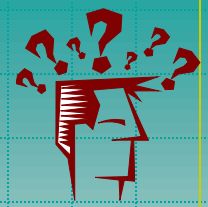
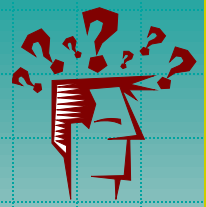
Bearing Capacity of Pile Foundation



EWC 343: Geotechnical & Foundation Engineering

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Bearing Capacity of Pile Foundation



Types of Foundations

Depending on depth at which it transfers load, foundations can be classified as:

- **Shallow Foundation:** Transfers load to a shallower strata;
  - Depth less than breadth, B
  - Depth less than 3 meters
- **Deep Foundation:** Transfers load to a deeper strata.

Bearing Capacity of Pile Foundation



Types of Foundation

There is another classification scheme where foundations may be categorized also by their depth below ground level. There are three arbitrary general categories:

1. Shallow: $\frac{D_f}{B} \leq 1$
2. Moderately deep: $1 < \frac{D_f}{B} \leq 5$
3. Deep: $\frac{D_f}{B} > 5$

Where, D_f is the depth of foundation and B is its width (minimum dimension). For moderately deep foundation, Meyerhof introduced factors that to be used with bearing capacity factors, taking into account the depth and shape of a footing.

Bearing Capacity of Pile Foundation

Defining Deep Foundation

A deep foundation, Fig. 1, is one that transmits some or all of the applied loads to soils well below the ground surface.

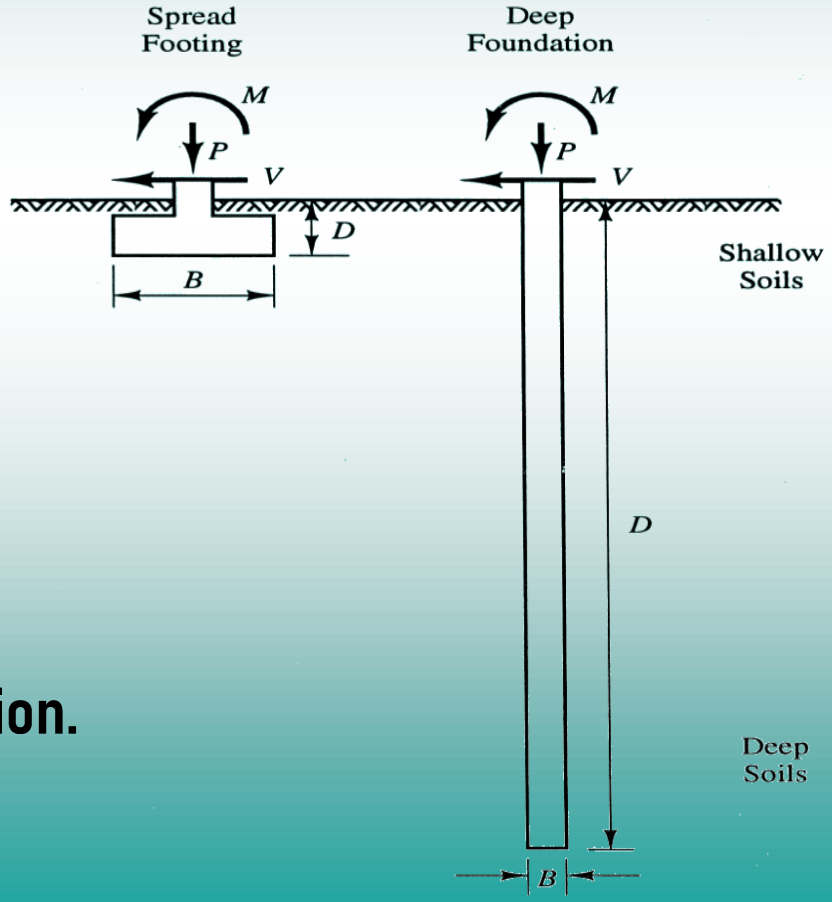


Fig. 1: Deep Foundation.

Bearing Capacity of Pile Foundation

Classification of Deep Foundation

Deep foundation can be generally classified as indicated in Fig. 2.

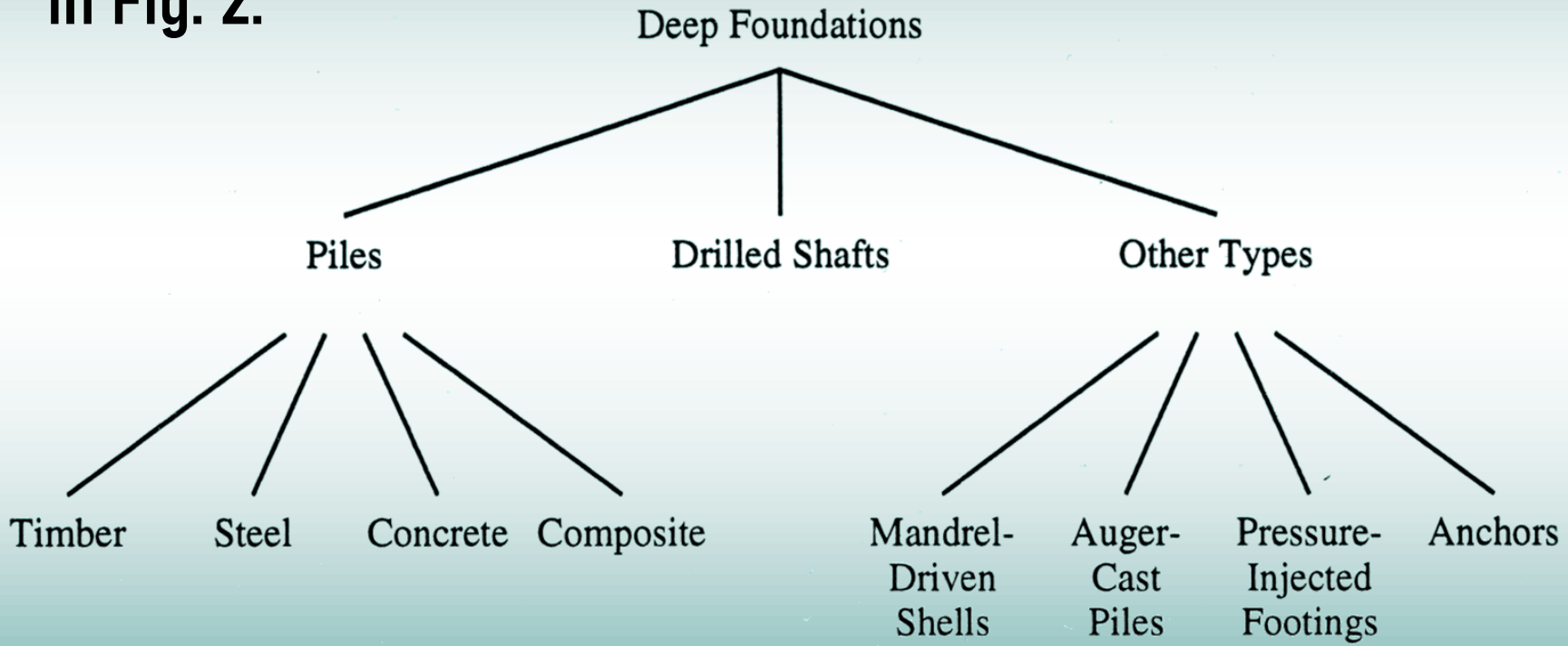
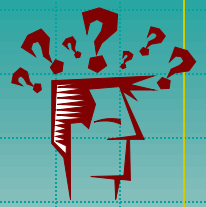


Fig. 2: Classification of Deep Foundation.

Bearing Capacity of Pile Foundation



Defining Pile Foundation

A pile is a columnar type of foundation that performs the function of transferring the load from the super structure to underlying strong or less compressible strata through weak and compressible layers.

Bearing Capacity of Pile Foundation



Functions of Pile Foundation

The functions of a pile foundation are mainly to:

- transfer load to a stronger and/or less compressible stratum
- distribute load into thick homogeneous soil
- carry uplift and lateral forces
- increase stability of tall structures
- avoid scour damage
- compact loose sand

Bearing Capacity of Pile Foundation

Functions of Pile Foundation

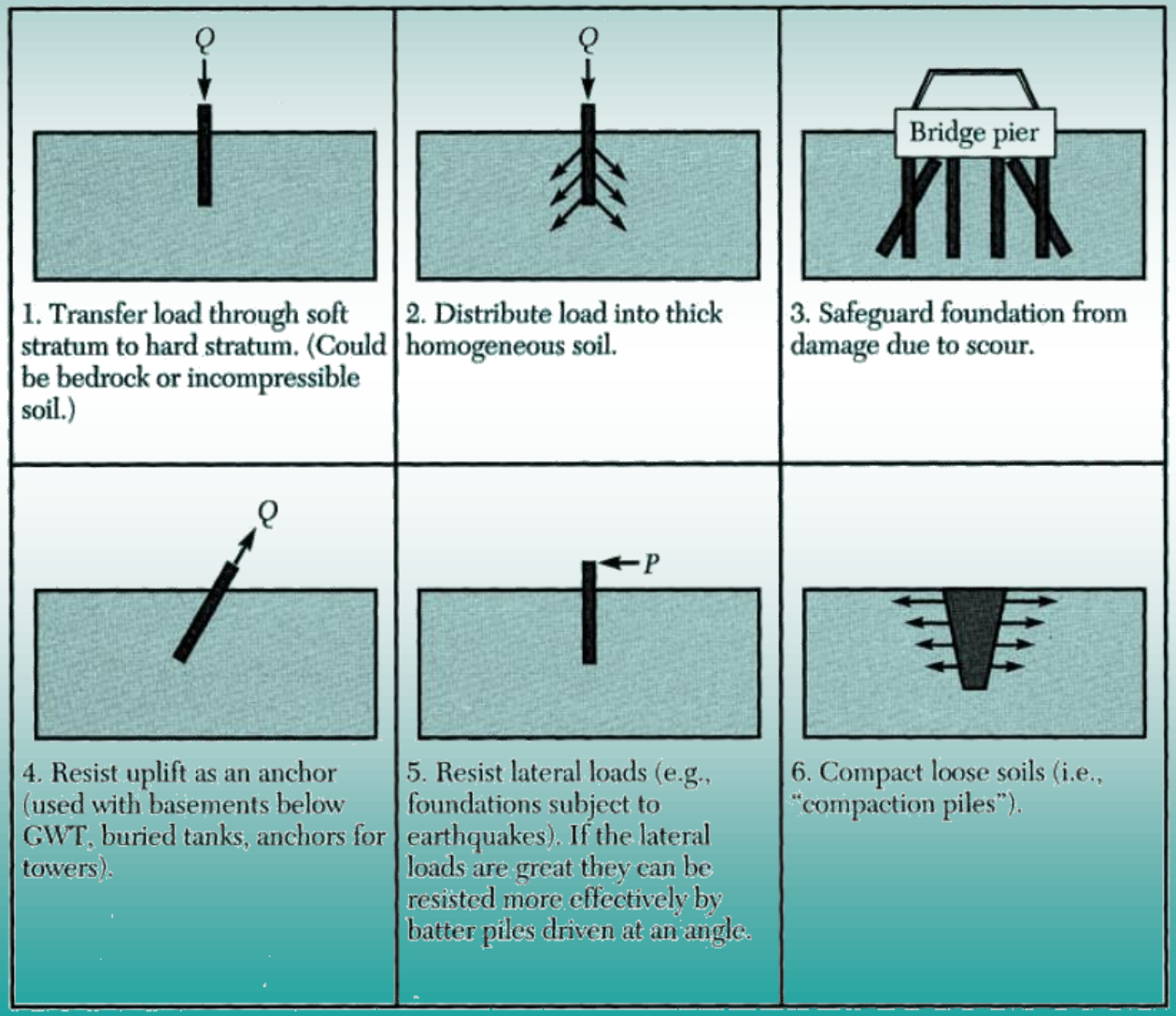
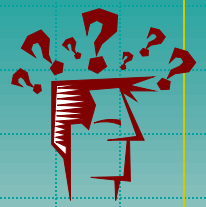


Fig. 3: Functions of a pile foundation.

Bearing Capacity of Pile Foundation

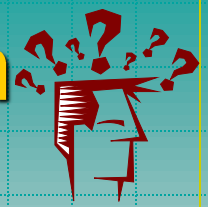


Functions of Pile Foundation

Pile foundations must satisfy five requirements:

- The pile foundation must possess sufficient structural strength to carry the applied loads.
- It must be possible to install the piles without damage to them.
- The soil must be able to develop the required resistance to transfer the load.
- The total and differential settlements of the foundation must be less than the permissible settlement of the structure the foundation is supporting.
- It must be possible to install the foundation without damage to adjacent structures.

Bearing Capacity of Pile Foundation



Functions of Pile Foundation

While designing pile foundations to satisfy the necessary requirements, one must know the following.

- The magnitude and direction of loads on the pile.
- The soil characteristics, including depth to the GWT and depth to the bearing strata, if applicable.
- The cost of materials and installation.
- The types of piles and driving equipment available.
- The locations and characteristics of adjacent structures.
- The building code requirements.

Bearing Capacity of Pile Foundation



Classification of Piles

Piles can be classified in a number of ways depending on many factors like:

- Installation procedure
- Manner of soil displacement
- Pile material
- Use
- Capacity
- Shape
- Environment
- Load inclination
- Length
- Type of structure
- Transmission of load

Bearing Capacity of Pile Foundation



Classification of Piles

Pile foundations can be classified as according to various criteria shown in Fig. 4.

Material	Shape	Cross-section	Load transfer	Method of forming	Method of installation	Use	Length
Timber	Cylindrical	Circular	Friction	Cast-*in-situ*	Bored	Load bearing	Short
Steel	Tapered	Square	Bearing	Pre-cast	Driven	Load test	Long
Concrete	Under-reamed	Hexagonal	Uplift	Prestressed	Jacked	Offshore	Slender
Composite	Corrugated	Octagonal			Vibrated	Compac-tion	Large diameter
		I-section			Jetted		
		H-section			Tremie		
		Pipe					

Fig. 4: Classification of Pile Foundations.

Bearing Capacity of Pile Foundation

Classification Based on Soil Displacement

Considering the volume of soil displaced due to installation of piles, they can be classified as shown in Fig. 5.

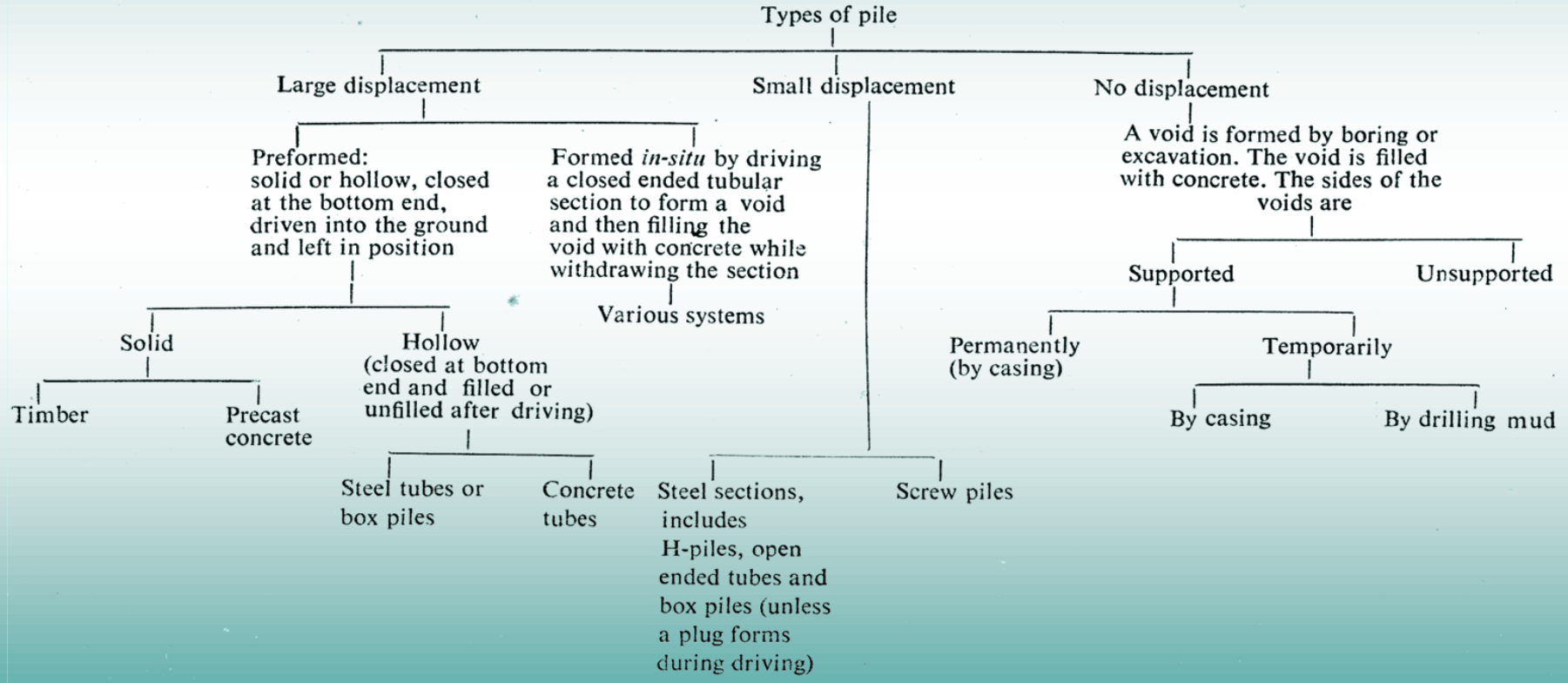


Fig. 5: Pile Foundations Based on Soil Displacement.

Bearing Capacity of Pile Foundation

Classification of Piles Based on Use

Pile foundations can be classified as according to use shown in Fig. 6.

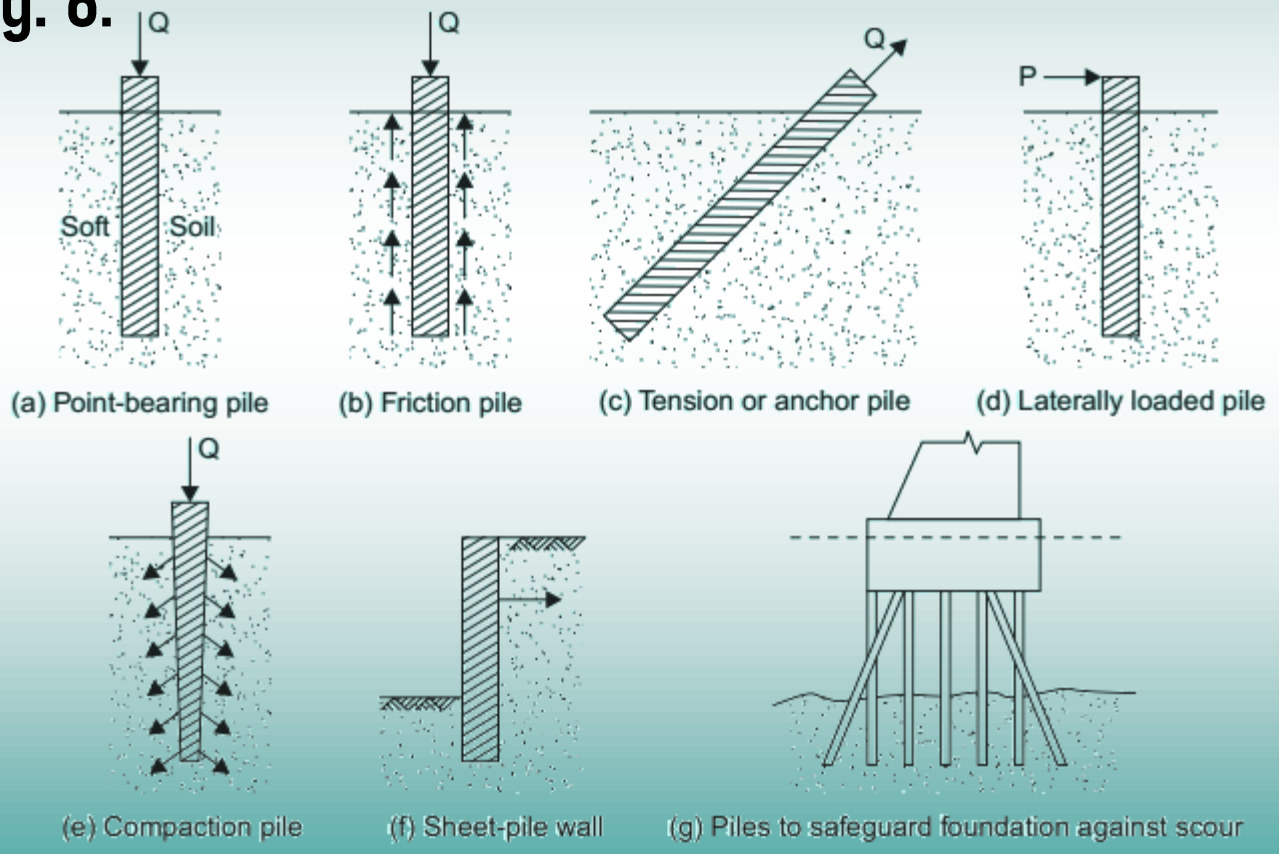
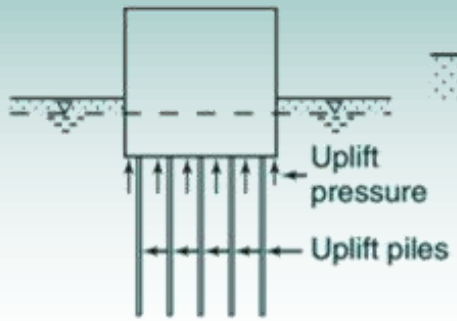


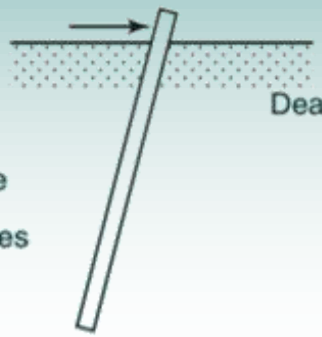
Fig. 6: Pile Foundations Classification Based on Their Use.

