SOIL MECHANICS II
CIVL 311
COURSE NOTES
2012

Module 8 – Segment B
Design of Deep Foundations

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Module 8

Design of Deep Foundations

Overall Learning Objectives

- Introduction – Deep Foundations (Book 2 pp. 274 - 280)
- Ultimate Limit State – load capacity of single piles (Book 2 pp. 280 – 297, 313 - 316)
- Serviceability Limit State – settlement of piles and pile groups (Book 2 pp. 298 – 313)
- Lateral Loading of Piles (Book 2 pp. 317 – 323)
- Design Issues and Procedures (Book 2 pp. 324 – 332)

Factors Leading to the Selection of Pile Foundations

- Near-surface soils are not competent
- Shallow foundation settlements excessive
- Potential for high differential settlements
- Uplift/lateral Loads
- Removal of upper soils not cost-effective

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Load Capacity of Single Piles

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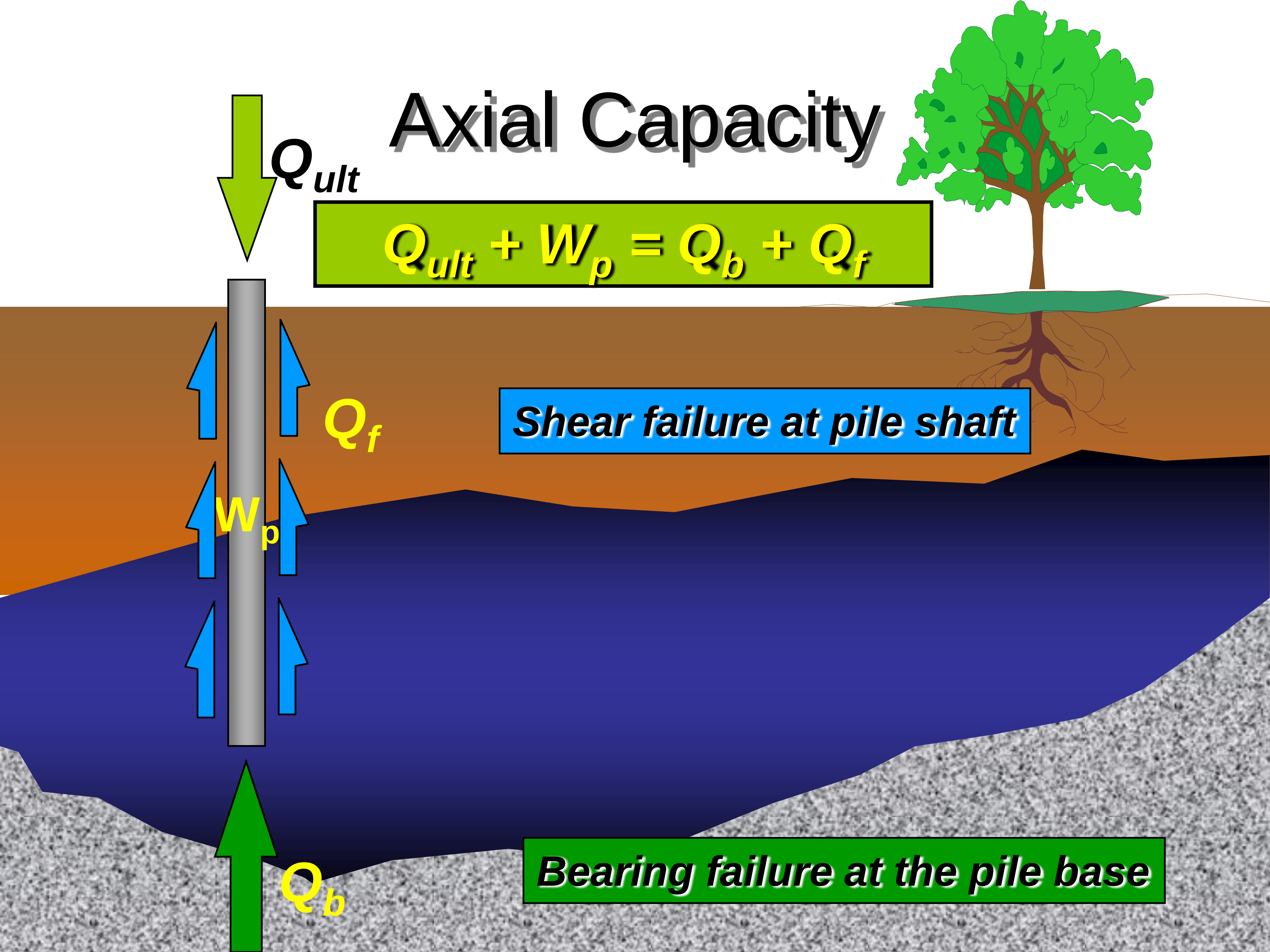
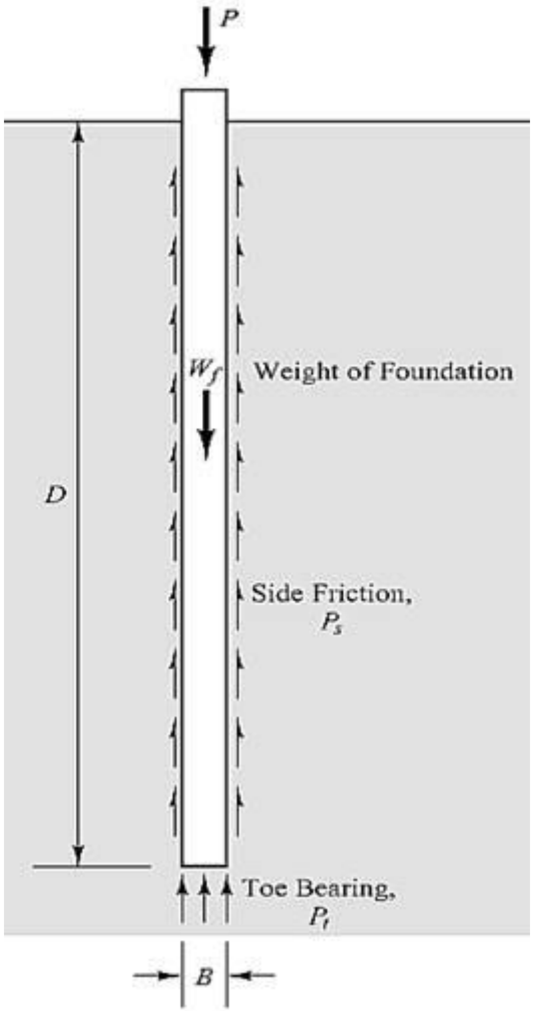
Axial Capacity

