GEOTECHNICAL Frontiers

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CPT & CPTu Application for Deep Foundations Geotechnical Design; Data-Based Approach

Short Course

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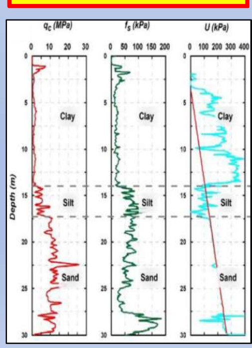
Short Course Outline

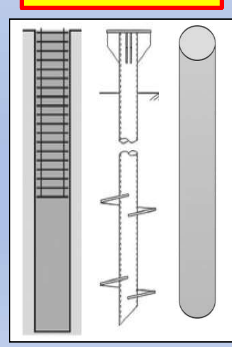
1. Background to CPT & CPTu Applications (Geotechnical & Foundation Engineering)
2. CPT-Based Methods for Deep Foundations (Demo Examples & Cases)
3. Cases: Histories & Studies
4. Summary & Conclusions

Aims, Scope & Objectives

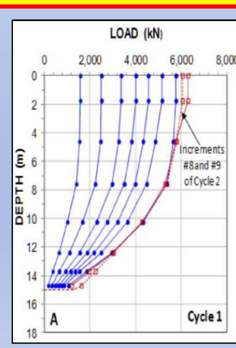
CPTu Log & Data



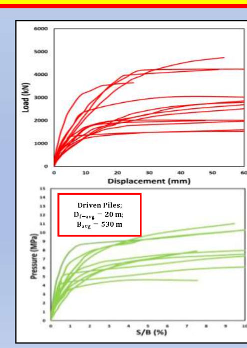
Various Piles



Resistance Distribution



Load-Displacement



Short Course Outline

1. **Background to CPT & CPTu Applications (Geotechnical & Foundation Engineering)**
2. **CPT-Based Methods for Deep Foundations (Demo Examples & Cases)**
3. **Cases: Histories & Studies**
4. **Summary & Conclusions**

Topic Subtitles

I. Geotechnical Engineering & Site Investigations

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

III. Applications of CPT & CPTu in Geotechnical Engineering

IV. Applications of CPT & CPTu in Foundation Engineering

V. Deep Foundations: Geotechnical Design

Data Sources

(1) Maps

(2) Aerial Photos

(3) Site Visit

(4) Non Destructive Tests

(5) Remote Sensing

(6) On-Situ Testing

(7) In-situ Penetration Testing

(8) Boring and Sampling

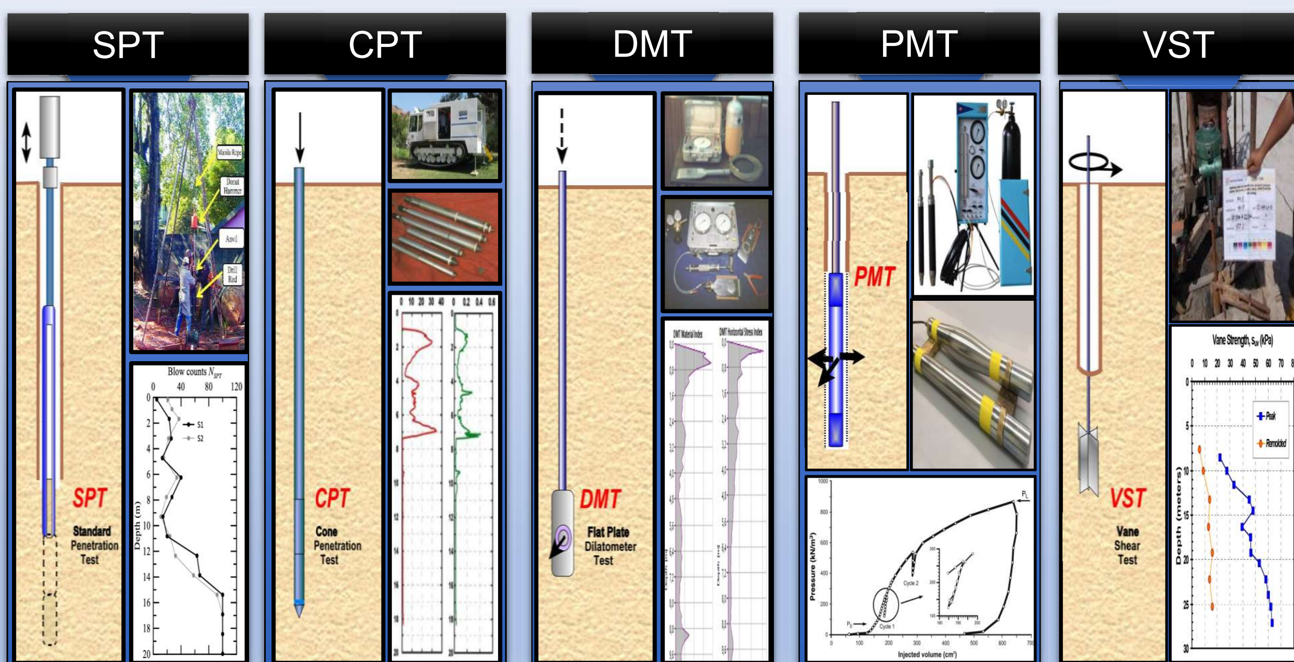
(9) Laboratory Testing

(10) Physical Modeling

(11) Full-scale Tests

(12) Instrumentation & Monitoring

Major Approaches: In Situ Penetration Tests



N | q_c, f_s & u_2 | K_D, I_D & E_D | P_L & P_0 | S_u

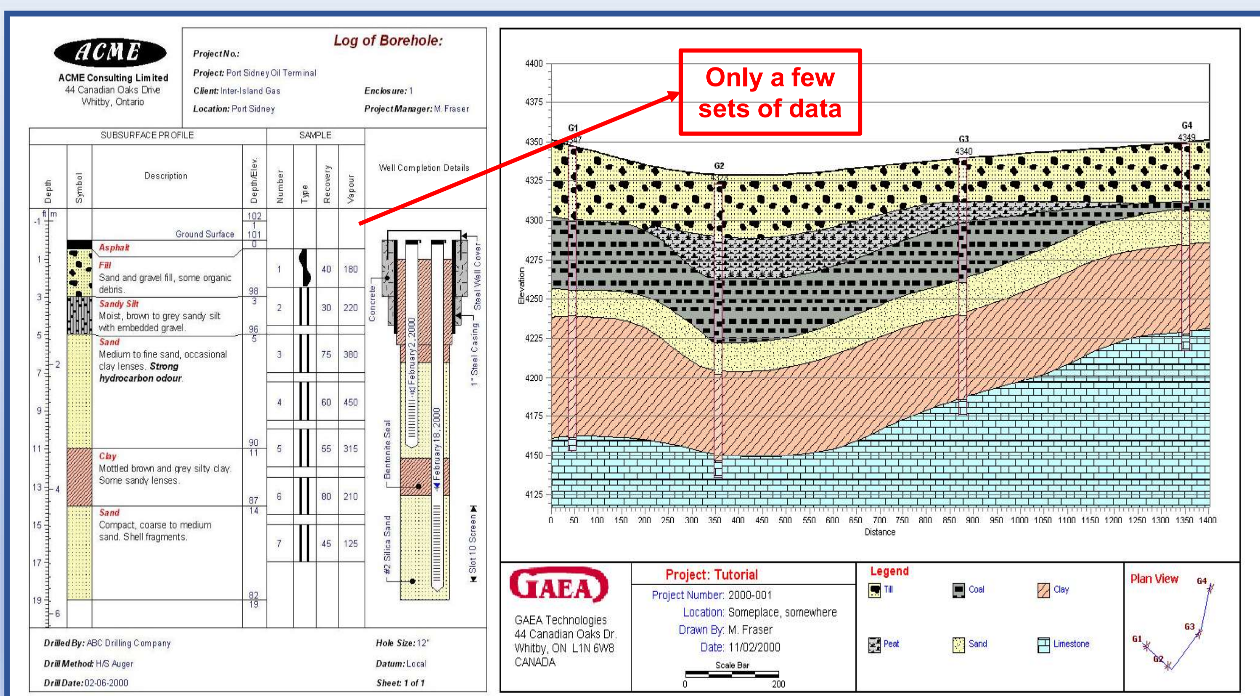
In-Situ Tests and Their Applicability

(Lunne et al., 1997)

Group	Device	Soil type	Profile	u	*φ'	S_u	ID	m_v	c_v	k	G_0	σ_h	OCR	σ-ε	Hard rock	Soft rock	Gravel	Sand	Silt	Clay	Peat
		Soil parameters													**Ground type**						
Penetro meters	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
	Peizocone (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
Pressure meters	Prebored (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 displace ment (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.
Soil parameter definitions: *u*, in situ static pore pressure; ϕ', effective internal friction angle; S_u, undrained shear strength; *ID*, density index; *mv*, constrained modulus; c_v, coefficient of consolidation; *k*, coefficient of permeability; *G0*, shear modulus at small strains; σ_h, horizontal stress; *OCR*, overconsolidation ratio; σ-ε, stress—strain relationship.

Typical Subsurface Log & Profile: Conventional Approach



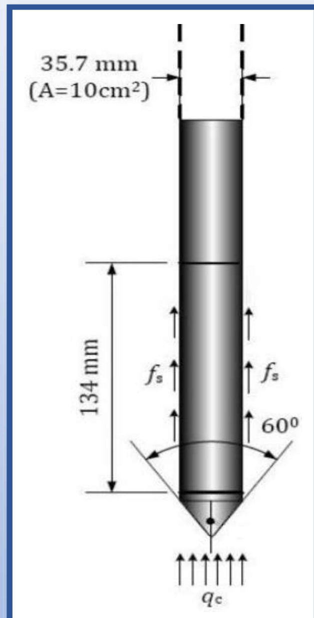
Why In-Situ Testing?