Bearing Capacity of Pile Foundation

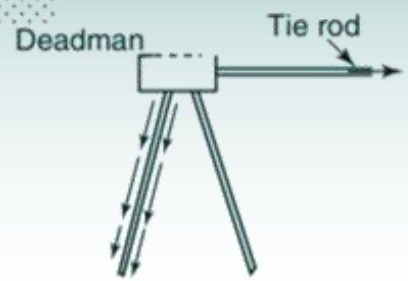
Classification of Piles Based on Use (Contd.)



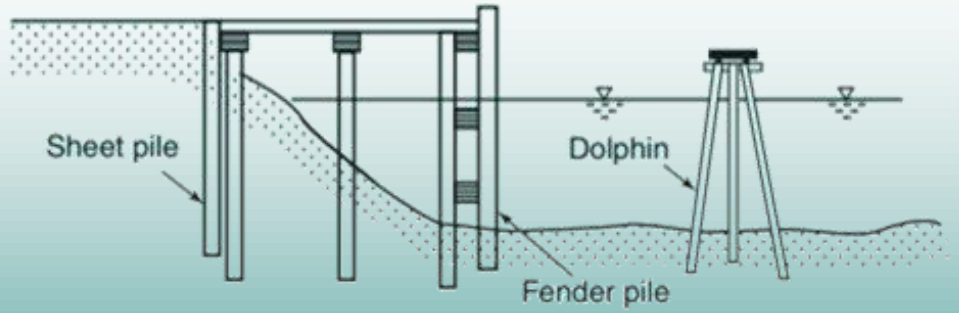
(h) Uplift piles



(i) Batter piles



(j) Batter pile as anchor



(j) Other functions of some piles

Fig. 6: Pile Foundations Classification Based on Their Use.

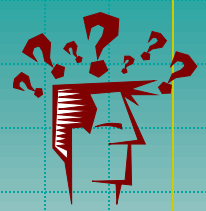
Bearing Capacity of Pile Foundation



Classification of Piles Based on Embedment

- Partially embedded or free standing pile
- Fully embedded or piled foundation

Bearing Capacity of Pile Foundation



Selection of Pile Type

- Method of support: Skin friction, End bearing
- Bearing soil layer depth
- Load type, direction and magnitude
- Constructability
- Cost

Bearing Capacity of Pile Foundation

Loading Type and Direction

- Permanent/ Transient/ Cyclic
- Vertical (Axial) or Horizontal (Lateral)

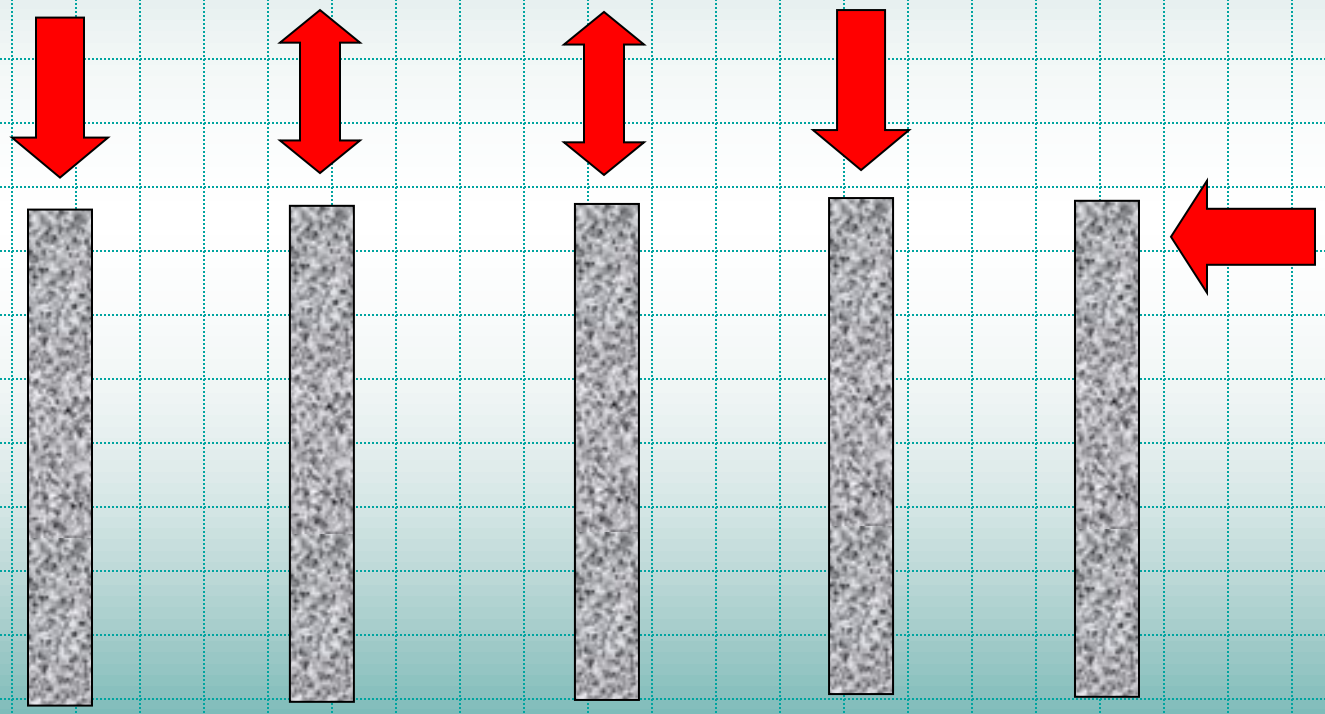


Fig. 7: Loading on Pile Foundations.

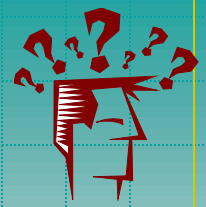
Bearing Capacity of Pile Foundation



Preferable Pile Type based on Loading Type

- Wood is better for transient resistance than permanent
- Steel pile is better for cyclic resistance
- High horizontal loads better resisted by stiffer piles or shafts

Bearing Capacity of Pile Foundation



Axial Load Carrying Capacity of Piles

- **Structural Capacity (Resistance):** Depends on load carrying capacity of pile material.
- **Geotechnical Capacity (Resistance):** Depends on properties of soil.
- **Driving/ Pushing Resistance (for driven or push-in piles):** Depends on impact/ pushing load resisting capacity of a pile.

Bearing Capacity of Pile Foundation

Driving Resistance

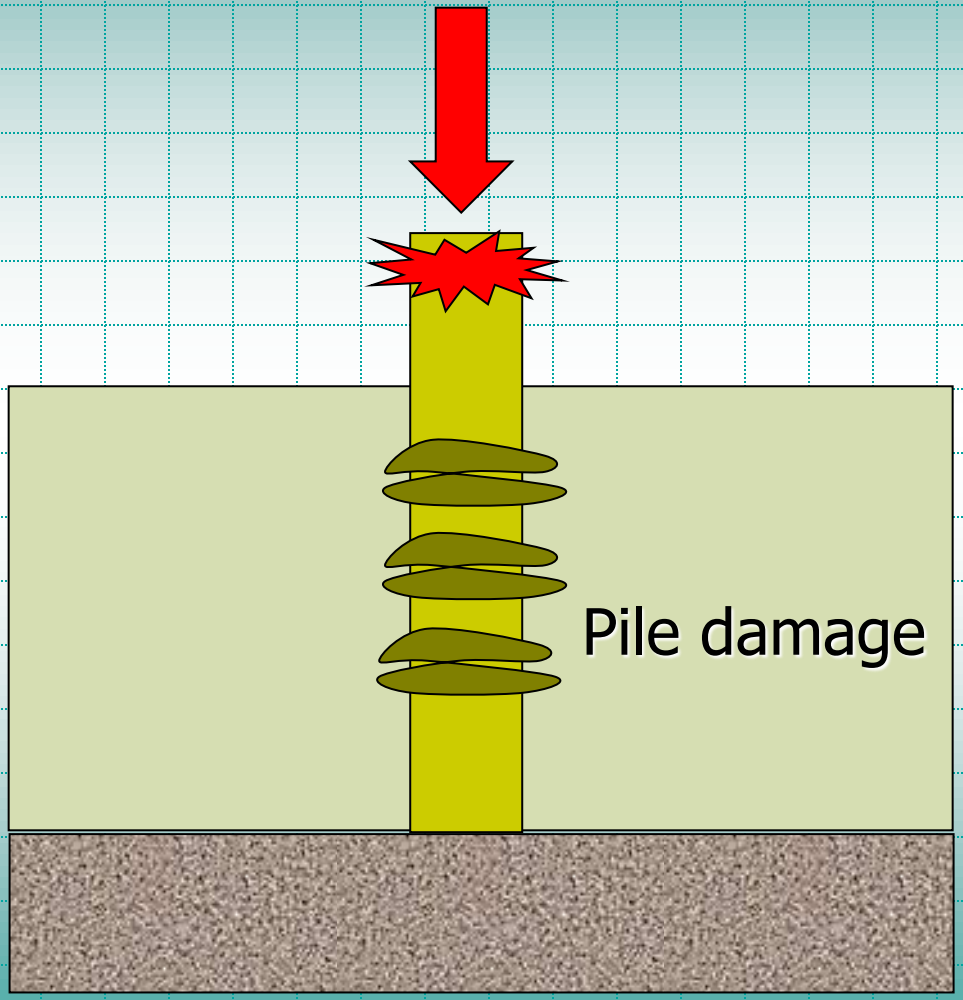
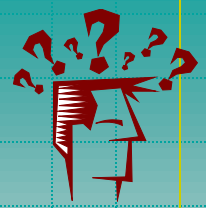


Fig. 8: Damage of Pile Foundations due to Loading.

Bearing Capacity of Pile Foundation



Determining Geotechnical Resistance of Piles

- Static Analysis Methods
- Field Methods
  - Static Load Test
  - Dynamic Load Test (PDA)
  - Driving Formulae

Bearing Capacity of Pile Foundation



Static Analysis Method: Bearing Capacity of a Single Pile

The load on a pile is:

- Partly carried by Skin Friction, and
- Partly by the resistance offered by the soil at the base of the pile, called the Point Resistance or End Bearing

Bearing Capacity of Pile Foundation

Method of Transferring Load and Support

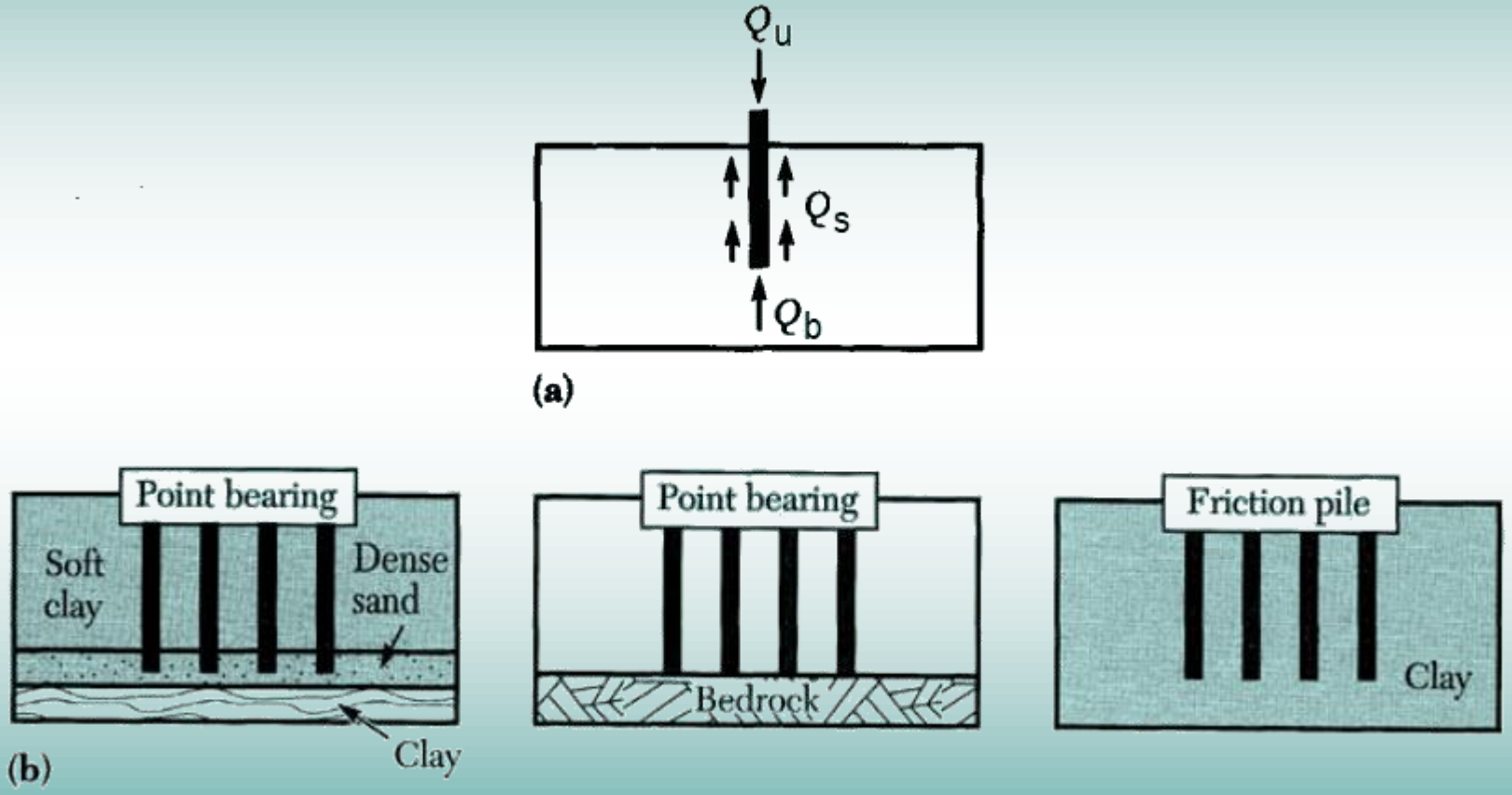


Fig. 9: Types of pile: (a) a typical pile; (b) friction and end-bearing piles.

Bearing Capacity of Pile Foundation

Method of Transferring Load and Support

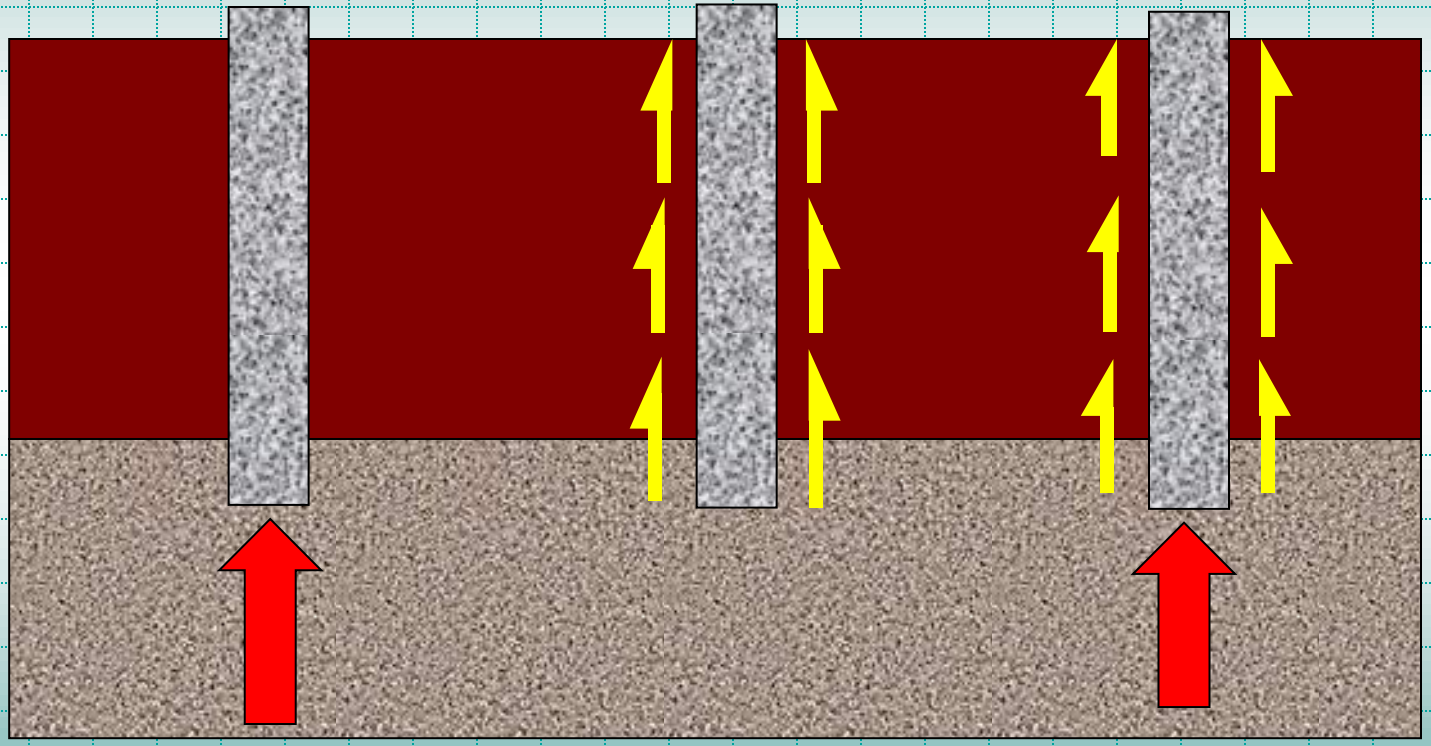
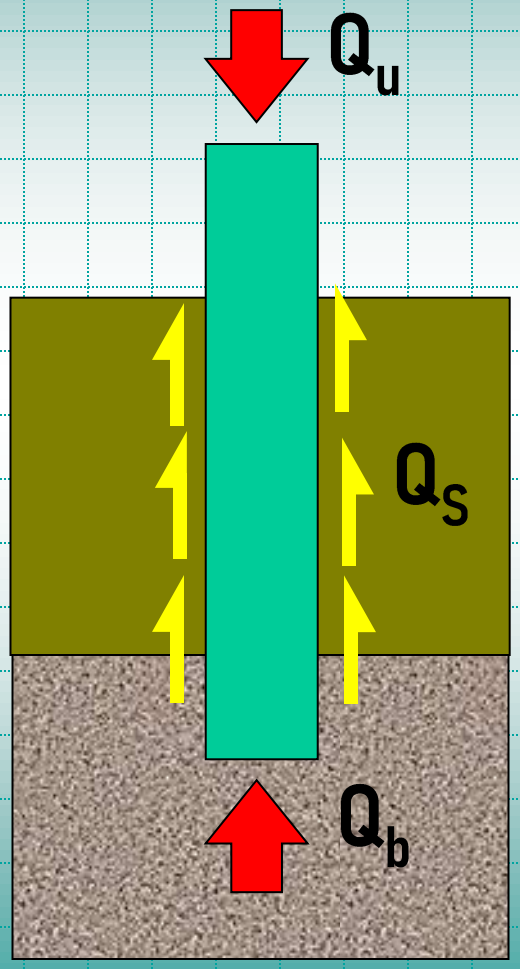


Fig. 10: End Bearing and Side Friction in Pile Foundation.

Bearing Capacity of Pile Foundation

Computation of Static Geotechnical Resistance



Ultimate Applied Load, Q_U

= Resisting Force

Ultimate Load/Capacity, Q_U

=Skin Friction + End Bearing

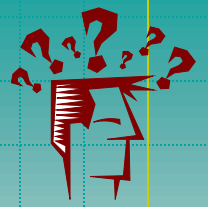
$= Q_S + Q_b$

$Q_S = A_S q_s$

$Q_b = A_b q_b$

Fig. 11: Analysis of End Bearing and Side Friction.

Bearing Capacity of Pile Foundation



Factors Affecting Skin Friction and End Bearing

Soil Types

- Cohesionless Soil (Sand)
- Cohesive Soil (Clay)
- c – ϕ soil
- Layered Soil

Pile Type

- High Displacement (Driven Pile)
- Low/No Displacement (Bored Pile)

Bearing Capacity of Pile Foundation

Static Analysis Method

For a driven pile in sand, the unit skin friction depends on the soil pressure acting normal to the pile surface, and the coefficient of friction between soil and pile material.