$$Q_{ult} + W_p = Q_b + Q_f$$



Axial Pile Load Transfer



$$Q_{ult} = Q_f + Q_b - W_p$$

$$Q_a = \frac{Q_{ult}}{FS}$$

Q_{ult} = Ultimate load capacity

Q_f = Shaft resistance ("skin friction")

Q_a = Allowable load capacity

Q_b = End bearing

W_p = Weight of pile

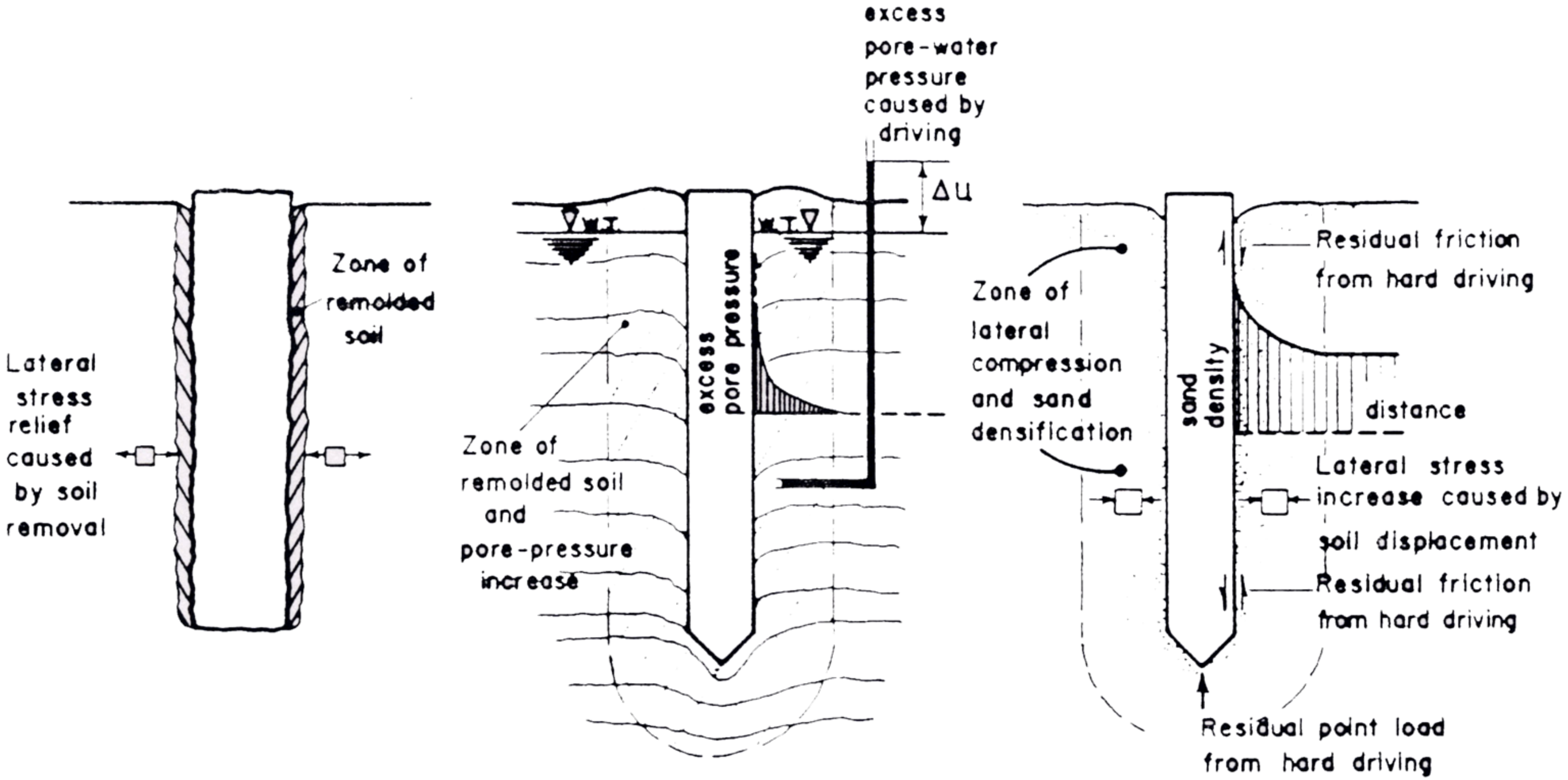
FS = Factor of safety

<u>Static Pile Analysis</u>

$Q_f > \sim 0.8\ Q_b$ → Friction Pile

$Q_f < \sim 0.8\ Q_b$ → End Bearing Pile

Pile Installation Effect on Soil Mass



(A)

BORED PILE

(B)

DRIVEN PILE - CLAY

(C)

DRIVEN PILE - SAND

Some Salient Points

- Displacement to mobilize full skin friction
  - 2.5 mm to 10 mm (independent of pile length/dia.)
- Displacement to mobilize end bearing
  - 8% to 10% of pile tip diameter
- Due to pile driving:
  - Loose coarse-grained soils (sands) could get dense
  - Dense coarse-grained soils (sands) could become loose
  - Fine-grained soils would get remolded (e.g., NC fine-grained soils could experience significant increase in pore water pressure)
  - Changes in density and effective stress → shear strength of soils could get changed

Approaches to design

- **Presumed values**
  - **Timber pile** Allowable load 180 to 200 kN
  - **Franki Pile** (PIF) - 750 kN for 400 mm dia., 1100 kN for 500mm diameter
- **Static methods**
  - **Indirect or analytical approach -** effective stress or total stress methods (α, β methods)
  - **Direct approach -**from CPT (LCPC) or N-value (Meyerhof)
- **Dynamic methods**
- **Load Testing**

Pile Capacity in fine-grained saturated soils – Shaft Friction

$$f_s = \alpha_u . s_u$$

$$Q_f = \Sigma\, (f_s)_i . \,(\text{perimeter})_i \,.\, (\text{length})_i$$
$$Q_f = \Sigma\, (\alpha_u)_i . \,(s_u)_i . \,(\text{perimeter})_i \,.\, (\text{length})_i$$