Laboratory Tests Limitations	Field Tests Advantages
Difficulties for undisturbed sampling	Overcome sampling difficulties
Soil disturbance & maintenance	Minimum changes in stress state
Soil volume change	Simple and fast
Omitting confinement pressure	Economical
Size effect and boundaries	Dominant applications in FE

CPT Device

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

- **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

Cone Penetrometer (CPTu) Probes and Terminology

- ASTM D 5778 procedures
- No boring, No samples, No spoil
- Hydraulic Push at 20 mm/s
- Range of sizes:10 cm² and 15 cm² probes

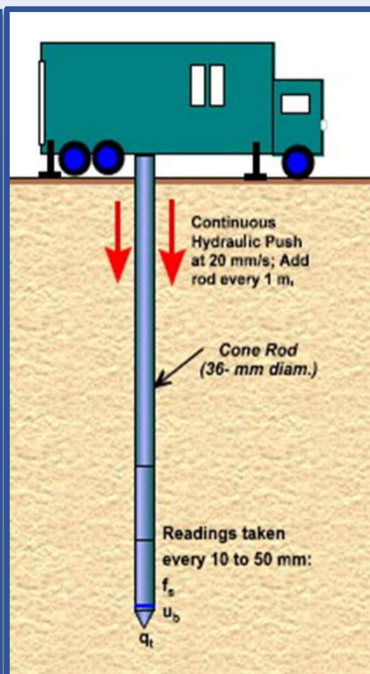
Advantages:

- Fast and continuous profiling
- Repeatable and reliable
- Continuous records of q_c, f_s, u per 2.5 cm
- Strong theoretical basis for interpretation

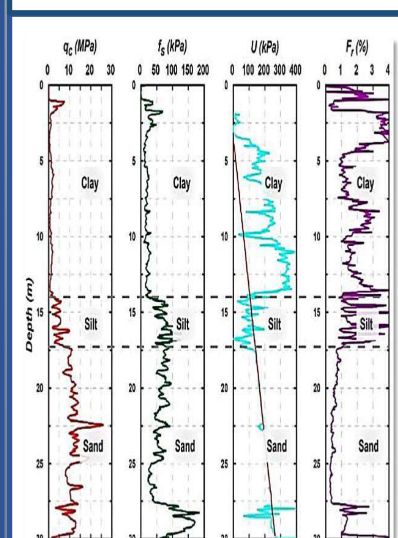
Disadvantages:

- High capital investment
- Requires skilled operators
- Limitation of use in gravel or cemented soils

Equipment & Procedure



Penetration rate: 20 mm/s
Set of data: per 25 mm or 1 inch



Data & Graphical Presentation

1. Measured Parameters

q_c, f_s, u

2. Corrected Parameters

- Corrected tip resistance:

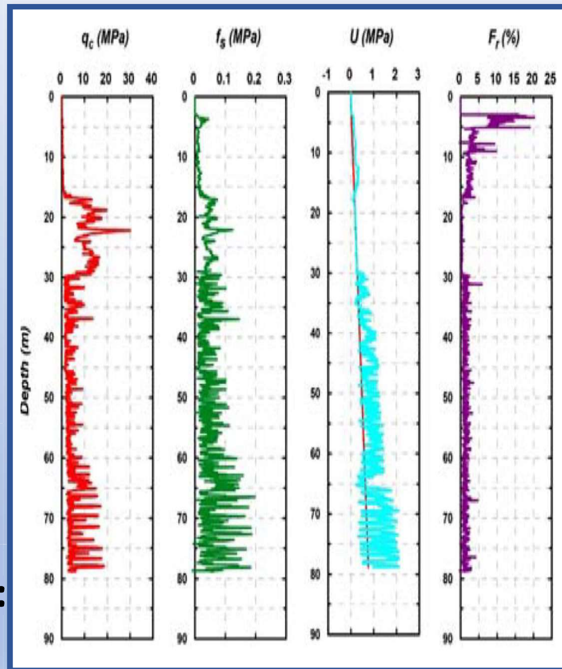
$$q_t = q_c + u_2(1 - a)$$

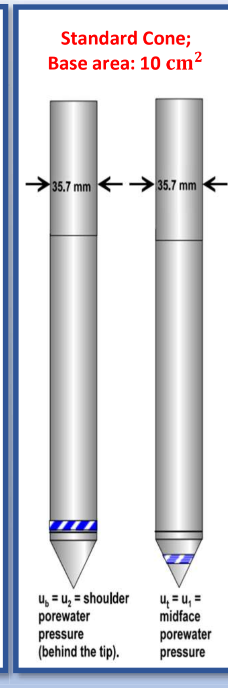
- **Friction ratio:**

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

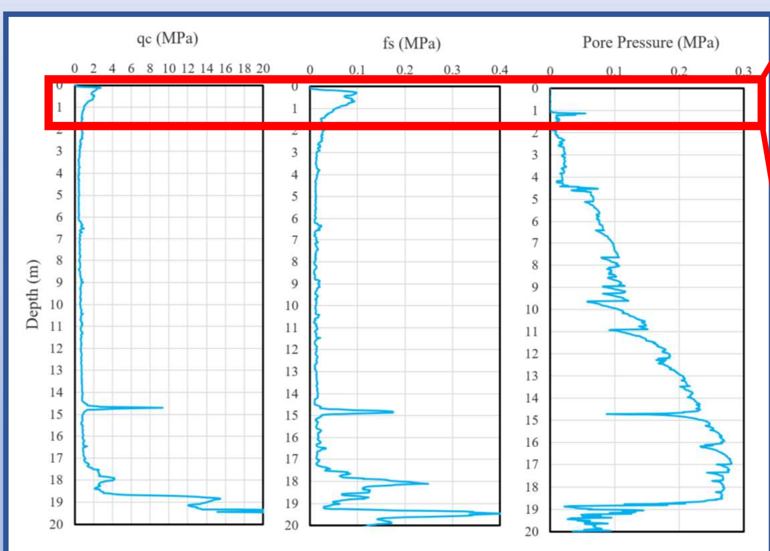
- **Pore pressure coefficient:**

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





Data & Graphical Presentation

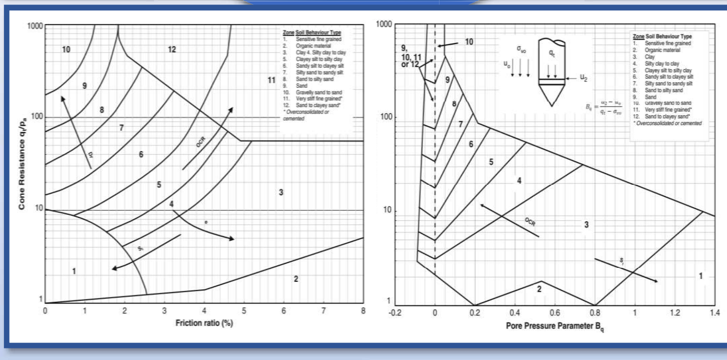


Tons of Data in 1 Meter !!!

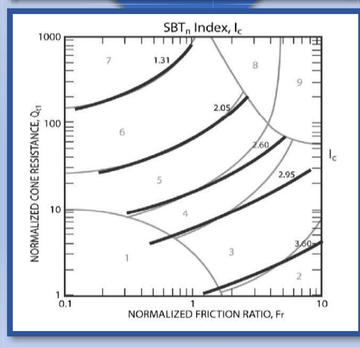
z	qc	fs	u2
m	MPa	MPa	MPa
0	0	0	0
0.02	0.06	-0.0003	0
0.04	0.24	-0.0003	0
0.06	0.16	0.001	-0.0005
0.08	0.71	0	-0.0005
0.1	2.47	0.0004	-0.001
0.12	2.79	0.0147	-0.001
0.14	2.48	0.0215	-0.001
0.16	2.31	0.0386	-0.001
0.18	2.32	0.0479	-0.001
0.2	2.27	0.0639	-0.001
0.22	2.27	0.0759	-0.0005
0.24	2.22	0.0878	-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 Behavior Classification and Profiling

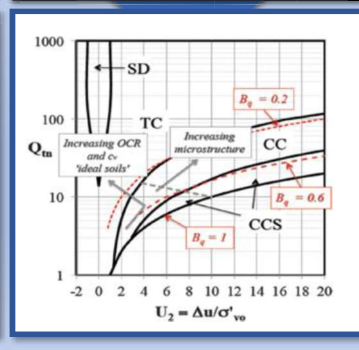
Robertson et al. (1986)



Robertson (2010)

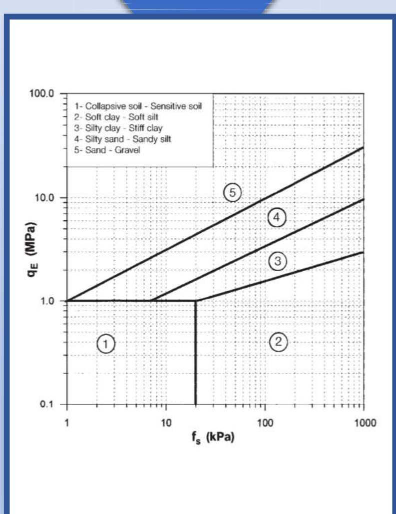


Robertson (2016)