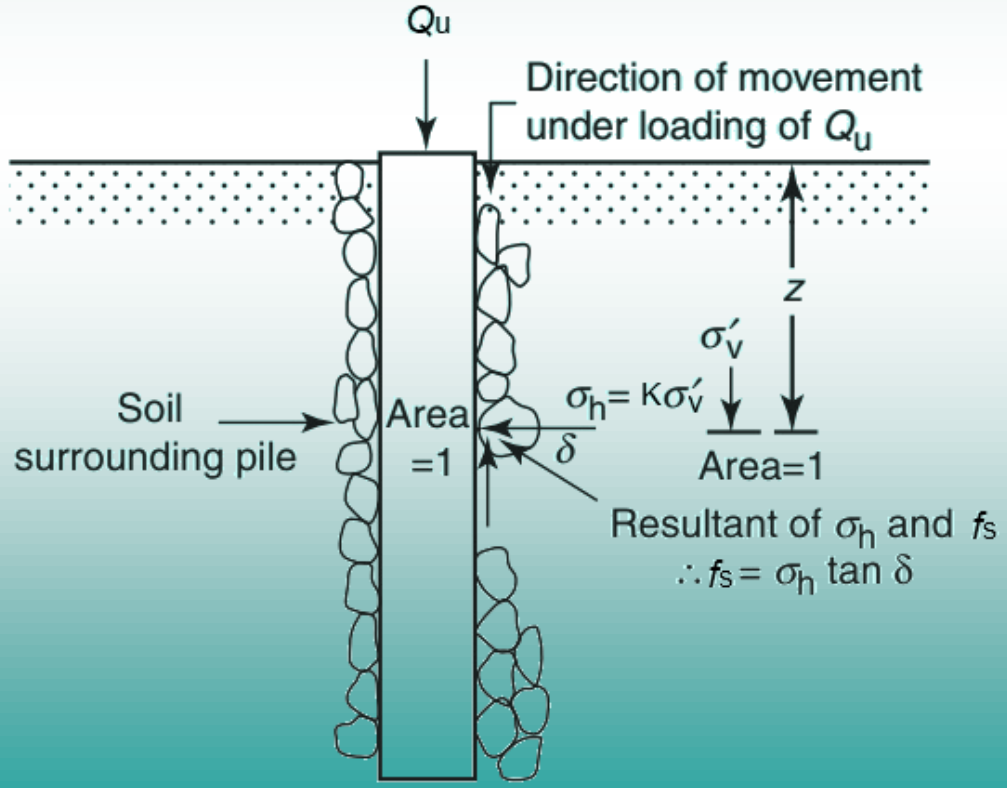


Fig. 12: Static Analysis of Pile Foundations

Bearing Capacity of Pile Foundation



Skin Friction of a Driven Pile in Sand

The unit skin friction, f_s, acting at any depth in sand is given by:

$$f_s = \sigma_h' \tan\delta$$

or,

$$f_s = K\sigma_v' \tan\delta$$

Where,

σ_h' = horizontal soil pressure acting at any depth z in a soil mass

σ_v' = effective overburden pressure acting at the same depth z

K = lateral earth pressure coefficient

δ = angle of friction between soil and the pile surface

Bearing Capacity of Pile Foundation



Skin Friction of a Driven Pile in Sand

Hence, the total skin friction acting along the embedded length of the pile is computed considering an average effective vertical overburden pressure. Thus,

$$Q_s = f_s A_s = \left(K\sigma'_{v(av)} \tan\delta\right) A_s$$

Where,

$\sigma'_{v(av)}$ = average effective vertical overburden pressure

A_s = perimetric area of pile shaft ($= \pi dL \; for \; circular \; pile$)

L = length of pile

Bearing Capacity of Pile Foundation



Skin Friction of a Driven Pile in Sand

Values of coefficient of friction $(tan\delta)$ depend on the type of soil and pile material. McCarthy (1982) reports values varying from 0.20 to 0.45 depending on the roughness of pile surface. Some standards recommend $\delta = \varphi$.

Bearing Capacity of Pile Foundation



Skin Friction: Driven Pile in Cohesionless Soil

The earth pressure is approximately equal to the passive condition at the top of the pile and may be near to the at-rest condition at the pile tip. The method of installation also influences the earth pressure coefficient as indicated below. Skin friction has limiting values depending on soil and method of construction.

Bearing Capacity of Pile Foundation



Skin Friction: Driven Pile in Cohesionless Soil

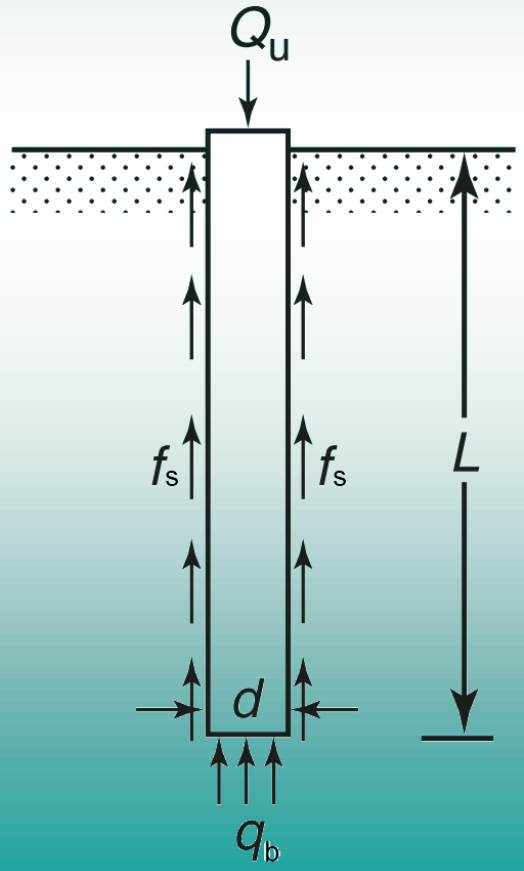
Pile Type	Coefficient of earth pressure, K
Bored or jetted piles	$K_o = 1 - \sin\varphi$
Low displacement driven piles	$K_o - 1.4K_o$
High displacement driven piles	$K_o - 1.8K_o$
Piles in loose to medium dense sand	$1 - 3$

Bearing Capacity of Pile Foundation



End Bearing: Driven Pile in Cohesionless Soil

The end bearing component of pile capacity can be determined using a method similar to that of shallow foundations.



Bearing Capacity of Pile Foundation



End Bearing: Driven Pile in Cohesionless Soil

Thus, for deep foundation in cohesionless soil:

$$q_b = \frac{1}{2} d\gamma N_\gamma + q_o N_q$$

Where,

$N_q =$ bearing capacity factor (for pile)

$N_\gamma =$ bearing capacity factor for shallow foundation

$d =$ width/diameter of the pile tip

$q_o =$ effective overburden pressure at pile tip

Bearing Capacity of Pile Foundation



End Bearing: Driven Pile in Cohesionless Soil

The term $\frac{1}{2} d\gamma N_\gamma$ becomes insignificant in comparison with the term $q_o N_q$ for deep foundations, and is normally neglected.

As such, equation for end bearing resistance (load) will take the form,

$$Q_b = (q_o N_q) A_b = (\sigma_v' N_q) A_b$$

Bearing Capacity of Pile Foundation



Bearing Capacity Factor N_q : Pile in Sand

The bearing capacity factor, N_q values established by Berezantsev et al. (1961), Fig. 13a, take into account the depth to width ratio of the pile most nearly conform to the practical criteria of pile failure.

As per Tomlinson, the maximum base resistance q_b is normally limited to 10000 kN/m^2 (1000 t/m^2).

Bearing Capacity of Pile Foundation

Bearing Capacity Factor N_q : Piles in Sand

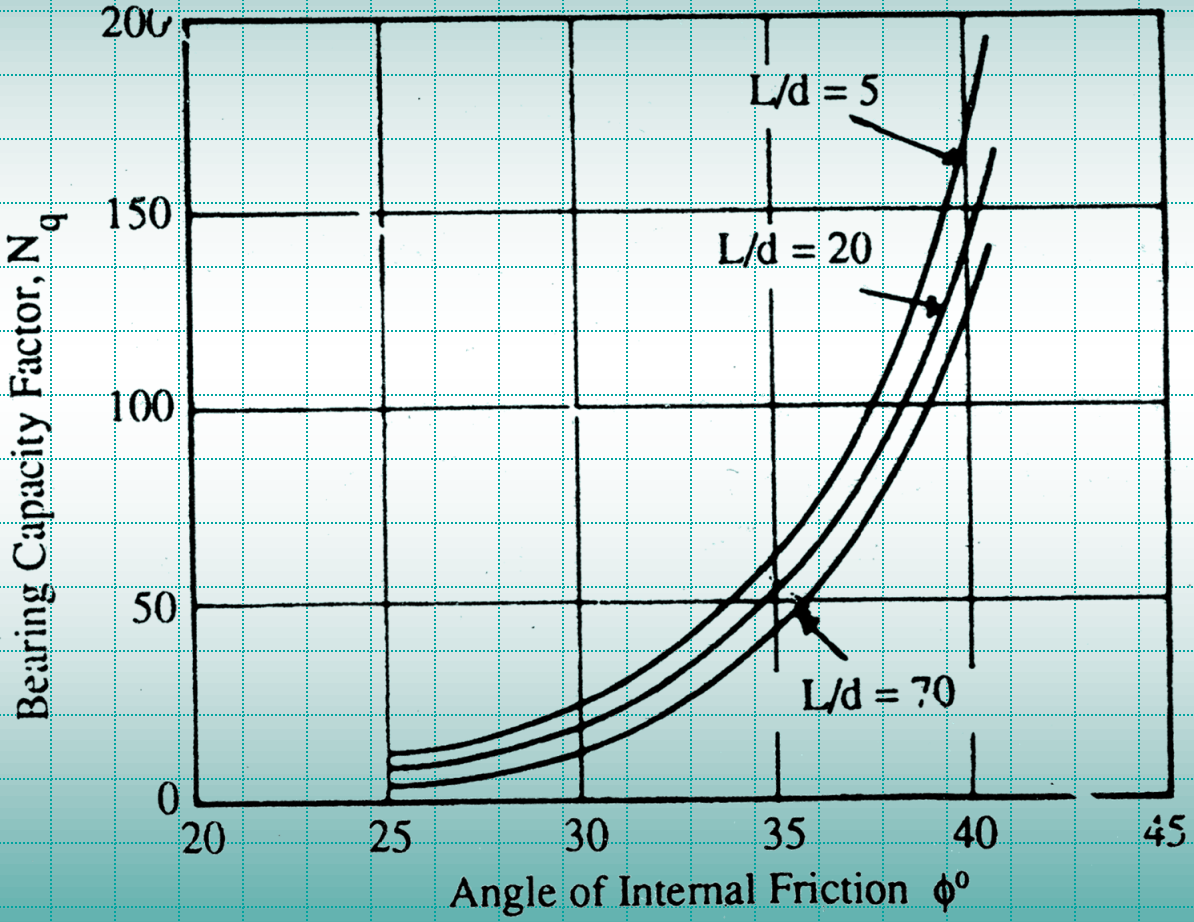


Fig. 13a: Bearing Capacity Factor, N_q for Pile Foundation.

Bearing Capacity of Pile Foundation



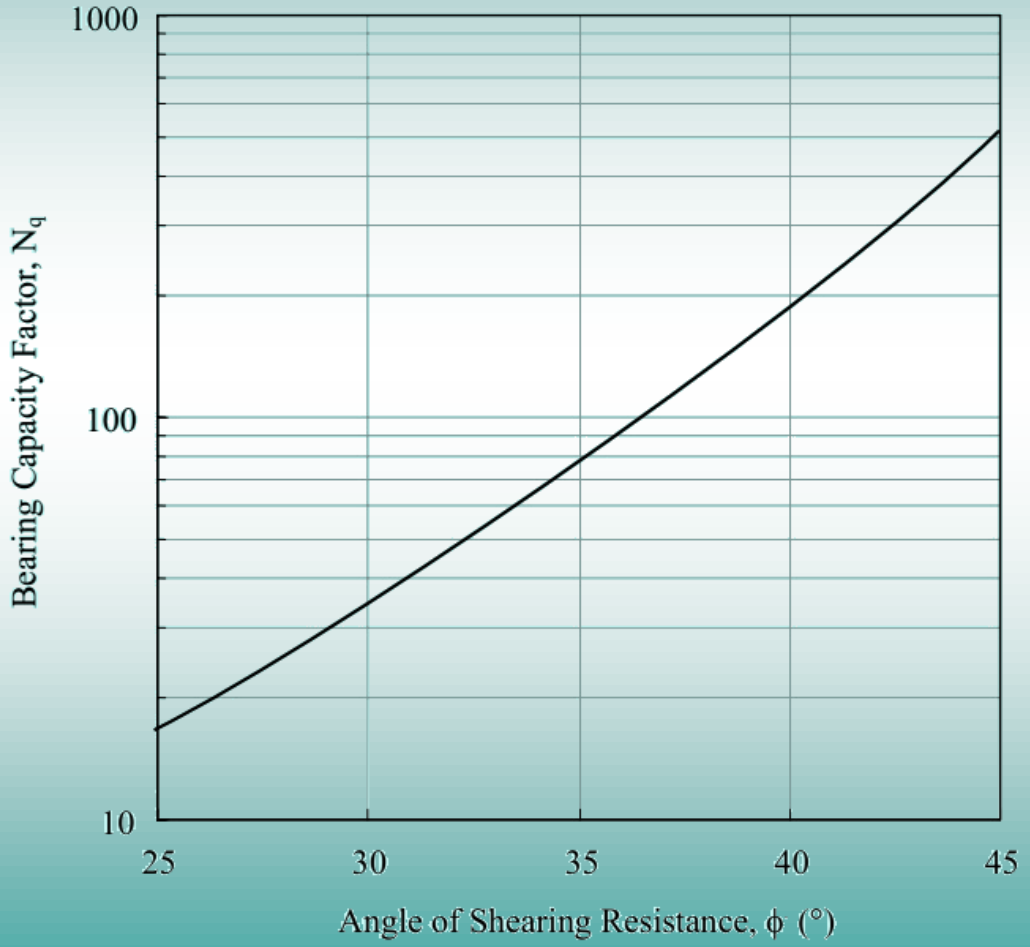
Bearing Capacity Factor N_q : Piles in Sand

Both Poulos & Davis (1980) and Fleming et al (1992) recommended the use of factors derived by Berezantzev et al (1961), which is also supported by Vesic (1967). Poulos & Davis (1980) further suggested that for the determination of N_q, the value of ϕ should be reduced by 3° to allow for possible loosening effect of installation.

For general design purposes, it is suggested that the N_q values based on Poulos & Davis (1980) as presented in Fig. 13b may also be considered.

Bearing Capacity of Pile Foundation

Bearing Capacity Factor N_q : Piles in Sand



For driven piles, $\phi = \frac{\phi'_1 + 40}{2}$

For bored piles, $\phi' = \phi_1 - 3$
where ϕ_1 is the angle of shearing resistance prior to installation.

Fig. 13b: Relationship between N_q and ϕ (Poulos & Davis, 1980)

Bearing Capacity of Pile Foundation

Effect on φ due to Pile Installation: N_q for Pile Foundations in Sand

The value of φ used in the calculation of f_s for a cohesionless soil depends on the method of installation. During driving of driven pile, the soil gets compacted up to a lateral distance of about 3.5d from the tip of the pile. As a result , there will be an increase in the value of φ within this zone. If φ is the maximum value at the tip of the pile and φ_1 the value of φ before installation, then we may write (Kishida, 1967):

$$\varphi = \frac{\varphi_1 + 40}{2}$$

Bearing Capacity of Pile Foundation



Effect on φ due to Pile Installation: N_q for Pile Foundations in Sand

When $\varphi_1 = \varphi = 40^o$, there will be no change of relative density of soil due to pile driving. Kishida (1967) has also suggested the following relationship between φ and SPT value, N.

$$\varphi^o = 15^o + \sqrt{20(N_1)_{60}}$$

Tomlinson (1986) suggested not to use higher values for φ due to driving. Sand may not get compacted always, especially when piles are driven to loose sand. He suggests insitu value of φ for the design purposes.

Bearing Capacity of Pile Foundation



Effect of Overburden on Skin Friction and End Bearing: Concept of Critical Depth

The vertical stress σ'_v increases linearly with depth from zero at ground level to a maximum value $\sigma'_{v(max)}$ or q_o at the pile tip. However, extensive research carried out by Vesic (1967) has indicated that σ'_v varies linearly from the ground surface up to a limited depth only beyond which, σ'_v, remains constant irrespective of the depth of embedment of pile. This phenomenon was attributed to arching of sand. This depth within which σ'_v varies linearly with depth may be called as critical depth, z_c as shown in Fig. 14, which is a function of φ.

Bearing Capacity of Pile Foundation

Effect of Overburden on Skin Friction and End Bearing: Concept of Critical Depth

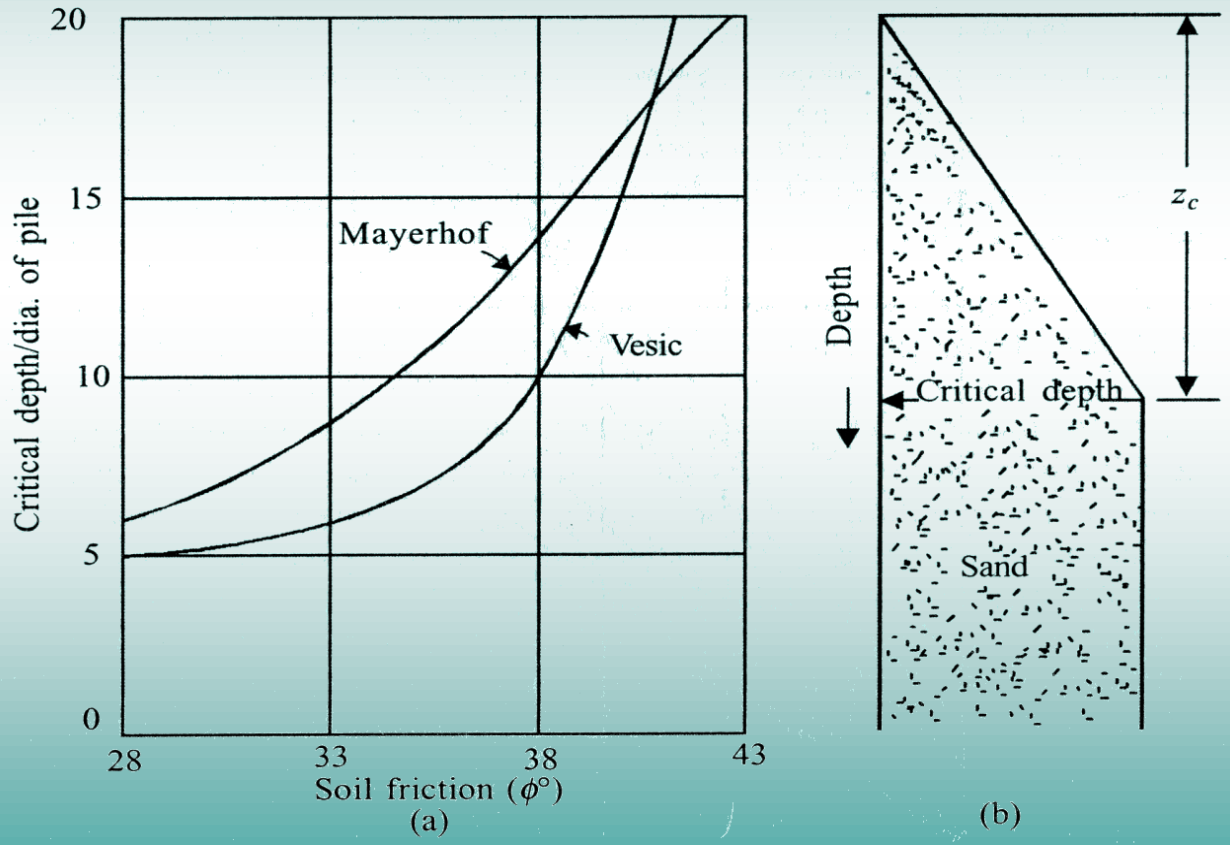


Fig. 14: Critical Depth in Pile Foundations.

Bearing Capacity of Pile Foundation



Effect of Overburden on Skin Friction and End Bearing: Concept of Critical Depth

Poulos (1980) suggests the following relationships for critical depth.

For $28^o < \varphi < 36.5^o$

$$\frac{z_c}{d} = 5 + 0.24(\varphi^o - 28^o)$$

For $36.5^o < \varphi < 42^o$

$$\frac{z_c}{d} = 7 + 2.35(\varphi^o - 36.5^o)$$

Critical depth concept implies that f_s varies with depth up to a depth z_c only and beyond this depth f_s remains constant. The overburden pressure for end bearing should be the same.

Bearing Capacity of Pile Foundation



Determinations of q_o , K_S and δ for f_s

The effective average overburden pressure $(\sigma'_v)_a$ for the whole depth of embedment L of the pile if the pile is driven to homogeneous soil or each layer of a layered system, may be calculated either by taking the critical depth concept or otherwise. If the soil is submerged, submerged unit weight of the soil is taken in the calculation.

Bearing Capacity of Pile Foundation



Determinations of q_o , K_S and δ for f_s

The values of K_s and δ vary not only with the relative density and the pile material but also with the method of installation of pile.