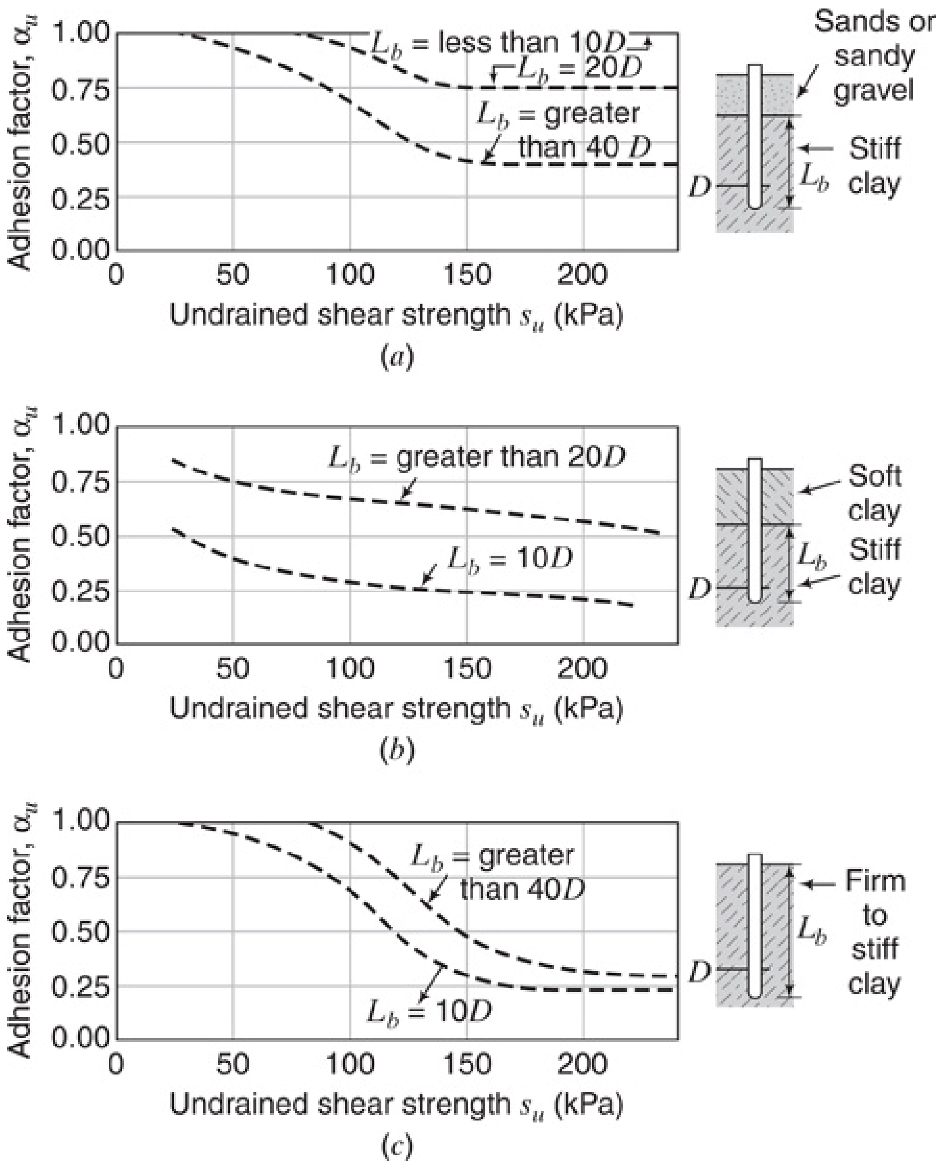
Where

f_s= skin friction stress along pile shaft

α = an empirical adhesion factor, α varies with soil strength and stress history

S_u = Undrained shear strength (use remolded S_u for driven piles)

Tomlinson (1987)



