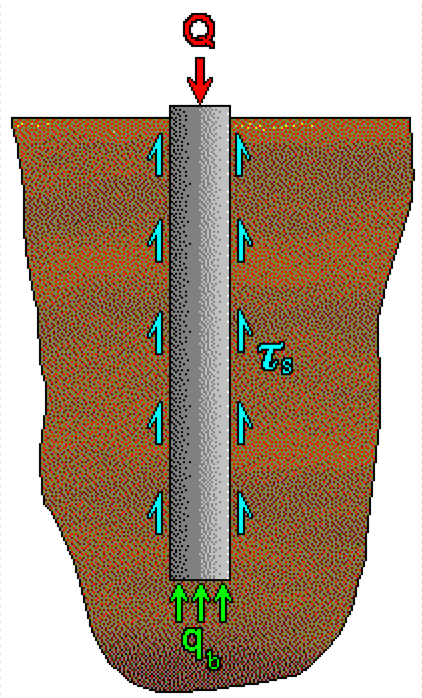
Principles of Foundation Engineering

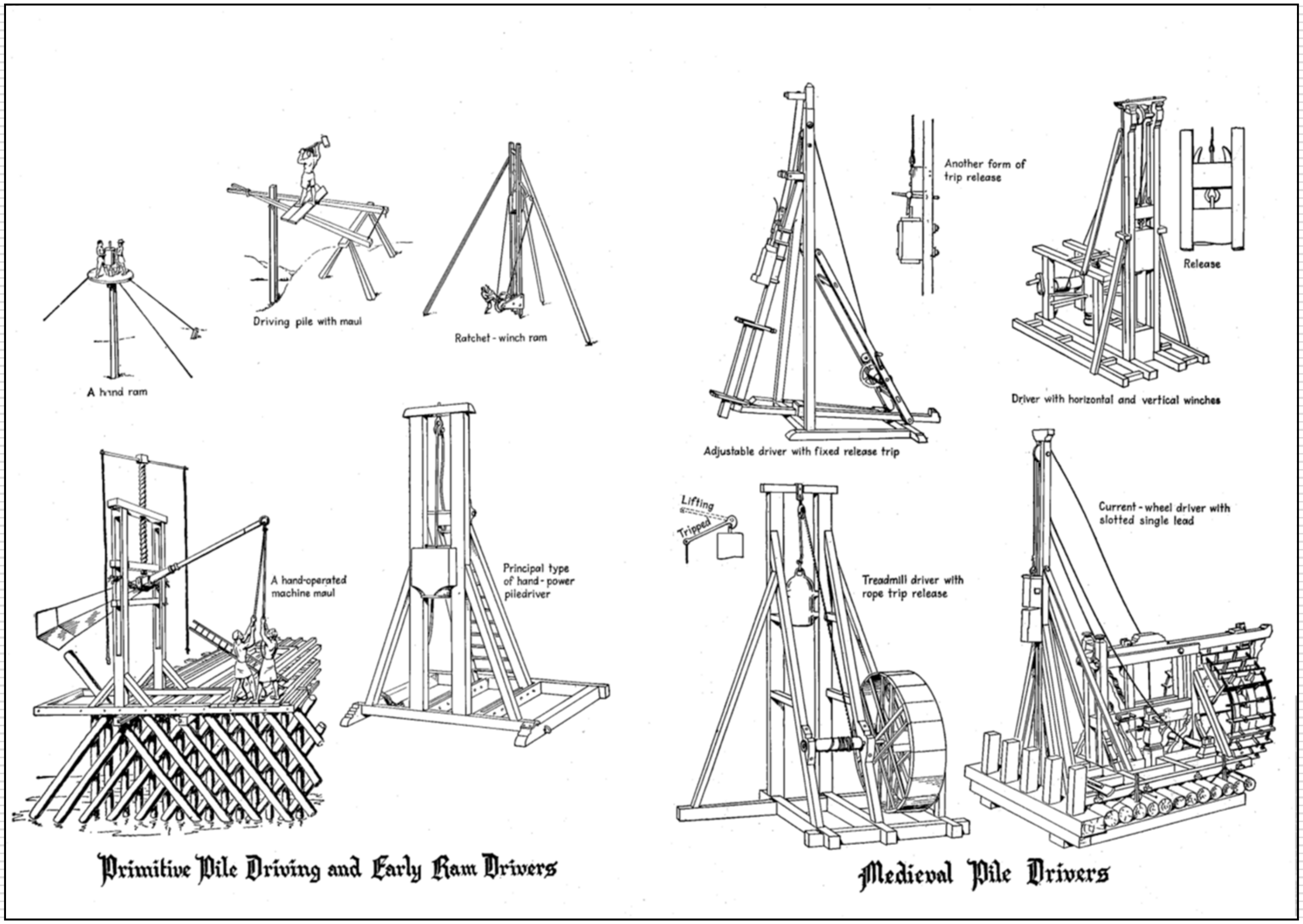
Braja M. Das



Chapter 11

Pile Foundations

Pile Driving Through The Ages

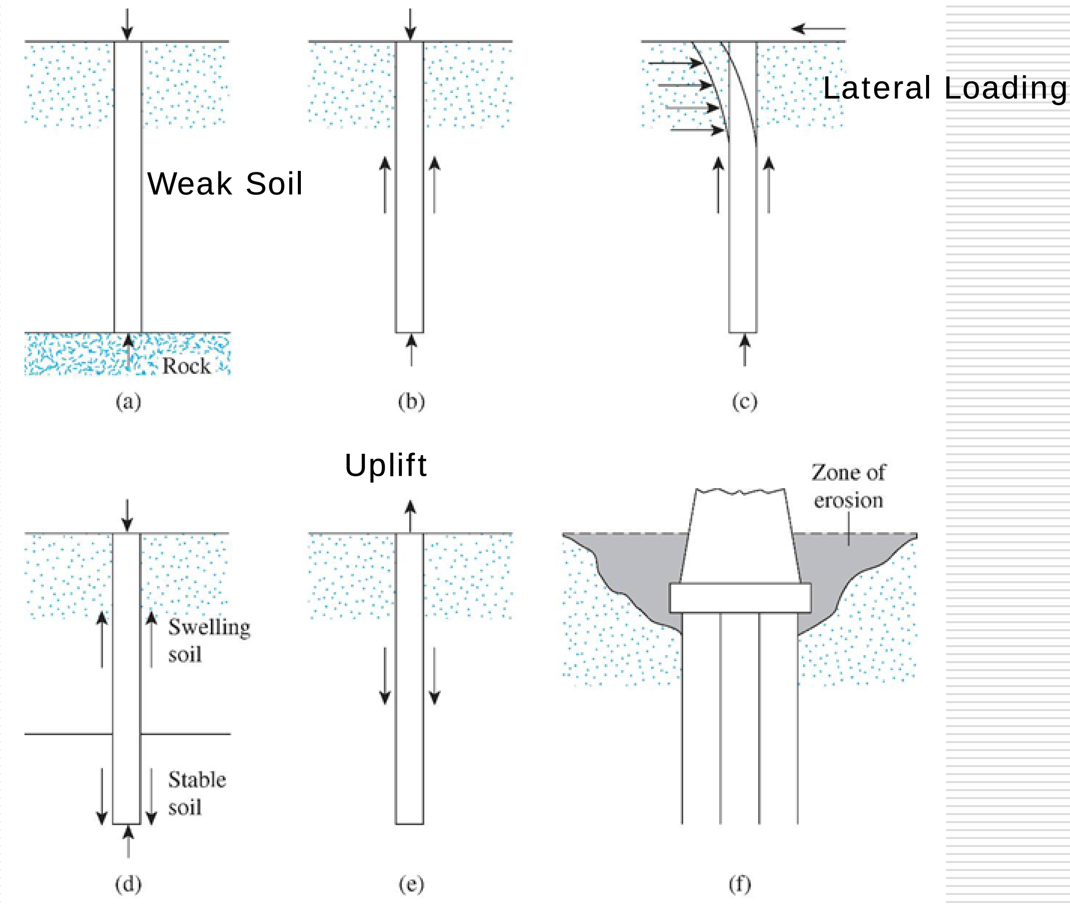


Pile Driving in Thailand

Why Piles?

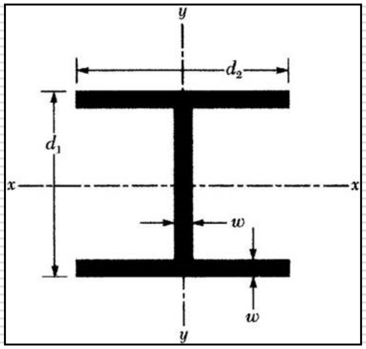
- Soils are insufficient in strength or compressibility characteristics to support shallow spread foundations.
- Structural loads are too high to be supported by reasonably size footings.
- Uplift forces are too high to be resisted by shallow foundations
- Lateral loads are too high to be resisted by shallow foundations

