Elective Course in Geotechnical Engineering

DESIGN OF PILE FOUNDATION

Civil Engineering Department

Fourth Year

Spring Semester

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Lecture 2

METHODS OF PILE DESIGN

Acknowledgment

- Prof Dr. Amr Elhakim
- Dr. Omar Ezzeldin

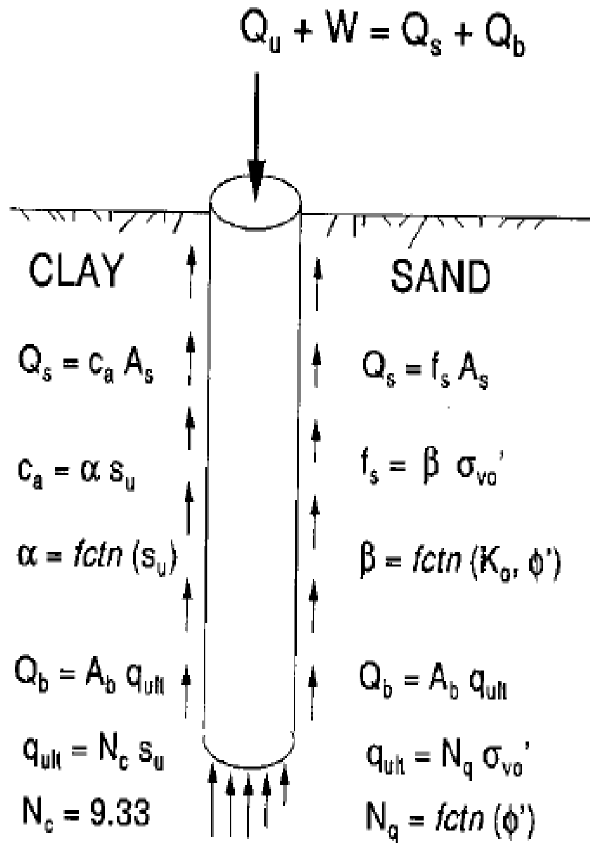
Lecture 1 Outline

- Small Diameter Piles Capacity
- Strain Compatability
- Large Diameter Piles Capacity
- Commentary on LRFD

Static Methods of Pile Design

Total stress Analysis

(α-method)



Effective stress Analysis

(β-method)

Capacity of Piles in Clay

- Ultimate Side Resistance in Cohesive Soil:
  - $Q_f = c_a(\pi D \, \Delta L)$
  - $c_a = \alpha c$
  - $\alpha = 0.3 \; to \; 0.4$ $\quad$ $Bored \; Piles$
  - $c_a \leq 100 \; kPa \; (1 \; kg/cm^2)$

- Ultimate Base Resistance in Cohesive Soil:
  - $Q_b = cN_c\left(\frac{\pi D^2}{4}\right)$
  - $N_c = 9$

ECP 202

Displacement Piles

نوع الخازوق	قوام التربة	إجهاد التماسك c * kN/m^2	إجهاد التلاصق C_a * الأقصى kN/m^2
خشب أو خرسانة	ضعيف التماسك جدا	٠,٠ –١٢,٥	٠,٠ –١٢,٥
	ضعيف التماسك	١٢,٥–٢٥,٠	١٢,٥ – ٢٤,٠
	متوسط التماسك	٢٥,٠–٥٠,٠	٢٤,٠–٣٧,٥
	متماسك	٥٠,٠–١٠٠,٠	٣٧,٥–٤٧,٥
	شديد التماسك	١٠٠,٠–٢٠٠,٠	٤٧,٥–٦٥,٠
صلب	ضعيف التماسك جدا	٠,٠ –١٢,٥	٠,٠ –١٢,٥
	ضعيف التماسك	١٢,٥–٢٥,٠	١٢,٥–٢٣,٠
	متوسط التماسك	٢٥,٠–٥٠,٠	٢٣,٠–٣٥,٠
	متماسك	٥٠,٠–١٠٠,٠	٣٥,٠–٣٦,٠
	شديد التماسك	١٠٠,٠–٢٠٠,٠	٣٦,٠–٣٧,٥

القيم الصغرى والعليا لإجهاد التلاصق C_a تناظر القيم الصغرى والعليا لإجهاد التماسك c

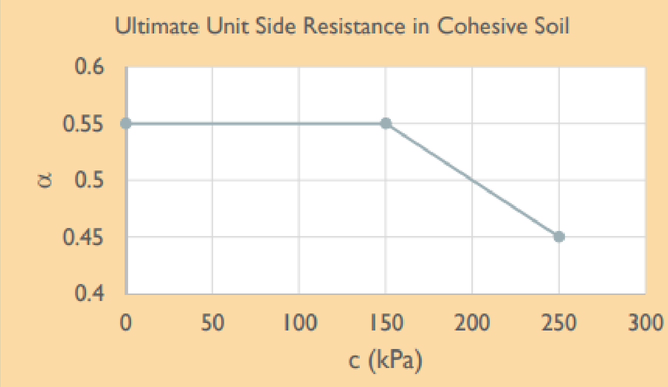
١ كجم / سم^{٢} $\simeq$ ١٠٠ ك . نيوتن/م^{٢}

VERTICAL CAPACITY OF CFA PILES
FHWA(1999) RECOMMENDATIONS

- Ultimate Side Resistance in Cohesive Soil:
  - $Q_f = c_a(\pi D\ \Delta L)$
  - $c_a = \alpha c$

c (kPa)	0-150.0	150.0-250.0
α	0.0-0.55	0.55-0.45

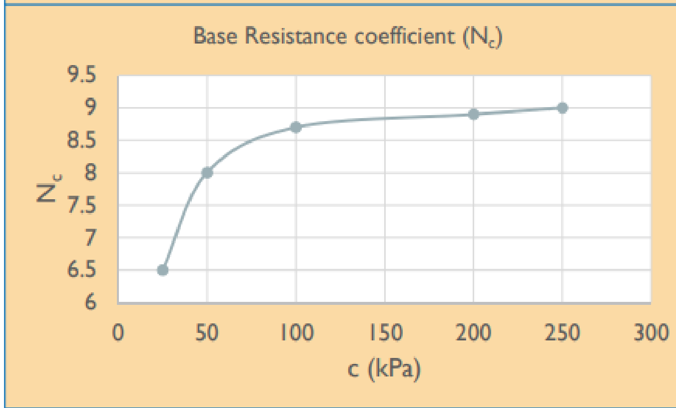
  - If the bearing layer is clay, the side friction at the bottom 1 D is neglected. If there is possibility of separation between the top portion of the pile and the cohesive soil around it, then side friction is neglected at the top 1.5 m.



- Ultimate Base Resistance in Cohesive Soil:
  - $Q_b = cN_c\left(\frac{\pi D^2}{4}\right)$

c (kPa)	25.0	50.0	100	200	250
N_c	6.5	8.0	8.7	8.9	9.0

  - If undrained cohesion (c) is higher than 250 kPa, then it is treated as an intermediate goe-material (soil/rock).



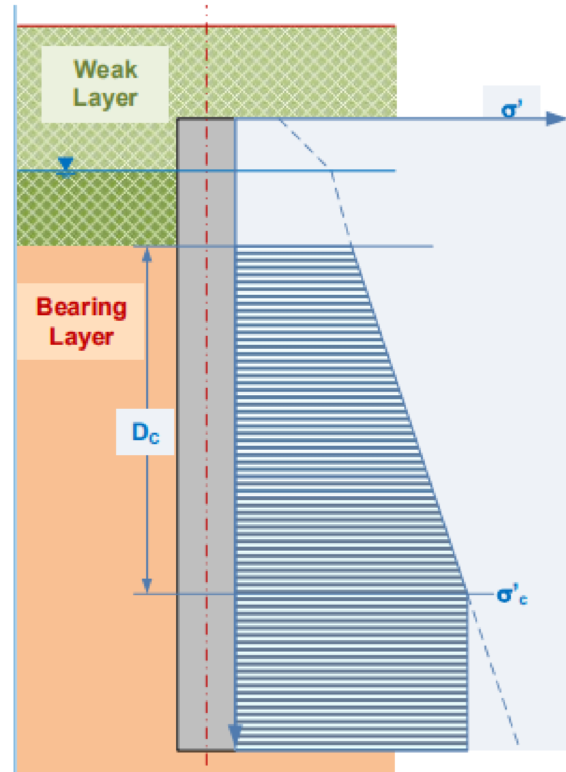
  - Factor of Safety:
  - Minimum factor of safety of 2.5 is used. This factor of safety could be reduced if high quality control and easy construction conditions are met.

