of q_c values in a zone including footing base and 1.5B beneath the footing. F.S. at least 3 is recommended
Bowles (1996)	$q_{ult} = 28 - 0.0052(300 - \bar{q}_c)^{1.5}$, for strip footings $q_{ult} = 48 - 0.0052(300 - \bar{q}_c)^{1.5}$, for square footings	$\bar{q}_c$= the arithmetic average of q_c values in an interval between footing base and 1.5B beneath, in terms of kg/cm^2.
CFEM (2006)	$q_{ult} = 0.30\ \bar{q}_c$ $q_{all} = 0.10\ \bar{q}_c$	a safety factor of 3 has been suggested
Tand et al. (1994)	$q_{ult} = R_k q_c + \sigma_{v0}$	R_k values range from 0.14 to 0.2, depending on the footing shape and depth, and σ_{v0} is the initial vertical stress at the footing base.
Eslami and Gholami (2006)	$q_{ult} = \bar{\alpha} \times \bar{q}_{cg}$ $\varphi = \dfrac{\log\left(\frac{\bar{q}_c}{\gamma' z}\right) + 0.5095}{0.0915}$	$\bar{q}_{c,g}$= geometric average of q_c values from footing base to 2B beneath footing depth.

7. Geotechnical Design: Bearing Capacity & Settlement

Case Study No.7: Eslami & Gholami (2006)

Scientia Iranica, Vol. 13, No. 3, pp 223 233

SCIENTIA IRANICA

Analytical Model for the Ultimate Bearing Capacity of Foundations from Cone Resistance

A. Eslami* and M. Gholami[1]

By application of Cone Penetration Test (CPT) data for shallow foundation (footing) design, the problems of providing representative undisturbed samples and, rather, φ N coefficient relations will be eliminated. An analytical model, based on a general shear failure mechanism of the logarithm spiral type, has been developed for calculating, directly, the bearing capacity of footings, q_{ult} from cone resistance, q_c. The transform of the failure mechanism from a shallow to a deep foundation and the scale effect have been considered in the proposed method. Six current CPT direct methods for determining the bearing capacity of footings have been investigated. The proposed method and others were compared to the measured capacity, ranging from 1.7 to 15 kg/cm^2, of 28 footings compiled in a database with a range of diameter from 0.3 to 3 m located in different soils. The graphical and cumulative probability approaches for the validation of the methods indicates optimistic results for the bearing capacity estimation of the proposed method, which is simple and routine.

Case Study No.7: Eslami & Gholami (2006)

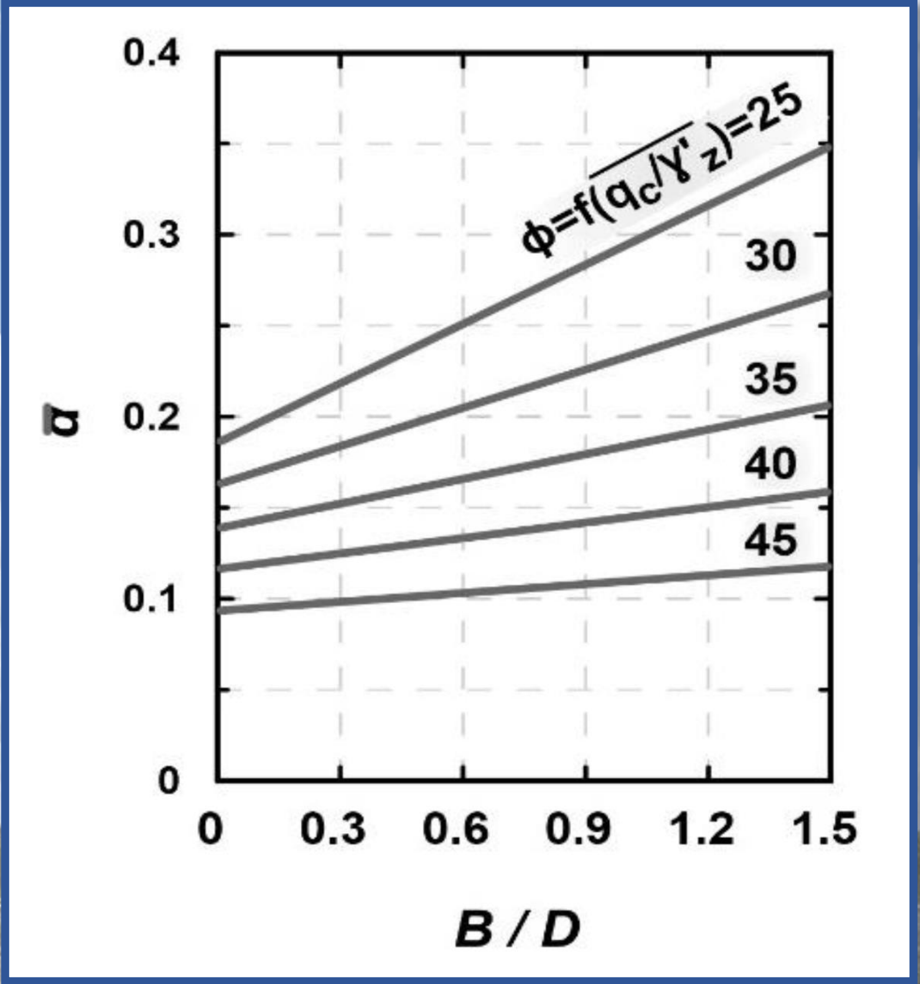
1. The zone located between the foundation base to 2B beneath can be divided into sublayers. The values of $\bar{q}_{cg}$ and $(\bar{q}_c/\gamma' z)_{cg}$ in this interval are calculated.