Conditions that require the use of pile foundations

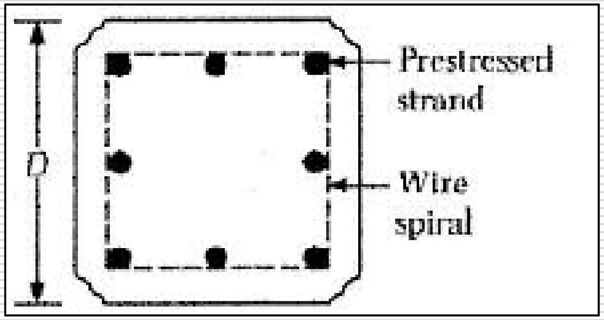


Typical Pile Types

Steel H-piles

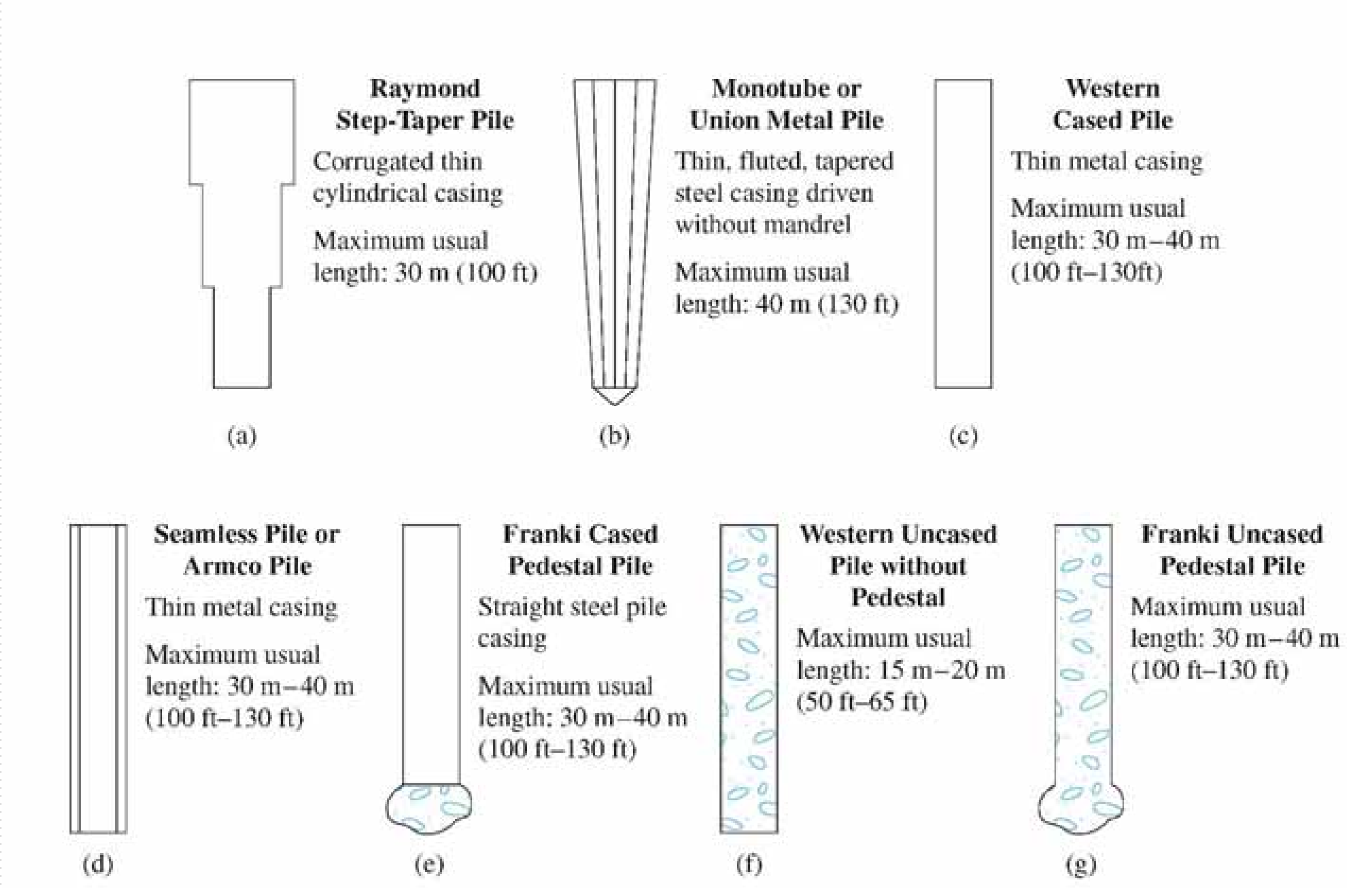


Prestressed Concrete Piles

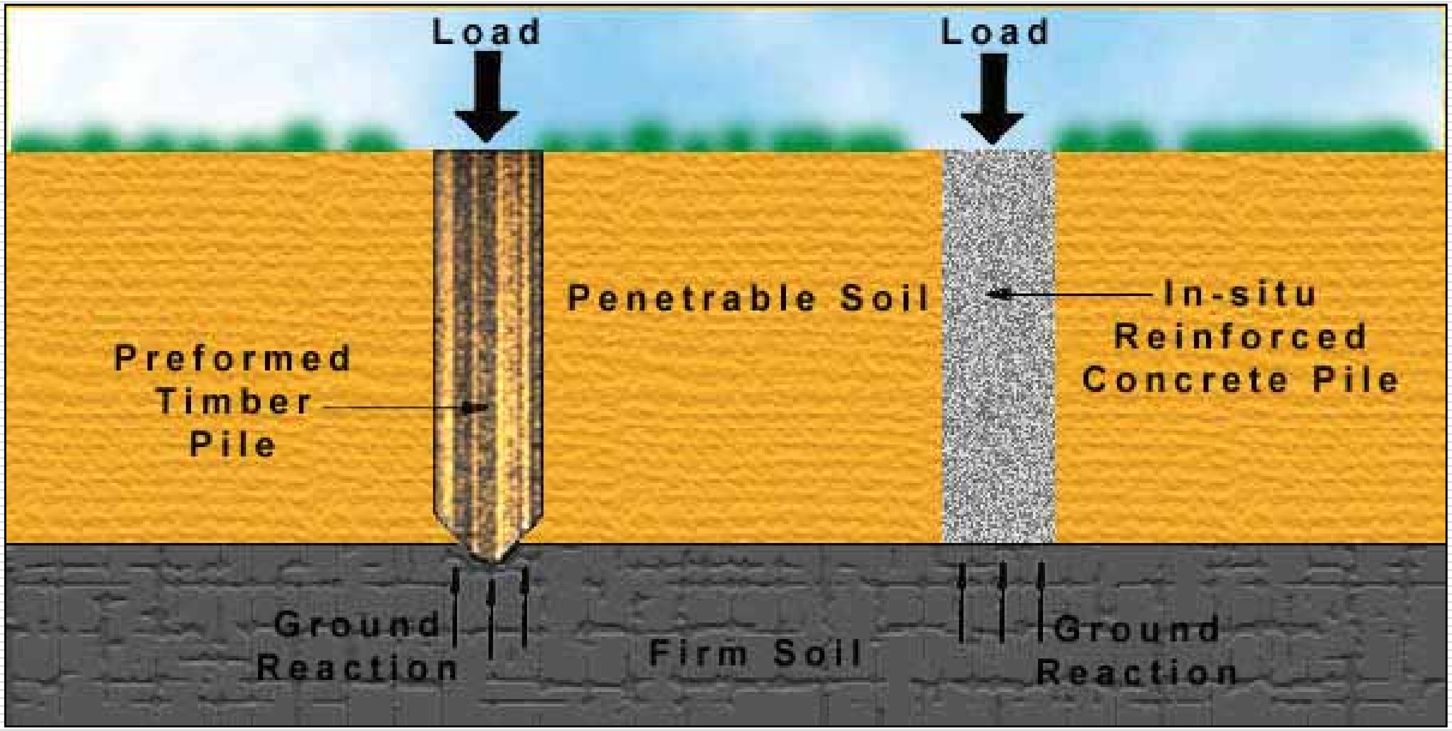


Timber Piles

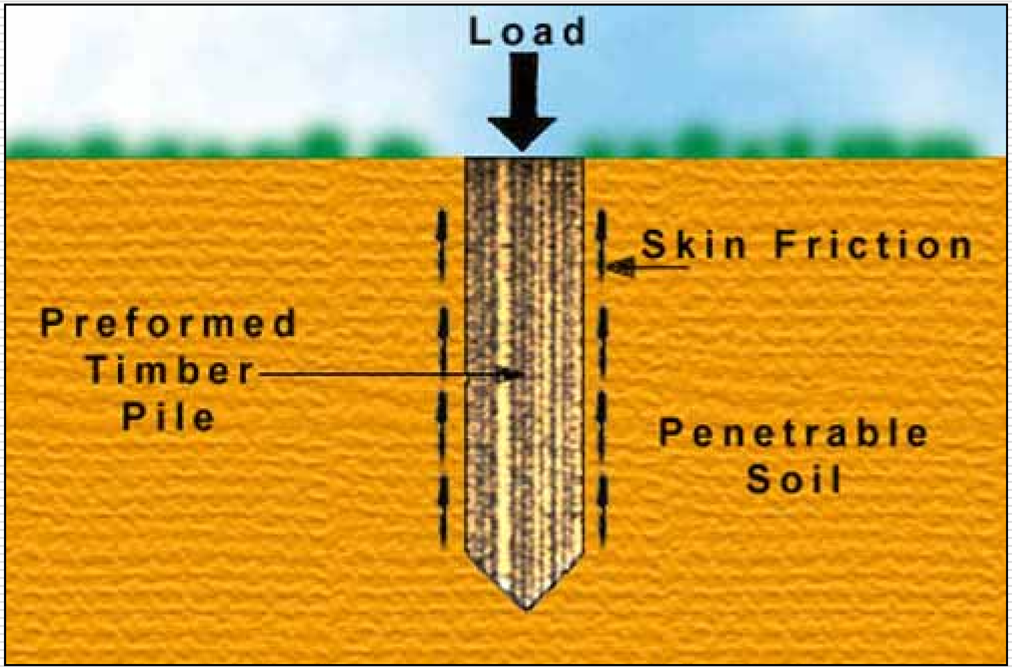
Cast-in-place concrete piles

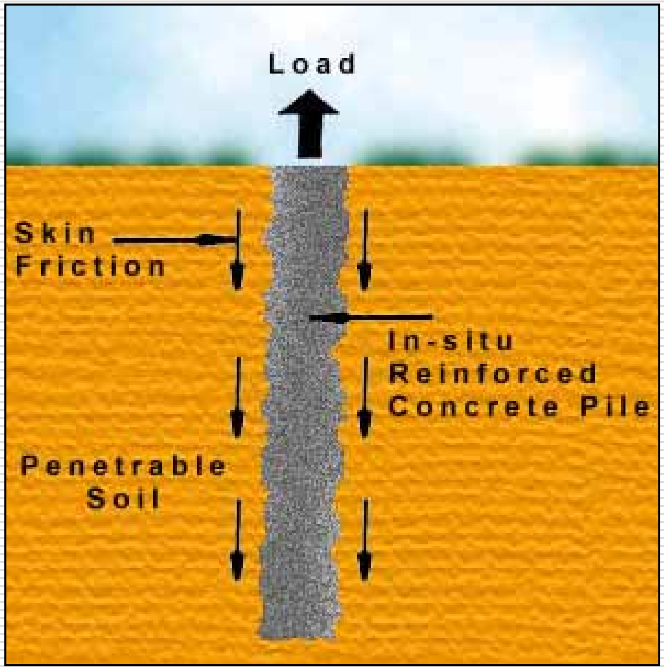


End Bearing Piles

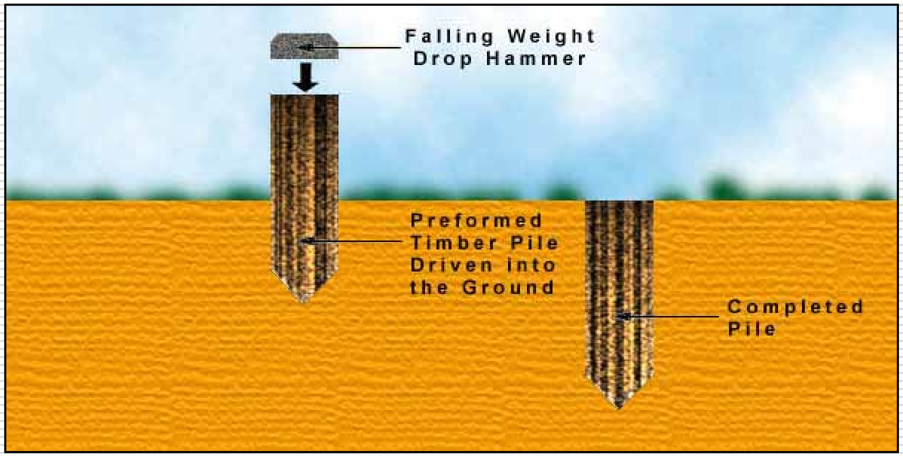


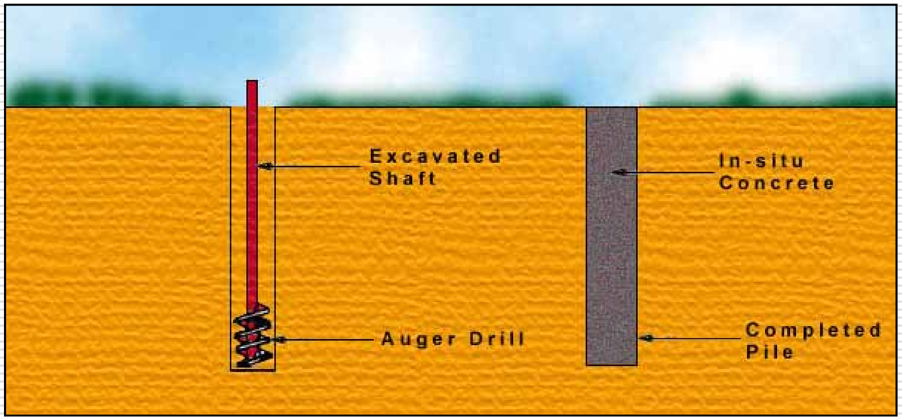
Friction Piles





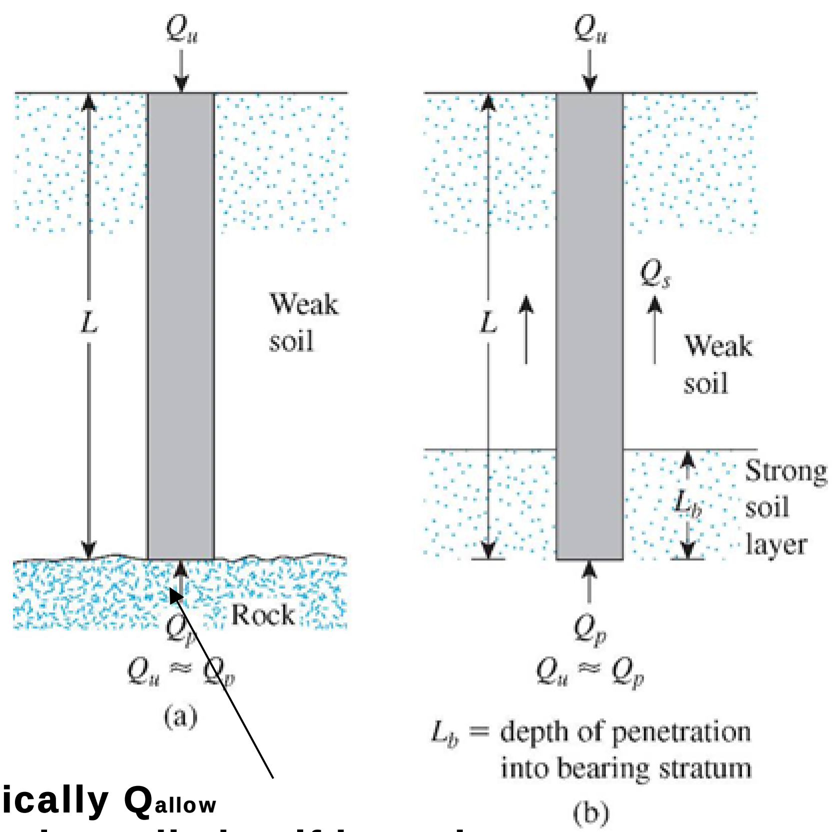
Displacement Versus Replacement





Tangent Wall Using Augercast

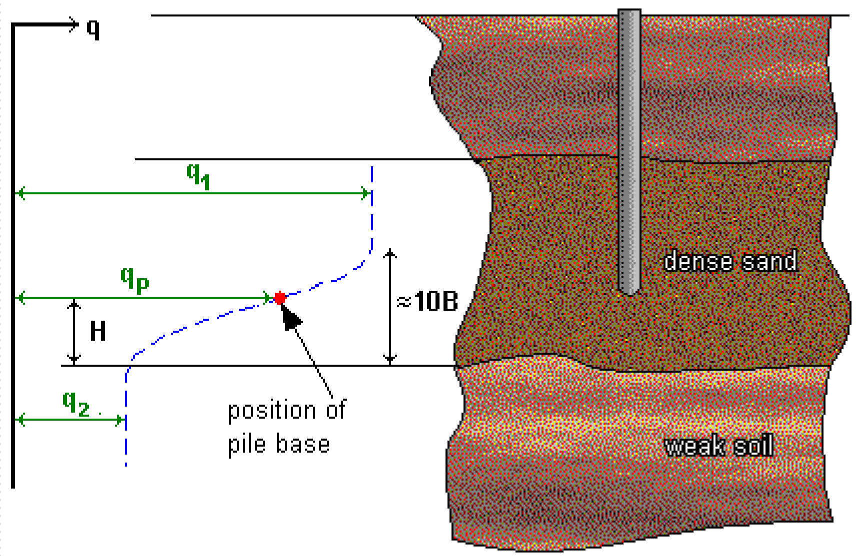
Point Bearing & Friction Piles



Typically Q_{allow}
Based on pile itself in rock

(a) and (b) Point bearing piles;
(c) friction piles