Capacity of Piles in Sand

- Ultimate Side Resistance in Cohesionless Soil:
  - $Q_f = K_{HC} p_o \tan\delta\,(\pi D\,\Delta L)$
  - $K_{HC} = 0.7\ to\ 1.5$
  - $\delta = 0.75\phi$ *for R.C. Piles*
  - p_o *is calculated considering the Critical Depth Proposition*

- Ultimate Base Resistance in Cohesionless Soil:
  - $Q_b = p_b N_q \left(\frac{\pi D^2}{4}\right)$
  - $\phi_{CFA} = (\phi - 3)$ *for Non Displacement Piles*
  - p_b *is calculated considering the Critical Depth Proposition*

ϕ (Degrees)	25	30	35	40
N_q	15	30	75	150



Critical Depth Proposition

ECP 202

جدول رقم (٤-٦): قيم المعاملات (K_{HC}) و(K_{HT})

K_{HT}	K_{HC}	نوع الخازوق
٠,٥-٠,٣	١,٠-٠,٥	خازوق ذو قطاع H
١,٠-٠,٦	١,٥-١,٠	خازوق ازاحة
١,٣-١,٠	٢,٠-١,٥	خازوق ازاحة متغير القطاع
٠,٦-٠,٣	٠,٩-٠,٤	خازوق ازاحة باستخدام النفاثات
١,٠-٠,٤	١,٥-٠,٧	خازوق تثقيب اعتيادى(قطر اقل من ٠,٦٠متر)

جدول رقم (٤-٧) : قيم زاوية الإحتكاك بين التربة وجزء الخازوق (δ)

δ (درجة)	نوع الخازوق
٢٠	حديد
٠,٧٥ (ϕ)	خرسانة
٠,٧٥ (ϕ)	خشب

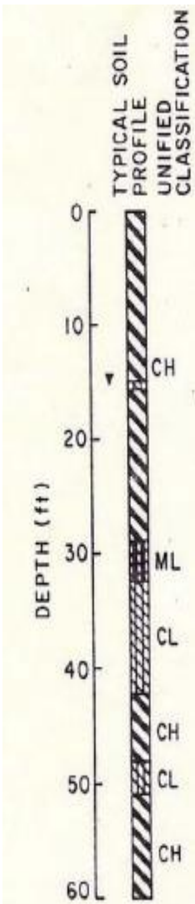
ϕ زاوية الاحتكاك الداخلى للتربة

VERTICAL CAPACITY OF CFA PILES FHWA(1999) RECOMMENDATIONS

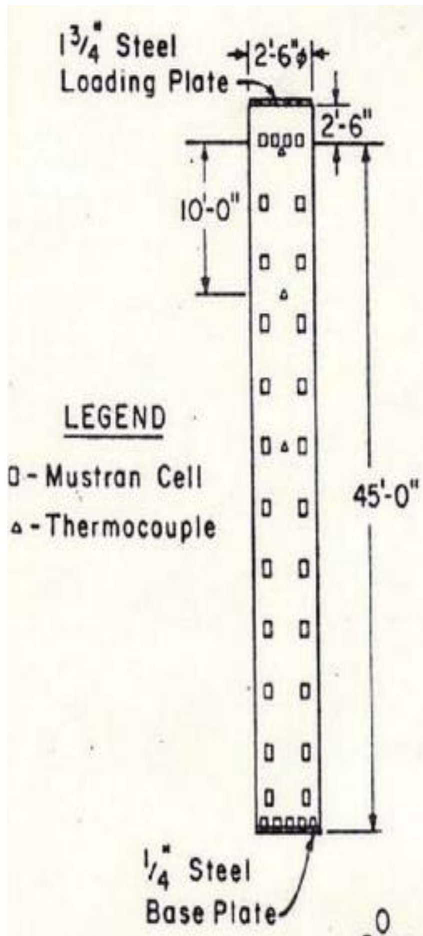
- Ultimate Unit Side Resistance in Cohesionless Soil q_f:
  - $q_f = \sigma' \beta$
  - $q_f \leq 200\ kPa$
  - σ': effective overburden stress at the level of the mid point of the segment at which the side resistance is calculated
  - $\beta = K\ tan\phi$
  - β ranges between 0.25 and 1.20
  - $\beta = 1.5 - 0.245\sqrt{z} \quad for\ N \geq 15$
  - $\beta = \frac{N}{15}(1.5 - 0.245\sqrt{z}) \quad for\ N < 15$
  - z= Depth in meters to the mid point of the segment at which the side resistance is calculated.
- Ultimate Unit Base Resistance in Cohesive Soil:
  - $q_b = 57.5\ N_{6\ 0}\ (kPa) \qquad N_{6\ 0} \leq 75$
  - $q_b = 4300\ kPa \qquad N_{6\ 0} > 75$

What to do with Large diameter piles?

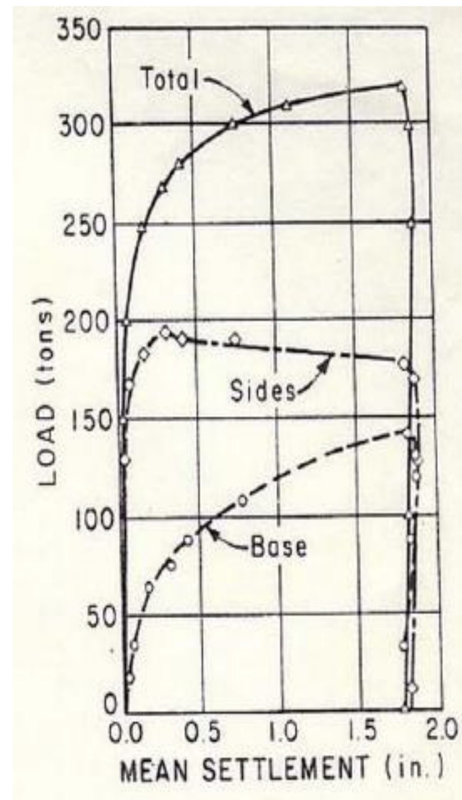
(Reese, L.C., Journal of Geotechnical Engineering, Jan. 1978)