Based on UK experience

Selection of α

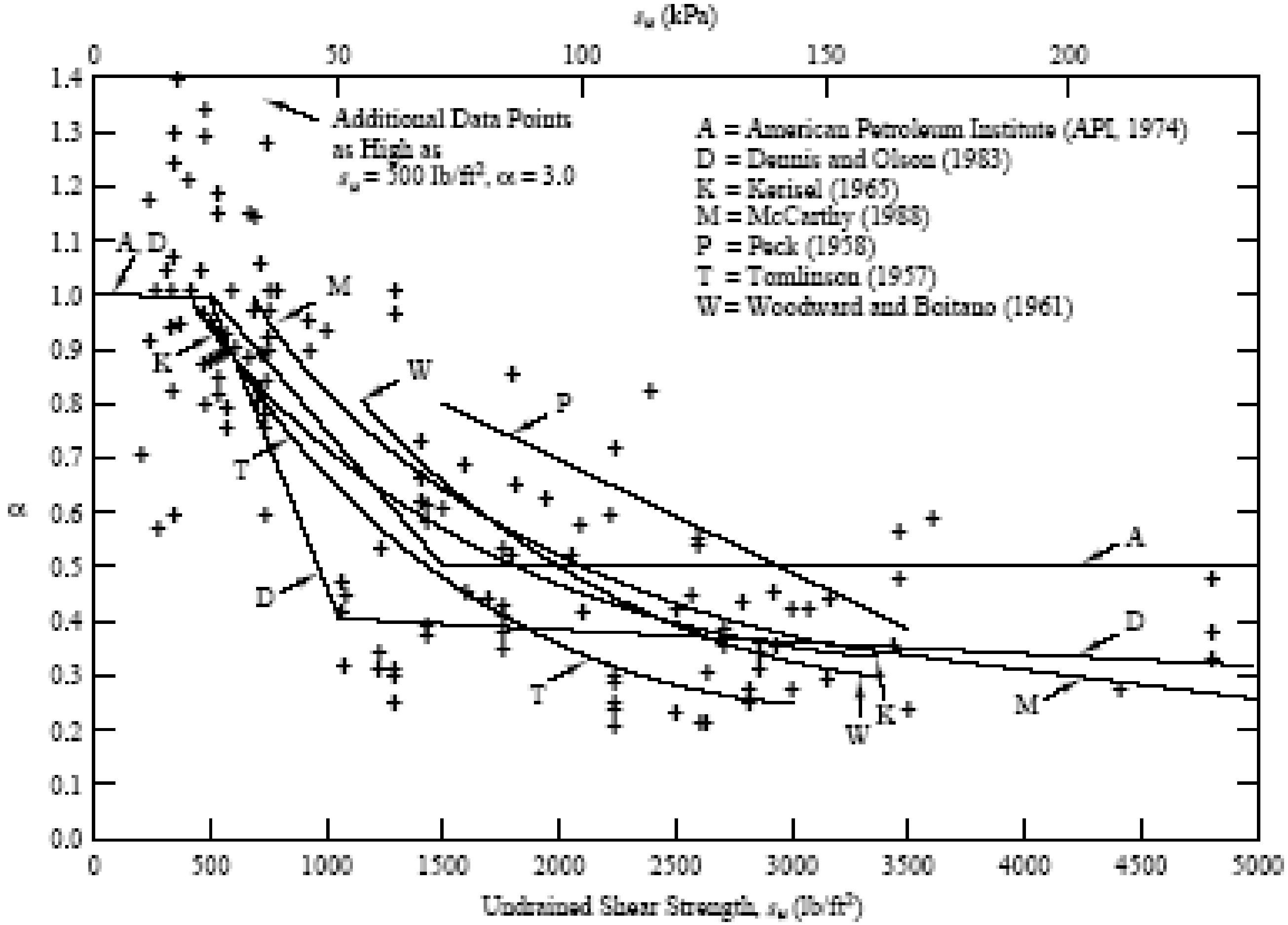


Figure 14.13 Measured values of α as backcalculated from full-scale static load tests compared with several proposed functions for α (Load test data adapted from Vesic, 1977).