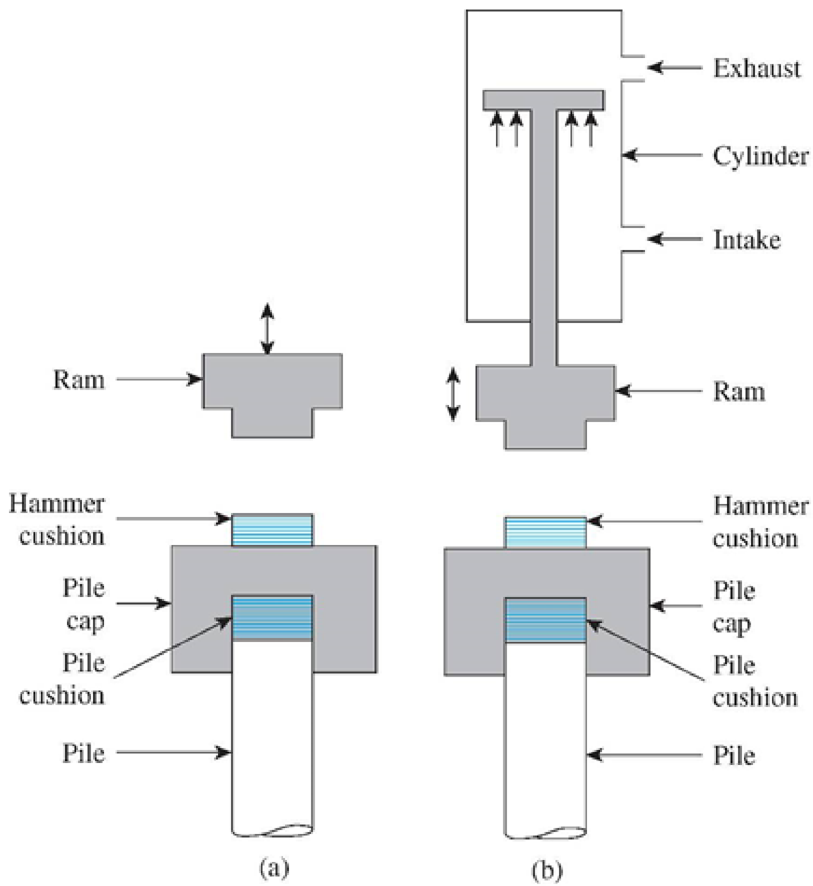
Weak Under Dense



Group Settlement Issues

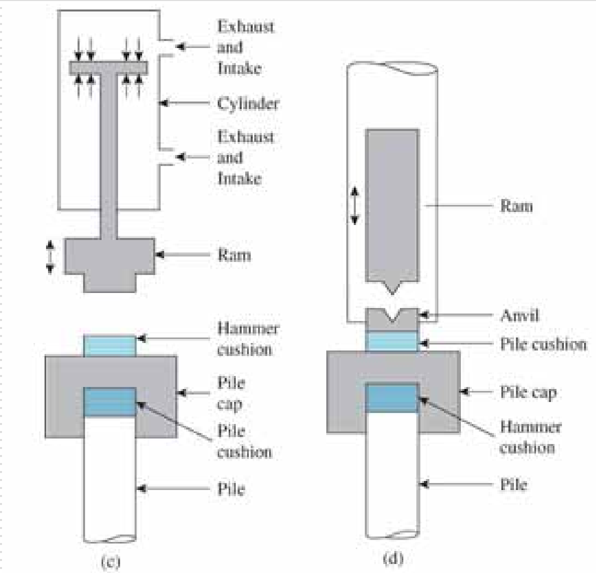
The base resistance at the pile toe is
$q_p = q_2 + (q_1 - q_2)H / 10B$ **but** $< q_1$
where B is the diameter of the pile, H is the thickness between the base of the pile and the top of the weaker layer, q_2 is the ultimate base resistance in the weak layer, q_1 is the ultimate base resistance in the strong layer.

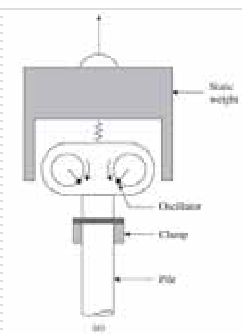
Pile-driving equipment



(a)drop hammer
(b)single-acting air or steam

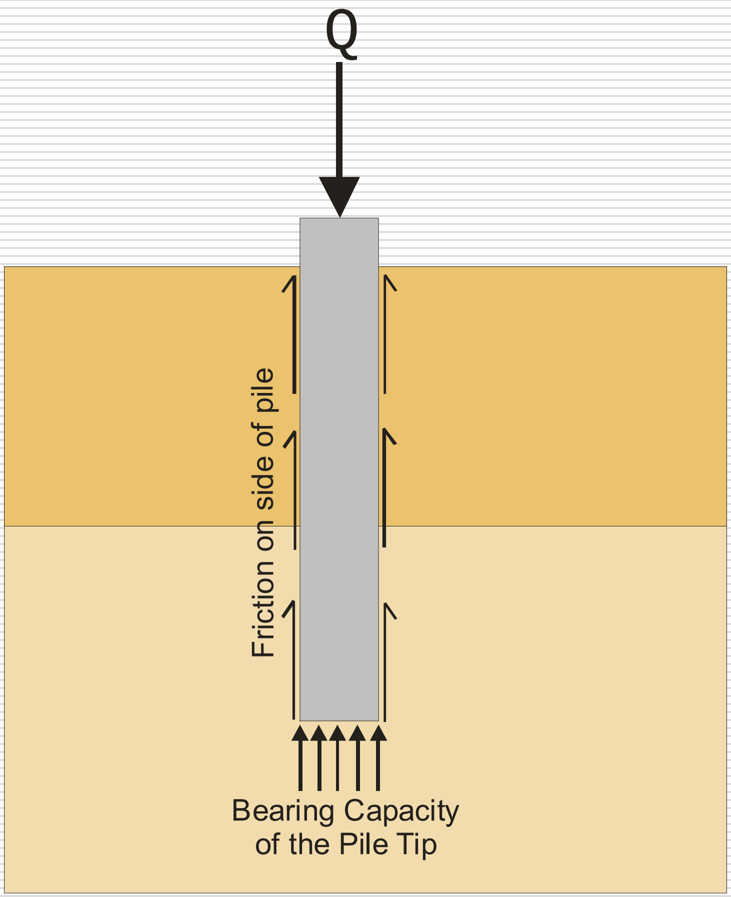
Pile-driving equipment



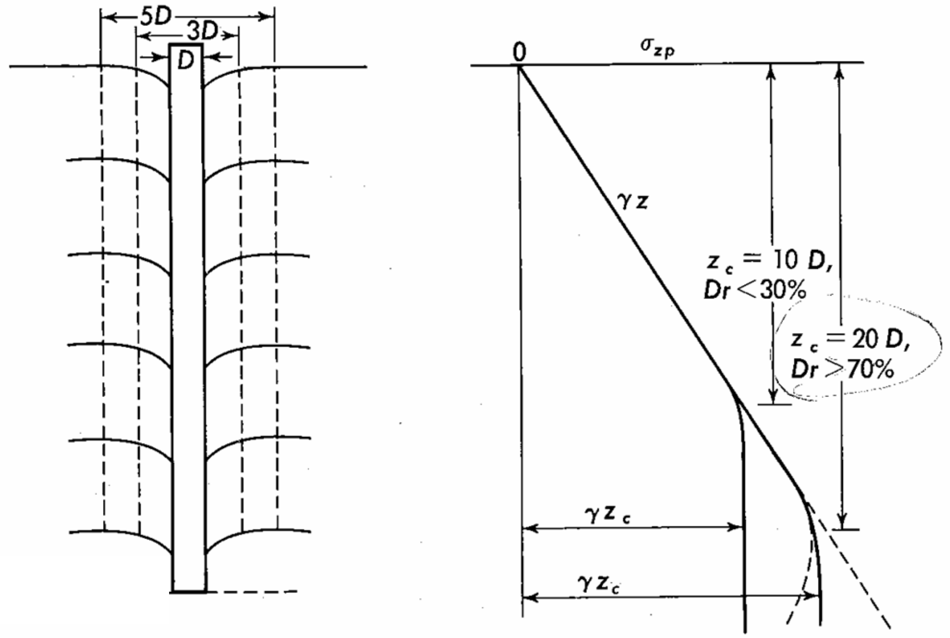


(c) double-acting and differential air or steam hammer
(d) diesel hammer
(e) vibratory pile driver