Ground Profile



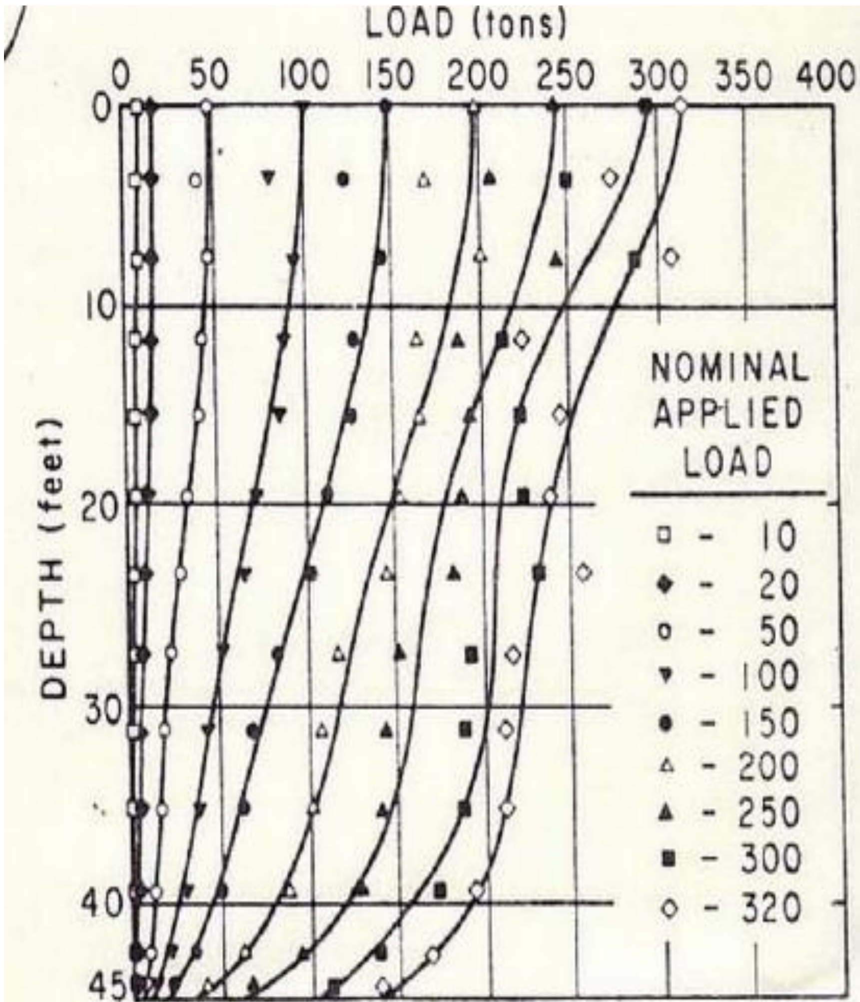
Instrumented Pile



Pile Load Tests

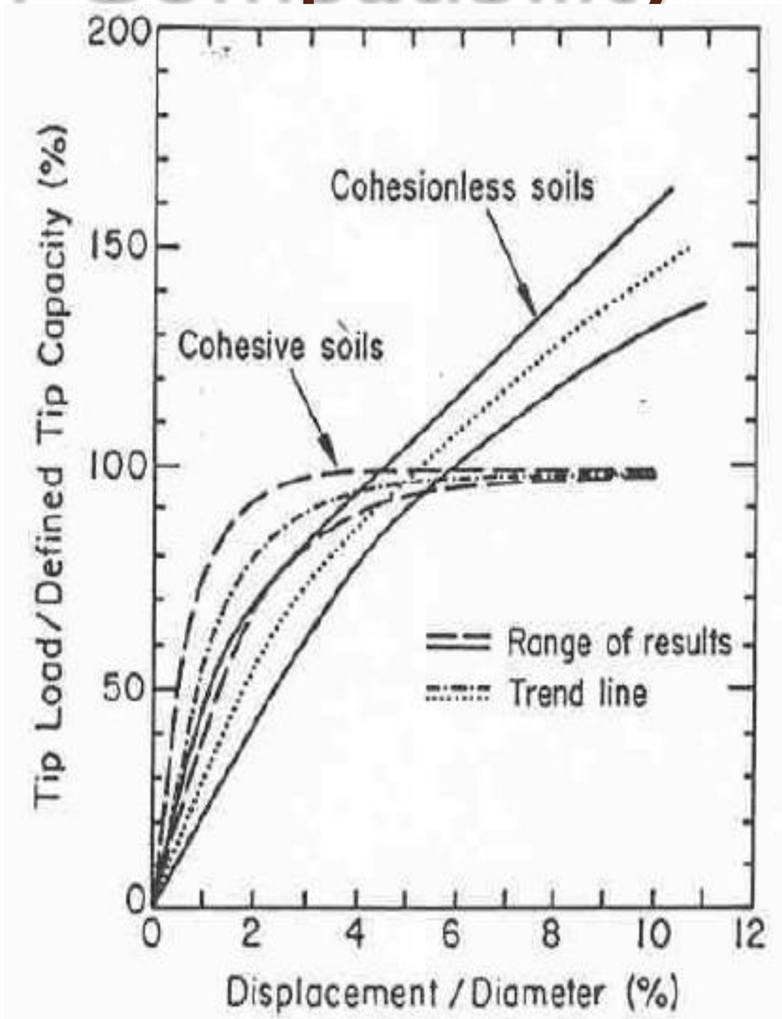
Axial load transfer

(Reese, L.C., Journal of Geotechnical Engineering, Jan. 1978)



Pile Load Transfer Curves

Strain Compatibility



Reese and O'Neill (1988)

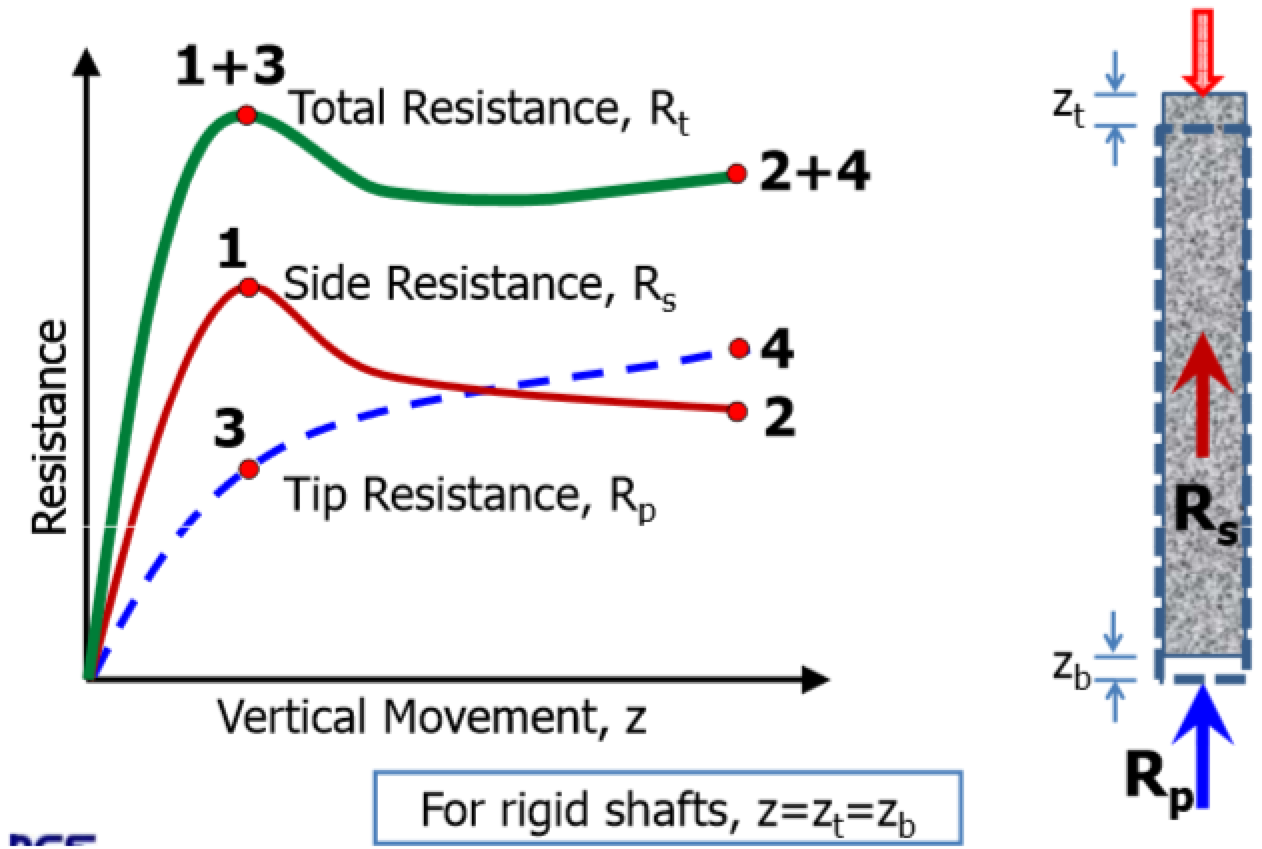
Strain Compatibility

- For the undrained loading (mainly fine-grained soils) beneath the base of drilled shafts, it is reasonable to assume the full mobilization of end bearing resistance at acceptable limits of vertical displacements ($s/B = 0.05$, where s = settlement and B = pile/shaft width).

Strain Compatibility

- For the drained loading (mainly coarse-grained soils) beneath the base of drilled shafts, it is unreasonable to assume the full mobilization of end bearing resistance at acceptable limits of vertical displacements.