Estimation of f_s Randolph and Murphy (1985)

$f_s = \alpha\, s_u$

Use lower of the following:

$f_s = 0.5 \sqrt{(s_u \sigma'_{zo})}$

or

$f_s = 0.5\, (s_u^{0.75})(\sigma'_{zo})^{0.25}$

Pile Capacity in fine-grained saturated soils – End Bearing

$$f_b = N_c(s_u)_b$$

$$Q_b = N_c(s_u)_b\,A_b$$

Where:

f_b = End bearing capacity (stress)

$(S_u)_b$ = Undrained shear strength at tip level

N_c = Bearing capacity coefficient

(N_c= 9 for $(S_u)_b$ > 25 kPa;
N_c= 6 for $(S_u)_b$ < 25 kPa)

A_b = Base ross sectional area

Some General Comments End Bearing

- Recall Bearing capacity equation

Effective Stress Analysis (ESA)

$$q_u = \gamma D_f (N_q - 1)(s_q d_q i_q b_q g_q r_q w_q) + 0.5\gamma B' N_\gamma (s_\gamma d_\gamma i_\gamma b_\gamma g_\gamma r_\gamma w_\gamma)$$

Total Stress Analysis (TSA)

$$q_u = 5.14 s_u (s_c d_c i_c b_c g_c r_c)$$

- For piles, failure is by punching or “cavity expansion” so formulae for N_c, N_q and N_γ are different.
  - When D/B>5, $\gamma' B N^*_\gamma$ is negligible.

Pile Capacity in coarse-grained soils – Shaft Friction

$$Q_f = \Sigma\, (\sigma'_x)_i .\ (\tan \phi'_i)\ .\ (\text{perimeter})_i\ .\ (\text{length})_i$$

$$Q_f = \Sigma\, K_i.(\sigma'_z)_i .\ (\tan \phi'_i)\ .\ (\text{perimeter})_i\ .\ (\text{length})_i$$

$$Q_f = \Sigma\, \beta_i .\ (\sigma'_z)_i .\ (\text{perimeter})_i\ .\ (\text{length})_i$$

Where

f_s= skin friction stress along pile shaft

ϕ'_i= pile-soil interface friction angle

β = empirical factor, α varies with soil strength and stress history

σ'_z = vertical effective stress

K = lateral stress coefficient

K after pile installation

- For other methods of installation

TABLE 14.5 APPROXIMATE RATIO OF COEFFICIENT OF LATERAL EARTH PRESSURE AFTER CONSTRUCTION TO THAT BEFORE CONSTRUCTION (Adapted from Kulhawy et al., 1983 and Kulhawy, 1991)

Foundation Type and Method of Construction	K/K_0
Pile—jetted	0.5–0.7
Pile—small displacement, driven	0.7–1.2
Pile—large displacement, driven	1.0–2.0
Drilled shaft—built using dry method with minimal sidewall disturbance and prompt concreting	0.9–1.0
Drilled shaft—slurry construction with good workmanship	0.9–1.0
Drilled shaft—slurry construction with poor workmanship	0.6–0.7
Drilled shaft—casing method below water table	0.7–0.9

Selection of δ or ϕ'_i

- Budhu recommends

Material	δ or ϕ'_i
Steel	$\frac{2}{3}\Phi'_{cs}$ to $0.8\ \Phi'_{cs}$
Concrete	$0.9\Phi'_{cs}$ to $1.0\ \Phi'_{cs}$
Timber	$0.8\Phi'_{cs}$ to $1.0\ \Phi'_{cs}$

Estimation of f_s Burland (1973)

Fine-grained soils

$$\beta = K_0^{OC} (\tan \phi'_i)$$
$$= (1 - \sin \phi'_{cs}) (OCR)^{0.5}. (\tan \phi'_i)$$