2. The average φ angle = $(\bar{q}_c/\gamma' z)_{cg}$

3. Based on D/B and φ values $\bar{\alpha}$ can be obtained

4. The ultimate bearing capacity is calculated as:

$$q_{ult} = \bar{\alpha} \times \bar{q}_{cg}$$



Bearing capacity correlation factor for relating q_{ult} to q_{cg} (Eslami & Gholami, 2006)

7.2. Direct Application of CPT Records for Bearing Capacity

- **Deep Foundations**

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)

Relevant Data Base Design: RDBD (Eslami & Heidarie, 2021)

7.2. Direct Application of CPT Records for Bearing Capacity

- **Deep Foundations**

Meyerhof (1956, 1976, 1983)

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

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

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

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

B = pile diameter (m)

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

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

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

Case Study No.8: Eslami & Fellenius (1997)

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

Abolfazl Eslami and Bengt H. Fellenius

Can. Geotech. J. **34**: 886–904 (1997). Received January 30, 1997. Accepted June 25, 1997.

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.

Case Study No.8: Eslami & Fellenius (1997)

- Toe Capacity

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

$$q_E = q_t - u$$

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

- Shaft Capacity

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

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

Shaft coefficient correlation

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

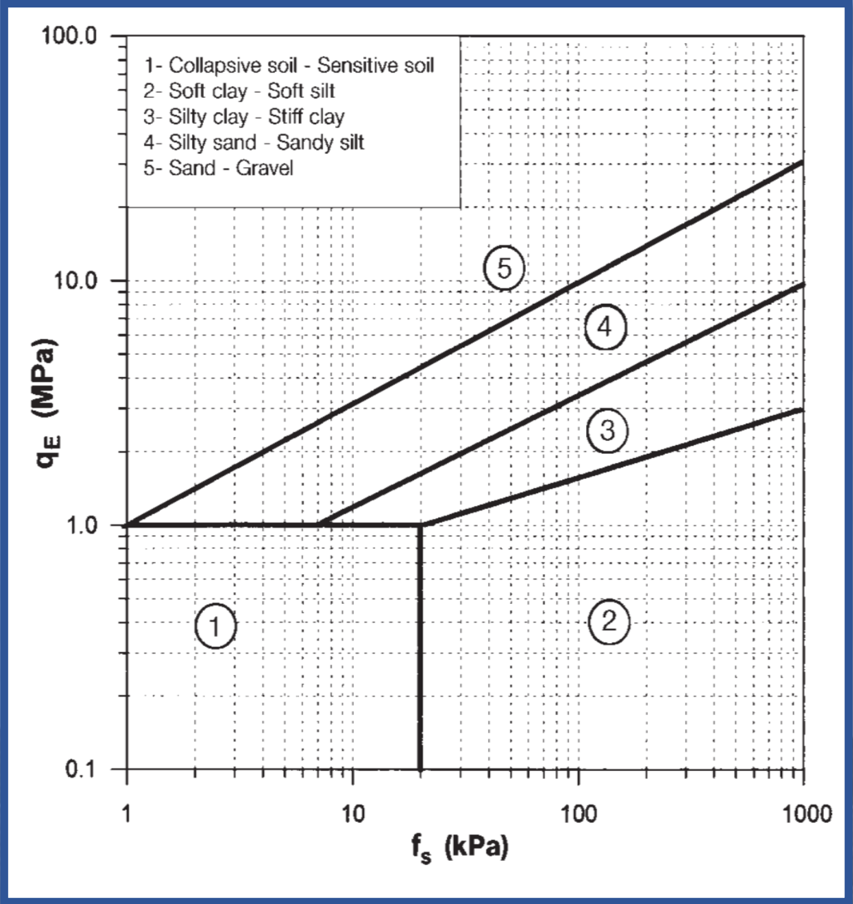
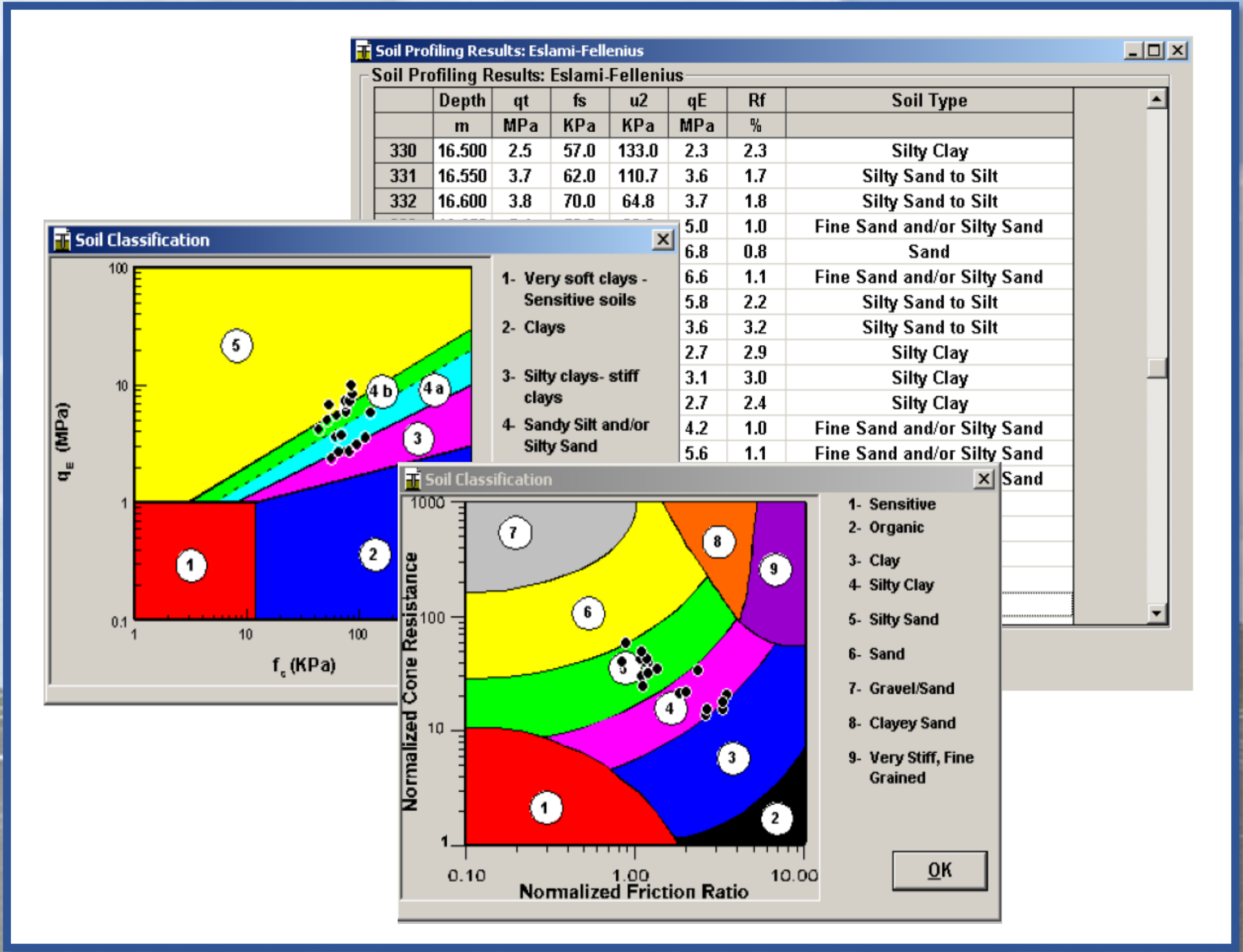