Pile Capacity Components



Stress Adjacent to Pile

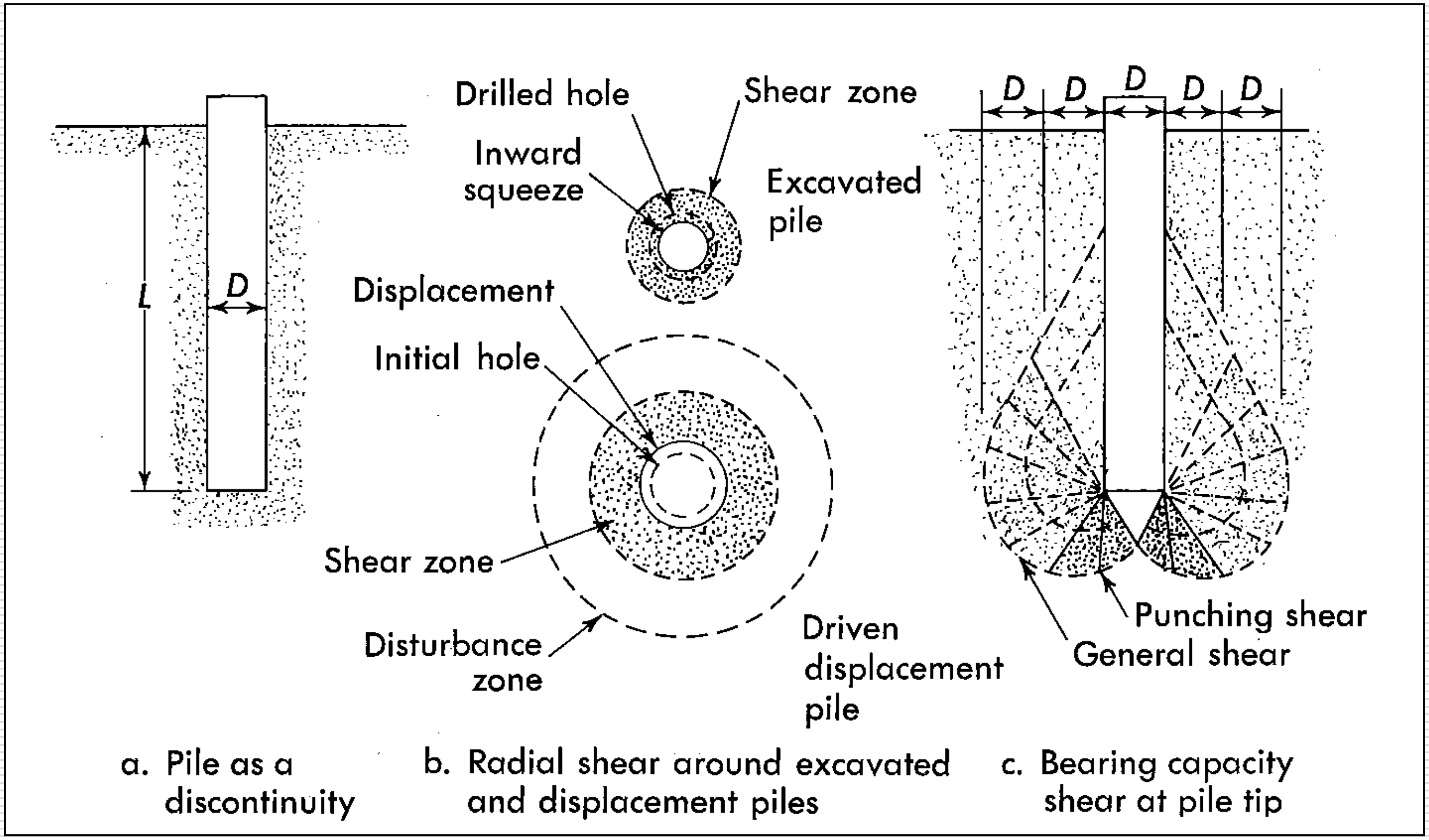


a. Distortion and zone of arching around a pile approaching failure

b. Vertical stress, σ_{zp}, adjacent to a loaded pile approaching failure

Vertical stress adjacent to a loaded pile. (Adapted from Vesic.[10:7]*)*

Pile Behavior



Zones Around a Driven Pile

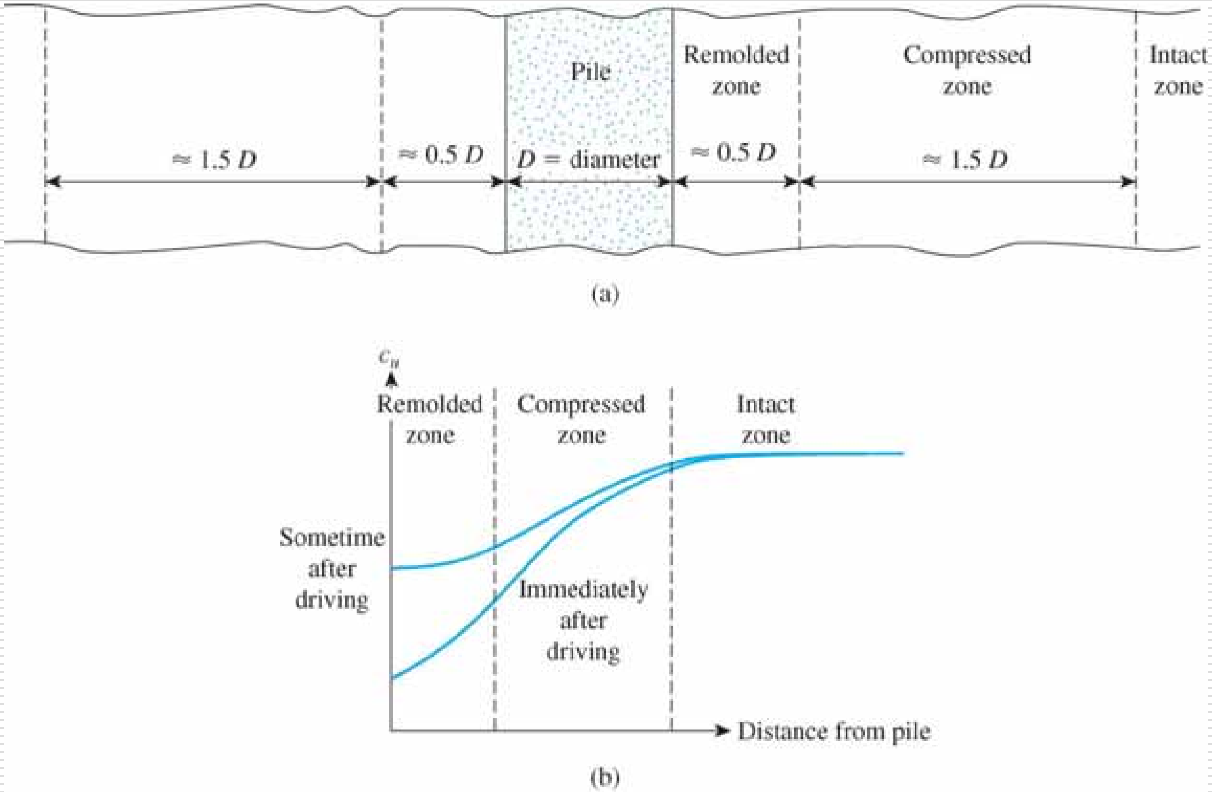


Figure 11.25 (a) Remolded or compacted zone around a pile driven into soft clay; (b) Nature of variation of undrained shear strength (c_u) with time around a pile driven into soft clay

Stresses Around Pile

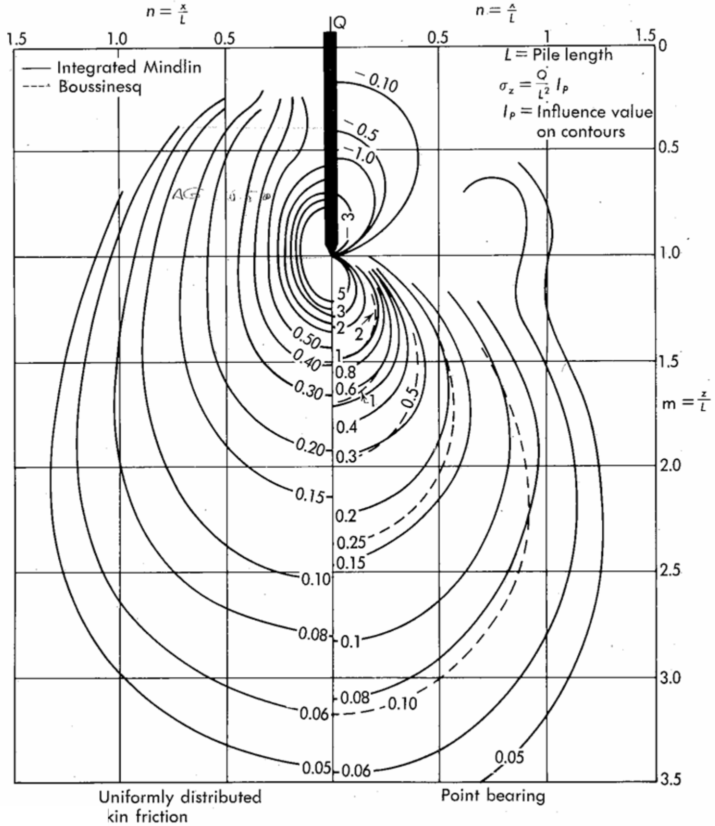
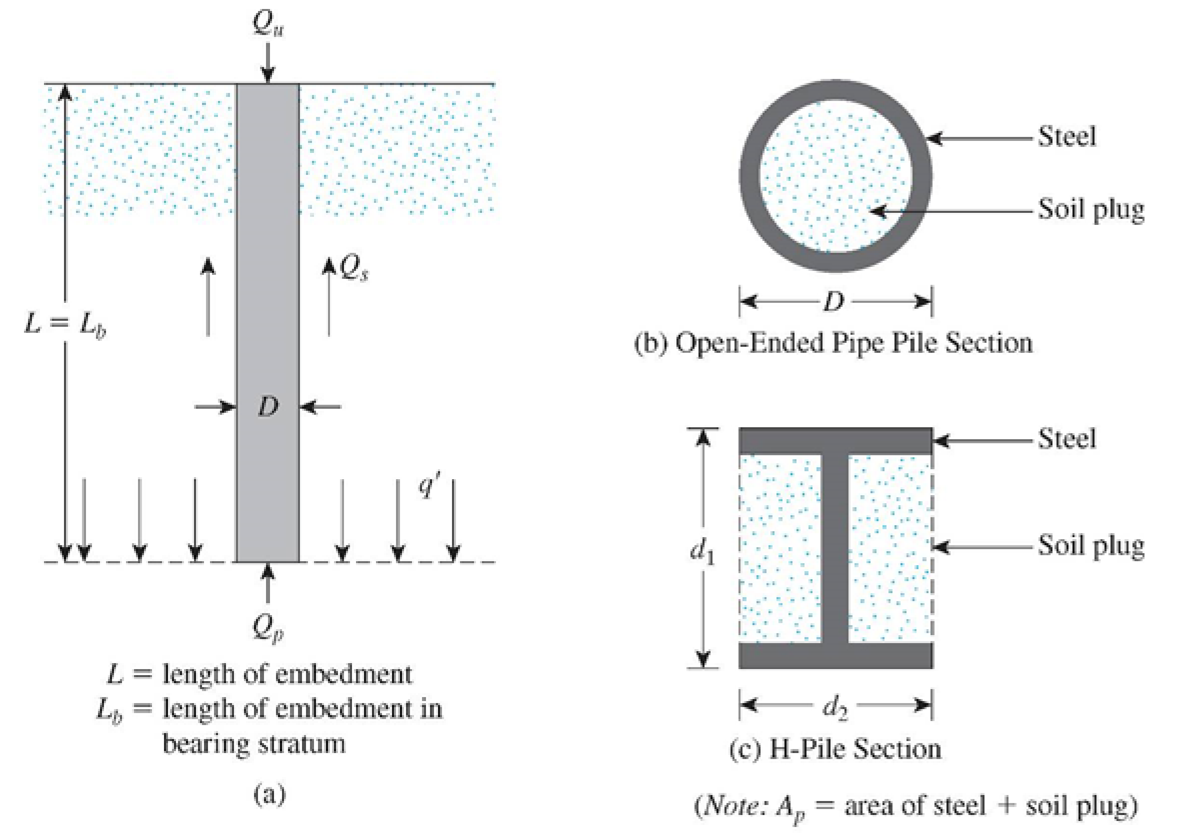
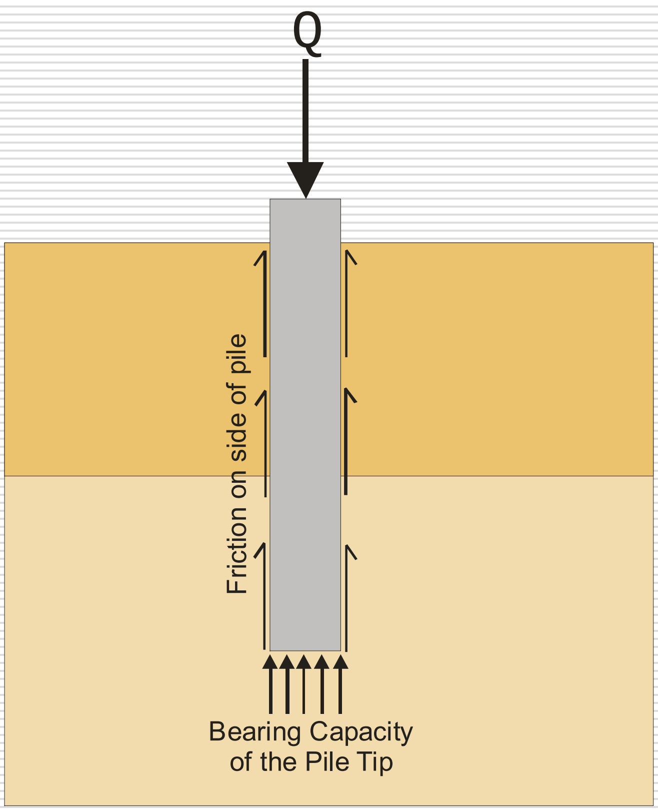