Sands

$$\beta = (1 - \sin \phi'_{cs}) (\tan \phi'_i)$$

Pile Capacity in coarse-grained soils – End Bearing

$$Q_b = f_b . A_b$$

$$Q_b = N_q . (\sigma'_z)_b . A_b$$

Where

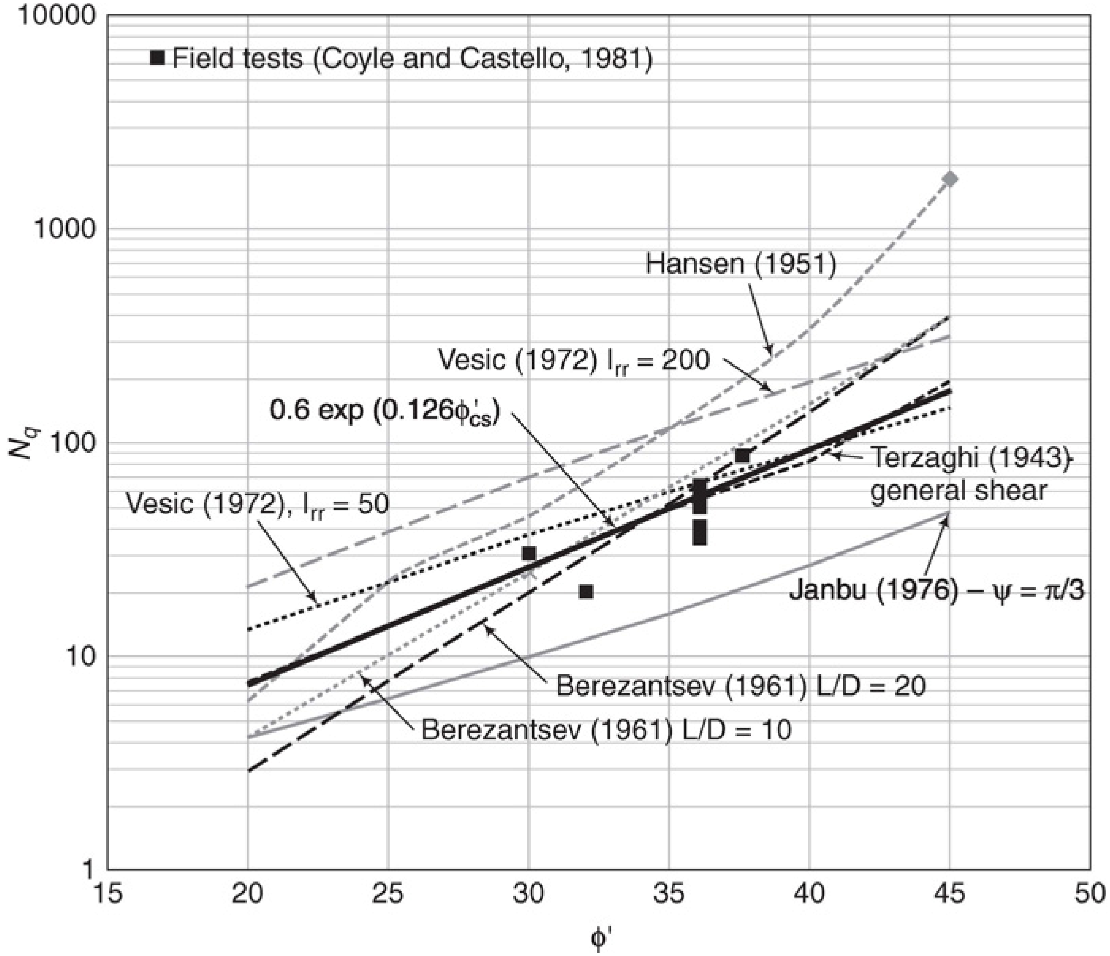
N_q = Bearing capacity coefficient
(function of $(\sigma'_z)_b$ and ϕ')

ϕ' = pile-soil interface friction angle

σ'_z = vertical effective stress
at end bearing level

A_b = Base cross sectional area

Range of N_q values



API design parameters

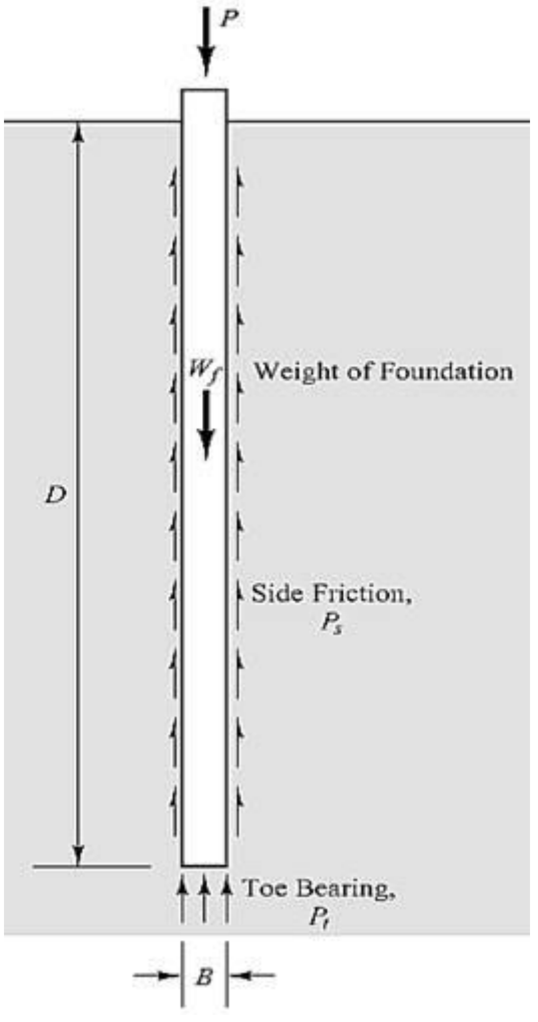
TABLE 2.6.4-1
Design Parameters for Cohesionless Siliceous Soil*

Density	Soil Description	Soil-Pile Friction Angle, δ Degrees	Limiting Skin Friction Values kips/ft² (kPa)	N_q	Limiting Unit End Bearing Values kips/ft² (MPa)
Very Loose Loose Medium	Sand Sand-Silt** Silt	15	1.0 (47.8)	8	40 (1.9)
Loose Medium Dense	Sand Sand-Silt** Silt	20	1.4 (67.0)	12	60 (2.9)
Medium Dense	Sand Sand-Silt**	25	1.7 (81.3)	20	100 (4.8)
Dense Very Dense	Sand Sand-Silt**	30	2.0 (95.7)	40	200 (9.6)
Dense Very Dense	Gravel Sand	35	2.4 (114.8)	50	250 (12.0)

*The parameters listed in this table are intended as guidelines only. Where detailed information such as in situ cone tests, strength tests on high quality samples, model tests, or pile driving performance is available, other values may be justified.