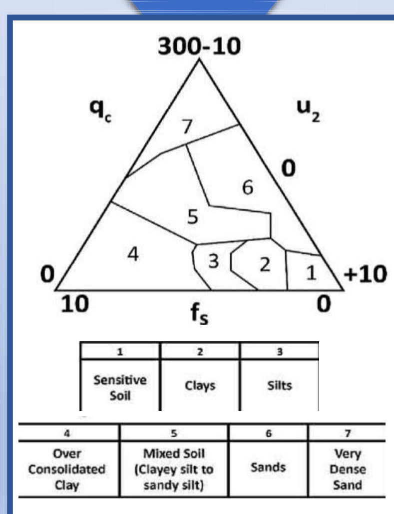


Soil Behavior Classification and Profiling

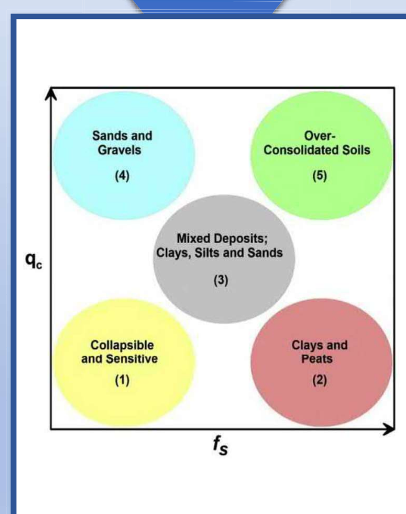
Eslami and Fellenius (1997)



Eslami et al. (2016 & 2022)



Eslami et al. (2018)



Estimating Soil Engineering Parameters

- ❖ 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

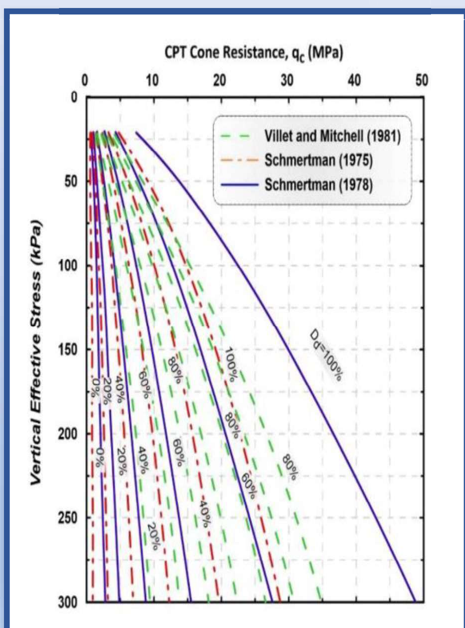
RHEOLOGICAL
- Strain rate, $\delta\varepsilon/\delta t$
- Time since consolidation (T)
- Secondary compression, $C_{\alpha\varepsilon}$
- Creep rate, α_R
- Time to failure, t_f

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

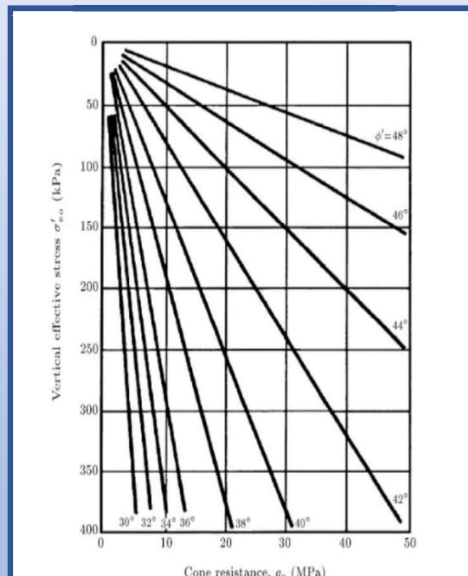
STRENGTH
- Drained and Undrained, τ_{max}
- Peak (s_u, c', ϕ')
- Post-peak, τ'
- Remolded strength
- Softened or critical state, $s_{u\,(rem)}$
- Residual (c_r', ϕ_r')
- Cyclic Behavior (τ_{cyc}/σ_{vo}')

Estimating Soil Engineering Parameters

Relative Density (D_r)



Friction Angle (φ)



(Mitchell & Durgunoglu, 1983)

Estimating Soil Engineering Parameters (Eslami et al., 2020)

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	$E_s = (3-8)q_c$

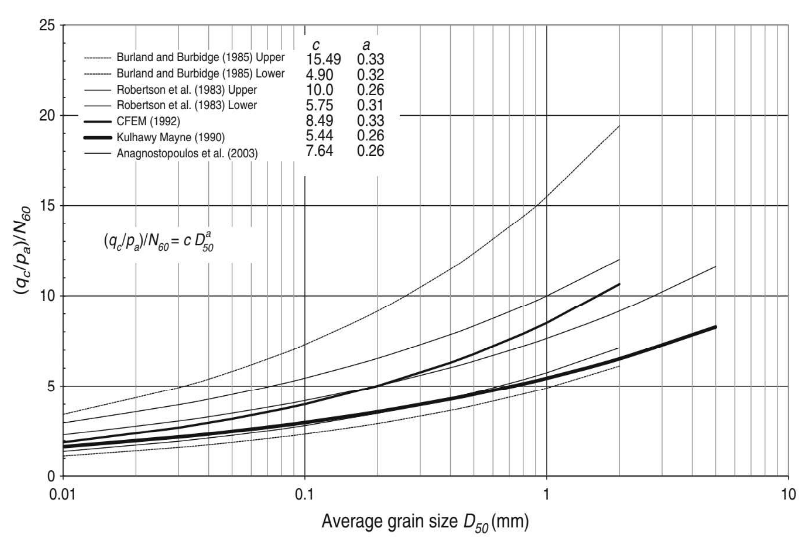
Undrained Shear Strength (S_u)

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

Estimating Soil Engineering Parameters (Eslami et al., 2020)

CPT (q_c) correlations with SPT (N)



(Adopted from Robertson & Campanella, 1983)

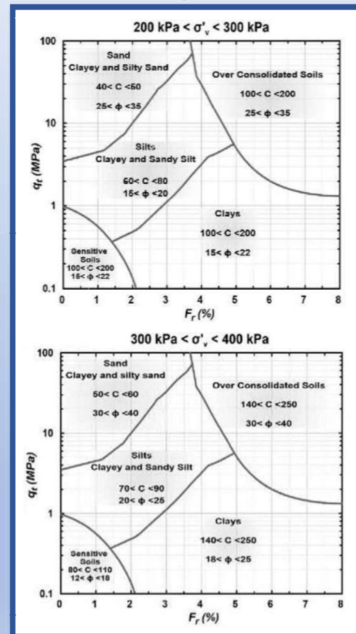
Estimating Soil Engineering Parameters

- **Eslami & Mohammadi (2016)**

q_c, f_s, u_2 → **C' , ϕ'**

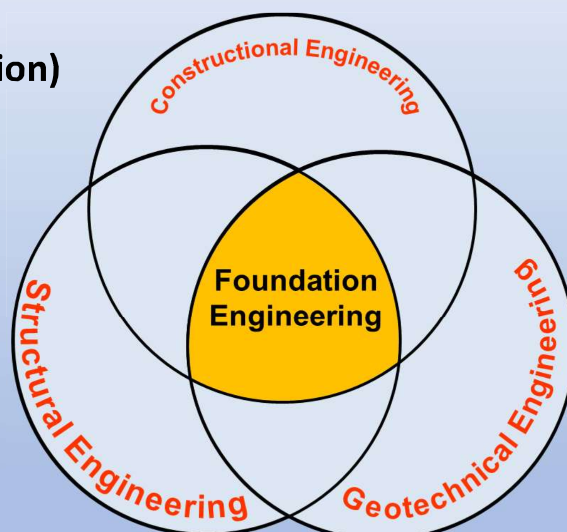
$$\begin{cases} C + 0.000789(1-\sin\phi)\sigma'_{v_0}\tan\left(\frac{2}{3}\phi\right)\left[\dfrac{q_c-\left(\dfrac{\sigma_{v_0}-2\sigma_{h_0}}{3}\right)}{\left(\dfrac{\sigma'_{v_0}-2\sigma'_{h_0}}{3}\right)}\right]^{1.44} = f_s \\ \left(\tan^2\left(\frac{\pi}{4}+\frac{\phi}{2}\right)e^{\pi\tan\phi}-1\right)C\cot\phi+\bar{q}.\tan^2\left(\frac{\pi}{4}+\frac{\phi}{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

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



Major Analysis & Design Requirements

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

Foundations Classification

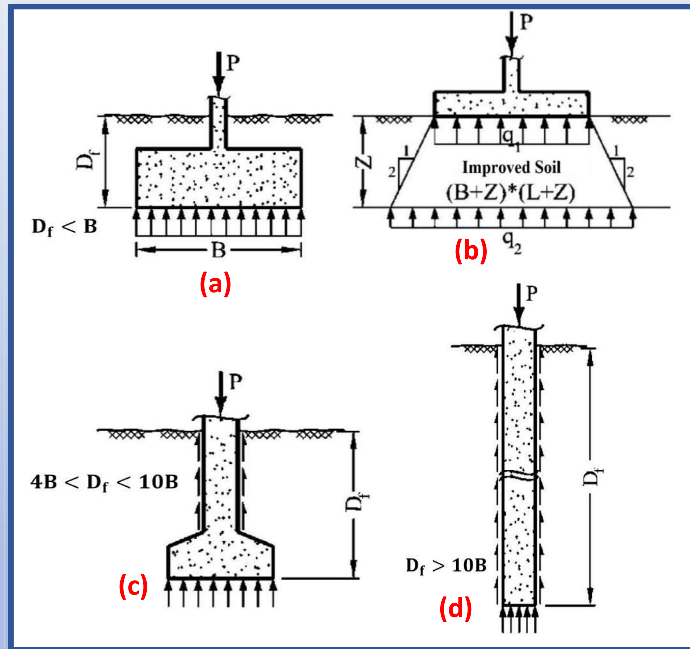
- **Embedment Depth**

✓ Shallow Foundations (a)