Figure 10.9 *Contours of equal stress around and below the point of a pile in a semi-infinite homogeneous elastic solid: The Mindlin analysis.* (*After O. Grillo.* 10:10)

Ultimate load-carrying capacity of a pile



Pile Capacity Design



$$Q_{ult} = Q_{point} + Q_{friction}$$

Many Methods

- As shown in the textbook, there are many methods for determining compressive and tensile pile capacity.
- Many of them are soil specific – either all clay or all sand. Rarely do you have that situations (off shore platforms)
- Many methods are testing specific – CPT, CPTu, SPT, laboratory test results, correlations with field testing, etc.
- We are going to limit the number of methods covered.
- One method – Sower's Method is not in the book, but allows layered soils with traditional c-ϕ soil types.

Point Resistance - Sowers

Limiting Values

$$q_u = q_p = c' \cdot N_c^* + q \cdot N_q^* + (\gamma \cdot B) / 2 \cdot N_\gamma^*$$

N_c^* = cohesion bearing capacity factor for piles
N_q^* = embedment bearing capacity factor for piles
N_γ^* = pile diameter bearing capacity factor for piles

$$Q_p = q_p * A_p$$

Limiting Values for Overburden

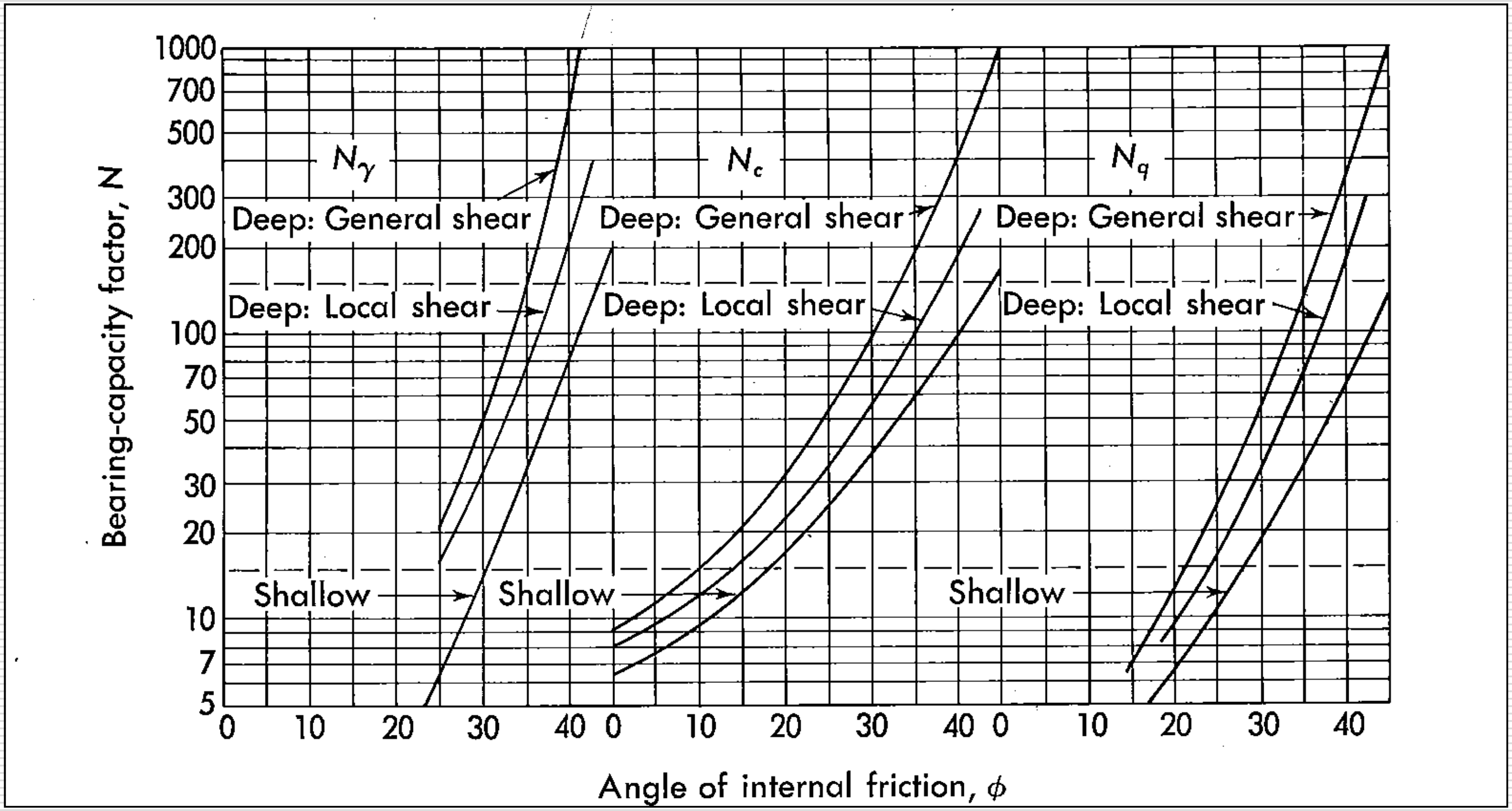
Use lower of these values

$\tan \phi \cdot N_q^*$ (kips/sq.ft.) ← **Typically lowest**

$\sigma'_o \cdot N_q^*$

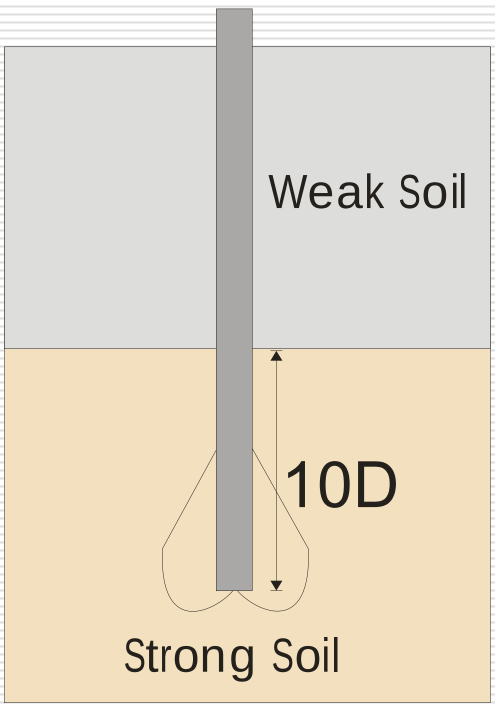
$\sigma'_{crit} \cdot N_q^*$

Sowers Pile Capacity Factors

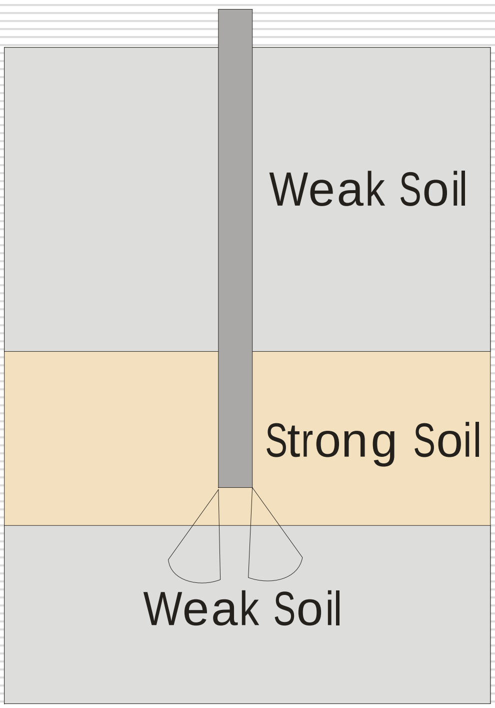


General, Local, or Shallow?