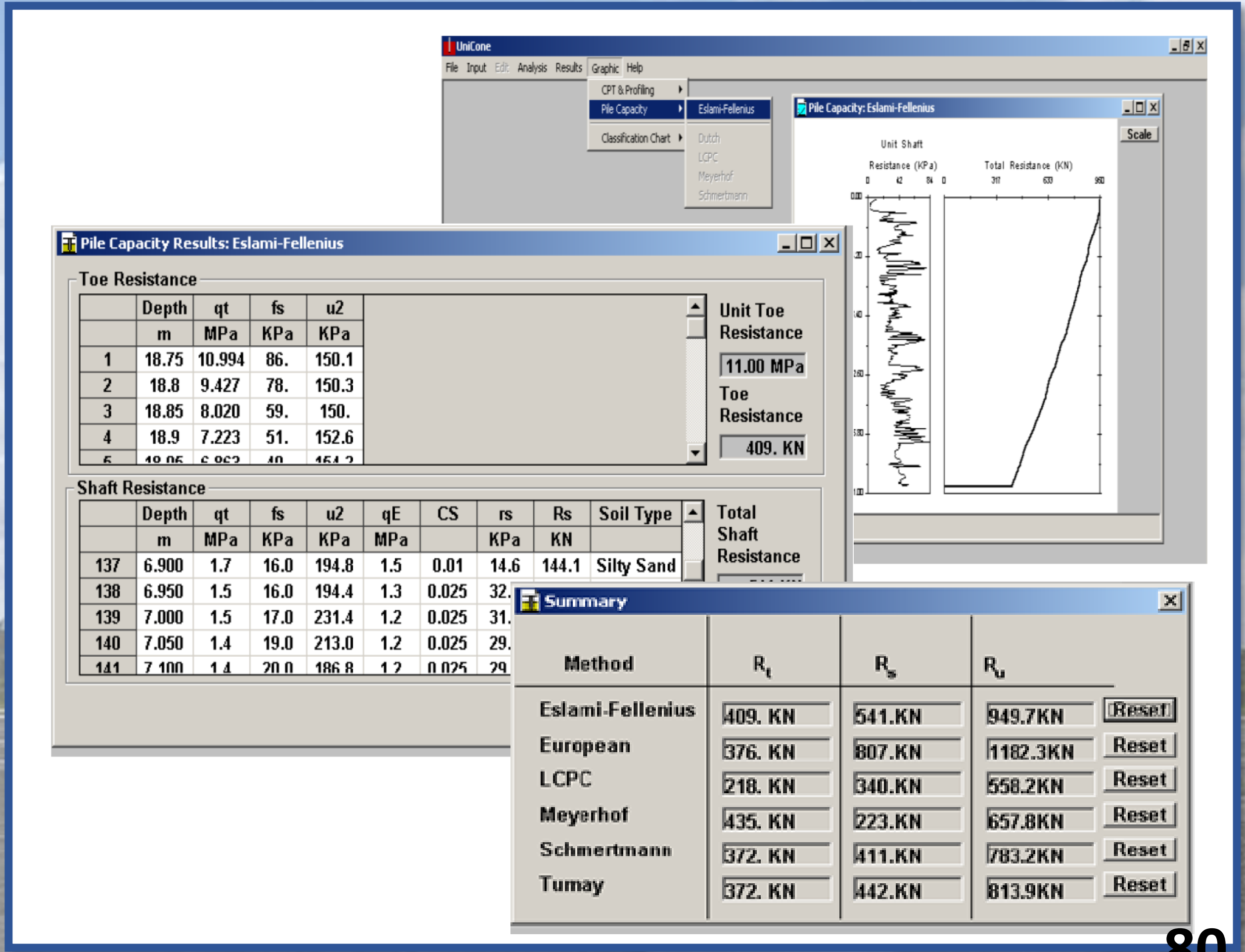
Chart for soil classification (Eslami & Fellenius, 1997)