✓ Shallow + Soil Improvement (b)

✓ Semi-deep Foundations (c)

✓ Deep Foundations (d)



Current categories of foundations
(Eslami et al., 2019)

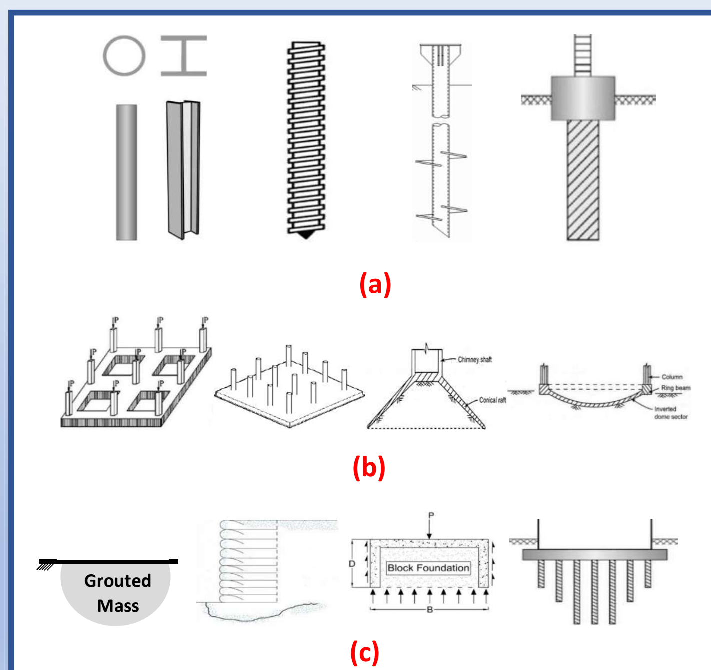
Foundations Classification

- **Form & Function**

✓ Linear (1D) Foundations (a)

✓ Planar (2D) Foundations (b)

✓ Volumetric (3D) Foundations (c)



(Eslami & Ebrahimipour, 2023)

Foundations Classification

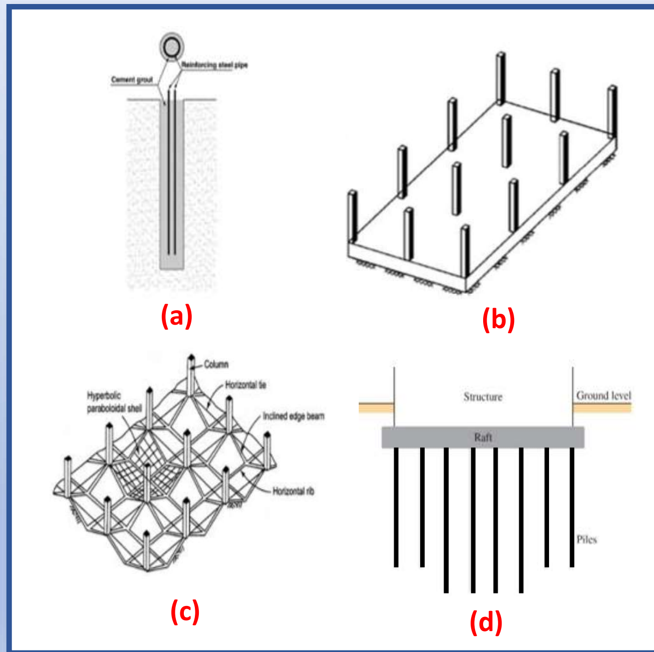
- **Load Transfer System**

✓ **Vector-act Foundations (a)**

✓ **Section-act Foundations (b)**

✓ **Surface-act Foundations (c)**

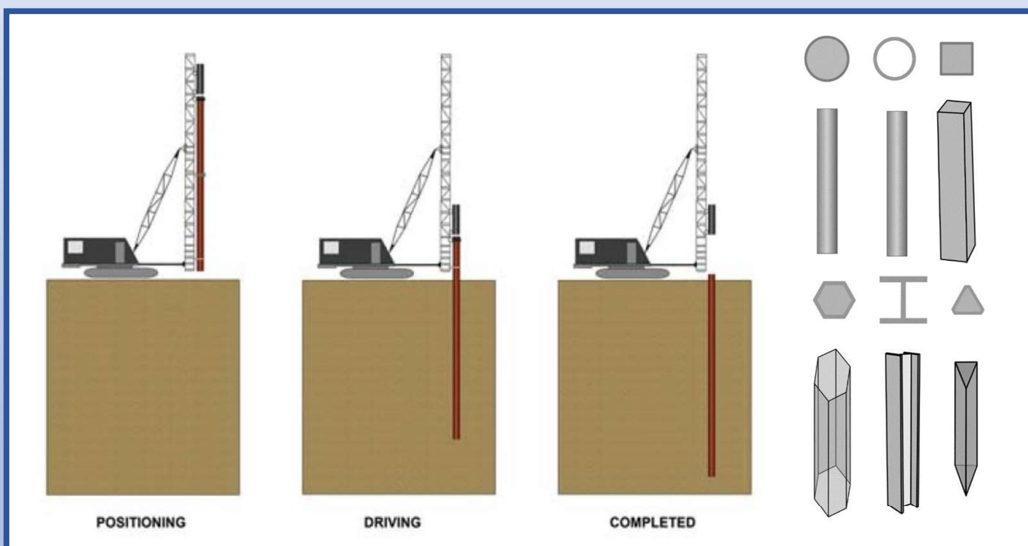
✓ **Block-act (Hybrid) Foundations (d)**



(Eslami & Ebrahimipour, 2023)