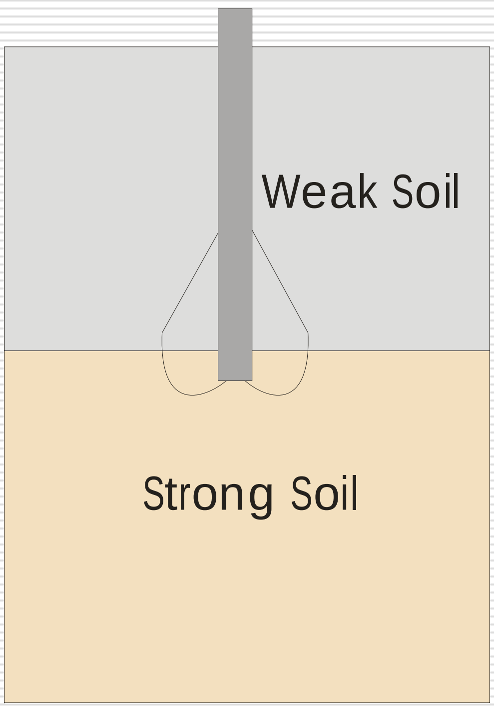
General Shear



Deep Local Shear



Shallow Shear

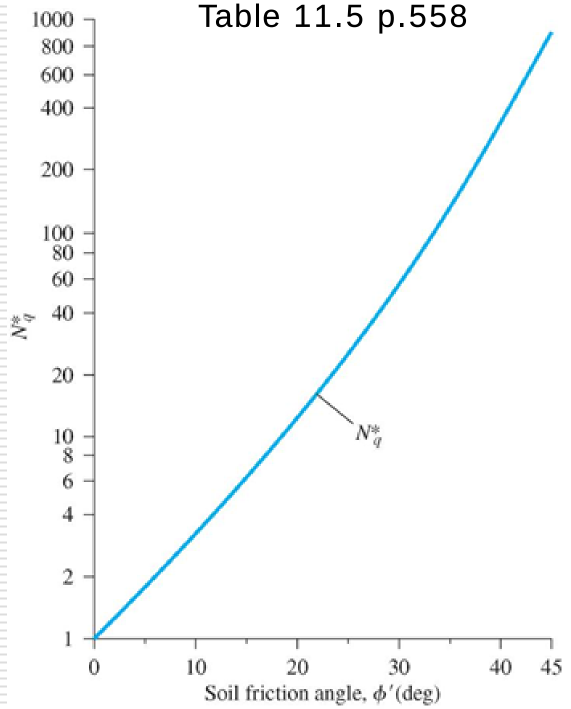


also short pile

Meyerhof Methods

For sands

$$q_p = \sigma'_o \cdot N_q^* =< 0.5 \cdot p_a \cdot N_q^* \cdot \tan\phi'$$



For saturated clays

$$q_p = 9c_u$$

$$Q_p = q_p \cdot A_p$$

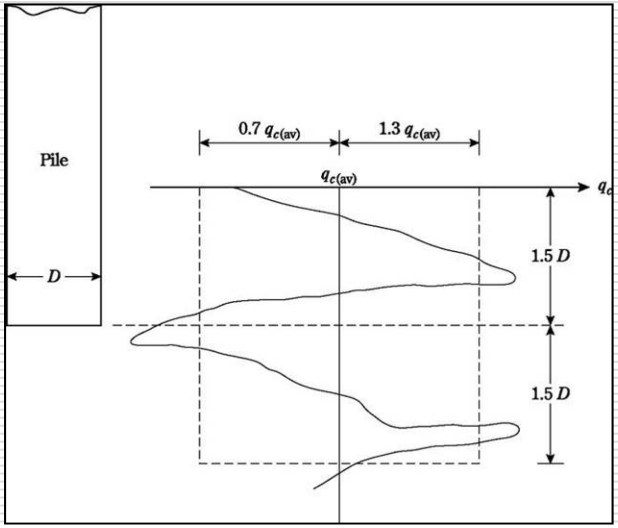
SPT Method

$$q_p = 0.4p_a(N1)_{60}\ L/D =< 4p_a(N1)_{60}$$

where p_a is atmospheric pressure – 1 tsf

CPT Method

$$Q_p = q_{c(eq)} \cdot k_b$$



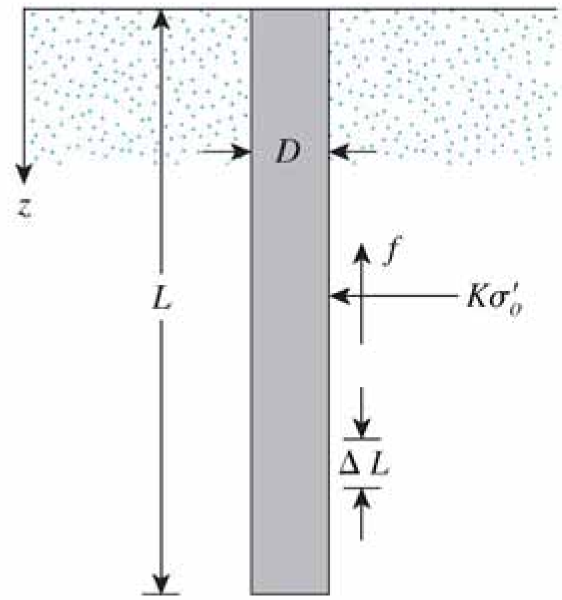
1 - Use q_c 1.5D above & 1.5 D below pile tip
2 - Calculate average q_c
3 - Eliminate qc values higher than $1.3qc_{(avg)}$ & values that are lower than 0.7 $q_{c(avg)}$
4 - Recalculate $q_{c(avg)}$

k_b = 0.6 for clays and silts
= 0.375 for sands and gravels

Frictional Resistance

$$Q_s = \Sigma(p \cdot \Delta L \cdot f)$$

where p = perimeter of the pile
ΔL = incremental pile length over which p and *f* are taken
f = unit friction resistance



Sowers Equations

Frictional Capacity

$f = c' + \sigma'_h \cdot \tan\phi'$ **Augercast Piles or Drilled Shafts**

or

$f = c_a + \sigma'_h \cdot \tan\delta$ **Timber, Steel and Concrete Piles**

$c_a = 0.9c'$ **when** $c' < 1$ **ksf**

$c_a = 0.9 + 0.3(c' - 1)$ **when** $c' > 1$ **ksf**

$\sigma'_h = K_s \cdot \sigma'_o$ **with** $\sigma'_o <= \gamma_{sat} \cdot z_{crit}$ 10 to 20 D

Sowers Frictional Factors

COEFFICIENT OF LATERAL EARTH PRESSURE IN COHESIONLESS SOILS ADJACENT TO PILE AT FAILURE		
Soil	**Displacement Condition**	**Ks**
Loose Sand	Jetted Pile	0.5 to 0.75
Dr<30%	Drilled Pile	0.75 to 1.5
	Driven Pile	2 to 3
Dense Sand	Jetted Pile	0.5 to 1
Dr>70%	Drilled Pile	1 to 2
	Driven Pile	3 to 5

COEFFICIENT OF FRICTION, COHESIONLESS SOILS AGAINST PILES AND SIMILAR STRUCTURES		
Material	**Coefficient of Friction $\tan\delta$**	**δ (deg)**
Wood	0.4	22
Rough Concrete Cast Against Soil	$\tan\phi$	ϕ
Smooth Formed Concrete	0.3 to 0.4	17
Clean Steel	0.2	11
Rusted Steel	0.4	22
Corrugated Metal	$\tan\phi$	ϕ

Sowers Method

PROJECT INFORMATION	PILE INFORMATION	BORING INFORMATION
Project Name : ______	Pile Type: ______	Boring : ______
Project #: ______	Pile Shape: ______	Critical Depth : ______ Pile D
	Installation Method : ______	Groundwater Depth : ______ feet
alculations By : ______	Top Dimension: ______ Inches	P'critical : ______ ksf
Date : ______	Bottom Dimension: ______ Inches	
	Pile Length: ______ Feet	**SAFETY FACTOR**
Checked By : ______	Skin Friction - tan δ: ______	
ate Checked : ______	Tip Area : ______ Square Feet	Safety Factor : ______
	Average Pile Dimension : ______ Inches	

CAPACITY CALCULATIONS

STRATA DEPTHS (FEET)		Cohesion	φ	Ks	Zavg.	P'o	P'o OR P'crit.	Tan φ or	Total Shear	Pile Area	Skin Friction	Downdrag		Remarks
Top	Bottom	(ksf)			(feet)	(ksf)	(ksf)	Tan δ	(ksf)	(Sq. Ft.)	(kips)	Yes	Kips	
END BEARING	C & PHI =												*Total Downdrag*	

Skin Friction (less downdrag) .... kips
tons

End Bearing
Local .... tons
General .... tons
Shallow .... tons

BEARING CAPACITY FACTORS

Factor	General Shear	Local Shear	Shallow Shear
Ng			
Nc			
Nq			

% Friction	*% End Bearing*

Total Pile Capacity - Local .... tons
Total Pile Capacity - General .... tons
Total Pile Capacity - Shallow .... tons
Allowable Pile Capacity - Local .... tons
Allowable Pile Capacity - General .... tons
Allowable Pile Capacity - Shallow .... tons
Allowable Uplift Capacity .... tons

Notes:
Cohesion values automatically reduced by 33% in drilled piles
Uplift = Full skin friction / F.S. excluding downdrag

$$q_p = c' \bullet N_c^* + q \bullet N_q^* + q \bullet D \bullet N_\gamma^*$$

Remember limiting factors for q•Nq

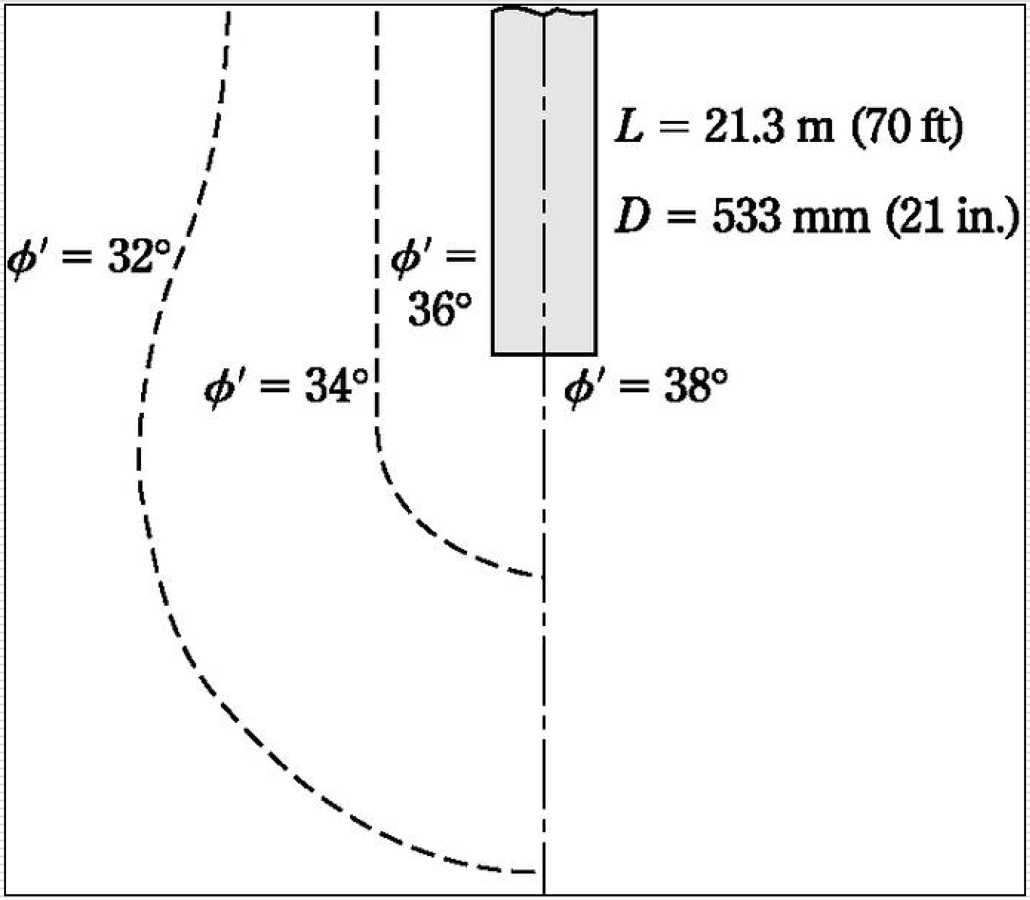
Note About Pile Weight

- Pile weight should be subtracted from Q_u especially for very large piles.

- Pile weight should be added to uplift capacity.