**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.

Axial Pile Load Transfer



$$Q_{ult} = Q_f + Q_b - W_p$$

$$Q_a = \frac{Q_{ult}}{FS}$$

Q_{ult} = Ultimate load capacity

Q_f = Shaft resistance ("skin friction")

Q_a = Allowable load capacity

Q_b = End bearing

W_p = Weight of pile

FS = Factor of safety

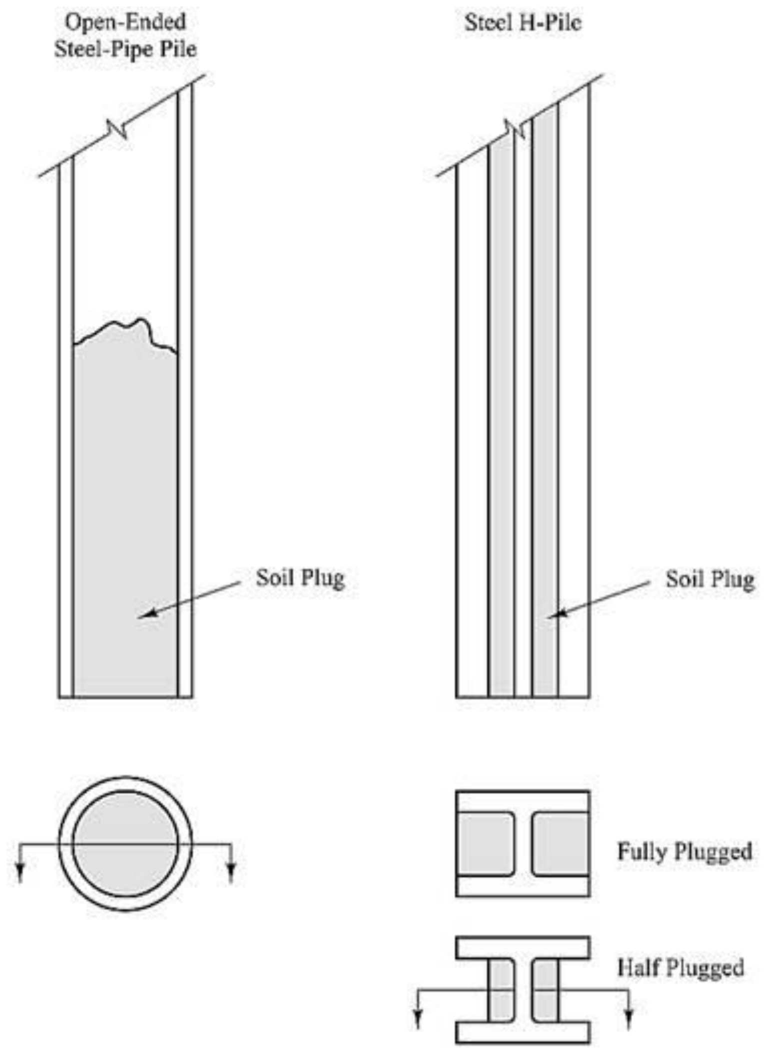
Toe (A_t) and Side (A_s) Contact Areas

Closed-section foundations: A_t=toe area of foundation

Open-section foundations:

For shorter piles, A_t=steel cross-section only

If the pile plugs, it may be possible to use A_t= area of steel + plug. This requires careful consideration.



Negative Skin Friction

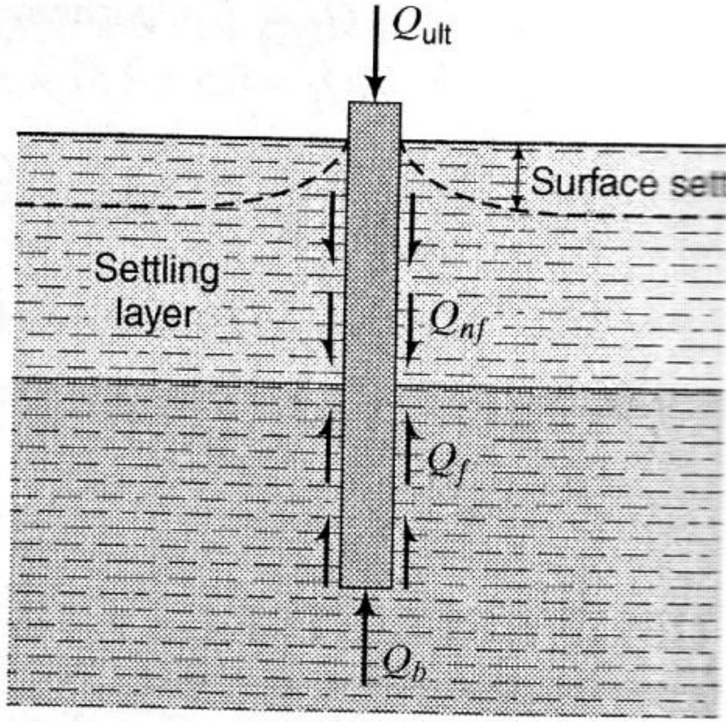


FIGURE 8.8 Negative skin friction.

$$Q_a = \frac{Q_b + Q_f}{\text{FS}} - Q_{nf}$$

EXAMPLE

<u>Example 8.1</u> Load capacity of a timber pile in a homogeneous clay

A cylindrical timber pile of diameter 457 mm (18 in) is driven to a depth of 11 m (40 ft) into homogeneous normally consolidated clay. The soil parameters are: s_u = 40 kPa (835 psf), ϕ'_{cs} = 28°, γ_{sat} = 18 kN/m^3 (110 pcf). Groundwater level is at the surface. Estimate the allowable pile load capacity for a factor of safety of 2. Is the pile a friction pile?