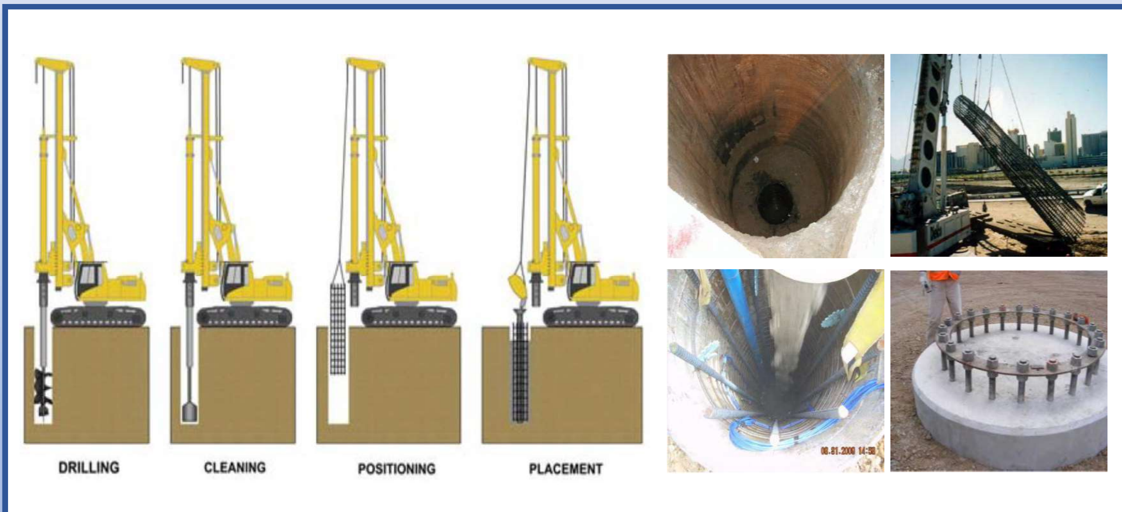
Different Types of Deep Foundations

- **Driven Piles**



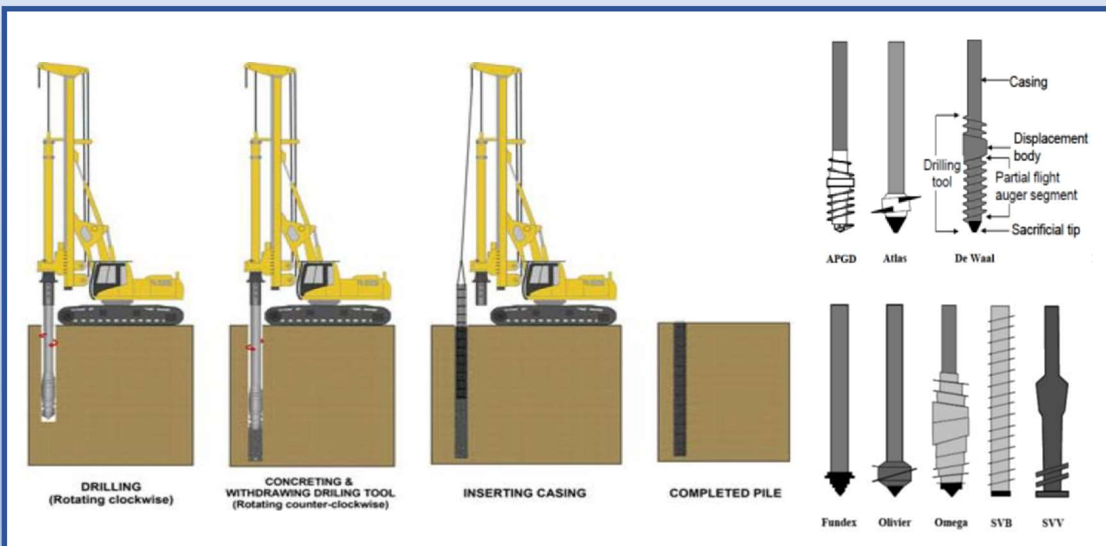
Different Types of Deep Foundations

- **Drilled Shafts**



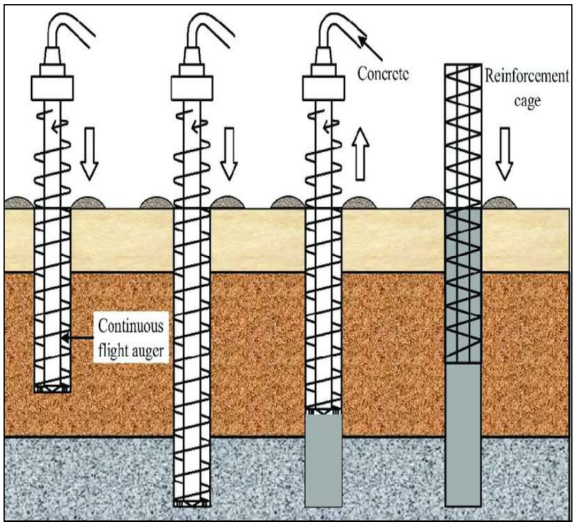
Different Types of Deep Foundations

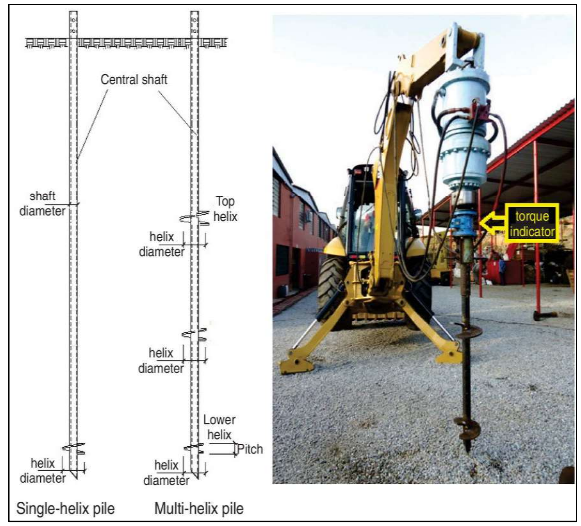
- **Drilled Displacement Piles (DDP)**



Different Types of Deep Foundations

- **Special Piles: Continuous Flight Auger (CFA) & Helical Piles**





Schematic of CFA pile installation **Helical piles installation**

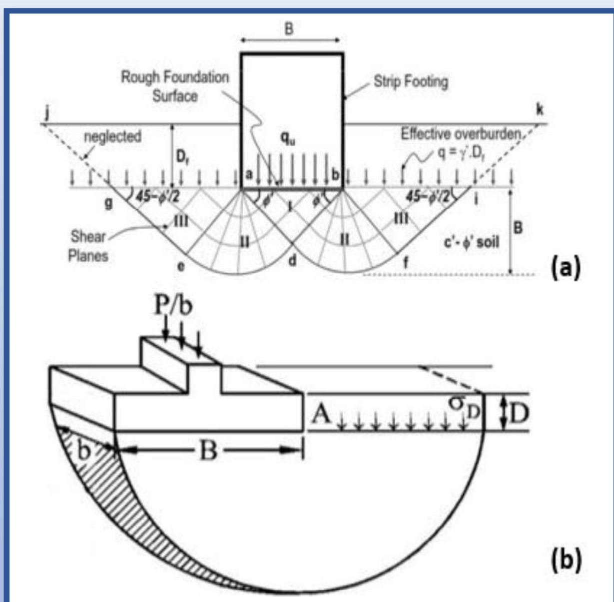
Necessity & Requirements of Deep Foundations Construction

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

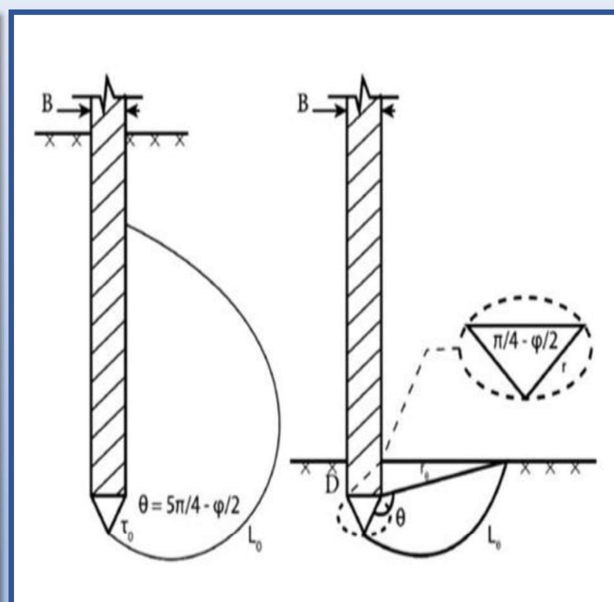
Necessity & Requirements of Deep Foundations Construction

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

Shallow Foundations: Direct Application for Bearing Capacity & Settlement



Shear failure zone, a) drained condition, b) undrained condition (Terzaghi, 1943)



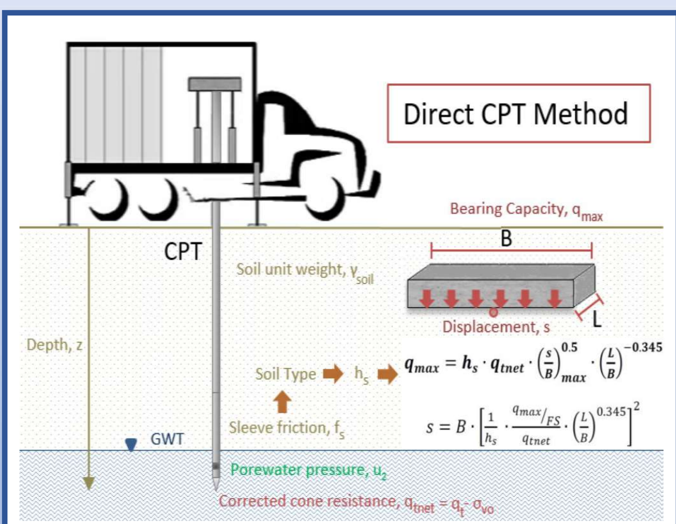
Comparison of rupture surface length for shallow and deep conditions

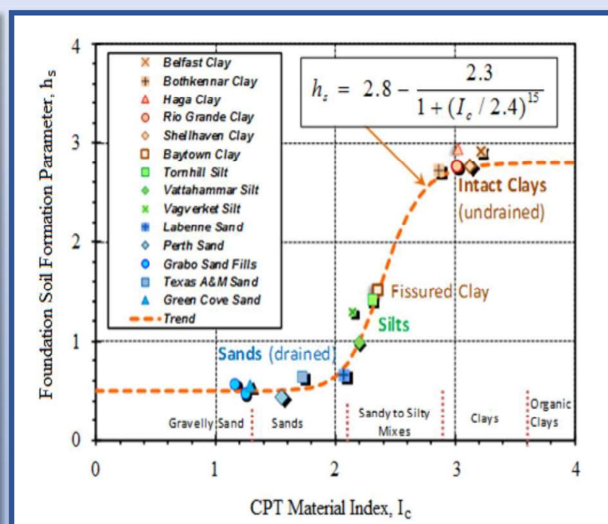
Shallow Foundations: Direct Application for Bearing Capacity & Settlement

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)	$q_{ult} = \bar{\alpha} \times \bar{q}_{cg}$ $\varphi = \frac{\log\left(\frac{\bar{q}_c}{\gamma z}\right) + 0.5095}{0.0915}$	$\bar{q}_{c,g}$= geometric average of q_c values from footing base to 2B beneath footing depth.

Shallow Foundations: Direct Application for Bearing Capacity & Settlement

- **Minnesota CPT Design Guide (2018)**



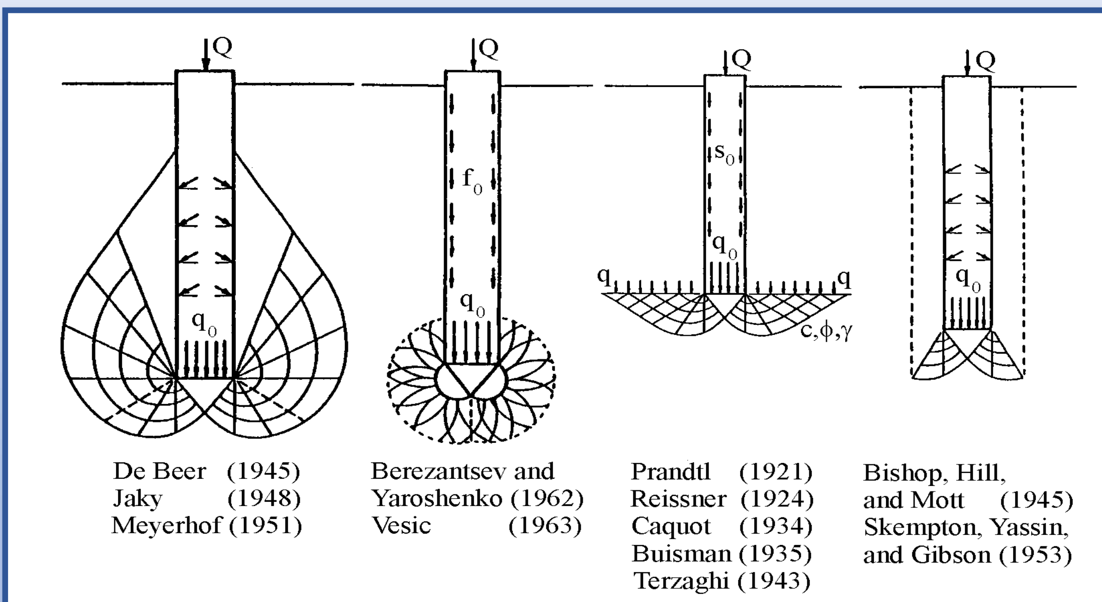


Foundation soil formation parameter h_s versus CPT material index, I_c (Mayne, 2017)