Concept of Mobilized Resistance



Friction vs Settlement in Cohesive Soils

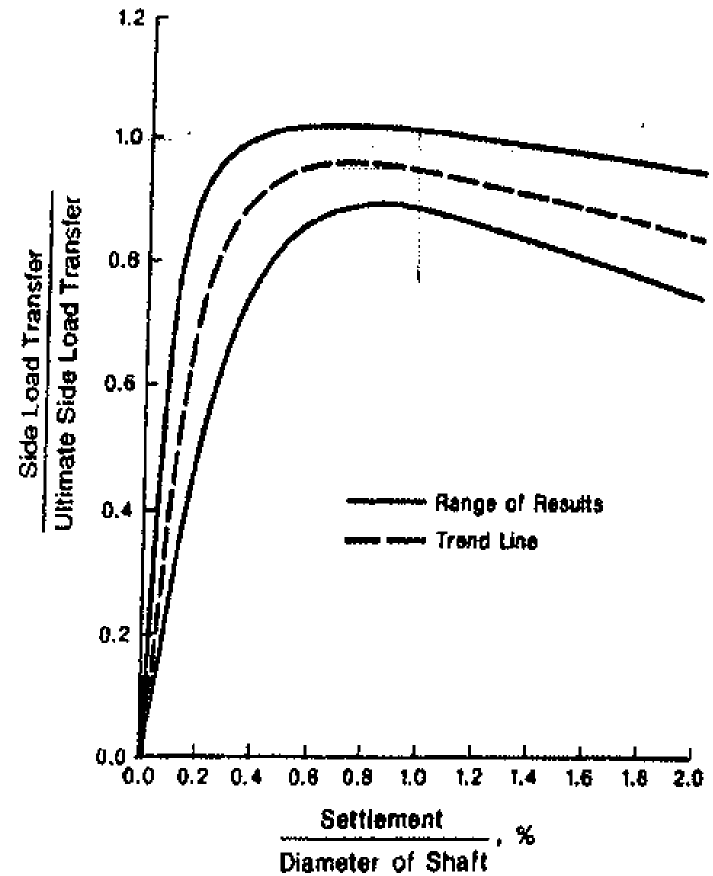


Figure 10.8.2.2.2-1 Normalized Load Transfer in Side Resistance versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

Bearing vs Settlement in Cohesive Soils

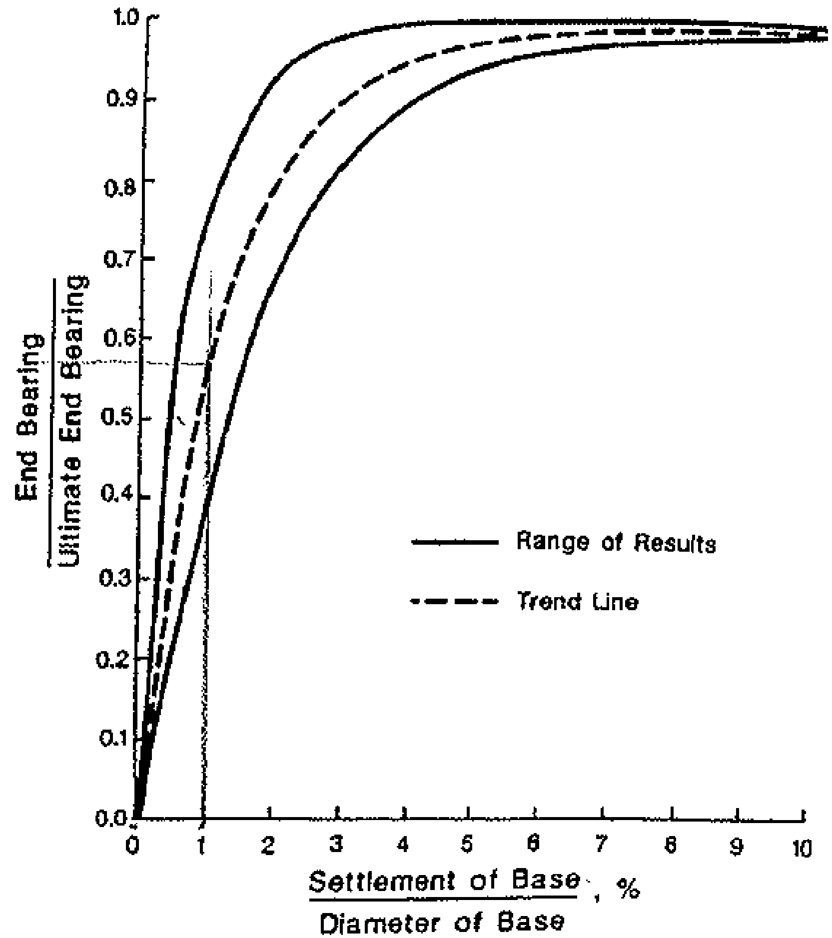


Figure 10.8.2.2.2-2—Normalized Load Transfer in End Bearing versus Settlement in Cohesive Soils (from O'Neill and Reese, 1999)

Friction vs Settlement in Cohesionless soils

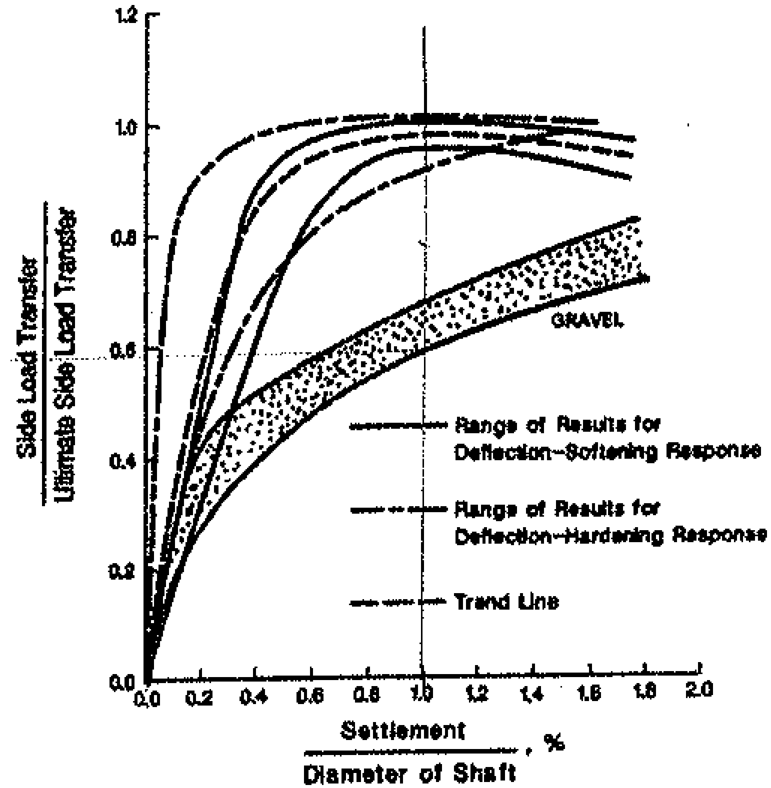


Figure 10.8.2.2.2-3—Normalized Load Transfer in Side Resistance versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

Bearing vs Settlement in Cohesionless soils

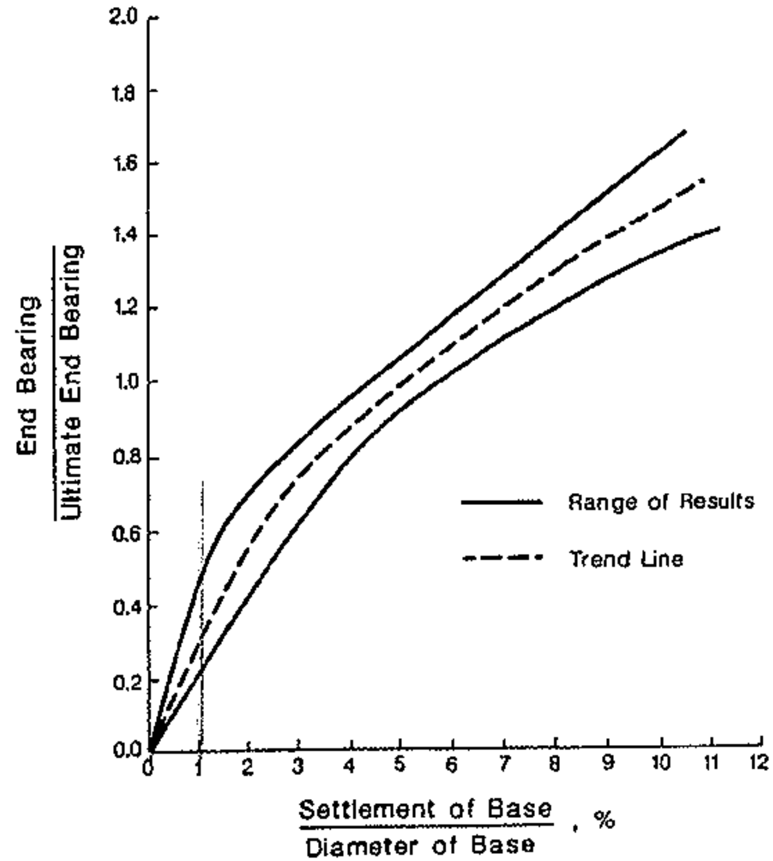


Figure 10.8.2.2.2-4—Normalized Load Transfer in End Bearing versus Settlement in Cohesionless Soils (from O'Neill and Reese, 1999)