Pile material	δ	*Values of* K_s *Low* D_r	*High* D_r
Steel	20°	0·5	1·0
Concrete	3/4 ϕ	1·0	2·0
Wood	2/3 ϕ	1·5	4·0

Bearing Capacity of Pile Foundation



Determinations of q_o , K_S and δ for f_s

Poulos and Meyerhof have proposed the relationship between $K_s \tan\delta$ and φ^o as shown in Figs. 15(a) and 15(b). Meyerhof assumes $\delta = 0.75\varphi$. The angle φ to be used in Figures are:

For driven pile

$$\varphi = 0.75\varphi_1 + 10^o$$

For bored pile

$$\varphi = \varphi_1 - 3^o$$

Bearing Capacity of Pile Foundation

Determinations of q_o , K_S and δ for f_s

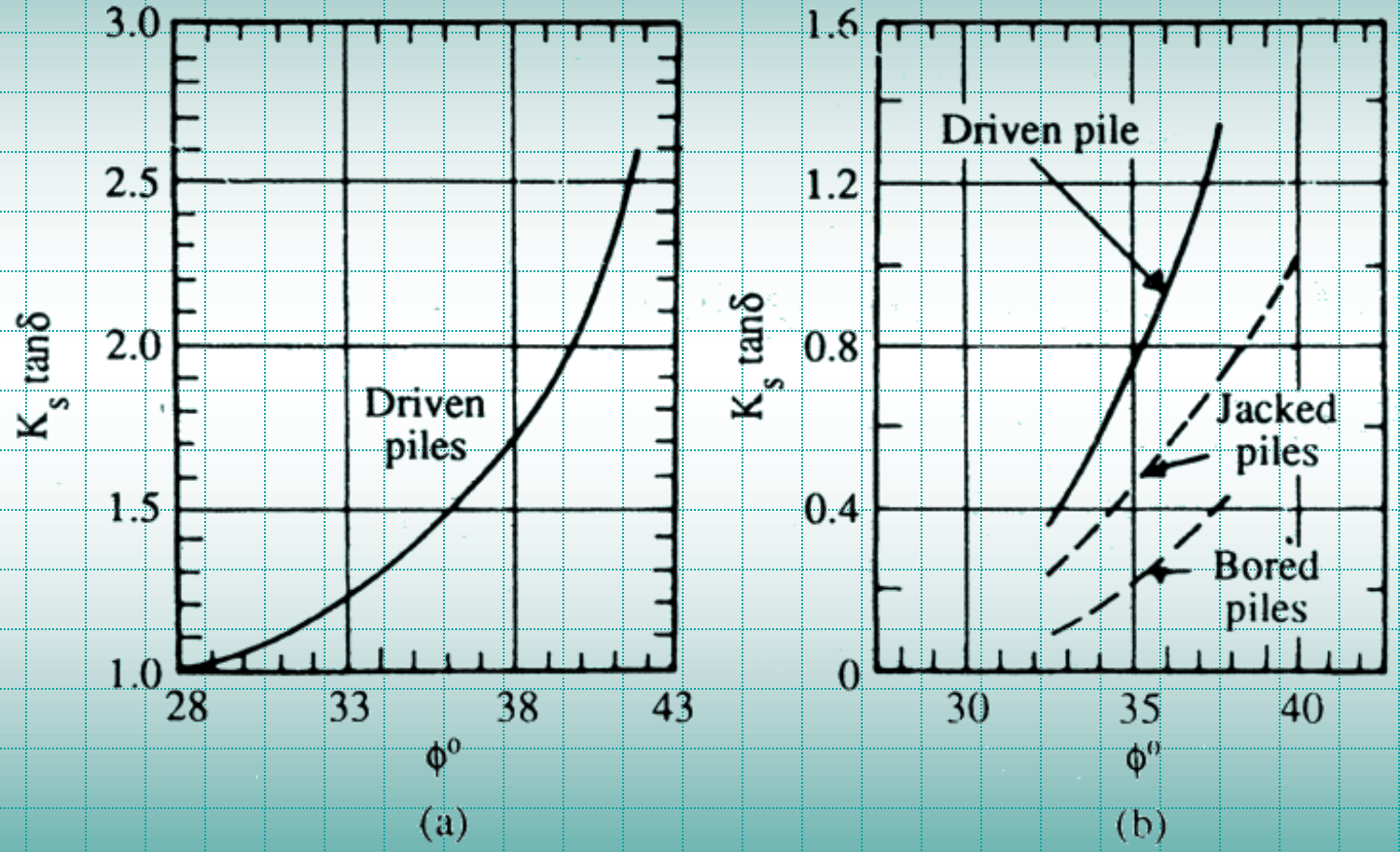


Fig. 15: Values of $K_s \tan \delta$ in cohesionless soil; (a) Poulos; (b) Meyerhof

Bearing Capacity of Pile Foundation



Static Analysis Method: Cohesive Soil

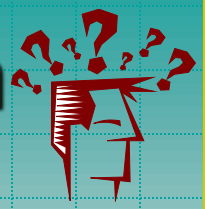
Driven Piles

- α – method
- β – method
- λ – method
- Nordlund – Thurman method
- SPT-method
- CPT-method

Bored Pile

- α – method
- β – method
- Side friction in Rock
- Tip Resistance in Rock

Bearing Capacity of Pile Foundation



Static Analysis: Cohesive Soil: α – Method

In α – method unit skin friction resistance on the pile f_s is expressed as the adhesion between pile and soil, c_a.

$$f_s = c_a = \alpha c_u$$

c_u = undrained cohesion/ shear strength (s_u) of soil

α = a factor called adhesion factor

The value of adhesion factor α depends on the undrained strength of soil. Smaller the undrained strength more is the tendency for the soil to adhere to the pile surface and α also tends to be close to 1.

Bearing Capacity of Pile Foundation

Static Analysis: Cohesive Soil: α – Method

Of the several correlations of α and c_u shown in the Fig. 16, Tomlinson's curve is most popularly used.

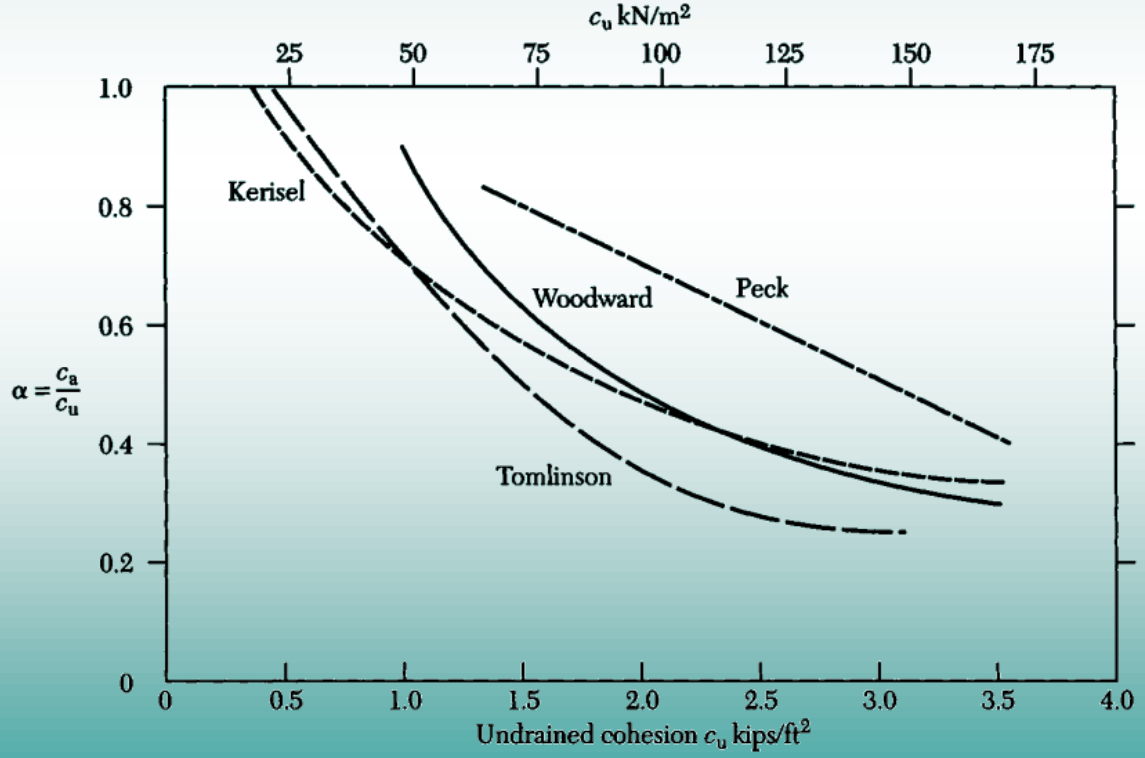


Fig. 16: Adhesion Factor,α in Pile Foundation.

Bearing Capacity of Pile Foundation



Static Analysis: Cohesive Soil: α – Method for Layered Soil

The total skin friction load on the pile is obtained as,

$$Q_s = \int_0^L P f_s dz$$

Where P = perimeter of the pile

L = length of the pile

When piles are driven through layered soils the skin friction resistance f_s varies over the length of pile. Also, when the pile penetrates into a lower layer it drags along with it a skin upper layer for some distance.

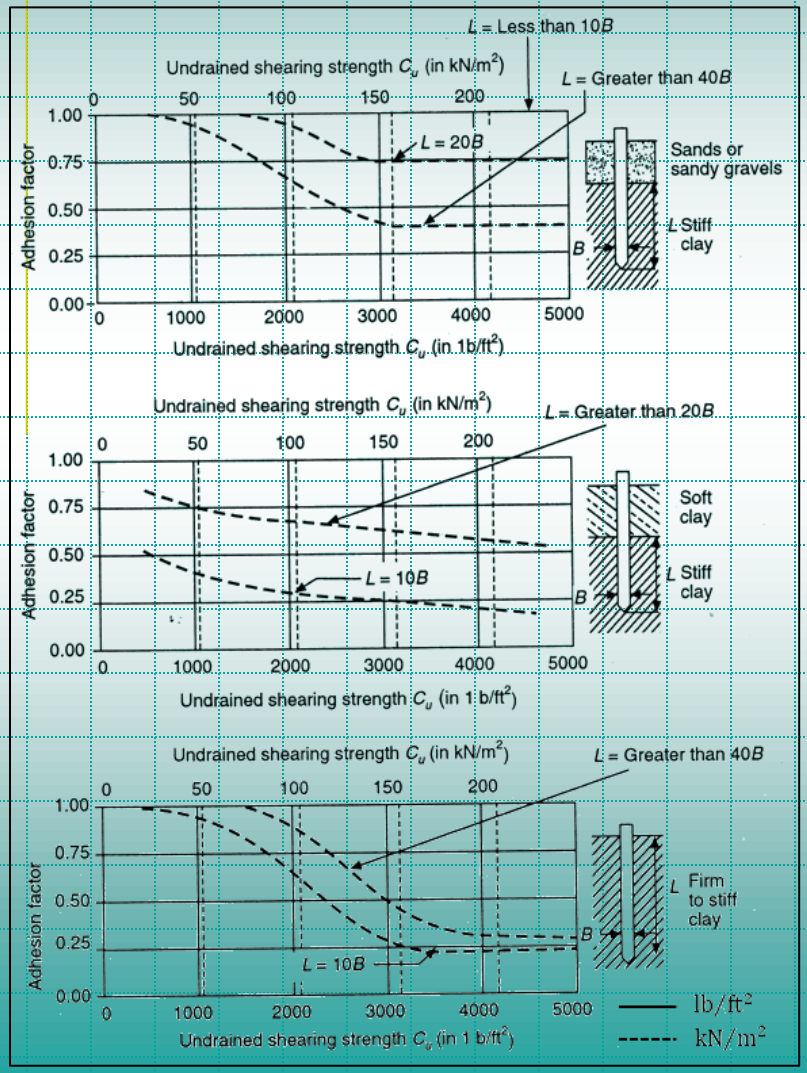
Bearing Capacity of Pile Foundation

Static Analysis: Cohesive Soil: α – Method for Layered Soil

The skin friction resistance f_s in the lower layer will depend on the characteristics of the upper layer in relation to the lower layer and the length penetration into the lower layer. Tomlinson (1970) recommends modified α values as shown in Fig. 17 and the following Table.

Bearing Capacity of Pile Foundation

Static Analysis: Cohesive Soil: α – Method for Layered Soil



← Fig. 17: Adhesion Factor, α for Pile Foundations in Layered Soil.

Bearing Capacity of Pile Foundation



Static Analysis Methods: Cohesive Soil: α – Method for Layered Soil

Case	Soil conditions	Penetration ratio†	c_a/c_u
I	Sands or sandy soils overlying stiff	< 20	*
	cohesive soils	> 20	See Fig. 15.6
II	Soft clays or silts overlying stiff	< 20 (> 8)	0.40
	cohesive soils	> 20	0.70
III	Stiff cohesive soils without	< 20 (> 8)	0.40
	overlying strata	> 20	See Fig. 15.6

$$\text{Penetration Ratio} = \frac{\text{Depth of penetration in stiff clay}}{\text{Pile diameter}}$$

Note 1: Adhesion factors are not applicable to H-section piles.

Note 2: Shaft adhesion for Cases I and II must be calculated separately.

Bearing Capacity of Pile Foundation



Static Analysis: Cohesive Soil: β – Method

Skin friction is expressed in this method similar to that of cohesionless soil as,

$$f_s = K_s \sigma_v' \tan\delta$$

The value, $K_s \tan\delta$ is often taken as a single term β that is,

$$\beta = K_s \tan\delta$$

The value, β is often taken as

$$\beta = (1 - \sin\varphi)\tan\varphi$$

Bearing Capacity of Pile Foundation



Static Analysis: Cohesive Soil: β – Method

For normally consolidated clay β varies from 0.25 to 0.29. There are evidences to indicate that value of β decreases for very long piles, otherwise very high skin frictional resistance will be indicated at large depth.

The value of β could be as low as 0.15 for piles exceeding 60 m length. Bowles (1982) recommends that the β value should be multiplied by a correction factor which has been modified by Kaniraj (1988).

Bearing Capacity of Pile Foundation



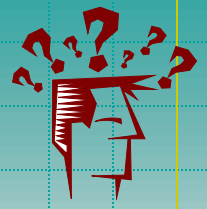
Static Analysis: Cohesive Soil: β – Method

Correction factor for $\beta = \log_{10}\left(\frac{180}{L}\right) \leq 0.5$

For piles in stiff over-consolidated soils the lower limit of β **is given by**

$$\beta = (1 - \sin\varphi)\tan\varphi\sqrt{OCR}$$

Bearing Capacity of Pile Foundation



Static Analysis: Cohesive Soil: λ– Method

This method is a pseudo-effective stress approach developed by Vijayavergia and Focht (1972) who suggest an empirical relationship for average skin friction, $f_{s(av)}$, from analysis of load test results on steel pipe piles and is given by,

$$f_{s(av)} = \lambda\left(\sigma'_{v(av)} + 2c_{u(av)}\right)$$

Where,

$\sigma'_{v(av)}$ = average effective vertical stress between the ground surface and pile tip;

$c_{u(av)}$ = average undrained shear strength along the length of the pile, and

λ = an empirical factor that depends on pile length, Fig. 18

Bearing Capacity of Pile Foundation

Static Analysis: Cohesive Soil: λ– Method

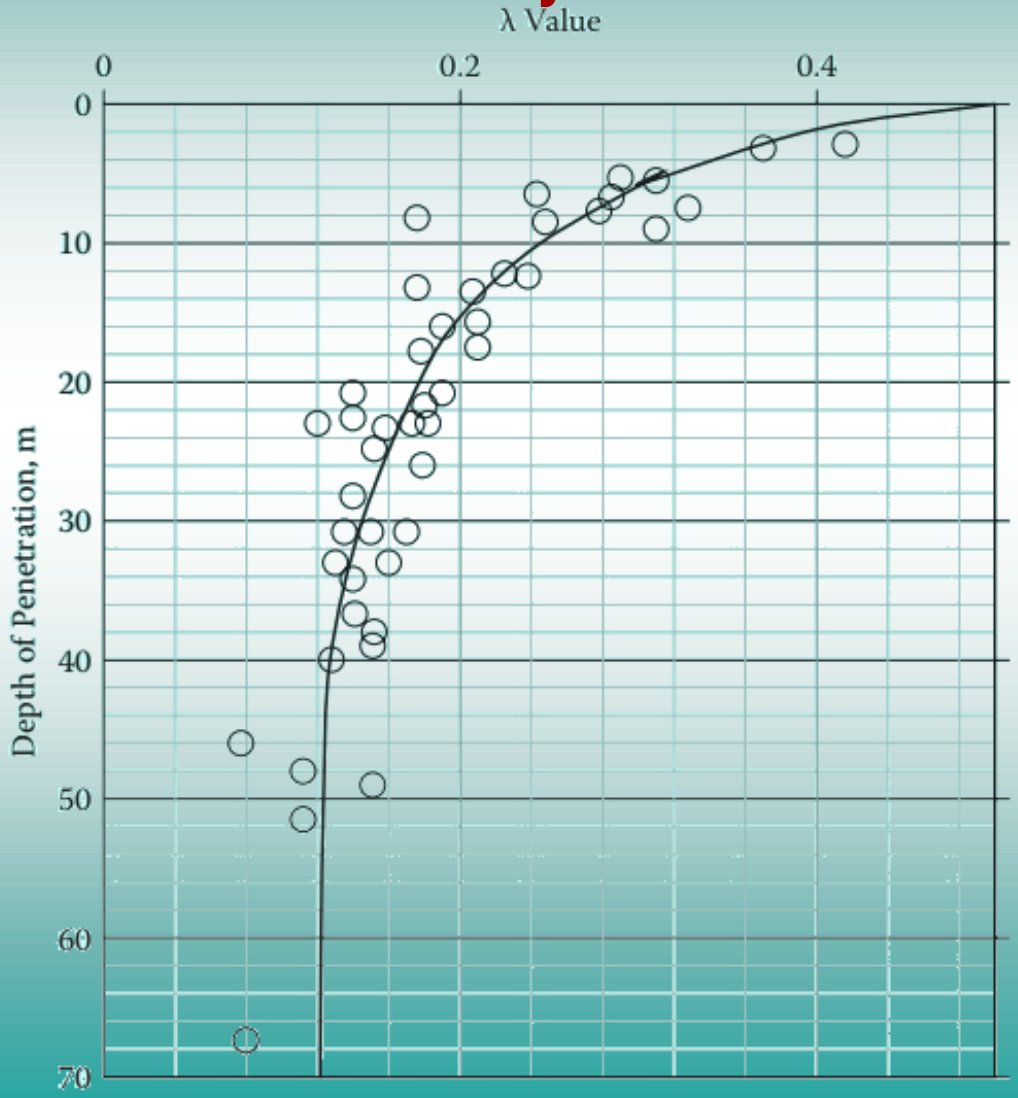


Fig. 18 Value of λ for Different Pile Penetration Length,

Bearing Capacity of Pile Foundation



Static Analysis: Cohesive Soil: End Bearing: Total Stress Approach

In the total stress approach, the ultimate point load of piles driven into clays is given by,

$$Q_b = c_{u(b)} N_c A_b$$

The value of N_c is generally taken as 9 for square and circular pile sections. Note that pile should be driven to a distance of at least 5d (d is the diameter of pile) into the bearing stratum.

Bearing Capacity of Pile Foundation

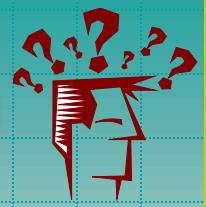


Static Analysis: Cohesive Soil: End Bearing: Effective Stress Approach

In the effective stress approach, Q_b may be expressed in a similar fashion of that of sandy soil as:

$$Q_b = (q_o N_q) A_b = (\sigma'_v N_q) A_b$$

Bearing Capacity of Pile Foundation



Static Analysis: Bored Cast in Situ Piles

The load carrying capacity of bored and cast-in-situ pile is significantly less than that the capacity of driven pile. It also depends on the method of construction.

For preliminary estimates one-third of f_s and one-half of q_b as calculated for driven piles can be used for unit skin friction and unit point resistance, respectively.

Bearing Capacity of Pile Foundation



Static Analysis: Piles in $c - \varphi$ Soil

For piles constructed in soils having both cohesion and friction, the principle of superposition considering both cohesion (c) and friction (φ) explained for clays and sands can be adopted.

Bearing Capacity of Pile Foundation



Safe Compressive Load: Cohesionless Soil

For piles in sand with shaft diameter less than 600 mm a factor of safety (FS) of 2.5 on Q_u computed using static formulae is satisfactory. The safe load Q_{safe} is given by:

$$Q_{safe} = \frac{Q_u}{FS} = \frac{Q_s + Q_b}{FS}$$

Bearing Capacity of Pile Foundation



Safe Compressive Load: Cohesive Soil

The ultimate skin-frictional load is calculated in two ways:

(i) using average shear strength and adhesion factors as in Fig. (Slide 44), say Q_s'; and

(ii) using lowest range of shear strength and adhesion factors of the Fig., say Q_s''.

The safe load Q_{safe} is the smaller of the two values:

$$Q_{safe} = \frac{Q_s' + Q_b}{2.5} \quad or, \quad Q_{safe} = \frac{Q_s''}{1.5} + \frac{Q_b}{3.0}$$

Bearing Capacity of Pile Foundation



Bearing Capacity Based on SPT Value: Driven Pile: Cohesionless Soil

Meyerhof (1959) proposed the formula:

For $L/D < 10$

$$Q_u = 4(L/D)N_{(b)}A_b + \frac{\bar{N}A_s}{5}$$

For $L/D \geq 10$

$$Q_u = 40N_{(b)}A_b + \frac{\bar{N}A_s}{5}$$

An upper limit of 6 ton/m^2 is suggested for the shaft resistance, and 1000 ton/m^2 is suggested for base resistance.

Bearing Capacity of Pile Foundation



Bearing Capacity Based on SPT Value: Driven Pile: Non-plastic Silt & Fine Sand

IS proposed the formula:

For $L/D < 10$

$$Q_u = 3(L/D)N_{(b)}A_b + \frac{\bar{N}A_s}{6}$$

For $L/D \geq 10$

$$Q_u = 30N_{(b)}A_b + \frac{\bar{N}A_s}{6}$$

An upper limit of 6 ton/m^2 is suggested for the shaft resistance, and 1000 ton/m^2 is suggested for base resistance.

Bearing Capacity of Pile Foundation



Bearing Capacity Based on SPT Value: Driven Pile: Plastic Silt & Clays

Meyerhof's formula may be extended for pile in clay of any length assuming adhesion factor α as 1.00.

$$Q_u = 4.5N_{(b)}A_b + \frac{\bar{N}A_s}{2}$$

An upper limit of 7 ton/m^2 is suggested for the shaft resistance, and 1000 ton/m^2 is suggested for base resistance.???????

Bearing Capacity of Pile Foundation



Bearing Capacity Based on SPT Value: Driven Pile:

All Soils: $L/D \geq 10$

End Bearing (kN/m²)

Sand = 400 N ($\leq$ 10000 kN/m²)

Silt = 300 N

Clay = 45 N

Skin Friction (kN/m²)

Sand $= 2\overline{N}$ ($\leq$ 60 kN/m²)

Silt $= 2.67\overline{N}$

Clay $= 5\overline{N}$ ($\leq$ 70 kN/m²)

Steel H-piles $= 1.0\overline{N}$

Bearing Capacity of Pile Foundation



Group Piles

Piles are commonly used in groups with a foundation slab or pile cap cast over the pile heads to distribute the load to the piles.

Although the piles in a group are generally identical, the group capacity is not necessarily the same as the capacity of single pile multiplied by the number of piles in the group.