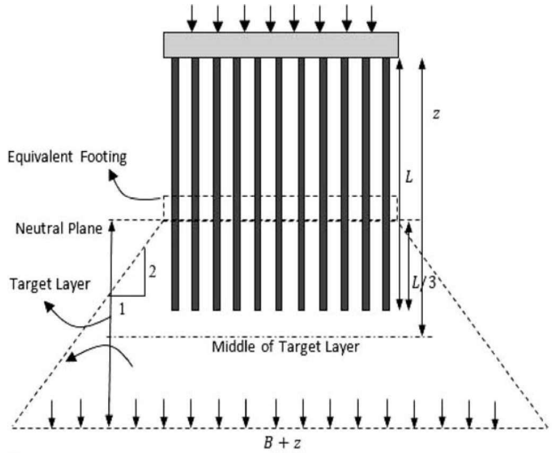
Geotechnical Design Aspects

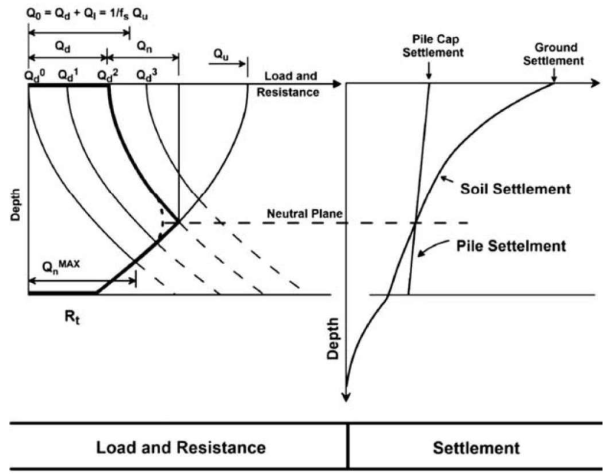
1. **Installation Method & Location**
2. **Bearing Capacity**
3. **Resistance Distribution**
4. **Settlement**
5. **Load - Displacement**

Failure Mechanisms for Bearing Capacity



Settlement & Resistance Distribution





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

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

Direct Application for Settlement & Load-Displacement

- **Valikhah & Eslami (2019)**

$$\Delta H = \left(\frac{1}{mj}\left[\left(\frac{\sigma'_0+\Delta\sigma'}{\sigma'_r}\right)^j - \left(\frac{\sigma'_0}{\sigma'_r}\right)^j\right]\right)\times H$$

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

b: penetration cone diameter

B: foundation width

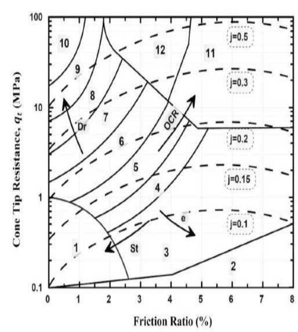
(b and B are in m and q_c is in kPa)

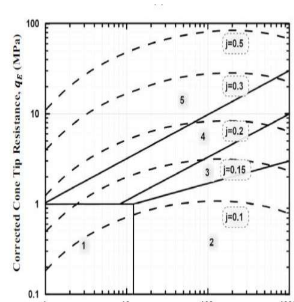
$j = \frac{q_c}{x+yq_c}$

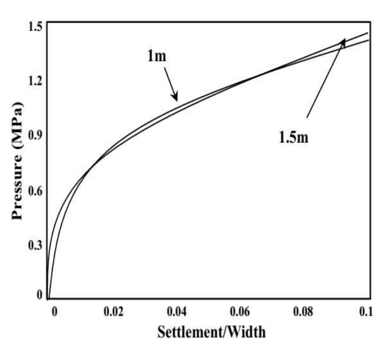
$x = 0.02R_f + 0.5$

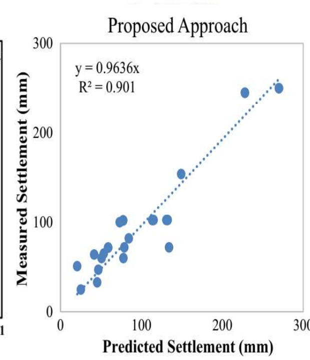
$y = 7.53(\sigma'_0)^{-0.25}$

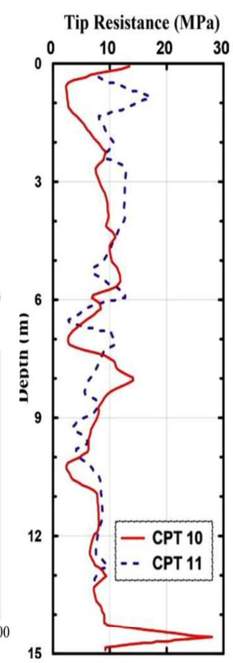
(q_c and σ'_0 are in kPa)



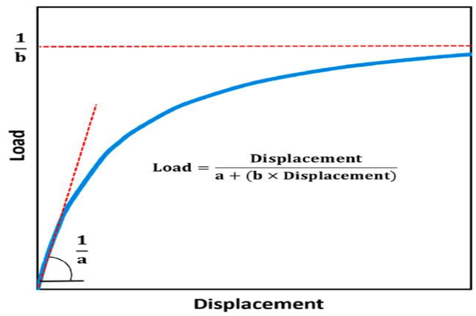




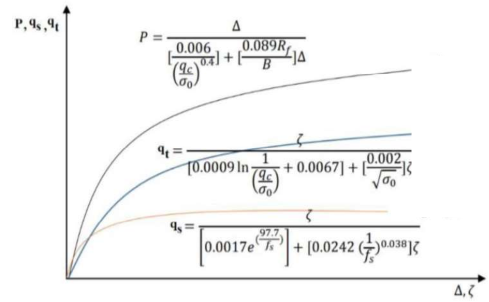




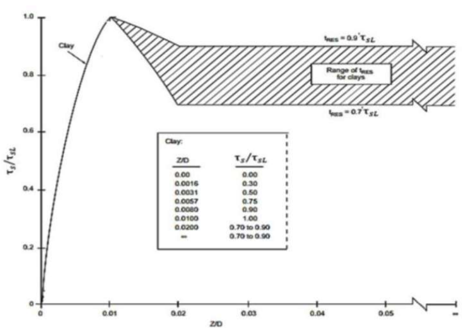
Different Approaches for Load-Displacement Behavior



Mathmatical Hyperbolic function

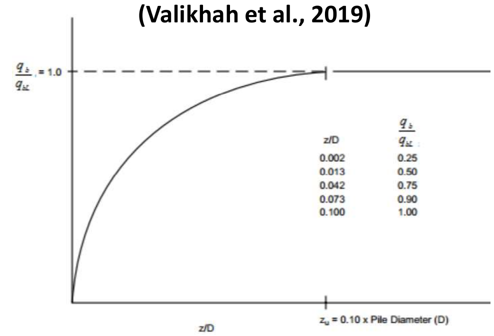


load-displacement based on CPTu records (Valikhah et al., 2019)



Z/D	τ_z/τ_{SL}
0.00	0.00
0.0016	0.30
0.0031	0.50
0.0057	0.75
0.0080	0.90
0.0100	1.00
0.0200	0.70 to 0.90
∞	0.70 to 0.90

t-z curve for cohesive soils (API, 2014)

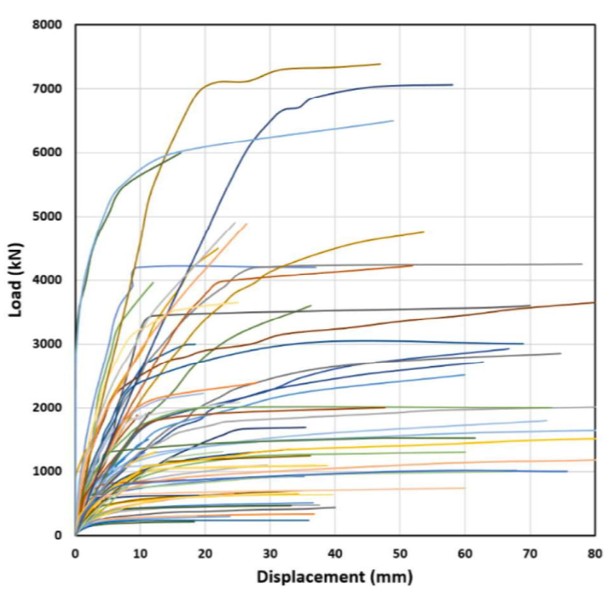


z/D	q_b/q_{sc}
0.002	0.25
0.013	0.50
0.042	0.75
0.073	0.90
0.100	1.00

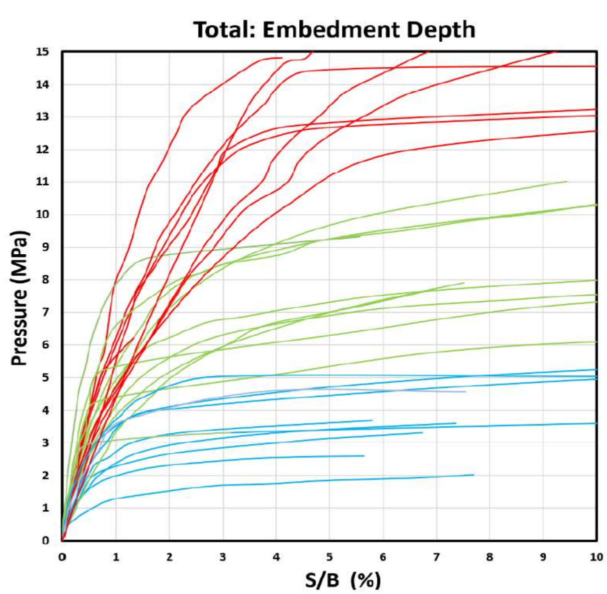
q-z curve for cohesive and non-cohesive soils (API, 2014)