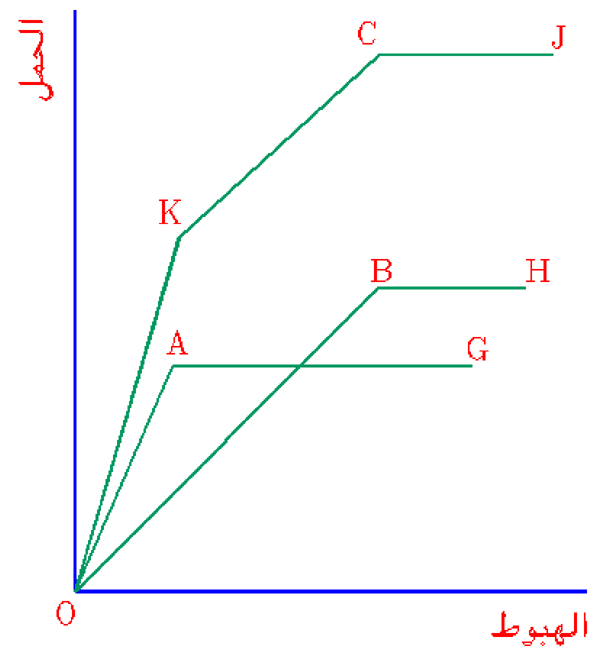
ECP for large diameter piles

The Egyptian code recommends the application of a graphical procedure in which the following graph is constructed:

OBH: end bearing-settlement relationship.

OAG: shaft resistant-settlement relationship.

OKCJ: total load-settlement relationship.



End Bearing Curve in Cohesive soils

Movement (mm)	End Bearing (MPa)
0.2 S_G	0.50
0.3 S_G	0.70
S_G	1.20

S_G is the expected pile movement at the ultimate bearing capacity and is equal to 5% of the pile diameter.

End bearing Curve in Cohesionless soils

Movement (cm)	End Bearing (MPa)	
	Enlarged End	Regular End
1	0.35	0.50
2	0.65	0.80
3	0.90	1.10
15	2.40	3.40

Friction Curve in cohesive soils

Undrained Shear Strength (kPa)	Maximum shaft resistance (kPa)
0	0
25	25
100	40
200	50

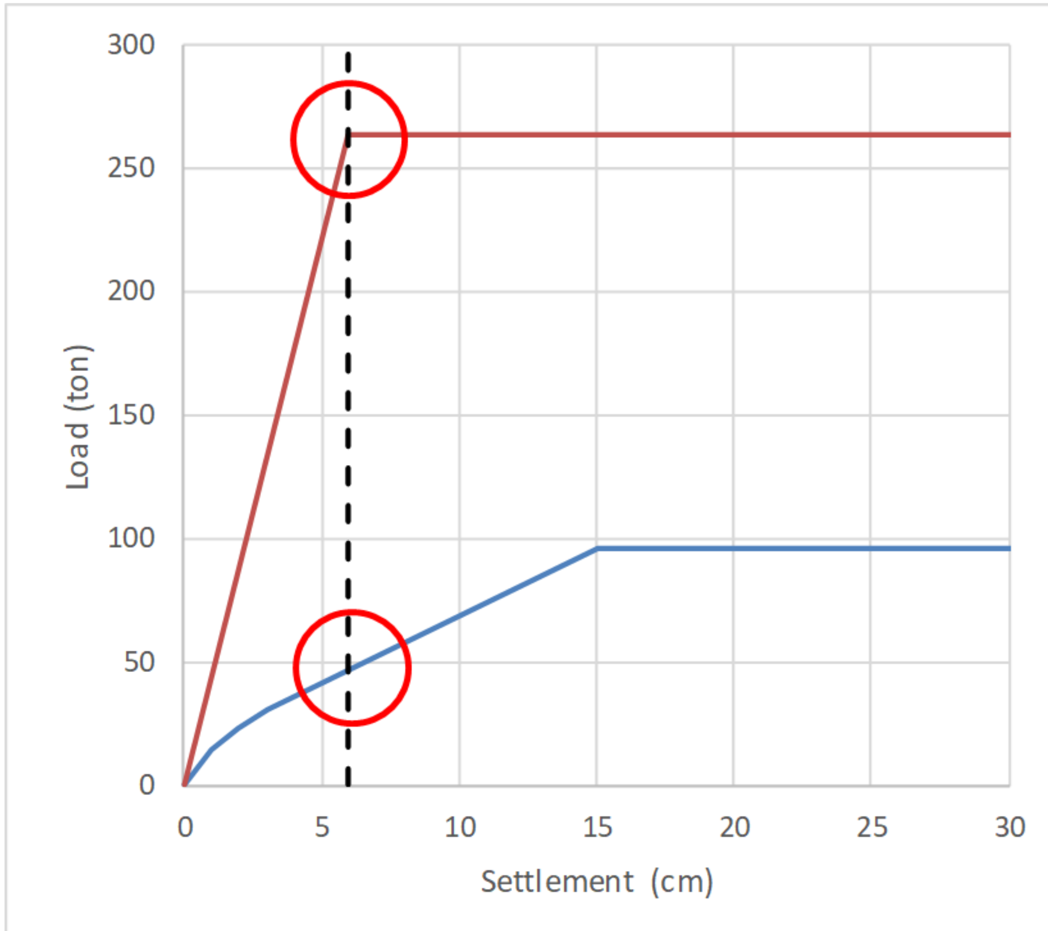
Friction Curve in Cohesionless soils

N	Depth below natural ground surface (m)	Maximum shaft resistance (kPa)
<10	----	zero
10–20	0–2	0
	2–5	30
	>5	50
20–30	0–2	0
	2–7.5	45
	>7.5	75
>30	0–2	0
	2–10	60
	>10	100