Strategy

The solution is a straightforward application of the static pile load capacity equations. Since the soil is fine-grained, you should check both the short-term and long-term pile load capacity. US units will be used for this solution.

Muni Budhu "Foundations and Earth Retaining Structures", John Wiley & Sons, NY, 2007

SOLUTION

Solution 8.1

Step 1: Calculate geometric and other parameters for driven piles.

L = 40 ft; diameter = 1.5 ft, perimeter = πD = π × 1.5 = 1.5π ft, $A_b = \frac{\pi D^2}{4} = \frac{\pi \times 1.5^2}{4} = 0.563\pi\, ft^2$

$\gamma' = 110 - 62.4 = 47.6\, pcf$

At center of clay: σ'_{zo} = 47.6 x 40/2 = 952 psf

Eq. (8.6) $f_s = 0.5(835 \times 952)^{0.5}$ = 446 psf (okay)

Eq. (8.7) $f_s = 0.5 \times 835^{0.25} \times 952^{0.25}$ = 431 psf

Use f_s= 431 psf

API approach - TSA

s_u/σ_z'=835/952=0.88

α=0.5x(0.88)$^{-0.5}$=0.53<1

f_s=0.53x853=446 psf

Budhu uses the lower of eqns 8.6 or 8.7

Eq. (8.14): $\beta = (1 - \sin\phi'_{cs})(OCR)^{0.5}\tan\phi'_i$. Assume $\phi'_i = \phi'_{cs}$ ESA

For OCR = 1 (normally consolidated soil), $\beta = (1 - \sin 28^o)(1)^{0.5}\tan 28^o = 0.28$

SOLUTION

Step 2: Calculate Q_a using TSA.

$$Q_f = f_s(\pi D)L = 431 \times 1.5\pi \times 40 = 81.3kips$$

$$Q_b = 9s_u A_b = 9 \times 835 \times 0.563\pi \approx 13.3kips$$

$$Q_{ult} = Q_f + Q_p = 81.3 + 13.3 \approx 94kips$$

$\dfrac{Q_f}{Q_{ult}} = \dfrac{81.3}{94.6} = 0.86 > 0.8$. Therefore, based on TSA, we have a friction pile.

$$Q_a = \frac{Q_{ult}}{FS} = \frac{94.6}{2} = 47.3kips$$

Muni Budhu "Foundations and Earth Retaining Structures", John Wiley & Sons, NY, 2007

SOLUTION

Step 3: Calculate Q_a using an ESA.

$$Q_f = \beta \sigma'_z (\pi D L)$$

Use an average value of σ'_z over the embedded length,

$$\sigma'_z = \gamma' \frac{L}{2} = 47.6 \times \frac{40}{2} = 952\, psf$$

$$Q_f = 0.28 \times 952 \times 1.5\pi \times 40 = 50252 lbs = 50.3 kips$$

<u>End bearing</u>

Eq. (8.21): $N_q = 0.6 \exp\,(0.126\phi'_{cs}) = 0.6 \exp(0.126 \times 28) = 20.4$

$$Q_b = N_q \left(\sigma'_z\right)_b A_b = 20.4 \times \left(47.6 \times 40\right) \times 0.563\pi = 68709 lbs = 68.7 kips$$

Muni Budhu “Foundations and Earth Retaining Structures”, John Wiley & Sons, NY, 2007

SOLUTION

If you use Janbu's equation, $N_q = \left(\tan\phi'_{cs} + \sqrt{1+\tan^2\phi'_{cs}}\right)^2 \exp\left(2\psi\tan\phi'_{cs}\right)$, you will get for

$\phi'_{cs} = 28^o$ and $\psi = \frac{\pi}{3}$ (soft clay), $N_q = \left\{\tan 28^o + \sqrt{1+\tan^2\left(28^o\right)}\right\}^2 \exp\left(\frac{2\pi}{3}\tan 28^o\right) = 9.5$

This gives a 53% decrease in end bearing capacity and illustrates the uncertainties in estimating the end bearing resistance.

$$Q_{ult} = Q_f + Q_b = 50.3 + 68.7 = 119 kips$$

$$\frac{Q_f}{Q_{ult}} = \frac{50.3}{119} = 0.42 = 42\%$$

Therefore, based on ESA, we have a combination of friction and end bearing pile.

$$Q_a = \frac{Q_{ult}}{2} = \frac{119}{2} = 59.5 kips$$

Q_a from the TSA is less than from the ESA. Use Q_a=47.3 kips

In practice, use FS ≥ 2.5.

Muni Budhu "Foundations and Earth Retaining Structures", John Wiley & Sons, NY, 2007