Load-Displacement Behavior of Driven Piles

- 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



Load – Displacement for all piles



Pressure – S/B for all piles records

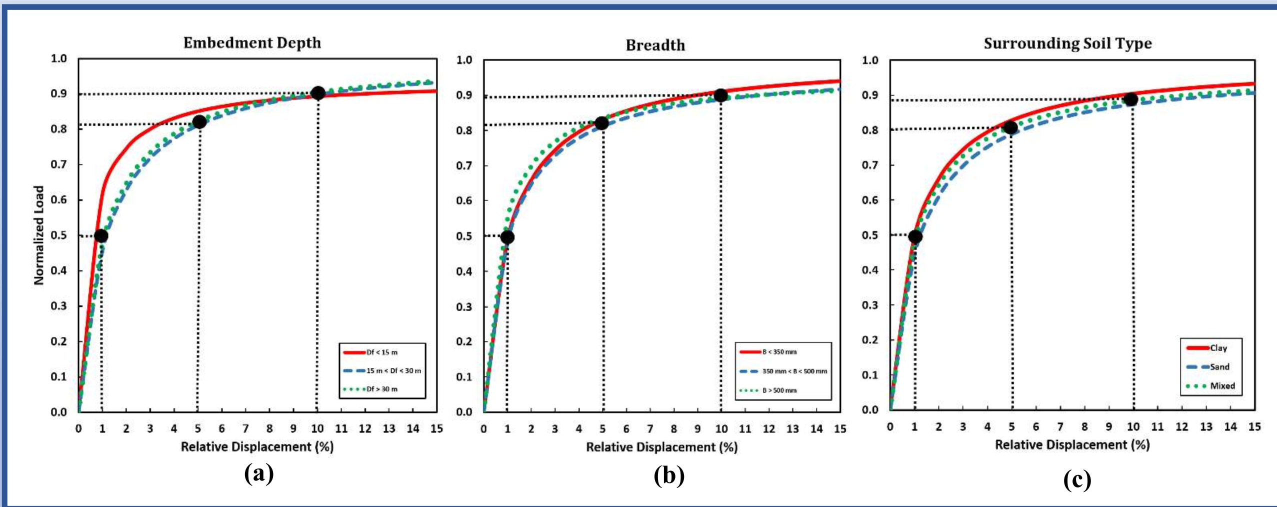
Load-Displacement Behavior of Driven Piles

Normalization Approach:

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

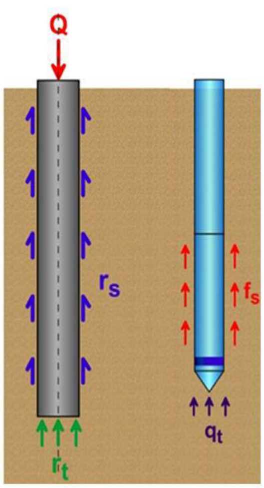
Relative Displacement & Normalized Load:

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



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

CPT & Pile Analogy

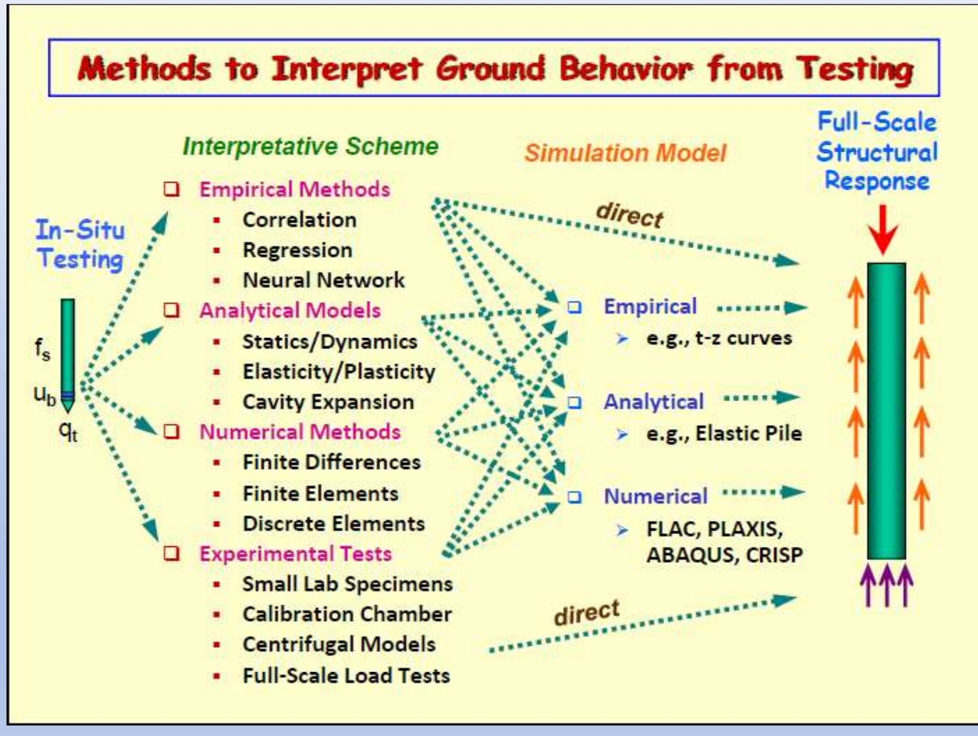


Indirect Approach q_c & f_s → C, φ → r_t, r_s

Direct Approach
- q_c → r_t
- f_s → r_s

Penetrometers can be realized as a *model pile*

CPT Approaches for Deep Foundations



Mayne (2009)

Pile Bearing Capacity

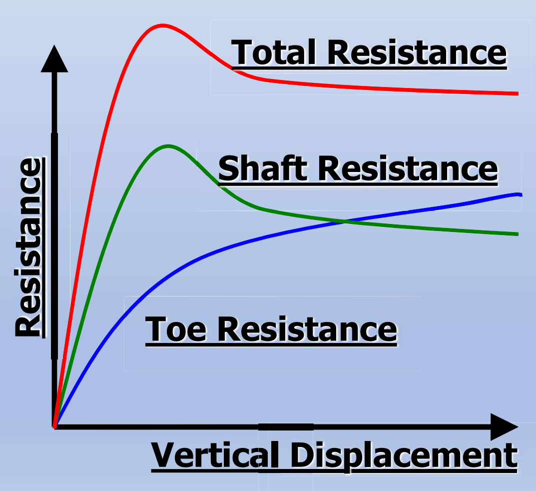
Ultimate Bearing Capacity = **Toe Capacity** + **Shaft Capacity**

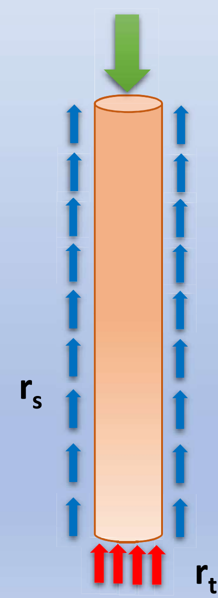
$$R_t = r_t . A_t$$

$$R_s = r_s . A_s . D_f$$

$$R_u = R_t + R_s$$

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





Static Analysis – Indirect Approach

Toe Resistance

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

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

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

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

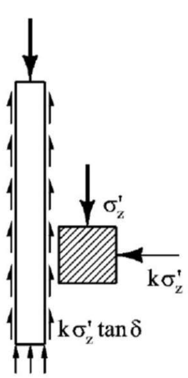
Static Analysis – Indirect Approach

Shaft Resistance

Effective stress analysis (ESA)

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

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



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

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

Wei Dong Guo (2012)

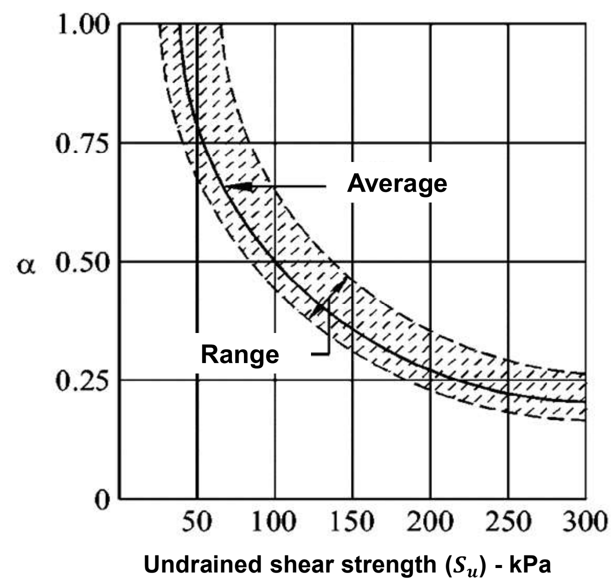
Static Analysis – Indirect Approach

Shaft Resistance

Total stress analysis (TSA)

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

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



Static Analysis – Indirect Approach

Unified Pile Design (CFEM)

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

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

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

The values of β and N_t are as given in Table

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

Static Analysis – Indirect Approach

API (2011)

For cohesive soils

$$r_t = 9S_u$$

$$r_s = \alpha S_u$$

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

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

For cohesionless soils

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

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

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

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

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

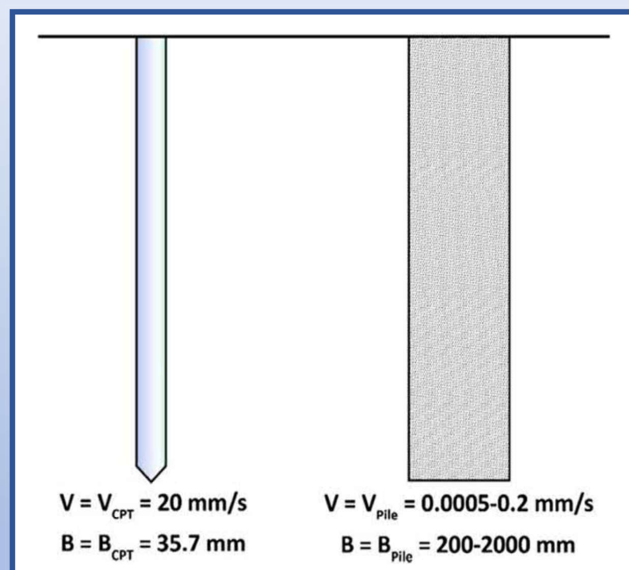
b: Sand-silt includes those soils with significant fractions of both sand and silt. Strength values generally increase with increasing sand fractions and decrease with increasing silt fractions.