Compaction of sand near driven piles

(after Meyerhof, 1961)

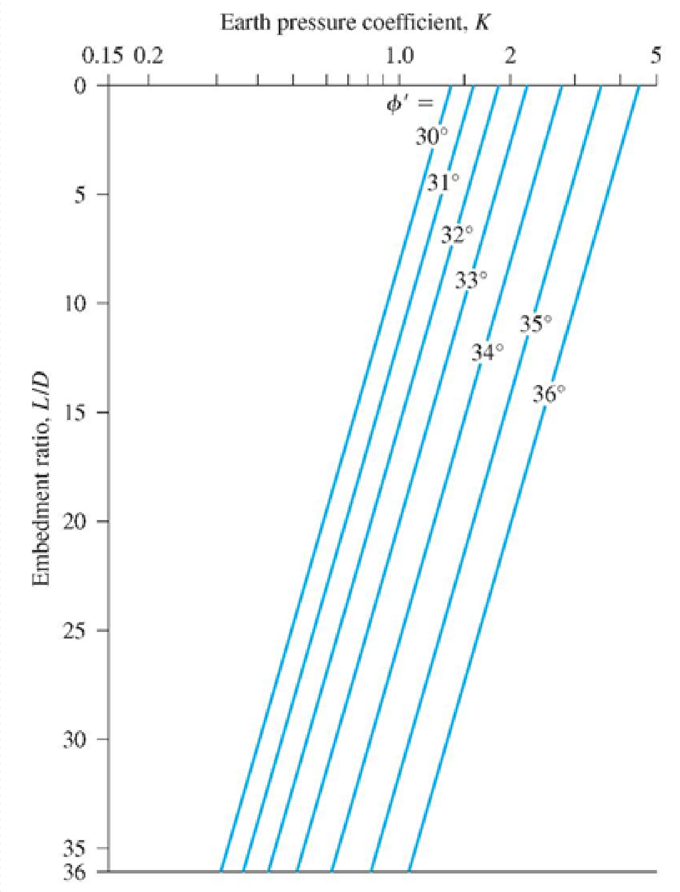


Valid only for displacement piles

Lateral Earth Pressure Method

Sands Only

$$Q_s = \sum K \cdot \sigma'_o \cdot \tan(0.8\phi') \cdot p \cdot \Delta L$$



Meyerhof SPT Method

High Displacement Piles

$$f_{av} = 0.02p_a(N_1)_{60}$$

where p_a = atmospheric pressure = 1 tsf

Low Displacement Piles

$$f_{av} = 0.01p_a(N_1)_{60}$$

$$Qs = \Sigma f_{av} \cdot p \cdot \Delta L$$

λ method - Clays

$$f_{av} = \lambda(\overline{\sigma'_o} + 2c_u)$$

$$Q_s = f_{av} \cdot p \cdot L$$

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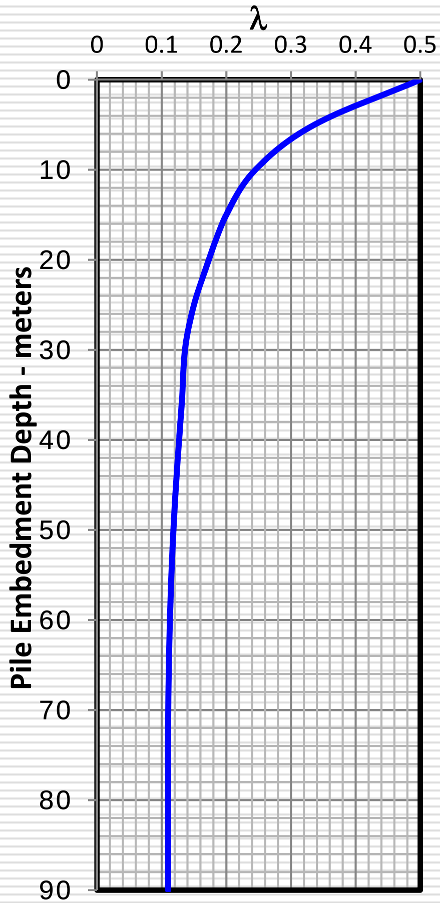
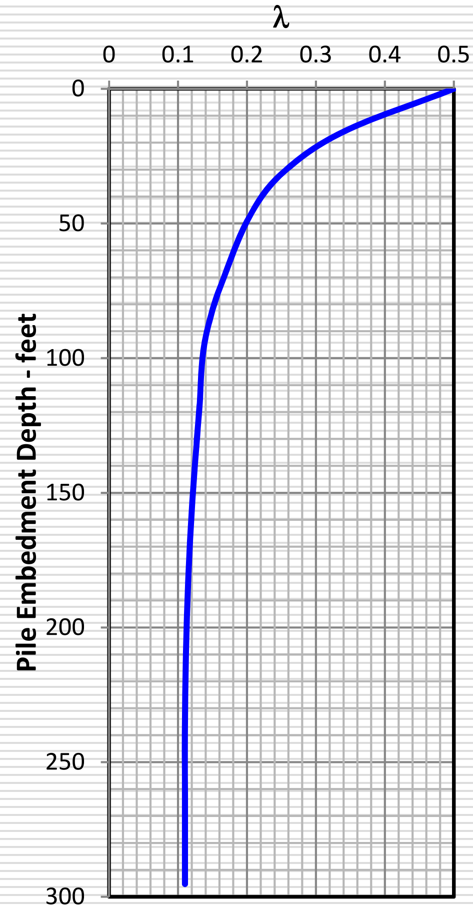
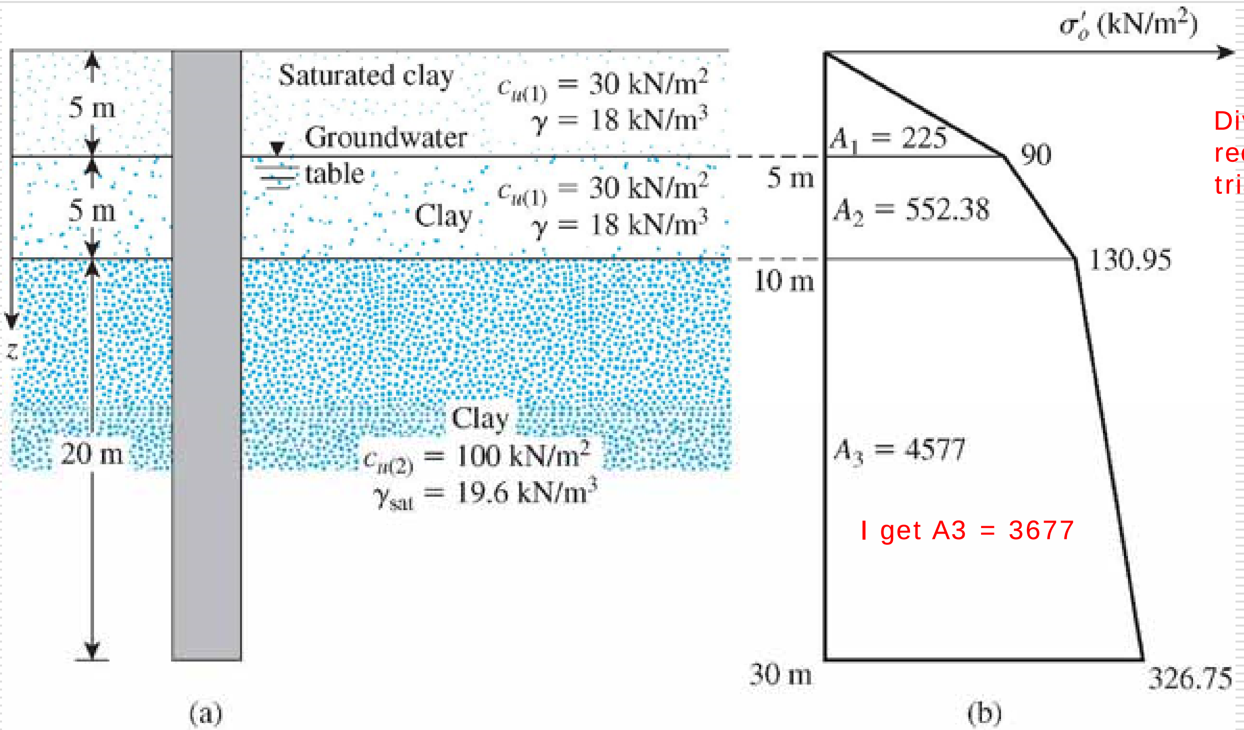
Drilling Platforms





λ Method Example



Divide A_2 and A_3 into a rectangle and a triangle

I get A3 = 3677

Figure 11.22 Estimation of the load bearing capacity of a driven-pipe pile

λ Example Continued

Table 11.9 Variation of λ with pile embedment length, L

Embedment length, L (m)	λ
0	0.5
5	0.336
10	0.245
15	0.200
20	0.173
25	0.150
30	0.136
35	0.132
40	0.127
50	0.118
60	0.113
70	0.110
80	0.110
90	0.110

Table 11.10 Variation of α (interpolated values based on Terzaghi, Peck and Mesri, 1996)

$\frac{c_u}{p_a}$	α
$\leq$ 0.1	1.00
0.2	0.92
0.3	0.82
0.4	0.74
0.6	0.62
0.8	0.54
1.0	0.48
1.2	0.42
1.4	0.40
1.6	0.38
1.8	0.36
2.0	0.35
2.4	0.34
2.8	0.34

Note: p_a = atmospheric pressure $\approx$ 100 kN/m^2 or 2000 lb/ft^2

λ Example Continued

(1) From Eq. (11.55),

$$Q_s = \Sigma \alpha c_u p \Delta L$$

[*Note*: $p = \pi(0.406) = 1.275$m] Now the following table can be prepared.

Depth (m)	ΔL (m)	c_u (kN/m^2)	α (Table 11.10)	$\alpha c_u p \Delta L$ (kN)
0–5	5	30	0.82	156.83
5–10	5	30	0.82	156.83
10–30	20	100	0.48	1224.0
				$Q_s \approx$ **1538 kN**

(2) From Eq. 11.51, $f_{av} = \lambda(\overline{\sigma}'_o + 2c_u)$. Now, the average value of c_u is

$$\frac{c_{u(1)}(10) + c_{u(2)}(20)}{30} = \frac{(30)(10) + (100)(20)}{30} = 76.7 \text{ kN/m}^2$$

To obtain the average value of $\overline{\sigma}'_o$, the diagram for vertical effective stress variation with depth is plotted in Figure 11.22b. From Eq. (11.52),

3677 148.48

$$\overline{\sigma}'_o = \frac{A_1 + A_2 + A_3}{L} = \frac{225 + 552.38 + 4577}{30} = 178.48 \text{ kN/m}^2$$

From Table 11.9, the magnitude of λ is 0.136. So

148.48 41.1

$$f_{av} = 0.136[178.48 + (2)(76.7)] = 45.14 \text{ kN/m}^2$$

Hence,

41.1 1572.6

$$Q_s = pLf_{av} = \pi(0.406)(30)(45.14) = \mathbf{1727\ kN}$$

Rock

$$q_p = q_u(N_\phi + 1)$$

where $N_\phi = \tan^2(45 + \phi'/2)$

q_u = unconfined compressive strength of rock

$$q_{u(design)} = q_{u(lab)} / 5$$

$$Q_p = q_p \cdot A_p$$

ROCK QUALITY[‡]

RQD (%)	DIAGNOSTIC DESCRIPTION	ROCK PARAMETER FIELD/LAB RATIO
0 - 25	Very Poor	0.15
25 - 50	Poor	0.20
50 - 75	Fair	0.25
75 - 90	Good	0.30 to 0.70
90 - 100	Excellent	0.70 to 1.00

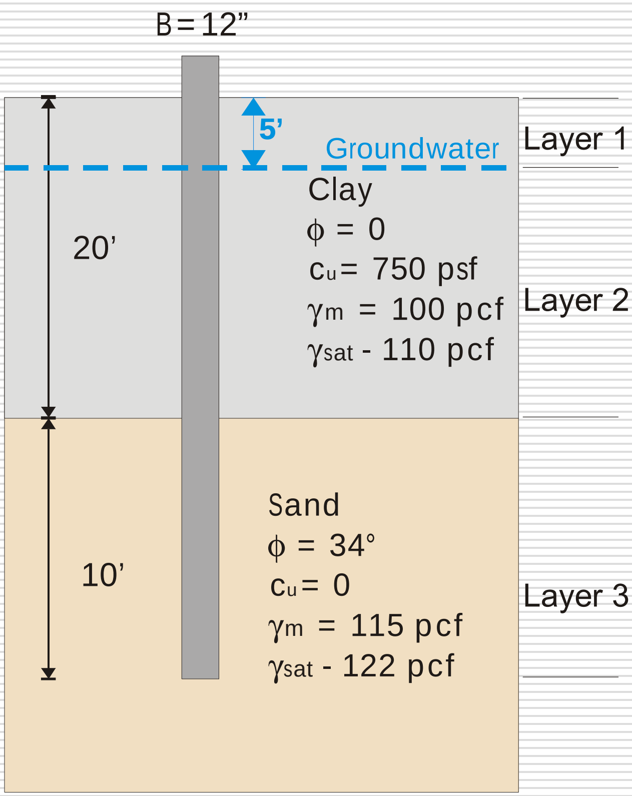
Rock Strength

Type of rock	q_u MN/m^2	q_u lb/in^2
Sandstone	70–140	10,000–20,000
Limestone	105–210	15,000–30,000
Shale	35–70	5000–10,000
Granite	140–210	20,000–30,000
Marble	60–70	8500–10,000