Soil disturbance caused by pile installation techniques and overlap of stresses between adjacent piles, may reduce the group capacity from the sum of individual capacities.

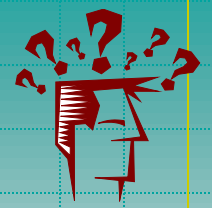
Bearing Capacity of Pile Foundation

Group Piles

On the other hand, soil between individual piles might get densified or increased with adhesion, and the group may tend to behave as an equivalent single large pile which may show higher group capacity.

The stress isobars of a single pile carrying a concentrated load will be somewhat as shown in Fig. 19(a). When piles are driven in a group, there is a possibility of stress isobars of adjacent piles overlapping each other as shown in Fig. 19(b).

Bearing Capacity of Pile Foundation



Group Piles

Since, the overlapping might cause failure either in shear or by excessive settlement, this possibility may be averted by increasing the spacing as shown in Fig. 19(c). Large spacing are not advantageous since a bigger size of pile cap would increase the overall cost of the foundation.

In case of a closely spaced group, the pressure bulb of individual piles overlap, Figs. 19 and 20 and a much larger bulb formed. Due to overlapping of stresses, the capacity of individual piles may decrease and result in a lower capacity of the group.

Bearing Capacity of Pile Foundation

Pressure Bulb in a Pile Group

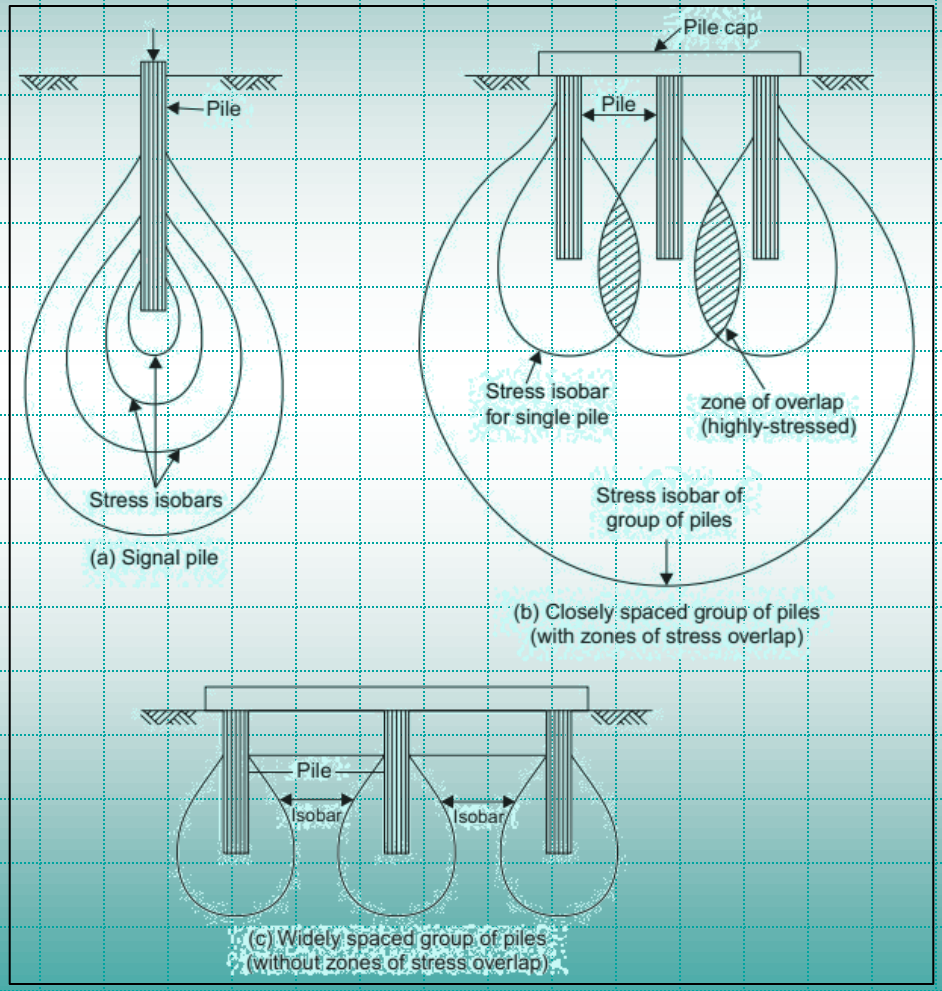
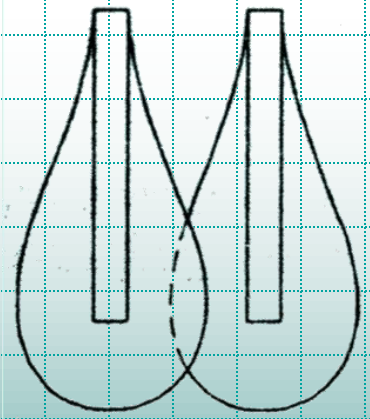
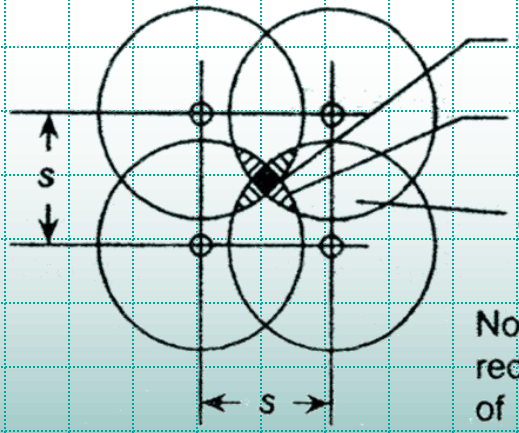


Fig. 19: Stress Isobars of Single Pile and Groups of Piles

Bearing Capacity of Pile Foundation

Pressure Bulb in a Pile Group





4 piles contribute to the stress in this zone.

3 piles contribute to the stress in this zone.

2 piles contribute to the stress in these zones.

Note that adequate spacing of the piles reduces the overlap zones and the number of piles contributing to any zone.

Fig. 20: Stress Isobars of Single Pile and Groups of Piles

Bearing Capacity of Pile Foundation



Spacing in Group Piles

The spacing of piles in a group is to be selected considering the following points:

Type of Soil and Method of Installation: Larger spacing may be required for driven piles in saturated clays to minimize the effect of heaving.

Minimum Spacing: Should be such that with due tolerance for error in layout and verticality, reasonable gap is maintained between adjacent piles.

Efficiency of the Group: The bearing should be such that the group efficiency is close to unity, that is, each pile can develop its full capacity.

Economy: Larger the group, costly is the cap.

Bearing Capacity of Pile Foundation



Spacing in Group Piles

In the case of driven piles there will be greater overlap of stresses due to the displacement of soil. If piles are driven in loose sands, compaction takes place and hence, the spacing may be small. However, if piles are driven in saturated silt or clay, compaction does not take place but the piles may experience uplift. To avoid this, greater spacing may be adopted.

Smaller spacing may be used for cast-in-situ piles in view of less disturbance.

Bearing Capacity of Pile Foundation

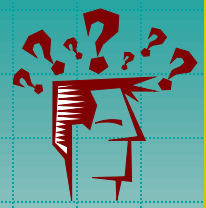


Spacing in Group Piles

Point-bearing piles may be more closely spaced than friction piles. The minimum spacing of piles is usually specified in building codes. The spacing may vary from 2d to 6d for straight uniform cylindrical piles, d being the diameter of the pile.

For friction piles, the recommended minimum spacing is 3d. For point-bearing piles passing through relatively compressible strata, the minimum spacing is 2.5d when the piles rest in compact sand or gravel; this should be 3.5d when the piles rest in stiff clay. The minimum spacing may be 2d for compaction piles.

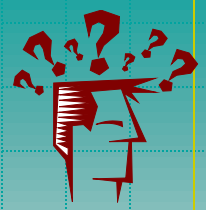
Bearing Capacity of Pile Foundation



Spacing in Group Piles

Piles should be, in general, driven proceeding outward from the centre, except in soft clay or very soft soil; in the latter case, the pile driving proceeds from the periphery of the foundation to the centre to prevent the lateral flow of soil during driving.

Bearing Capacity of Pile Foundation



Spacing in Group Piles: General Provisions

(i)	Bearing piles (general)	2.5d
(ii)	Bearing piles on rock	2.0d
(iii)	Friction piles	3.0d
(iv)	Piles driven through loose sand or fill	2.0d

Norwegian Code Provisions

Length of piles	Friction piles		Bearing piles
	in sand	in clay	
Less than 12 m	3*d*	4*d*	3*d*
12–24 m	4*d*	5*d*	4*d*
More than 24 m	5*d*	6*d*	5*d*

Bearing Capacity of Pile Foundation

Spacing in Group Piles

Sowers et al. (1961) have shown that the optimum spacing at which the group efficiency is unity for long friction piles in clay is given by:

$$s_o = 1.1 + 0.4n^{0.4}$$

Where, s_o =optimum spacing in terms of pile diameters; s_o is 2 to 3 pile diameters centre to centre, and n = number of piles in the group. The actual efficiency, η_g, at the theoretical optimum spacing is:

$$\eta_g = 0.5 + \frac{0.4}{(n - 0.9)^{0.1}}$$

This has been found to be 0.85 to 0.90, rather than unity. Since a factor of safety is used for design, the error in assuming the real efficiency to be 1.0 at optimum spacing is inconsequential.

Bearing Capacity of Pile Foundation



Group Capacity of Piles

The capacity of a pile group is not necessarily the capacity of the individual pile multiplied by the number of individual piles in the group. Disturbance of soil during the installation of the pile and overlap of stresses between the adjacent piles, may cause the group capacity to be less than the sum of the individual capacities.

Conversely, the soil between individual piles may become "locked in" due to densification from driving and the group may tend to behave as a unit or an equivalent single large pile.

Bearing Capacity of Pile Foundation



Group Capacity of Piles

Densification and improvement of the soil surrounding the group can also occur. These factors tend to provide the group a capacity greater than the sum of the capacities of individual piles.

The capacity of the equivalent large pile is analysed by determining the skin friction resistance around the embedded perimeter of the group and calculating the end-bearing resistance by assuming a tip area formed by this block, as in Fig. 21.

Bearing Capacity of Pile Foundation

Group Capacity of Piles

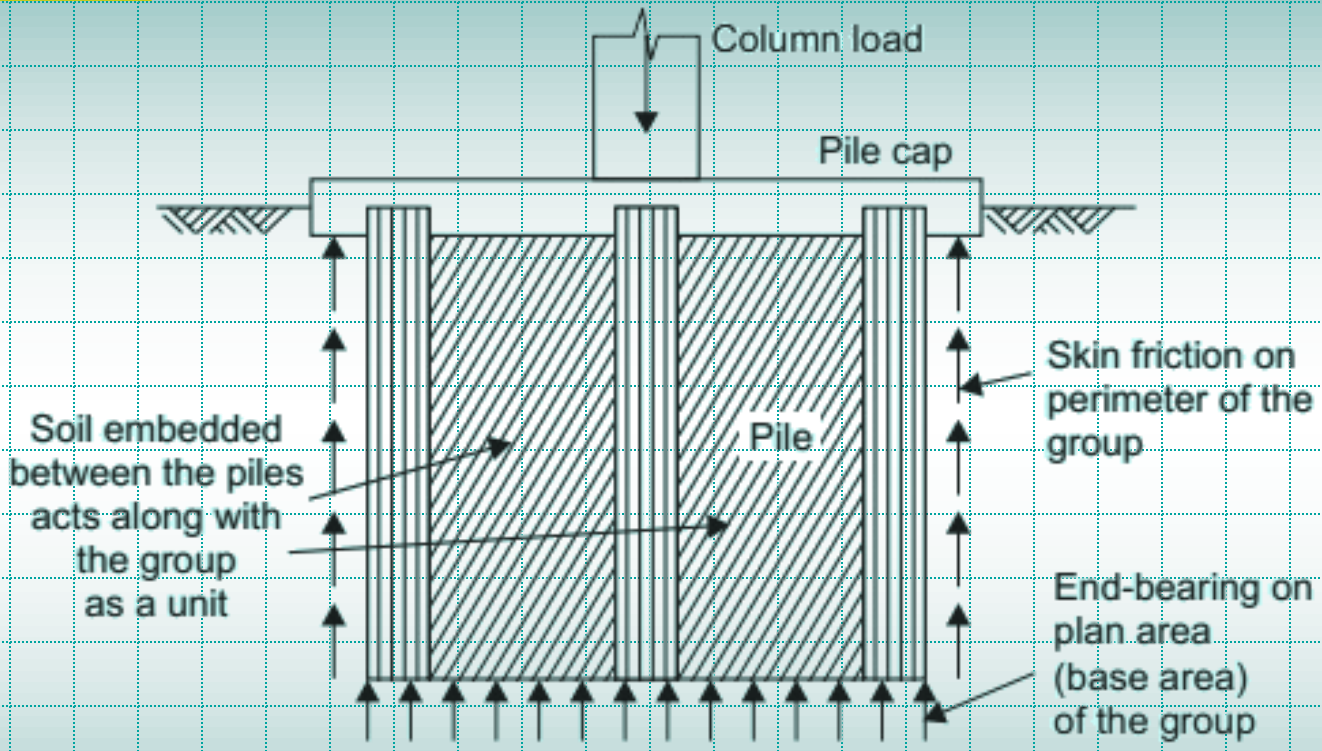


Fig. 21: Single Equivalent Large Pile Concept for a Group (Block Failure)

Bearing Capacity of Pile Foundation



Group Capacity of Piles

To determine the capacity of a pile group, the sum of the capacities of the individual piles is compared with the capacity of the single large equivalent (block) pile; the smaller of the two values is taken. Applying an appropriate factor of safety to this chosen value, the design load of the pile group is obtained.

The skin friction resistance of the single large equivalent pile (block) is obtained by multiplying the surface area of the group by the shear strength of the soil around the group.

Bearing Capacity of Pile Foundation



Group Capacity of Piles

The end-bearing resistance is computed by using the general bearing capacity equation of Terzaghi. The bearing capacity factors for deep foundations are used when the length of the pile is at least 10 (ten) times the width of the group; otherwise, the factors for shallow foundations are used.

The capacity of the equivalent large pile is affected by soil type and properties, besides spacing of piles. Generally speaking, there is a greater tendency for the group to act as a block or large single unit when the piles are close and the soil is firm or compact.

Bearing Capacity of Pile Foundation

Case I: Pile Group in Cohesive Soil: Pile Cap Above Ground Level

The ultimate bearing capacity of a group, Q_{ug} can be estimated using several approaches:

(a) Group capacity as the capacity of a single pile multiplied by the number of piles. This is true mostly for bearing piles and piles in sand.

(b) As mentioned before, the pile group with the soil enclosed within the group is considered as an equivalent pier of dimensions given by the peripheral dimensions of the group, Fig. 22.

Bearing Capacity of Pile Foundation

Case I: Pile Group in Cohesive Soil: Pile Cap Above Ground Level

The ultimate capacity of pile group is given by:

$$Q_{ug} = Q_{sg} + Q_{bg} \implies Q_{ug} = f_{sg}A_{sg} + q_{bg}A_{bg}$$

$$Q_{ug} = [2(a+b)L]c_u + cN_cA_b$$

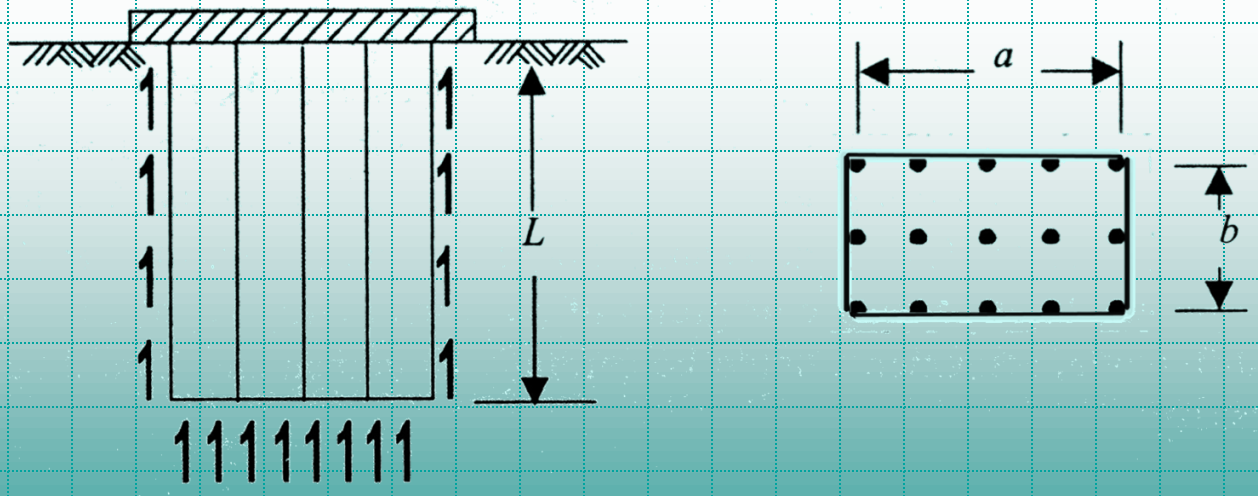


Fig. 22: Block Concept in Group Piles.

Bearing Capacity of Pile Foundation



Case II: Capacity of Pile Group: Pile Cap Cast Directly on Reasonably Firm Strata

The additional capacity of the pile cap on the ground is added to the capacity as worked out in Case I. Deductions in the pile cap bearing area are also to be made for the area occupied by the piles in the pile cap. This is often termed as piled foundation.

Bearing Capacity of Pile Foundation



Case III: Capacity of Pile Group: Presence of Weak Clay Layer under the Bearing Stratum

The strength of the foundation at the base with a spread of 2:1 at 2/3L (where L is the length of the pile) in the case of piles in clay and at the end of the piles in case of sand as shown in Fig. 23 should be examined to ensure the safety in bearing of the foundation.

Bearing Capacity of Pile Foundation

Case III: Capacity of Pile Group: Presence of Weak Clay Layer under the Bearing Stratum

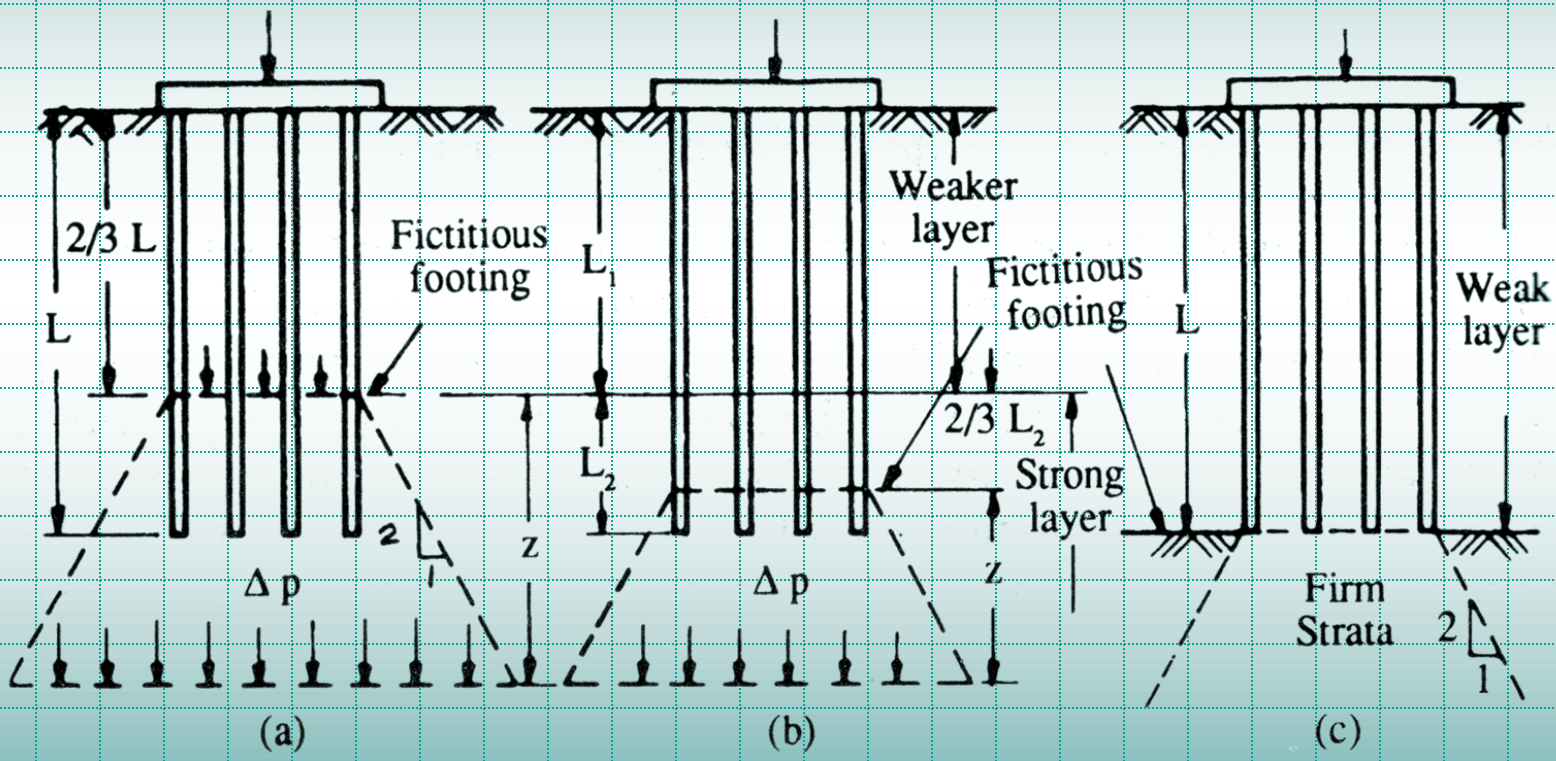


Fig. 23: Presence of weal Layer in Pile Foundations.