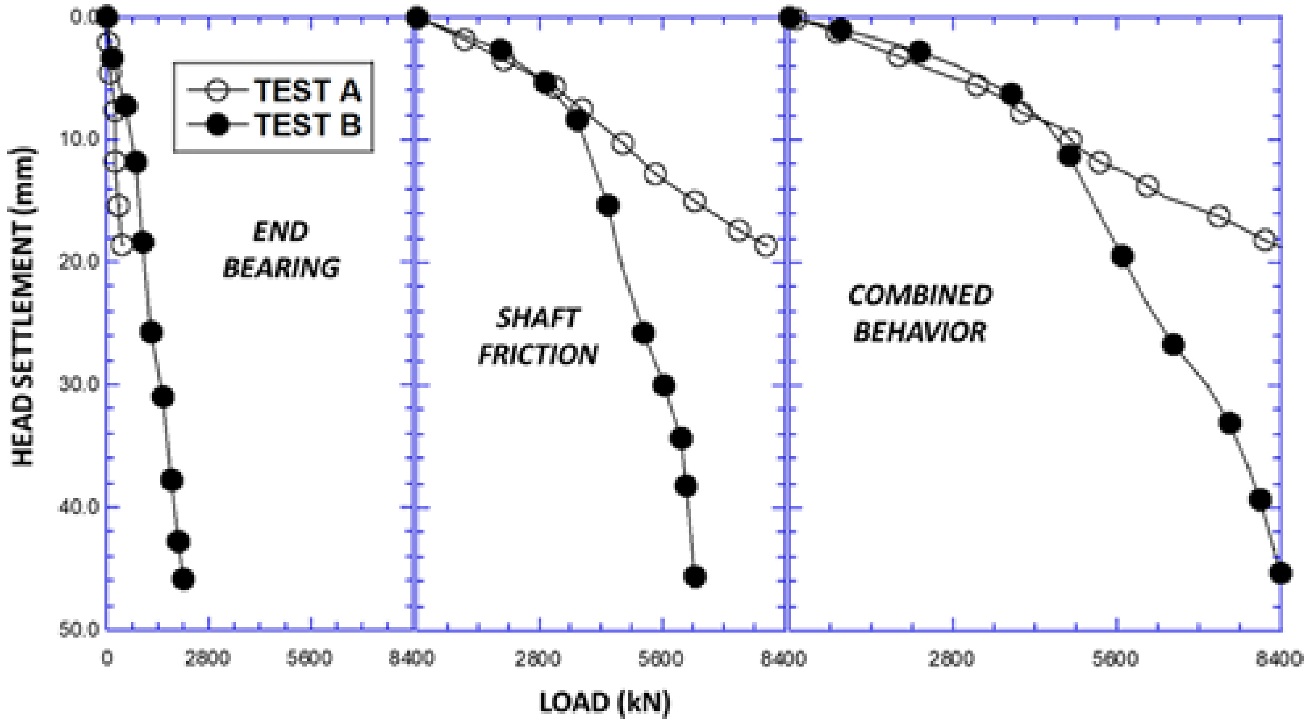
Important Notes

- According to the Egyptian Code of Practice, shaft resistance is neglected:
  - 2-m below natural ground surface.
  - One-pile diameter above pile tip.

Example



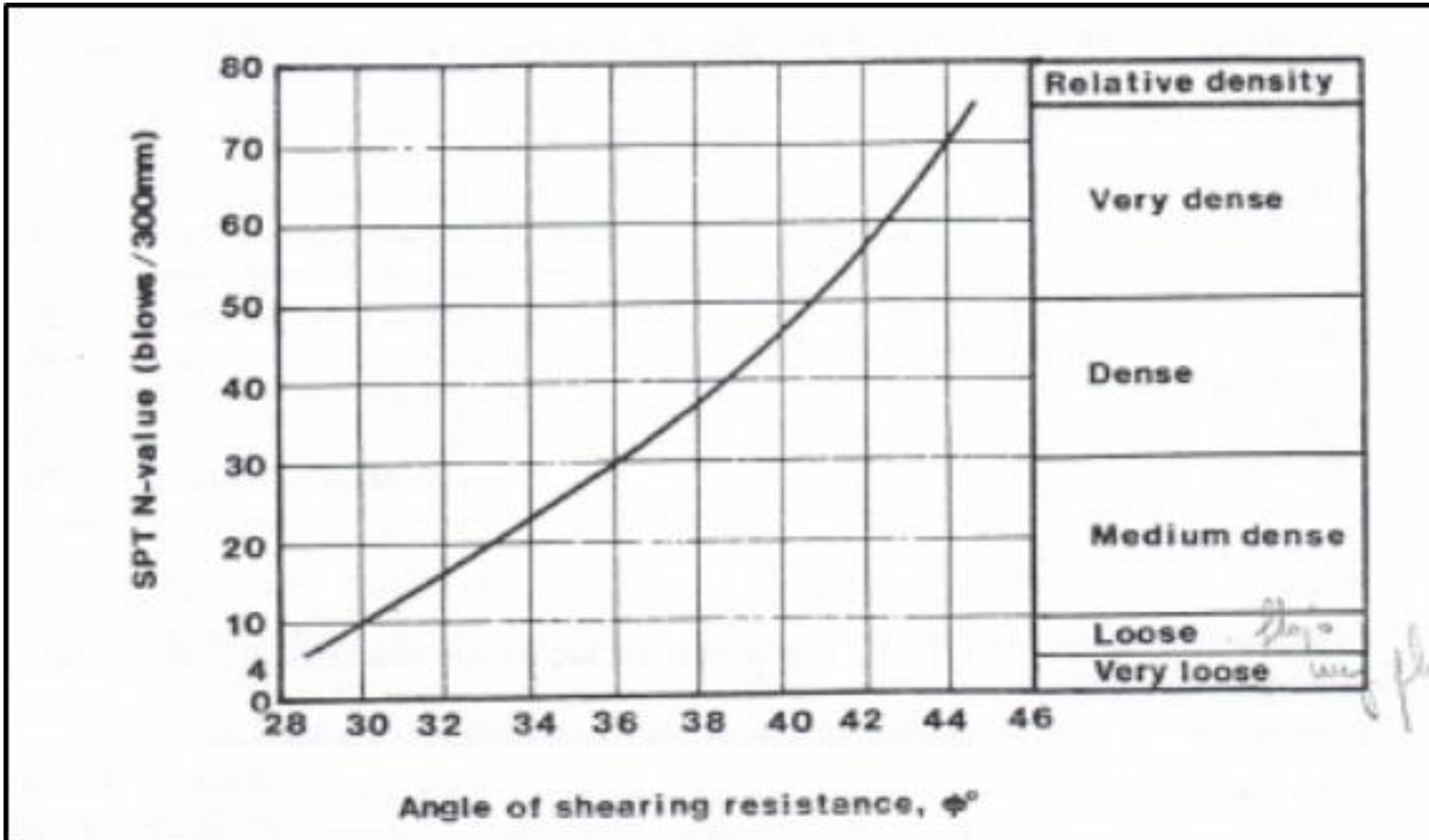
Failed Test vs Successful Test



VERTICAL CAPACITY OF BORED PILES
FHWA(1999) RECOMMENDATIONS

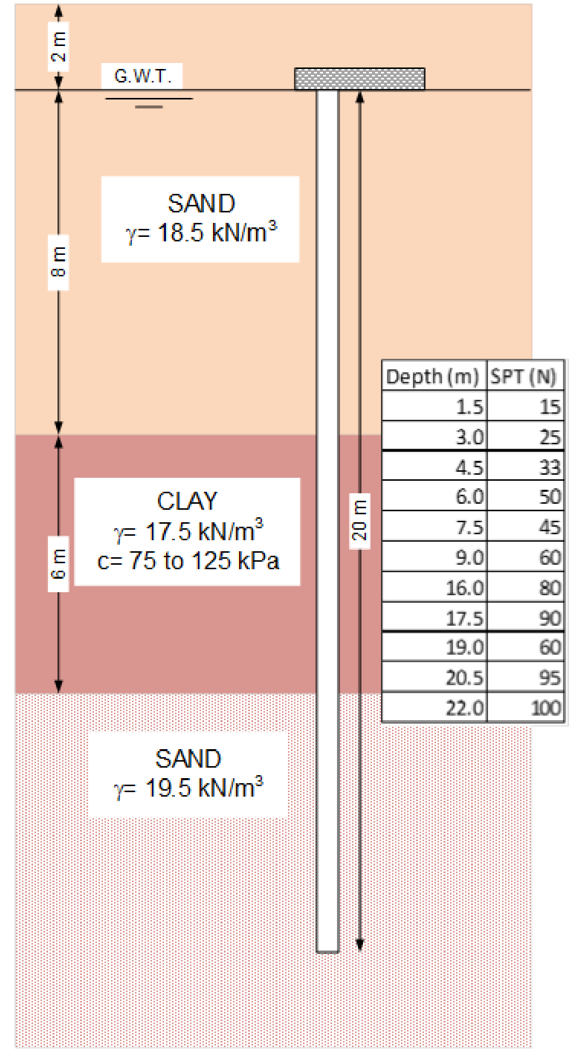
- Ultimate Unit Side Resistance in Cohesionless Soil q_f:
  - $q_f = \sigma' \beta$
  - σ': effective overburden stress at the level of the mid point of the segment at which the side resistance is calculated. It is calculated considering the Critical Depth Proposition
  - $\beta = K \tan\phi$
  - $K = K_o$
  - $K_o = (1 - \sin\phi)\ OCR^{\sin\phi}$ $\quad OCR: Overconsolidation\ Ratio$
  - $OCR = \frac{\sigma'_p}{\sigma'}$ $\quad \sigma'_p: overconsolidation\ pressure$
  - $\sigma'_p = 0.47 p_a (N_{60})^m$ $\quad p_a: atmospheric\ pressure\ (100\ kPa)$
  - $m = 0.6\ \ (Clean\ Quartizitic\ Sand), m = 0.8\ \ (Silty\ Sand)$
  - N_{60}: SPT (N) corrected to 60% theoretical energy
- Ultimate Unit Base Resistance in Cohesionless Soil:
  - $q_b = 57.5\ N_{60}\ \ (kPa)$ $\quad q_b \leq 2875\ kPa$
- Factor of Safety:

  Minimum factor of safety of 2.5 for side resistance and 2.75 for base resistance is used. This factor of safety could be reduced if high quality control and easy construction conditions are met.