Case Study No.9: UniCone (Fellenius, Infante and Eslami, 2002)

Pile Capacity Calculation



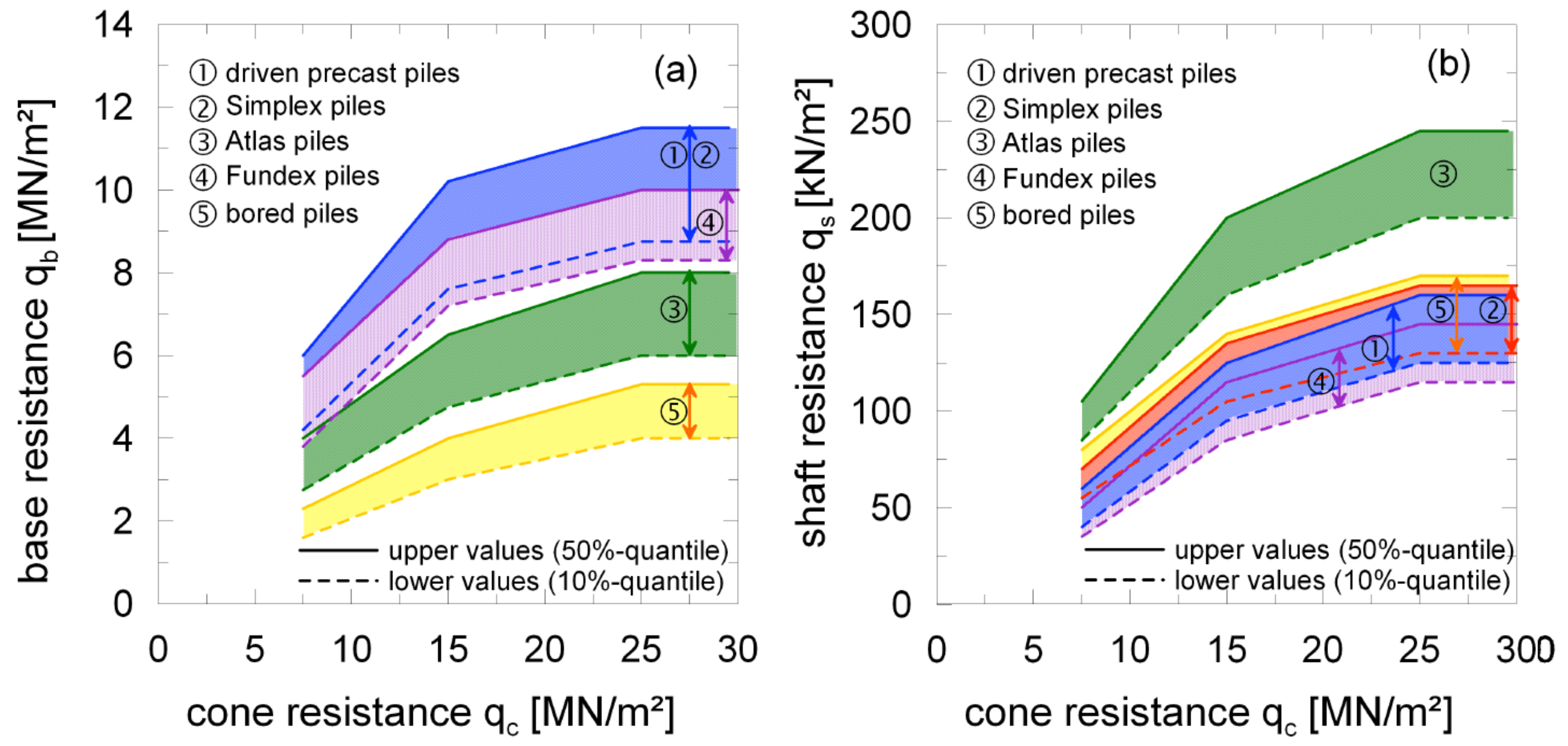
Soil Profiling



7.2. Direct Application of CPT Records for Bearing Capacity

- **Deep Foundations**

German Method
Kempfert & Becker (2010)



1000 case records

8. Case Studies

Case Study No.10: Heidarie, Jamshidi & Eslami (2019)

GEORISK
https://doi.org/10.1080/17499518.2019.1628281

Taylor & Francis
Taylor & Francis Group

Reliability based assessment of axial pile bearing capacity: static analysis, SPT and CPT-based methods

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ABSTRACT

Since piles are one of the major geotechnical foundation systems, estimation of their axial bearing capacity is of great importance. Employing different design methods, resulting in a wide range of bearing capacity estimations, complicates the selection of an appropriate design scheme and confirms the existence of model error along with the inherent soil variability in bearing capacity prediction. This paper tends to evaluate different predictive methods in Reliability-Based Design (RBD) framework. In