Bearing Capacity of Pile Foundation



Group Efficiency: Converse–Labarre Formula

$$\eta_g = 1 - \left[\frac{(n_1 - 1)n_2 + (n_2 - 1)n_1}{90 n_1 n_2}\right]\theta$$

Where,

η_g = efficiency of the group
n_1 = number of rows
n_2 = number of piles in a row
θ = $\tan^{-1}$ (d/s) in degrees
d = diameter of the piles
s = spacing of the piles

Bearing Capacity of Pile Foundation



Number of Piles in a Group and Arrangements

Usually driven piles are not used singly beneath a column or a wall because of the tendency of the pile to wander laterally during driving and consequent uncertainty with regard to centering the pile beneath the foundation. In cases where unplanned eccentricities result, failure may occur either at the connection between the pile and column or within the pile itself. Hence, piles for walls are commonly installed in a staggered arrangement to both sides of the centre line of the wall.

Bearing Capacity of Pile Foundation



Number of Piles in a Group and Arrangements

For a column, at least three piles are used in a triangular pattern, even for small loads. When more than three piles are required in order to obtain adequate capacity, the arrangement of piles is symmetrical about the point or area of load application.

Representative pile group patterns for wall and column loads are indicated in Fig. 24.

Bearing Capacity of Pile Foundation

Number of Piles in a Group and Arrangements

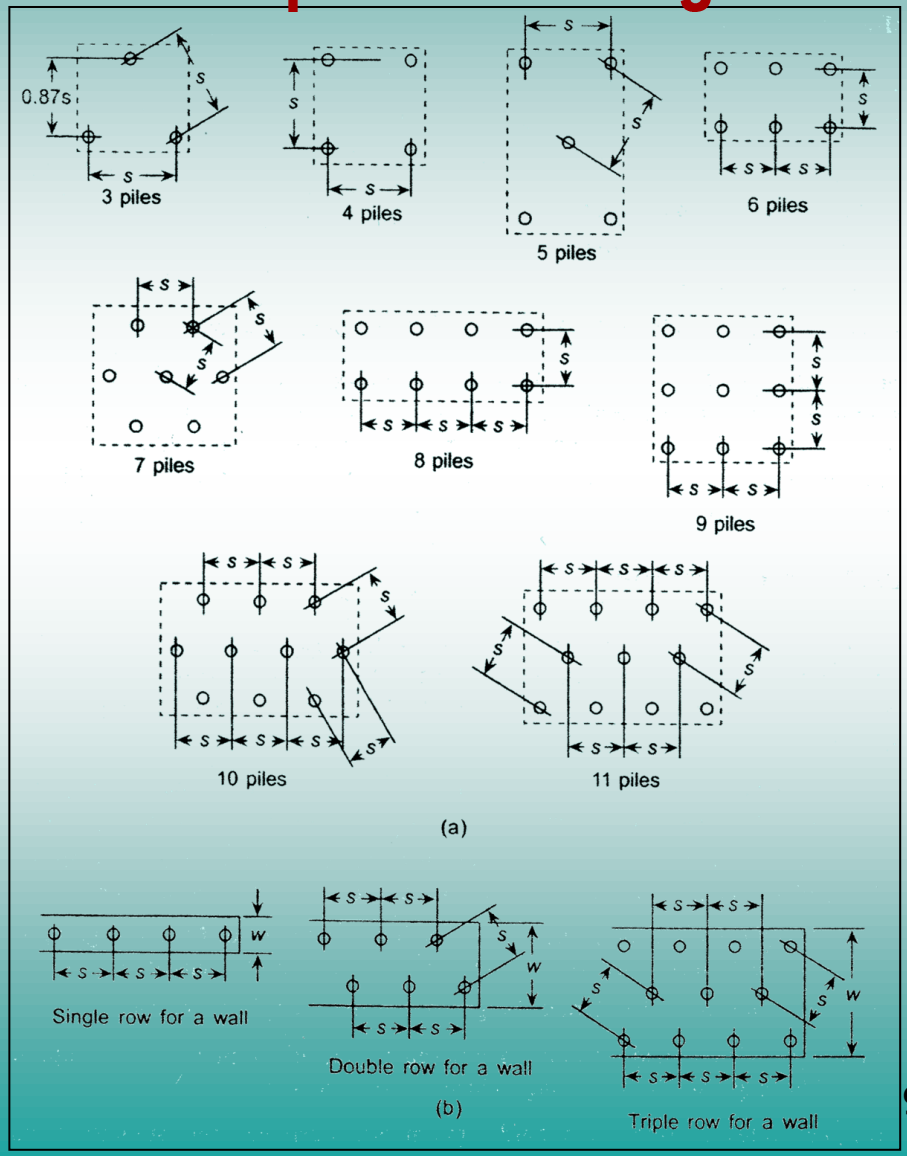


Fig. 24: Arrangements of Piles in a Group.

Bearing Capacity of Pile Foundation



Number of Piles in a Group and Arrangements

Column and wall loads are usually transferred to the pile group through a pile cap, which is typically a reinforced concrete slab structurally connected to the pile heads to help the group act as a unit. The requirement for group arrangement of driven piles does not apply to bored piles.

Drilled shafts can be installed quite accurately. A single large-diameter drilled shaft pile is commonly used to support columns in residential buildings. This may be used when the three pile configuration yields unnecessarily extra load carrying capacity in the case of driven piles.

Bearing Capacity of Pile Foundation



Pile Group Subjected to Eccentric Vertical Loads

The vertical reaction of a in a group subject to eccentric loading or moment can be expressed as (Fig.25).

$$R = \frac{Q}{n} \pm \frac{M_y x}{\sum x^2} \pm \frac{M_x y}{\sum y^2}$$

$R =$ total reaction at the pile head

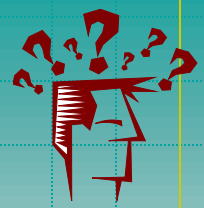
$Q =$ total vertical load acting on the pile cap

$n =$ number of piles in the group

$M_x =$ total moment about x-axis $= Q.e_y$

$e_y =$ eccentricity along y-axis.

Bearing Capacity of Pile Foundation



Pile Group Subjected to Eccentric Vertical Loads

$M_y =$ total moment about x-axis $= Q.e_x$

$e_x =$ eccentricity along x-axis.

$x =$ distance of the considered pile from y-axis.

$y =$ distance of the considered pile from x-axis.

$\sum x^2 =$ sum of the squares of the distances of all the piles from y-axis.

$\sum y^2 =$ sum of the squares of the distances of all the piles from x-axis.

Bearing Capacity of Pile Foundation

Pile Group Subjected to Eccentric Vertical Loads

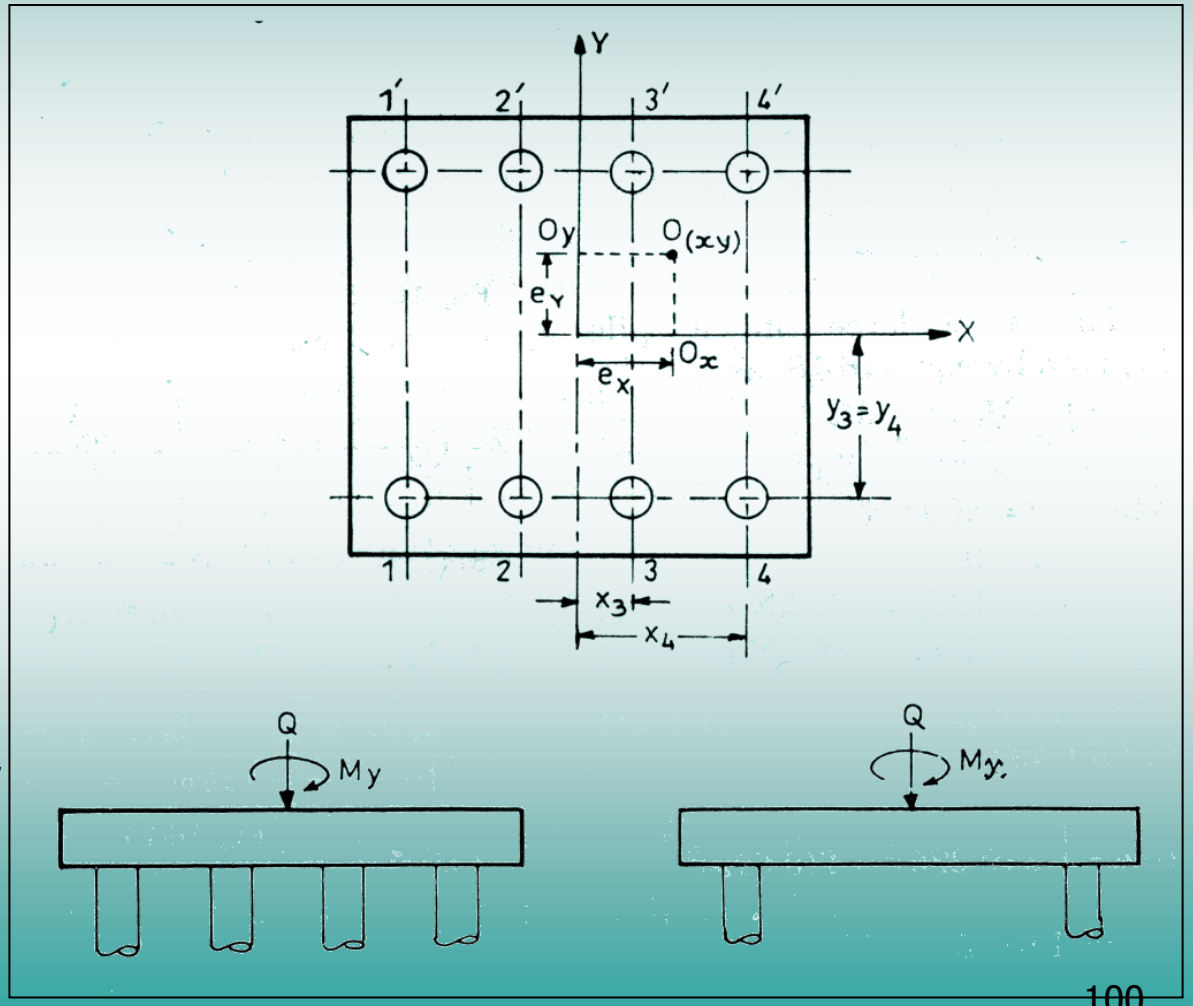


Fig. 25: Eccentrically Loaded Pile Group.

Bearing Capacity of Pile Foundation



Uplift Capacity of Pile

As a general rule, a factor of safety of 2-4 on the frictional resistance calculated for compression may be applied to determine the uplift capacity of piles. However, it should be noted that an upward movement of only 0.5-1% of the pile diameter is required to mobilize the peak frictional resistance. In case of pile groups, the uplift resistance may be calculated by taking the resistance of the soil enclosed within the groups as seen in Fig. 26.

Bearing Capacity of Pile Foundation

Uplift Capacity of Pile

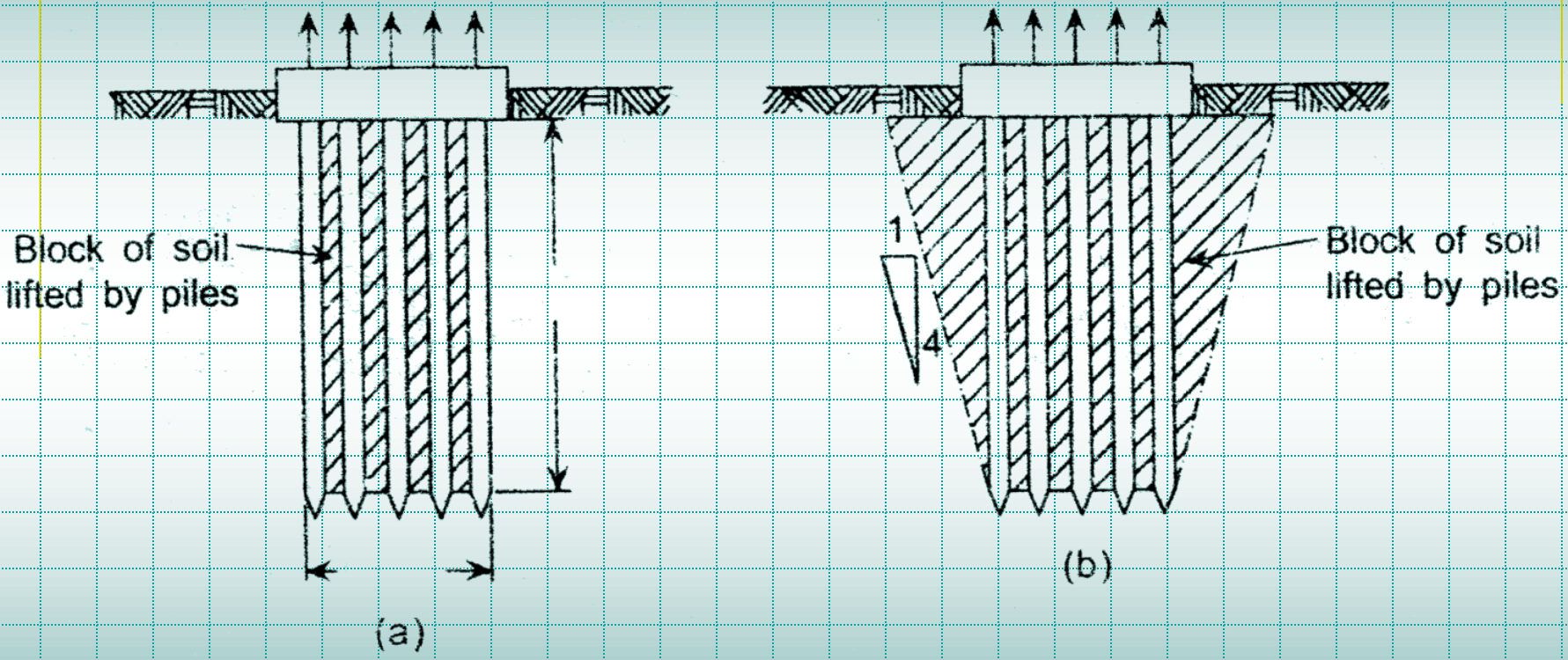


Fig. 26: Uplift Resistance of Pile Group; (a) in Clay; (b) in sand.

Bearing Capacity of Pile Foundation



Bearing Capacity of Pile from Pile Driving Formula

Except for the wave equation analysis, pile driving (or dynamic) formulae are normally applicable only for point bearing piles in cohesionless soils. These formulae are not applicable to clay soils as its strength is time dependent. In clays we may use the set condition to check only the quality of driving.

The components of a pre-cast pile driving system consist of several items, Fig. 27, which may be briefly described as follows:

Bearing Capacity of Pile Foundation

Bearing Capacity of Pile from Pile Driving Formula

- Pile Frame
- Pile Hammer
- Helmet
- Follower

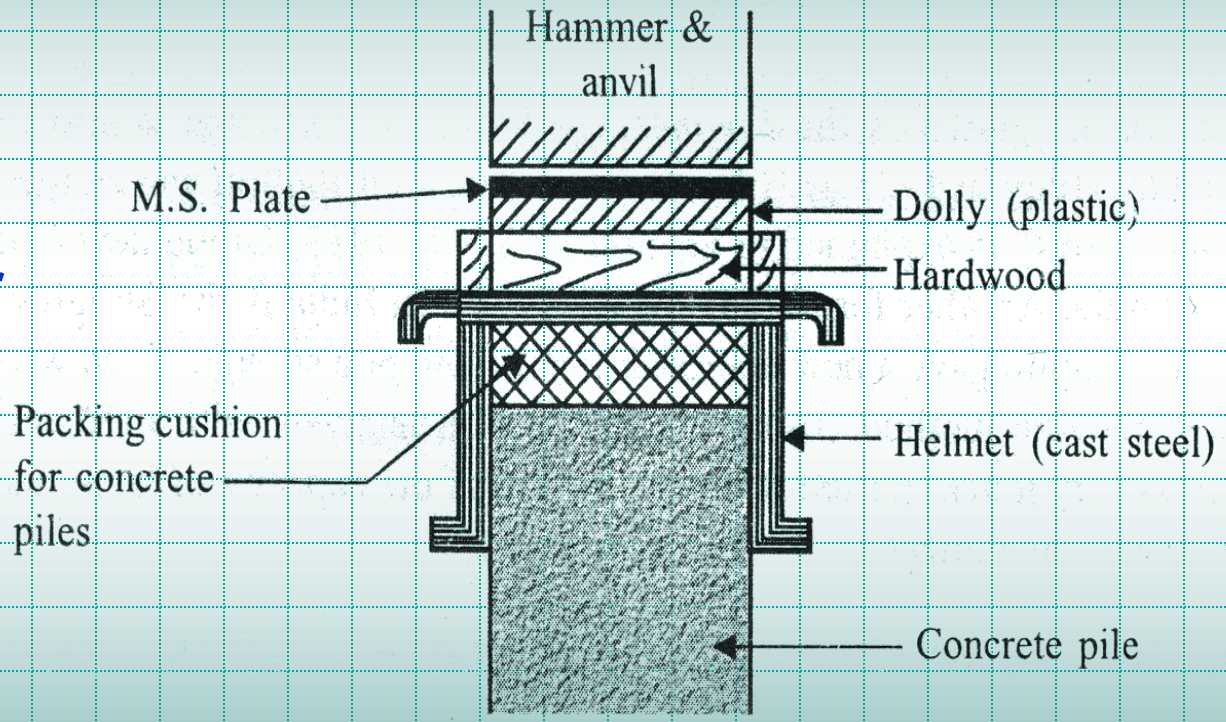


Fig. 27: Arrangements for Pile Driving Hammer.

Bearing Capacity of Pile Foundation



Pile Driving Hammers

Pile hammer consists of ram and anvil. These are made of cast iron or cast steel and are used to drive the pile. Hammer may be any one of the following types as illustrated in Figs. 28 and 29.

- Drop Hammer
- Single Acting Hammer (Steam or Pneumatic)
- Double Acting Hammer (Steam or Pneumatic)
- Diesel Hammer
- Vibratory Hammer

Bearing Capacity of Pile Foundation

Principles of Operation of Pile Driving Hammers

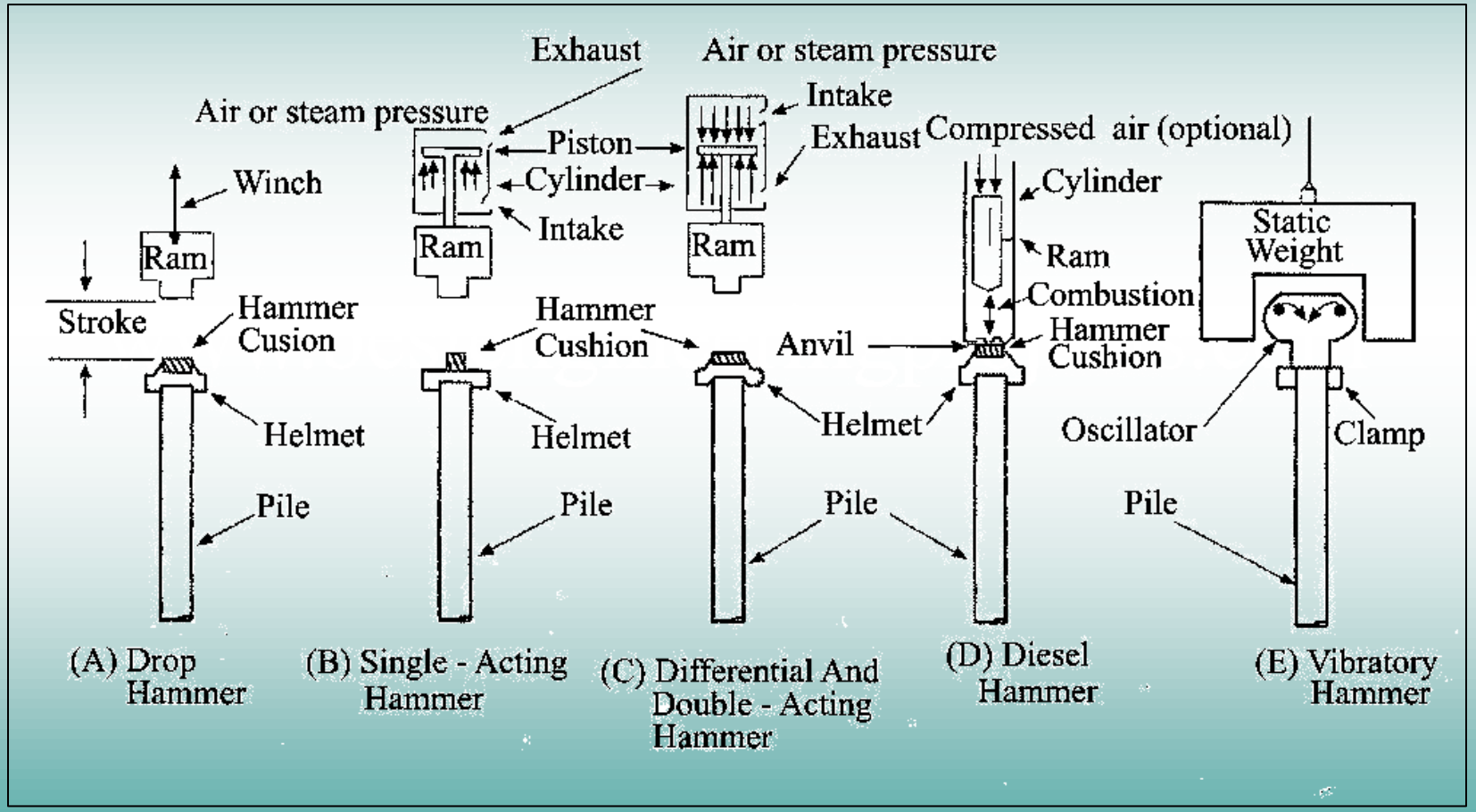


Fig. 28: Pile Hammers.