SPT N-value (blows / 300mm)
80
70
60
50
40
30
20
10
4
0
28
30
32
34
36
38
40
42
44
46
Angle of shearing resistance, φ°
Relative density
Very dense
Dense
Medium dense
Loose
Very loose

Homework



2 m
G.W.T.
SAND
γ= 18.5 kN/m3
8 m
CLAY
γ= 17.5 kN/m3
c= 75 to 125 kPa
6 m
20 m
SAND
γ= 19.5 kN/m3
Depth (m) SPT (N)
1.5 15
3.0 25
4.5 33
6.0 50
7.5 45
9.0 60
16.0 80
17.5 90
19.0 60
20.5 95
22.0 100

Example

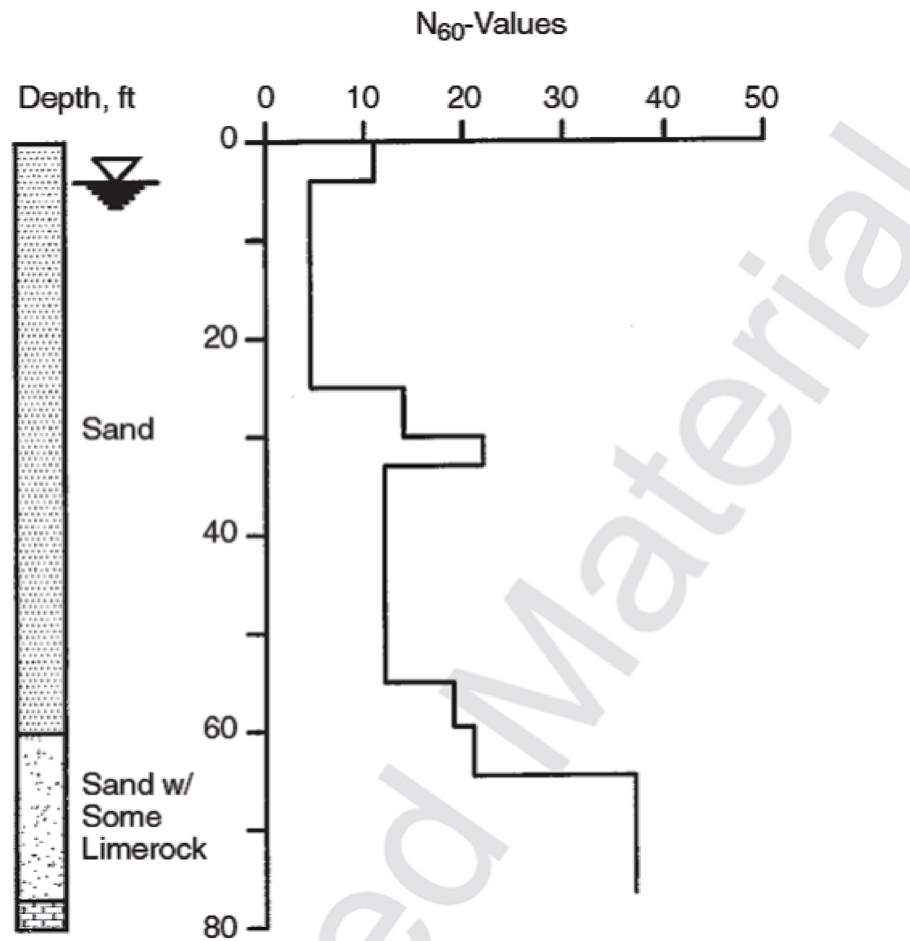


Figure 11.6 General soil description of Example Problem 2.

60cm, Friction according to ECP

1. Frictional Capacity

$$Q_f = K_{HC} p_o \tan \delta \, (\pi D \, \Delta L)$$

K_{HC}	0.70

Depth (m)	ΔL (m)	p_o kN/m2	N	φ (Degrees)	δ (Degrees)	tanδ	Q_f (kN)
0.563	1.125	10.367	11	30.320	22.740	0.419	6.452
4.314	6.377	48.061	4	28.034	21.026	0.384	155.449
8.300	1.594	82.102	14	31.211	23.408	0.433	74.771
9.566	0.938	92.767	22	33.730	25.298	0.473	54.258
13.364	6.659	104.992	12	30.520	22.890	0.422	389.474
17.350	1.313	113.268	19	32.824	24.618	0.458	89.916
							770.320