this regard, different static analyses, SPT and CPT-based methods are considered to evaluate which approaches collectively and which method individually, have more reliable predictions for compiled data bank. In order to assess reliability indices and resistance factors, two approaches have been considered, i.e. First Order Second Moment method (FOSM) and First Order Reliability Method (FORM). To investigate the reliability indices for different methods in both RBD approaches, various safety factors and loading ratios have been considered. Also, the Load and Resistance Factor Design (LRFD) resistance factors are calibrated for different target reliability indices and loading ratios. Results show that CPT-based methods are more reliable among other methods. Furthermore, the estimated efficiency ratio, i.e. the ratio of resistance factor to resistance bias factor, confirms this agreement.

ARTICLE HISTORY
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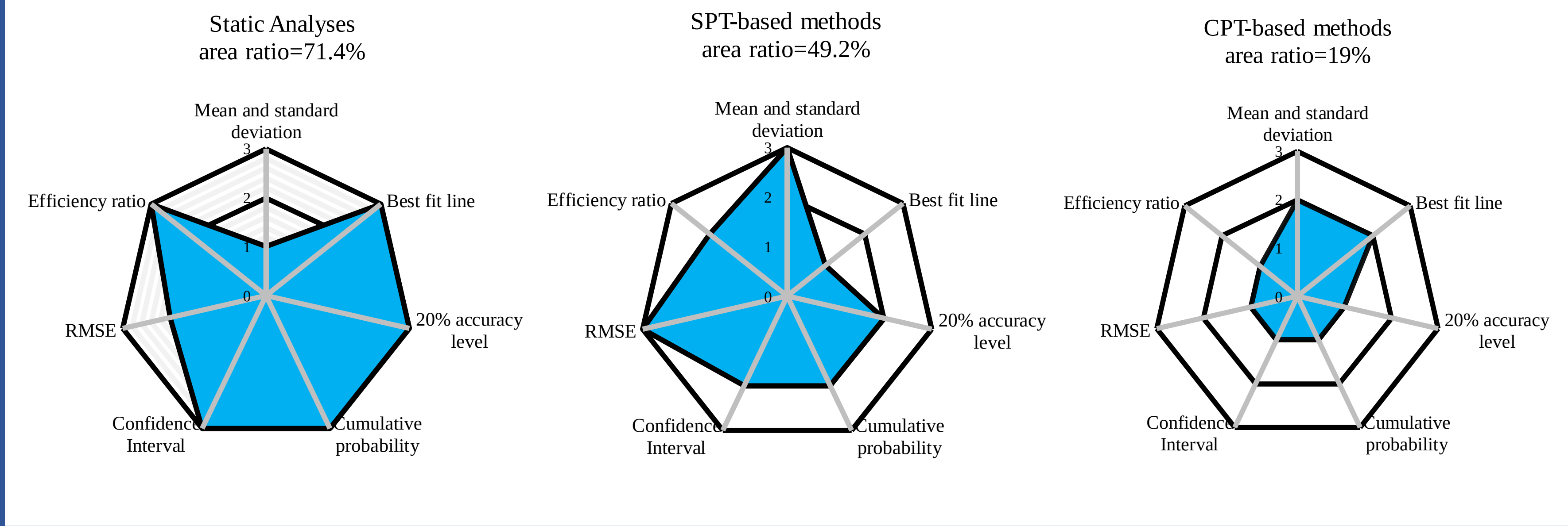
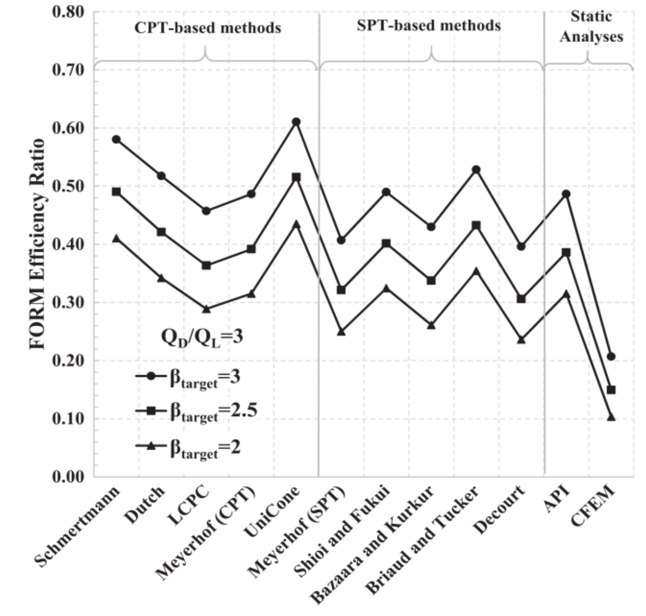
KEYWORDS
Axial pile bearing capacity; CPT; LRFD; pile foundation; reliability based design