Type of rock	Angle of friction, ϕ' (deg)
Sandstone	27–45
Limestone	30–40
Shale	10–20
Granite	40–50
Marble	25–30

Problem

Square Concrete Pile



Method Sowers

Z_{crit} = 20D = 20(12/12) = 20 feet σ'_{crit} = (5(100)+15(110-62.4))/1000 = 1.21 ksf

For clay, c_a = 0.9c' for c' < 1ksf so c_a = 0.9(0.75 ksf) = 0.675 ksf

σ'_o @ center of Layer 1 = 2.5(100)/1000 = 0.25 ksf

σ'_o @ center of Layer 2 = (5(100)+7.5(110-62.4))/1000 = 0.86 ksf

σ'_o @ center of Layer 3 = (5(100)+15(110-62.4)+5(122-62.4))/1000 = 1.51 ksf

> σ'_{crit} , so use σ'_{crit} for Layer 3 or 1.21 ksf

Friction Resistance – $c_a + \sigma'_H \cdot \tan\delta$ – for smooth concrete use 0.3 for $\tan\delta$

$\sigma'_H = Ks \cdot \sigma'_o$ unless $\sigma'_o > \sigma'_{crit}$, then $\sigma'_H = Ks \cdot \sigma'_{crit}$

Layer 1 & 2 friction (f) = c_a = 0.675 ksf (0.9•Cu)

Layer 3 friction (f) = $\sigma'_H \cdot \tan\delta = \sigma'_{crit}(Ks)(\tan\delta)$ – Use Ks = 3 from Table

σ'_H = (1.21)(3)(0.3) = 1.09 ksf

$Q_f = \Sigma\Delta L(p)(f)$ = 20(12/12)(4)(0.675) = 54 kips for clay

= 10(12/12)(4)(1.09) = 43.6 kips for sand

Total Q_f = 54 + 43.6 = 97.6 kips

Point Resistance

$q_u = q_p = c' \cdot Nc^* + q \cdot Nq^* + (\gamma \cdot B)/2 \cdot N\gamma^*$ (c'•Nc* → 0) $N\gamma^* = Nq^* = 130$

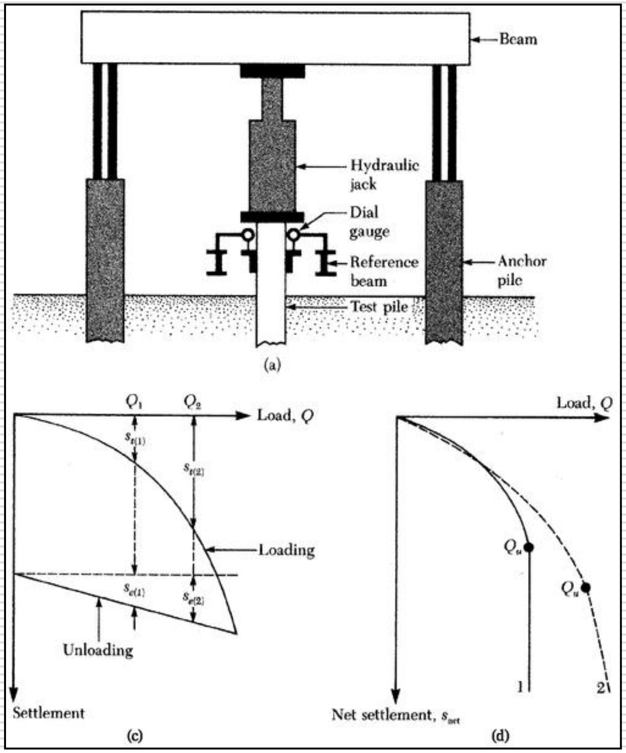
$q \cdot Nq^* = \tan\phi \cdot Nq^*$ = 87.7 ksf (limiting value)

$(\gamma' \cdot B)/2 \cdot N\gamma^*$ = ((122-62.4)(12/12)/2(130))/1000 = 3.9 ksf (could be ignored)

q_u = 87.7 + 3.9 = 91.6 ksf

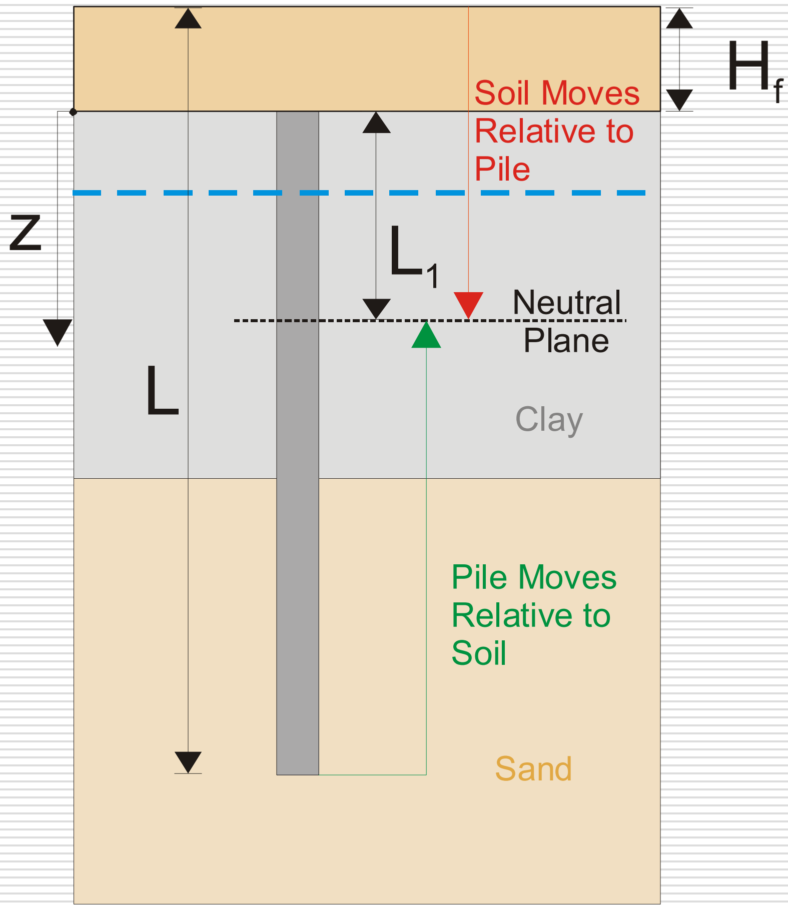
$Q_p = q_u \cdot A_p = 91.6(12/12)^2$ = 91.6 kips Qtotal = Qf + Qp = 97.6 + 91.6 = 189.2 kips

Pile Load Testing



(a) Schematic diagram of pile load test arrangement
(c) plot of load against total settlement
(d) plot of load against net settlement

Negative skin friction (Downdrag)



There will not be negative skin friction (downdrag) unless the soil moves more than the pile.

Negative skin friction

$$L_1 = \frac{(L - H_f)}{L_1} \cdot \left[\frac{(L - H_f)}{2} + \frac{(\gamma_f \cdot H_f)}{\gamma'} \right] - \frac{(2 \cdot \gamma_f \cdot H_f)}{\gamma'}$$

Solver in Excel

$$Q_d = \sum_{z=0}^{L_1} (p \cdot \Delta L \cdot f)$$

where p = perimeter of the pile

ΔL = incremental pile length over which p and f are taken

f **= unit friction resistance**

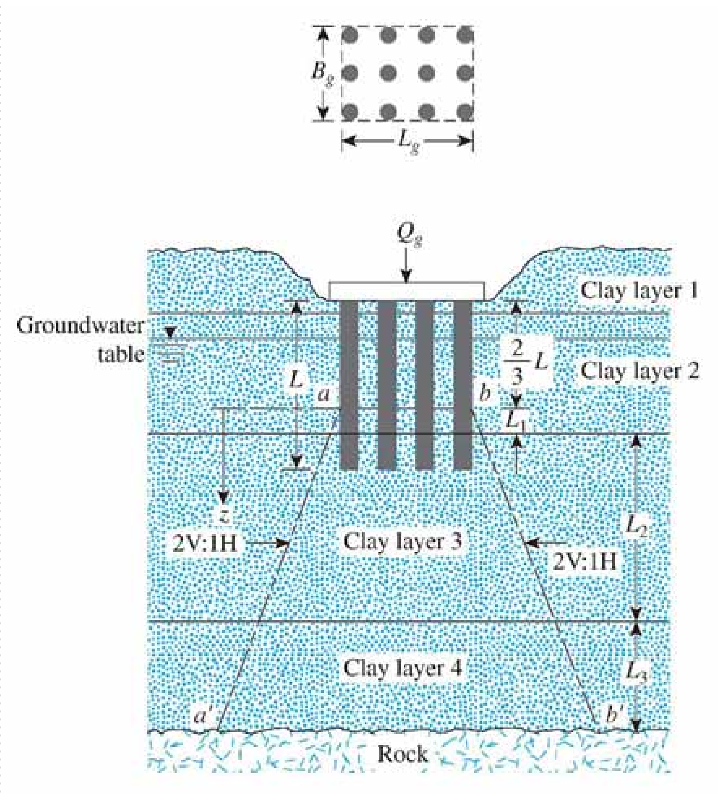
H_f = height of fill

γf = unit weight of fill

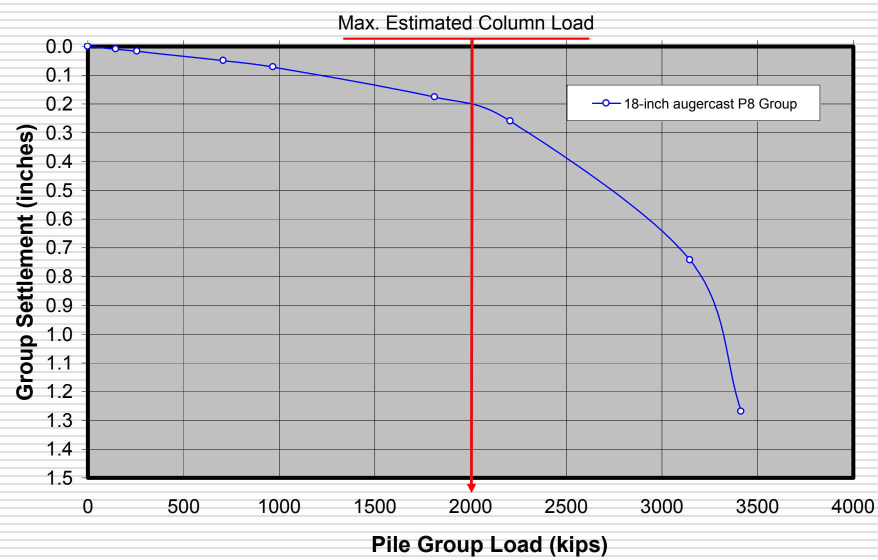
γ' = bouyany unit weight

L_1 – depth to neutral point

Consolidation settlement of group piles



Group Efficiency is 100% when pile to pile spacing is >= 3D



Homework

CE 430 & 530

- **11.1a**
- **11.8**
- **Calculate pile capacity shown below with 4 feet of sand fill creating negative skin friction for problem in figure below using Sowers.**