60cm, Friction according to FHWA

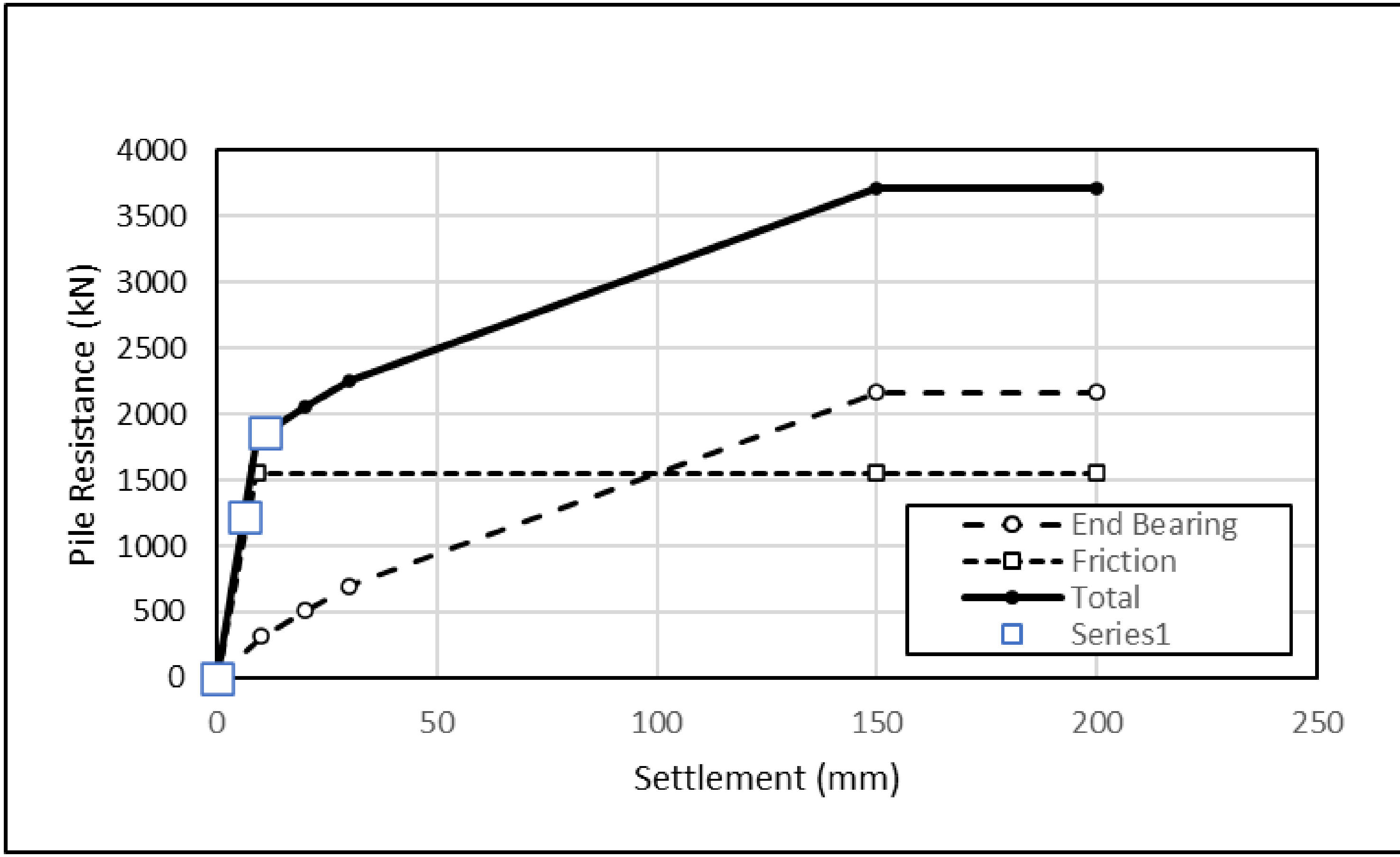
1. Frictional Capacity

$$q_f = \sigma' \beta$$

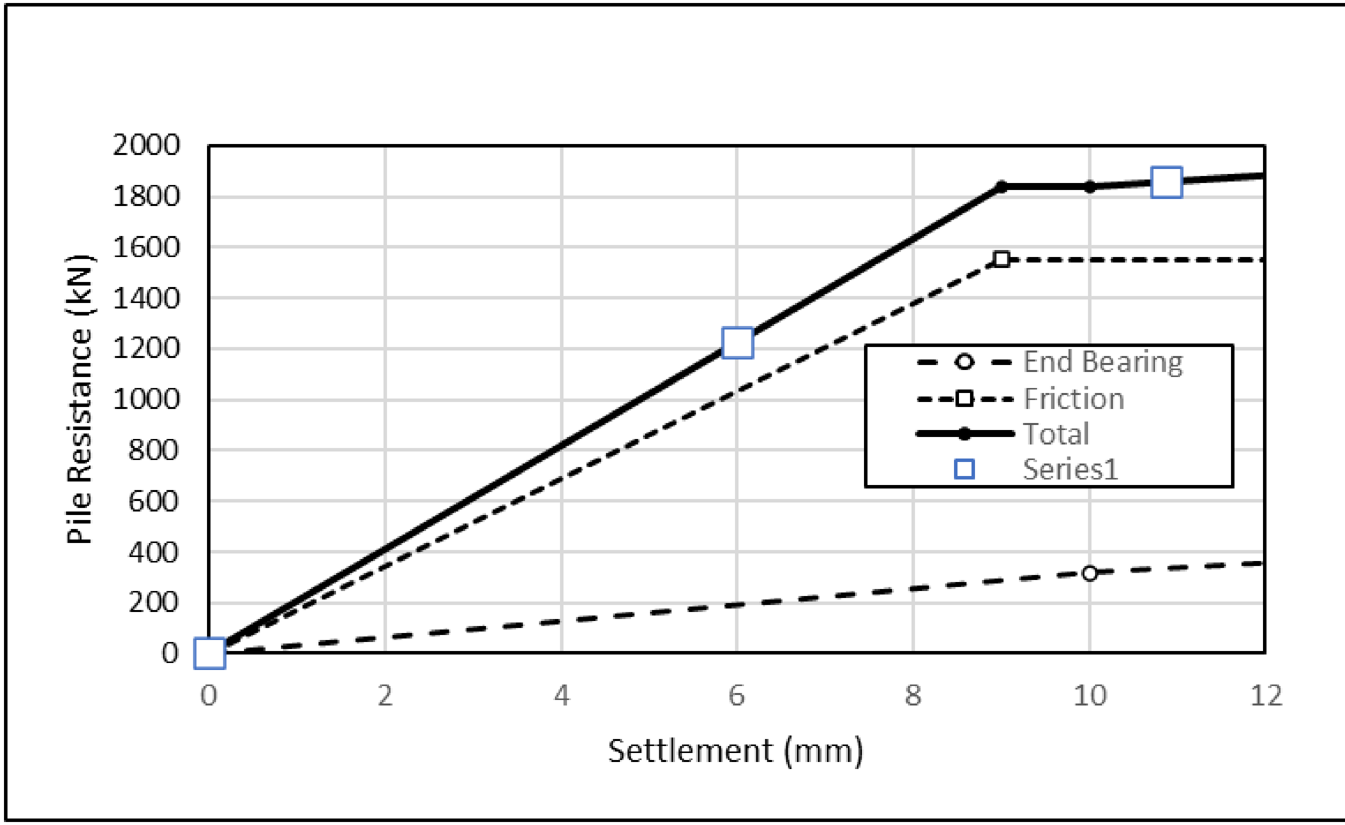
$$\beta = 1.5 - 0.245\sqrt{z} \quad for\ N \geq 15$$
$$\beta = \frac{N}{15}(1.5 - 0.245\sqrt{z}) \quad for\ N < 15$$

Depth (m)	ΔL (m)	σ' kN/m2	N	β	q_f (kN/m^2)	Q_f (kN)
0.563	1.125	10.367	11	0.963	9.984	21.180
4.314	6.377	48.061	4	0.282	13.554	162.938
8.300	1.594	82.102	14	0.726	59.637	179.224
9.566	0.938	92.767	22	0.742	68.855	121.722
13.364	6.659	124.760	12	0.467	58.234	730.921
17.350	1.313	158.332	19	0.479	75.919	187.892
						1403.877

90cm, Combined Curve



Zoom In



2000
1800
1600
1400
1200
1000
800
600
400
200
0
Pile Resistance (kN)
0
2
4
6
8
10
12
Settlement (mm)
End Bearing
Friction
Total
Series1

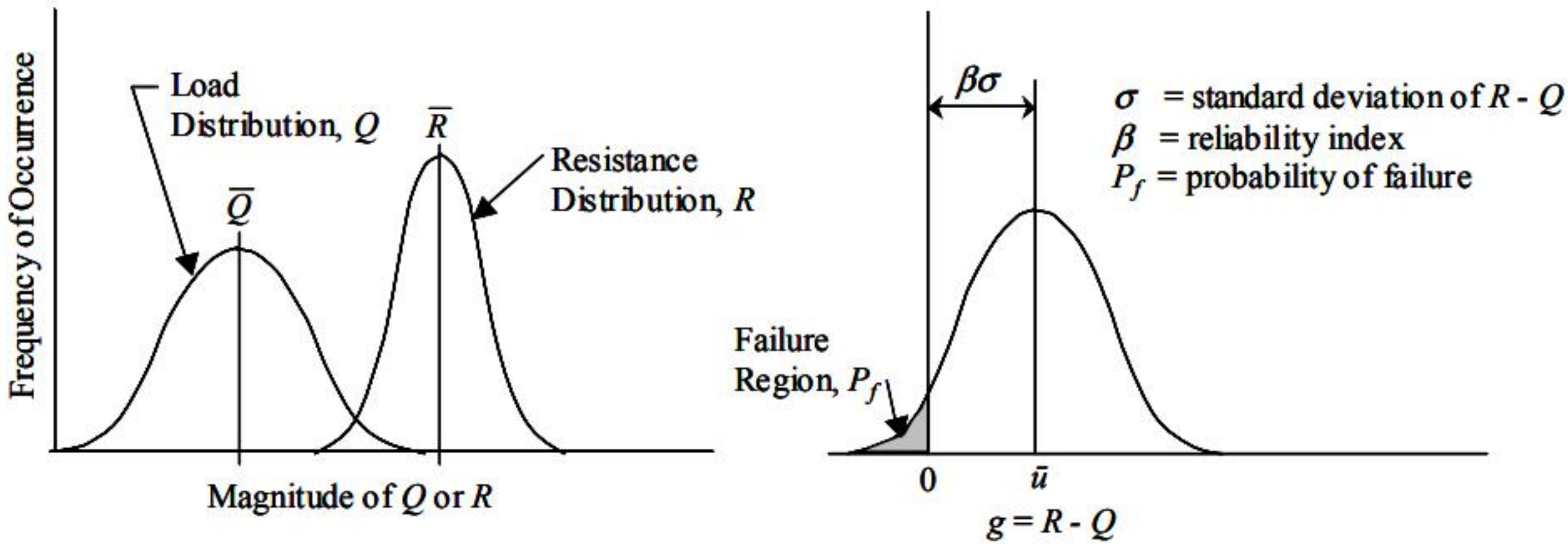
Main Idea of LRFD

$$\sum \gamma_i Q_{ni} \leq \varphi R_n \qquad (1)$$

where

γ_i = load factor applicable to a specific load component;
Q_{ni} = a specific nominal load component;
$\Sigma \gamma_i Q_{ni}$ = the total factored load for the load group applicable to the limit state being considered;
φ = the resistance factor; and
R_n = the nominal resistance available (either ultimate or the resistance available at a given deformation).

Bases of approach



a) Frequency distributions for random values of Q and R b) Distribution of limit state function values

Thank you