Case Study No.10: Heidarie, Jamshidi & Eslami (2019)



Associated error for approaches



Efficiency ratio for each method and different target reliabilities

9. Summary and Conclusions

- **Geotechnical Engineering (GE):**

❖ Team works & interactive
❖ Observational methods & engineering judgement
❖ Dealing with geomaterials & geosynthetics

Less artificial,
more geomaterial

- **Site Investigations (SI):**

❖ Collection & appraisal of data
❖ Recognition subsurface potentials & hazards
❖ Data sources:

  ➢ Site visit, maps & aerial photos
  ➢ Geophysics & remote sensing
  ➢ On situ & in situ tests
  ➢ Sampling, lab tests & physical modeling
  ➢ Full scale tests, instrument & monitoring

In-situ tests;
uncertainty reduction

9. Summary and Conclusions

- **Foundation Engineering (FE):**

❖ Knowledge-based & multidisciplinary
❖ Realized as artistic rather than routine
❖ Iterative practice in analysis & design

In-situ tests in FE more pronounced than laboratory tests

- **Cone & Piezocone Penetration Tests (CPT, CPTu):**

❖ Accurate & reliable data
❖ Simple, fast & economical
❖ Continuous records with depth

CPT & CPTu (q_t, f_s, u_2); fast, continuous & providing tons of data

- **Major Applications of CPT in GE:**

❖ Soil behavior classification & profiling
❖ Estimating soil engineering parameters
❖ Identification & modification of problematic deposits
❖ Foundation engineering

CPT; versatile tool for soft to medium, compressible & problematic deposits

9. Summary and Conclusions

- **CPT and FE: Scale Effects**

❖ Embedment & diameter
❖ Influence zone & data processing
❖ Penetration rate & failure mechanism
❖ Ultimate capacity interpretation & strain level

CPT; model Pile & source of relevant records

- **CPT and FE: Design**

❖ Construction & installation procedure
❖ Direct & indirect approaches for bearing capacity
❖ Settlement & load-displacement estimation
❖ Pile capacity: commonly used 25 direct methods

CPT; towards reliable foundation design

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