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

Scale Effect Correlations

- **Determinant Factors for Toe Capacity**

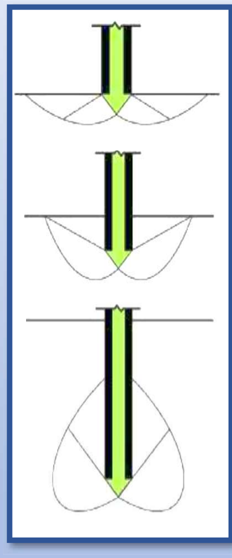
1. Embedment depth
2. Influence zone
3. Data production processing and averaging
4. Diameter
5. Nonhomogeneous condition
6. Penetration rate and failure mechanism
7. Ultimate capacity interpretation



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

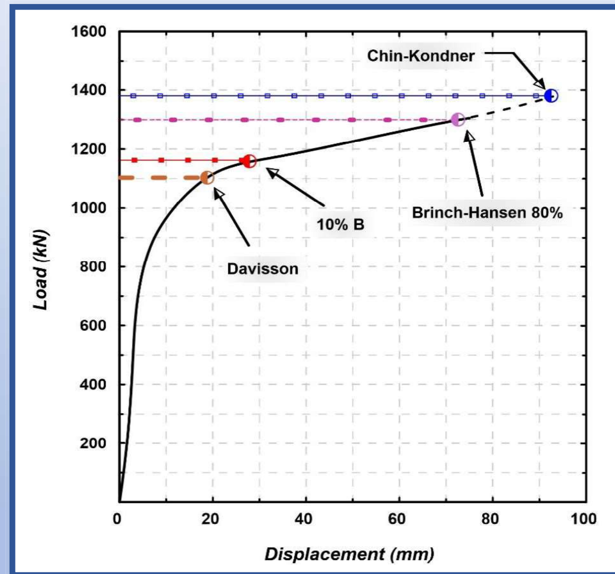
Scale Effect Correlations

Embedment Depth



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

Ultimate Capacity Condition

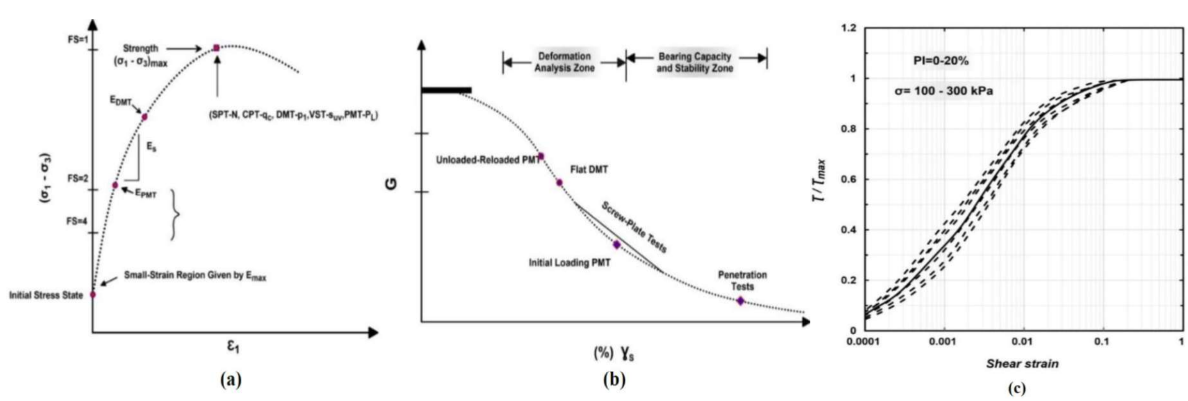


Interpretation of load displacement diagram for Case 001-L&D31 (Moshfeghi & Eslami, 2016)

Scale Effect Correlations

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

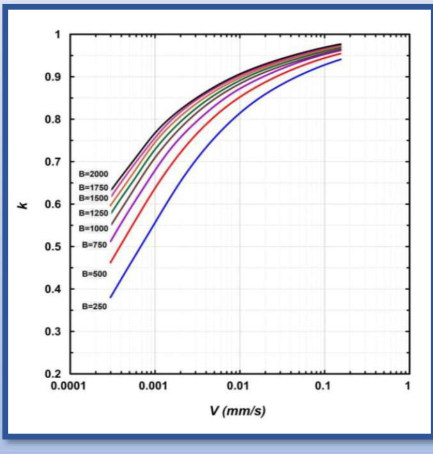
$$\gamma_{pile} = 0.0255\ (V_{pile})^{0.61} . (D_{pile})^{0.5}$$



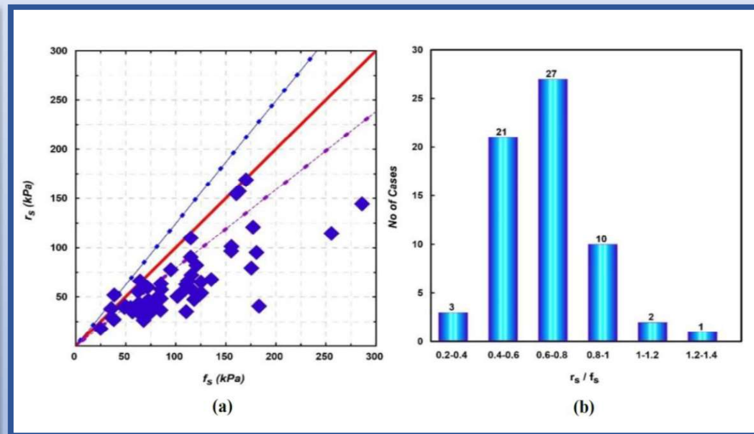
Stress strain Strength curves for different in situ tests; (a) strength measured by in situ tests at the peak of the stress strain curve, (b) variation of shear modulus with strain level (c) Variation of shear stress with shear strain (Sabatani et al., 2002)

Scale Effect Correlations

$$r_s = k.f_s$$



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



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

Direct Application for Deep Foundations Axial Capacity

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

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

Summary of Commonly Used CPT-Based Methods

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

Summary of Commonly Used CPT-Based Methods

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

Summary of Commonly Used CPT-Based Methods

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

Comments on the Current CPT-Based Methods for Pile Design

- **The methods developed in 70s and 80s do not consider the more accurate measurements achievable by CPTu, since, it was before the piezocone was generally available.**
- **While the recommendations are specified to soil type (clay and sand) for a few methods, none of them, except for Eslami and Fellenius (1997) and enhanced UniCone (Niazi and Mayne, 2016), include a means for identifying the soil type from CPT data. Instead, the soil profile governing the coefficients relies on information from conventional boring and sampling, and laboratory testing, which may not be fully relevant to the CPT data.**
- **All of the CPT-based methods include random smoothing and filtering of the CPT data, that is, elimination of peaks and troughs that exposes the results to considerable subjective operator influence.**

Comments on the Current CPT-Based Methods for Pile Design

- **The cone resistance (total resistance) has not been corrected for the pore pressure on the cone shoulder and, therefore, the data behind the methods include errors—smaller in sand, larger in clay. This matter, i.e., penetration pore pressure, u_2, is realized by Eslami and Fellenius (1997).**
- **Most of the older methods employ total stress values, whereas in long term, effective stress governs pile capacity.**
- **Some of the methods are locally developed, that is, they are based on limited types of piles and soils,**
- **The upper limit resistance imposed on the unit toe resistance in the Schmertmann is not reasonable in very dense sands where values of pile unit toe resistance, r_t, higher than 15 MPa frequently occur.**
- **Most of the direct methods involve a judgment in selecting the coefficient to apply to the average cone resistance to arrive at the unit toe resistance.**

Comments on the Current CPT-Based Methods for Pile Design

- **Some methods such as Eslami and Fellenius (1997), NGI (2005), ICP (2005), UWA (2005), specify a certain criterion for evaluating the pile capacity from static loading test results that can be used as reference to the pile capacity estimated from CPT data. While, other methods have not introduced any criteria for pile ultimate capacity. Yet, the capacity of a pile is determined from the results of static loading tests, varies considerably with the method used to evaluate the test (Fellenius, 1975).**
- **The NGI (2005), ICP (2005), Fugro (2005), and UWA (2005) methods are included in the commentary of the new 22nd edition of the API RP 2A Recommendations (2006) and are applicable for displacement piles in sand. They are more or less following a similar format. For instance, they all consider the effects of friction fatigue and toe condition in open end piles. Also, except for the Fugro method, the dilation effects during pile loading are accounted.**