Bearing Capacity of Pile Foundation

Principles of Operation of Pile Driving Hammers

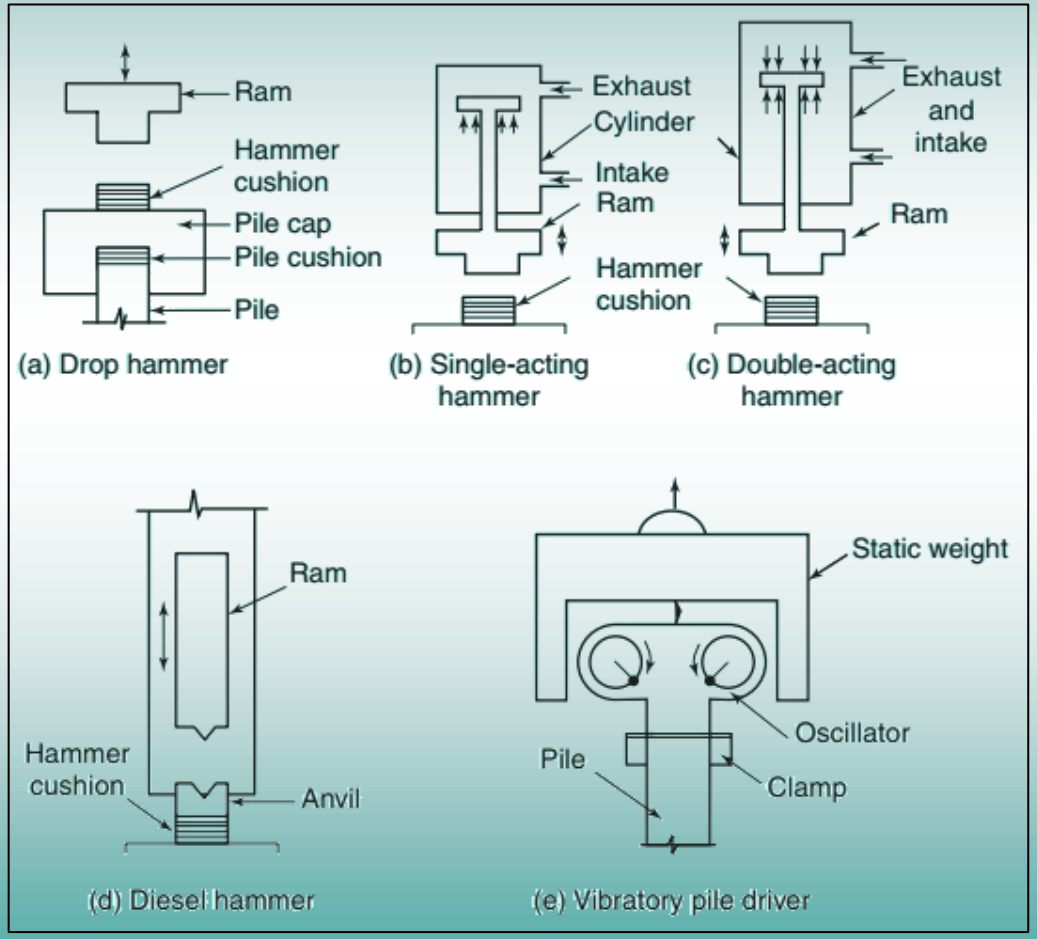


Fig. 28: Pile Hammers.

Bearing Capacity of Pile Foundation

Classification of Impact Pile Driving Hammers

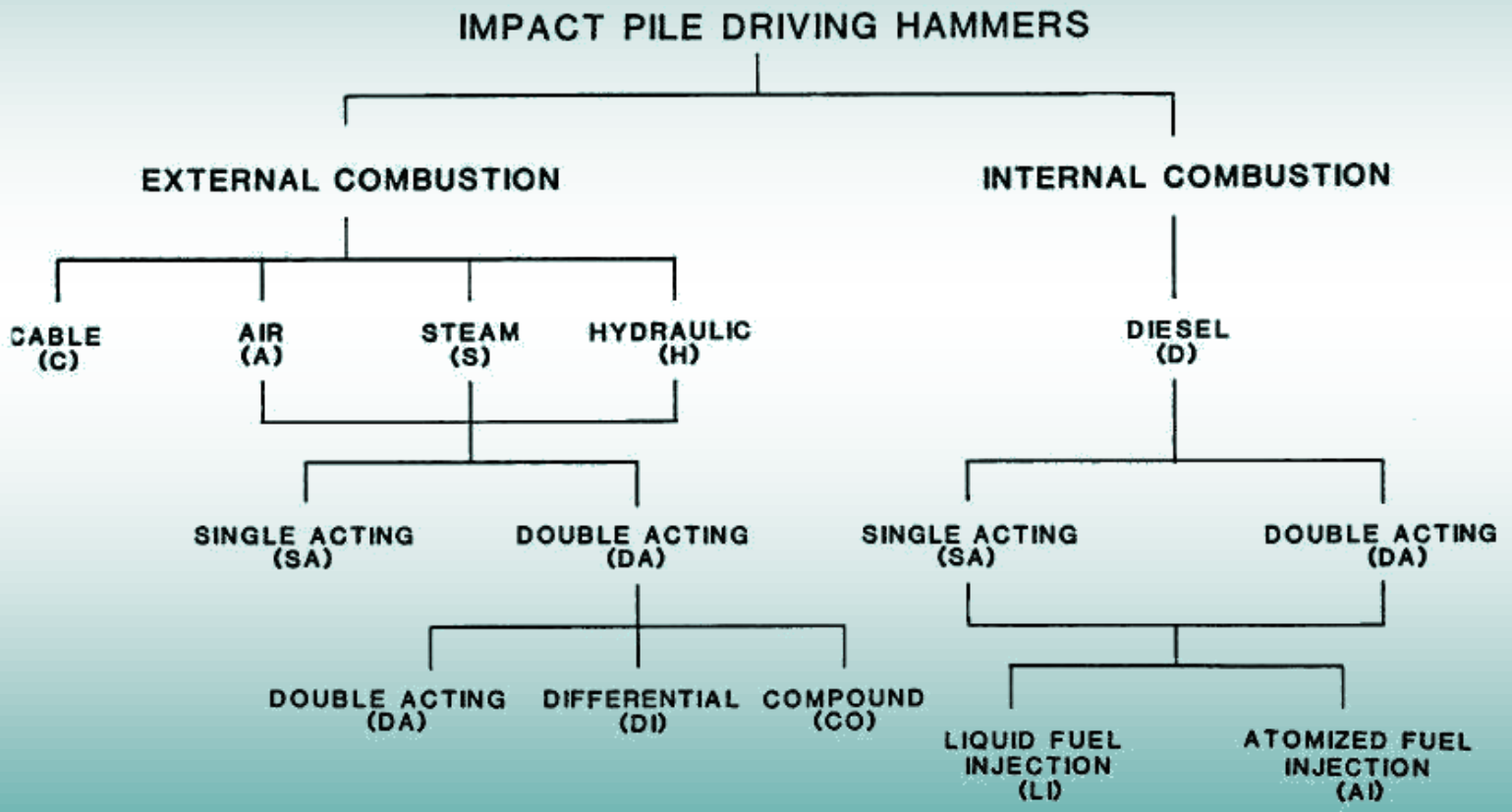


Fig. 29: Classification of Pile Hammers.

Bearing Capacity of Pile Foundation

Pile Driving Formulae



The resistance offered by the soil to penetration of pile during driving gives an indication of its bearing capacity. A pile that meets greater resistance during driving is capable of taking higher load. Dynamic pile formulae have been derived on the principle that the energy supplied to the pile is utilized in overcoming the resistance of the ground to the penetration of the pile.

The method to determine the ultimate capacity of piles by equating the energy imparted by the falling weight and the resistance to penetration of the pile during the last few blows (termed set) was first attempted by Sanders who published his formula in 1850.

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Engineering News

The prominent current pile driving formulae are:

- Engineering News Formula
- Hiley's Formula
- Janbu's Formula

The Engineering News formula given by Wellington (1886) and published in Engineering News Record is the simplest and most commonly used dynamic pile formula. Using a factor of safety of 6, the allowable pile capacity Q_a if given by:

$$Q_a = \frac{166.7WH}{(S + C)}$$

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Engineering News

Where,

Q_a = allowable pile load in kN

W = weight of hammer in kN

H = drop of hammer in m

S = average penetration of pile (in mm) per blow in final 150 mm of driving

C = additional penetration of the pile tip that would have been occurred if there were no energy loses.

= 25.4 mm for drop hammer

= 2.54 mm for steam hammer

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Modified Hiley

Modified Hiley formula is considered to be superior to the Engineering News formula, as it takes into account various energy losses during driving in a more realistic manner. According to this formula, the ultimate capacity of pile is given by:

$$Q_{ult} = \frac{WH\eta}{S + \frac{1}{2}(C_1 + C_2 + C_3)}$$

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Modified Hiley

Where,

Q_{ult} = ultimate pile capacity in kN

W = weight of hammer in kN

H = drop of hammer in m

η = efficiency of the blow that represents the ratio of energy after impact to the striking energy of ram.

S = final set or penetration per blow in mm

C_1 = temporary compression of dolly and packing

C_2 = temporary compression of pile

C_3 = temporary compression of ground

Bearing Capacity of Pile Foundation

Pile Driving Formulae: Modified Hiley

η = efficiency of the blow that represents the ratio of energy after impact to the striking energy of ram.

$$= \frac{(W + Pe^2)W}{(W + P)}, \quad for\ W > P$$

$$= \frac{(W + Pe^2)W}{(W + P)} - \left[\frac{(W - Pe)}{(W + P)}\right], for\ W \leq P$$

P = weight of pile, anvil, helmet and follower if any, kN.

e = coefficient of restitution of the materials under impact; recommended values of e are given in Table below.

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Modified Hiley

$C_1 = 0.177\frac{Q_{ult}}{A}$, where driving is done without dolly or helmet and cushion about 25 mm thick.

$C_2 = 0.905\frac{Q_{ult}}{A}$, where driving is done with short dolly up to 600 mm long, helmet and cushion up to 75 mm thick.

$C_2 = 0.0657L\frac{Q_{ult}}{A}$ and $C_3 = 0.355\frac{Q_{ult}}{A}$

L = length of the pile in metres, and

A = area of cross section of pile in mm.

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Modified Hiley

Table: Recommended values of coefficient of restitution, e		
Sl. No.	Description	e
1	For steel ram of double acting hammer striking on steel anvil and driving reinforced concrete pile	0.50
2	For cast iron ram of single acting or drop hammer striking on head of reinforced concrete pile	0.40
3	For single acting drop hammer striking a well conditioned driving cap and helmet with hard wood dolly while driving reinforced concrete piles or directly on head of timber pile	0.25
4	For deteriorated condition of head of pile or of dolly	0.00

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Modified Hiley

The recommended values of η in relations to e and P/W are given in Table. Where the pile finds refusal in rock, 0.5P should be substituted for P in the above expression for η. Safe load may be estimated by dividing Q_{ult} with a factor of safety of 2.5.

Bearing Capacity of Pile Foundation

Pile Driving Formulae: Modified Hiley

Table Values of η in Relation to e and P/W					
P/W	$e = 0.5$	$e = 0.4$	$e = 0.32$	$e = 0.25$	$e = 0.0$
0.5	0.75	0.72	0.70	0.69	0.67
1.0	0.63	0.58	0.55	0.53	0.50
1.5	0.55	0.50	0.47	0.44	0.44
2.0	0.50	0.44	0.40	0.37	0.33
2.5	0.45	0.40	0.36	0.33	0.28
3.0	0.42	0.36	0.33	0.30	0.25
3.5	0.39	0.33	0.30	0.27	0.22
4.0	0.36	0.31	0.28	0.25	0.20
5.0	0.31	0.27	0.24	0.21	0.16
6.0	0.27	0.24	0.21	0.19	0.14
7.0	0.24	0.21	0.19	0.17	0.12
8.0	0.22	0.20	0.17	0.15	0.11

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Jambu

Jambu's formula gives ultimate pile capacity (with a FS of 2.50) as:

$$R_u = \frac{1}{k_u} \times \frac{\alpha W H \eta}{S}$$

Where,

$$k_u = C_d \left(1 + \sqrt{1 + \frac{\lambda_c}{c_d}} \right)$$

$C_d = 0.75 + 0.15 \frac{P}{W}$, **and**

$$\lambda_c = \frac{\alpha W H}{A E S^2}$$

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Jambu

R_u = ultimate capacity (FS to be used, 2.5)

η = efficiency factor (0.70 for good, 0.55 for average and 0.40 for difficult or bad driving conditions)

W = weight of hammer/ram

P = weight of pile

H = height of fall of hammer in metres

α = hammer efficiency;

= 0.8 for winch operated drop hammers;

= 1.0 for trigger release drop hammers;

= 0.9 for single acting hammers;

= 1.0 for double acting hammers

Bearing Capacity of Pile Foundation



Pile Driving Formulae: Jambu

A = area of pile or casing tube in cast-insitu piles

E = elastic modulus of pile or casing tube in cast-in situ piles

S = set per blows (average of last four blows); L = length of pile

Bearing Capacity of Pile Foundation



Pile Load Test: Purposes

Given the many uncertainties inherent in the design and construction of piles, it is difficult to predict with accuracy the performance of a pile. Load test on a pile is one of the best methods of determining the load-carrying capacity of a pile.

It may be conducted on a driven pile or cast-in-situ pile, on a working pile or a test pile, and on a single pile or a group of piles. A working pile is one which forms part of the foundation, while a test pile is one which is used primarily to check estimated capacities (as predetermined by other methods).

Bearing Capacity of Pile Foundation



Pile Load Test: Types

The aim of a pile load test is invariably to determine the vertical load capacity; however, in certain special cases the test may be used to obtain the uplift capacity or lateral load capacity. Load test on a pile group is expensive and may be conducted only in the case of important projects. Depending on types of loading, understandably the pile load test can be of the following types:

- Vertical load tests or axial compression tests to determine the carrying capacity of vertical loads.
- Pull-out/uplift tests to evaluate tension capacity.
- Lateral load test for lateral capacity of pile.

Bearing Capacity of Pile Foundation



Pile Load Test: Types

The most common procedure is the test in which the load is maintained slowly (Maintained load method, ML). About five to eight equal increments are used until the load reaches about double the design value.

Time-settlement data are recorded for each load increment. Each increment is maintained until the rate of settlement becomes a value less than 0.25 mm per hour. The final load is maintained for 24 hours.

Another procedure is the constant-strain rate of penetration (CRP) method. In this method, the load is increased such that the settlement occurs at a predetermined rate such as 0.25 mm or 0.5 mm per minute. This test is considerably faster than the other approach.

Bearing Capacity of Pile Foundation



Pile Load Test: Types

Other procedures include cyclic loading, where each load increment is repeatedly applied and removed. Settlements are recorded at every increment or decrement of load. These help in separating elastic and plastic settlements, and also point-bearing and frictional resistances.

There are two broad types of pile loading tests, depending on the mode of application of load namely static and dynamic loading tests. Static loading tests are generally preferred because they have been traditionally used and also because they are perceived to replicate the long-term sustained load conditions.

Bearing Capacity of Pile Foundation



Pile Load Test: Types

Dynamic loading tests are usually carried out as a supplement to static loading tests and are generally less costly when compared with static loading tests. The failure mechanism in a dynamic loading test may be different from that in a static loading test.

Bearing Capacity of Pile Foundation



Pile Load Test: Timing

Both cohesive and cohesionless soils will have their properties altered by pile driving. In clays, the disturbance causes remoulding and consequent loss of strength. With passage of time, much of the original strength will be regained. The effect of pile driving in sand is to create a temporary condition wherein extra resistance is developed, which is lost later by stress relaxation. Hence, the test should be conducted only after a lapse of a few weeks in clays and at least a few days in sands, in order that the results obtained be more meaningful for design.

Pile capacity and load test are also attributed to the type of the pile and the construction procedures. As such, they must be concerns in deciding the timing of load tests.

Bearing Capacity of Pile Foundation



Pile Load Test: Timing

For cast-in-place piles, the timing of a loading test is usually dictated by the strength of the concrete or grout in the pile. Weltman (1980b) recommended that at the time of testing, the concrete or grout should be a minimum of seven days old and have a strength of at least twice the maximum applied stress.

Weltman (1980b) recommended that a driven pile should be tested at least 3 days after driving if it is driven into a granular material and at least 4 weeks after driving into a clayey soil, unless sufficient local experience or results of instrumentation indicate that a shorter period would be adequate for dissipation of excess pore pressure.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression

The basic principles of any pile load test mainly lies in the application of load and the measurement of pile head settlement or vice versa, using a suitable test assembly.

To ensure stability of the test assembly, careful consideration should be given to the provision of a suitable reaction system. The geometry of the arrangement should also aim to minimize interaction between the test pile, reaction system and reference beam supports. It is advisable to have, say, a 10% to 20% margin on the capacity of the reaction against maximum test load.

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Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression

Kentledge is commonly used for the application of load (Fig. 30a) on the pile. This involves the use of dead weights supported by a deck of steel beams sitting on crib pads. The area of the crib should be sufficient to avoid bearing failure or excessive settlement of the ground. The crib pads should be placed at least 1.3 m from the edge of the test pile to minimize interaction effects (ICE, 1988).

Alternatively, tension piles are used to provide reaction for the applied load (Fig. 30b) and they should be located as far as practicable from the test pile to minimize interaction effects.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression

A minimum centre-to-centre spacing of 2 m or three pile diameters, whichever is greater, between the test pile and tension piles is recommended.

A minimum of three reactions piles should be used to prevent instability of the set up during pile loading tests. Alternatively some from of lateral support should be provided.

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Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression

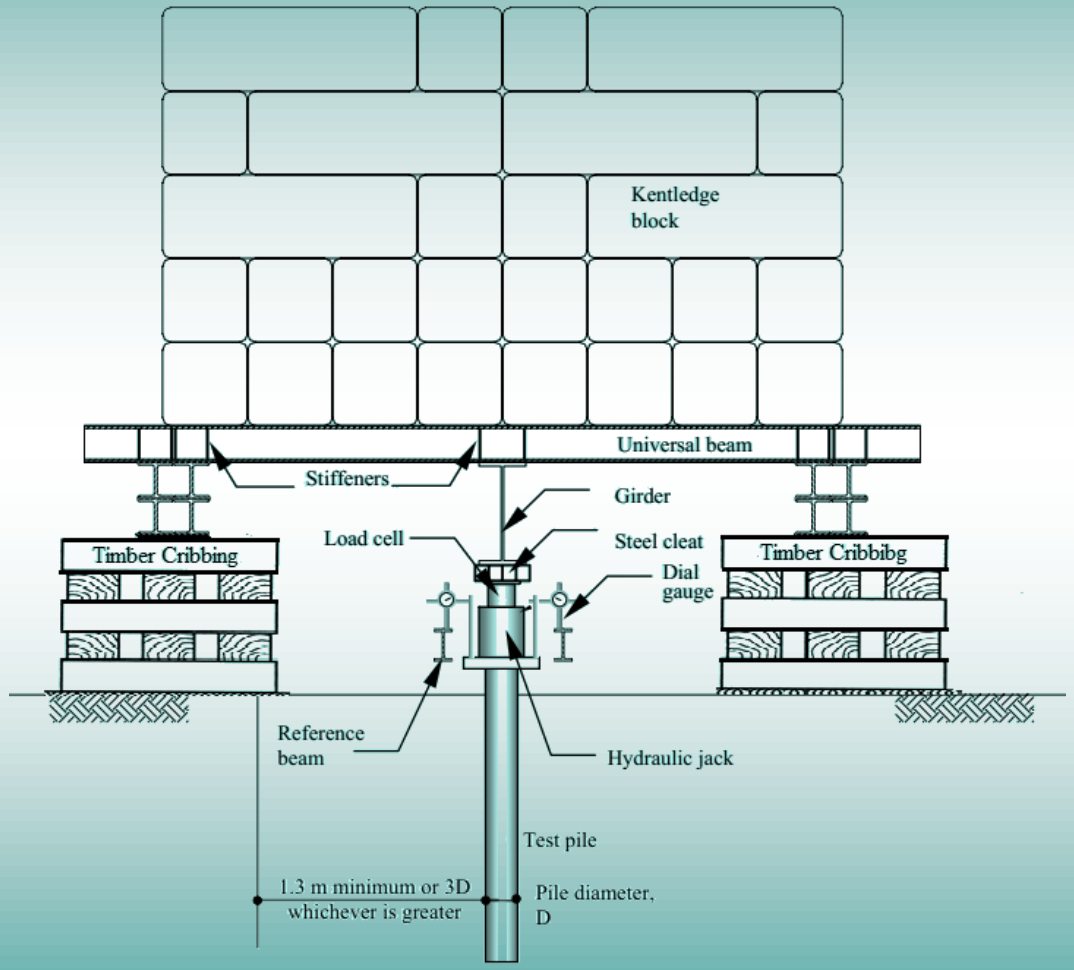


Fig. 30(a): Pile Load Test Arrangements of a Compression Test using Kentledge.

Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression

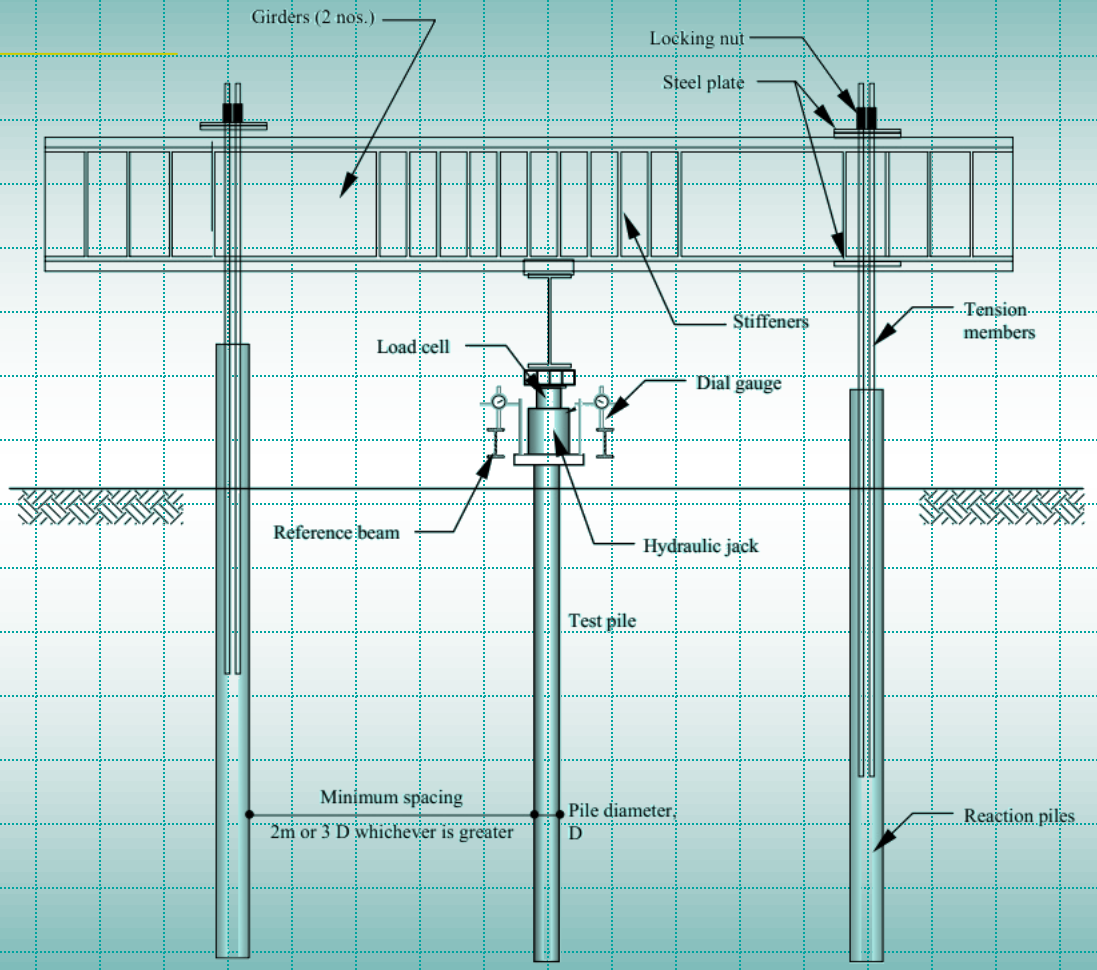


Fig. 30(b): Pile Load Test Arrangements of a Compression Test using Tension Piles.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression

For many years, slow ML tests have been the most common type of pile load tests. These tests typically use load increments of 25, 50, 75, 100, 125, 150, 175, and 200% of the proposed design load and hold each increment until the pile stops moving or until the rate of movement is acceptably small. This process generally requires holding each load for at least 1 or 2 hours, sometimes more. Thus, these tests may require 24 hours more to complete and the load-settlement curve is obtained from the obtained data. The nature of the curves depends on soil types as shown in Fig. 31.

Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression: Interpretation

For piles in soft clays, this is relatively straightforward. The load-settlement curve has a distinct plunge, as shown by curve A in Fig. The ultimate capacity is the load that corresponds to this plunge.

Piles in sands, intermediate soils, and stiff clays have a sloped curve with no clear point of failure, as shown by curve B.

Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression: Interpretation

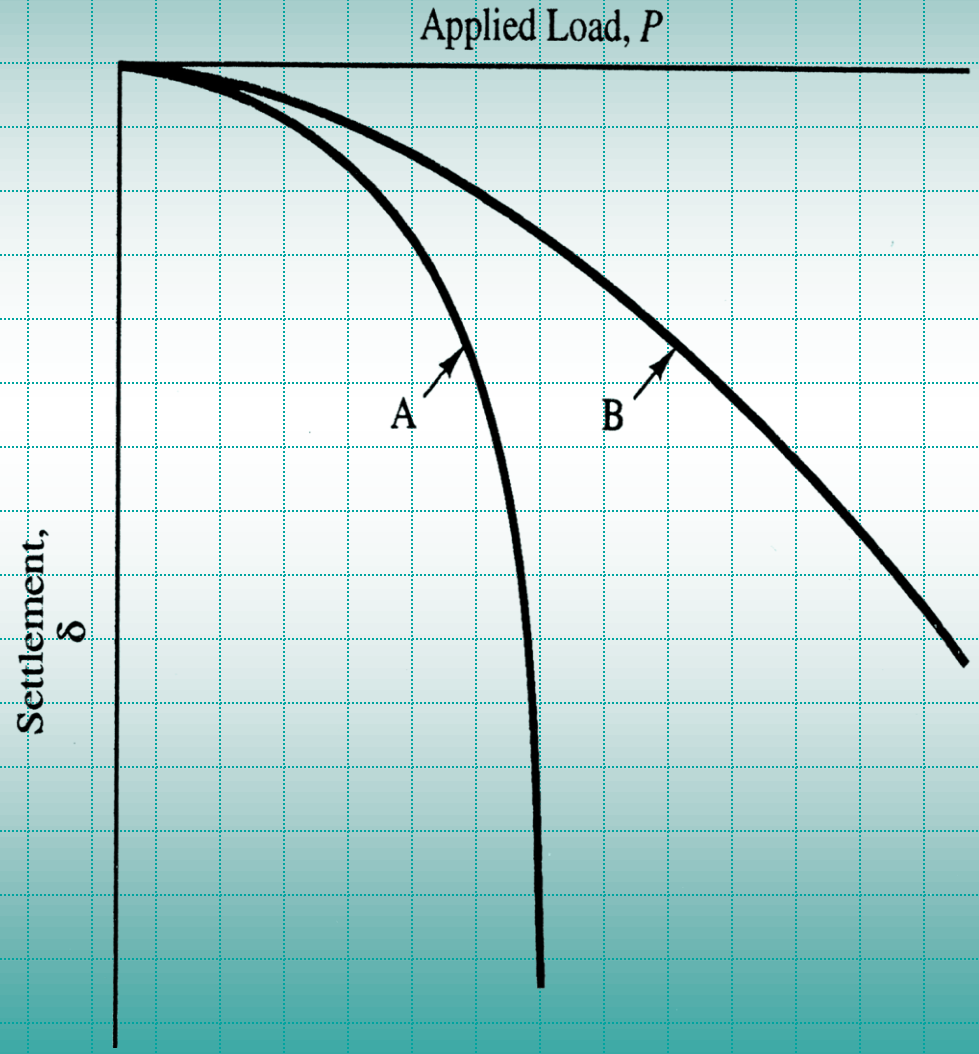


Fig. 31: Load Settlement Curves from Pile Load Test

Bearing Capacity of Pile Foundation



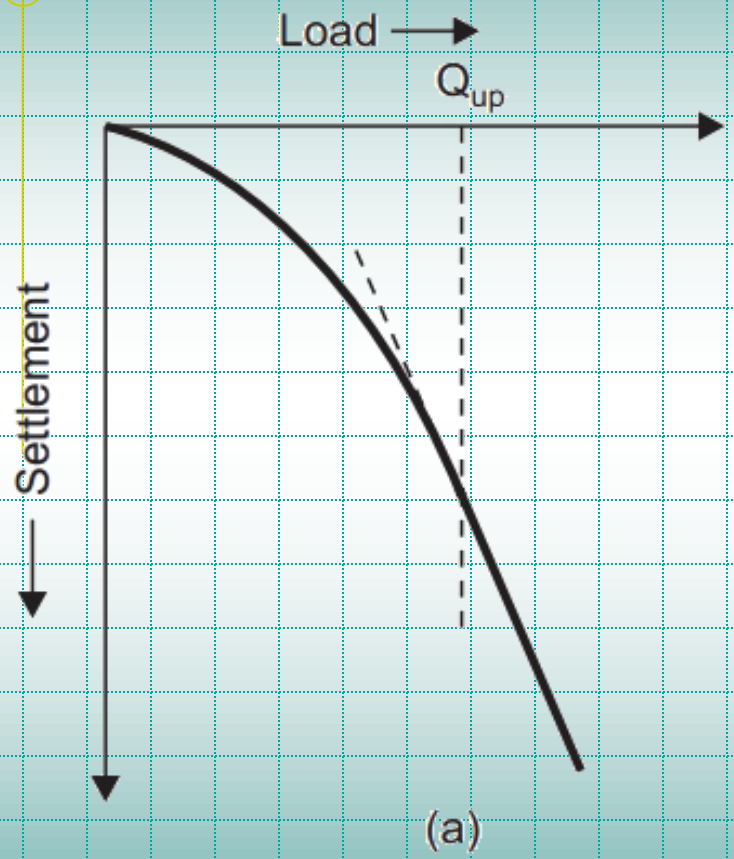
Pile Load Test: Axial Compression: Interpretation

Often the definition of 'failure load' is arbitrary. It may be taken when a predetermined amount of settlement has occurred or where the load-settlement plot is no longer a straight line. If the ultimate load could be found, a suitable factor of safety 2 to 3 may be used to determine the allowable load.

The ultimate load may be determined as the abscissa of the point where the curved part of the load-settlement curve changes to a steep straight line (Fig. 32a). Alternatively, the ultimate load is the abscissa of the point of intersection of initial and final tangents of load settlement curve (Fig. 32b).

Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression: Interpretation



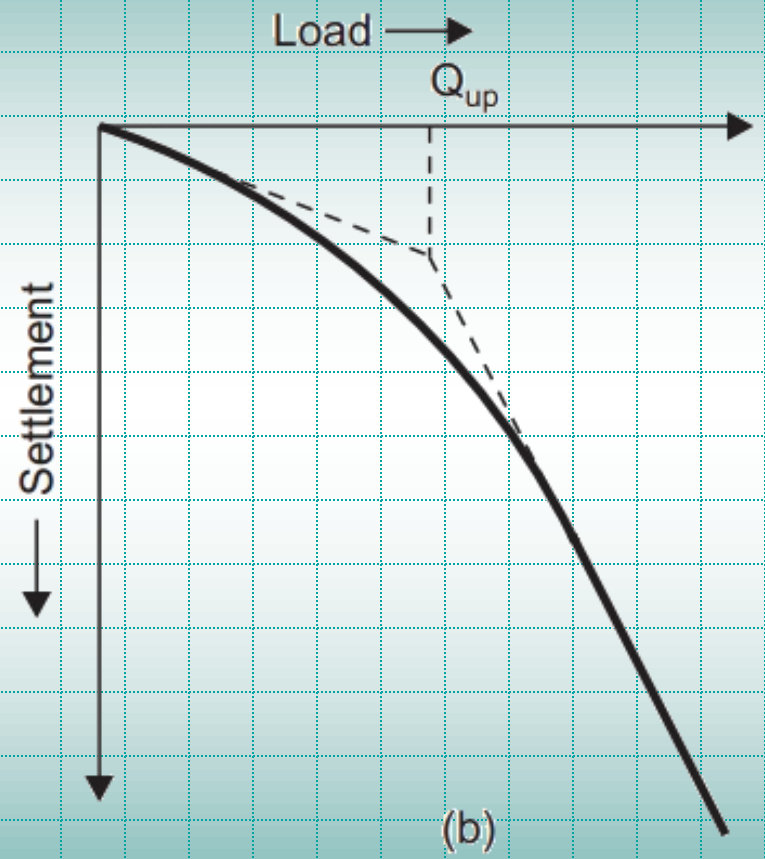


Fig. 32: Ultimate capacity from load-settlement curve for a pile.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Interpretation

Another method in use for the slow test is to plot both load and settlement values on logarithmic scale. The results typically plot as two straight lines (Fig. 32c). The intersection of the straight lines is taken as failure load for design purposes although this may not be the actual load at which failure occurs.

Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression: Interpretation

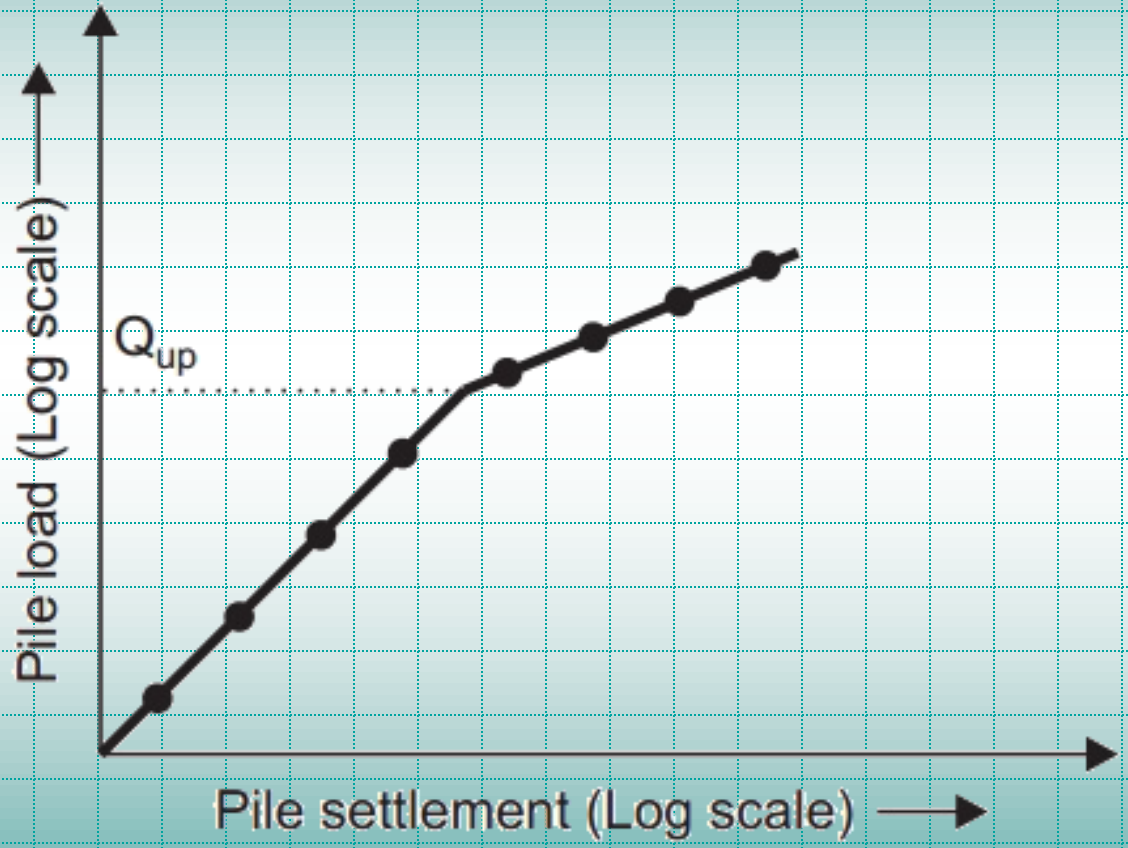
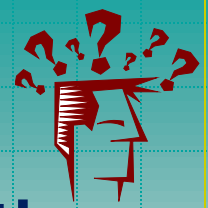


Fig. 32c: Log-Log of pile load-settlement curve and determination of failure.

Bearing Capacity of Pile Foundation



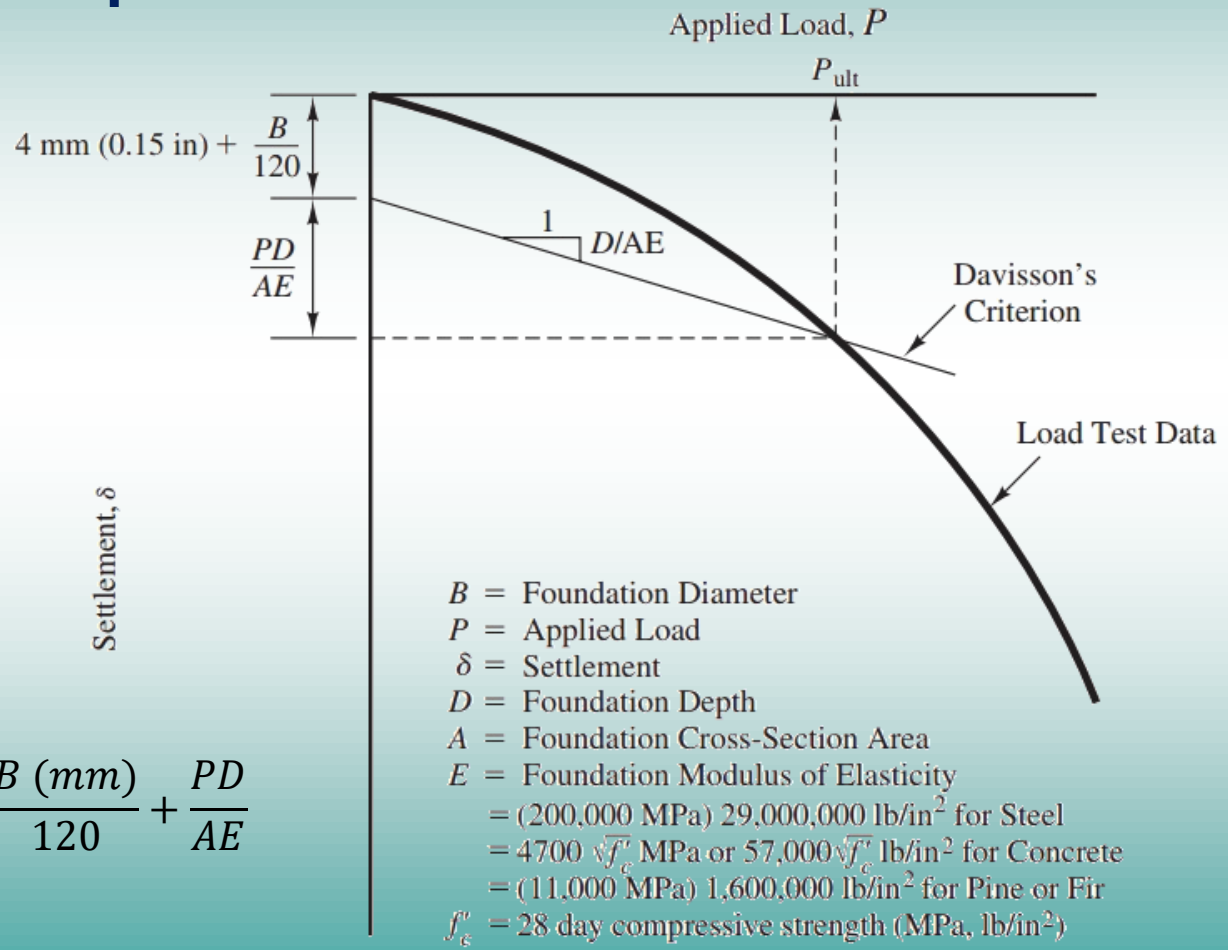
Pile Load Test: Axial Compression: Interpretation: Davisson's Method

A very useful method of computing the ultimate failure load has been reported by Davisson (1973). This method is based on offset method that defines the failure load. The elastic shortening of the pile, considered as point bearing free standing column is computed and plotted on the load-settlement curve, with the elastic shortening line passing through the origin. The scale of the load settlement curve should be so chosen that the slope of the elastic shortening line is approximately 20^o. An offset line is drawn parallel to the elastic line as shown in Figure 32d.

Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression:

Interpretation: Davisson's Method



$$\delta(mm) = 4.0 + \frac{B\ (mm)}{120} + \frac{PD}{AE}$$

Fig. 32d: Davisson's Method of Pile Load Test Interpretation

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Interpretation: Davisson's Method

Davisson method defines the nominal axial downward load capacity as that which occurs at a settlement of 4 mm (0.15 in) + B/120 + PD/(AE), Fig. 32d. The last term in this formula is the elastic compression of a pile that has no side friction. This method has the advantage of being very objective, so everyone analyzing the same data will obtain virtually the same value of P.

Bearing Capacity of Pile Foundation

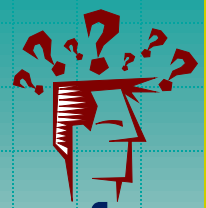


Pile Load Test: Axial Compression: Interpretation: BNBC 2020 Provisions

The allowable load on a single pile may be obtained as one of the following [BNBC 2020]:

1. 50% of the ultimate load at which the total settlement is equal to one-tenth the diameter of the pile.
2. Two-thirds of the load which causes a total settlement of 12 mm.
3. Two-thirds of the load which causes a net (plastic) settlement of 6 mm (total settlement minus elastic settlement).

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Constant Rate of Penetration Test (CRP)

The constant-rate-of-penetration test has the advantage that it is rapid. However, the mobilized pile capacity may be influenced by strain rate effects, particularly in cohesive soils.

A constant strain rate of 0.25 to 1.25 mm/min and 0.75 to 2.5 mm/min is commonly used for clays and granular soils respectively. The load should be supplied by a hydraulic power pack and by regulating the rate of oil flow to the jack and monitoring the pile movement with dial gauges. This procedure can control the rate of pile penetration better.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Constant Rate of Penetration Test (CRP)

Tsui (1968) reported that two piles at a building site which have been subjected to a maintained-load test followed by a CRP test showed similar capacities although the load settlement characteristics are different. In general, CRP tests are less suitable for piles founded on rock or granular soils and can constitute a safety hazard if the increase in loading becomes excessive. CRP tests are not suggested in Hong Kong given the ground conditions.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

In cyclic load test, the load is raised up to a particular level, released to zero and again raised to a higher value and released to zero. Settlements are recorded at each increment or decrement of load. A typical plot of a cyclic load test data will look as shown in Fig. 33.

The total settlement S of the pile head at any load may be written as:

$$S = S_e + S_p$$

and,

$$S = \Delta L + S_{es} + S_p$$

Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression: Cyclic Load Test

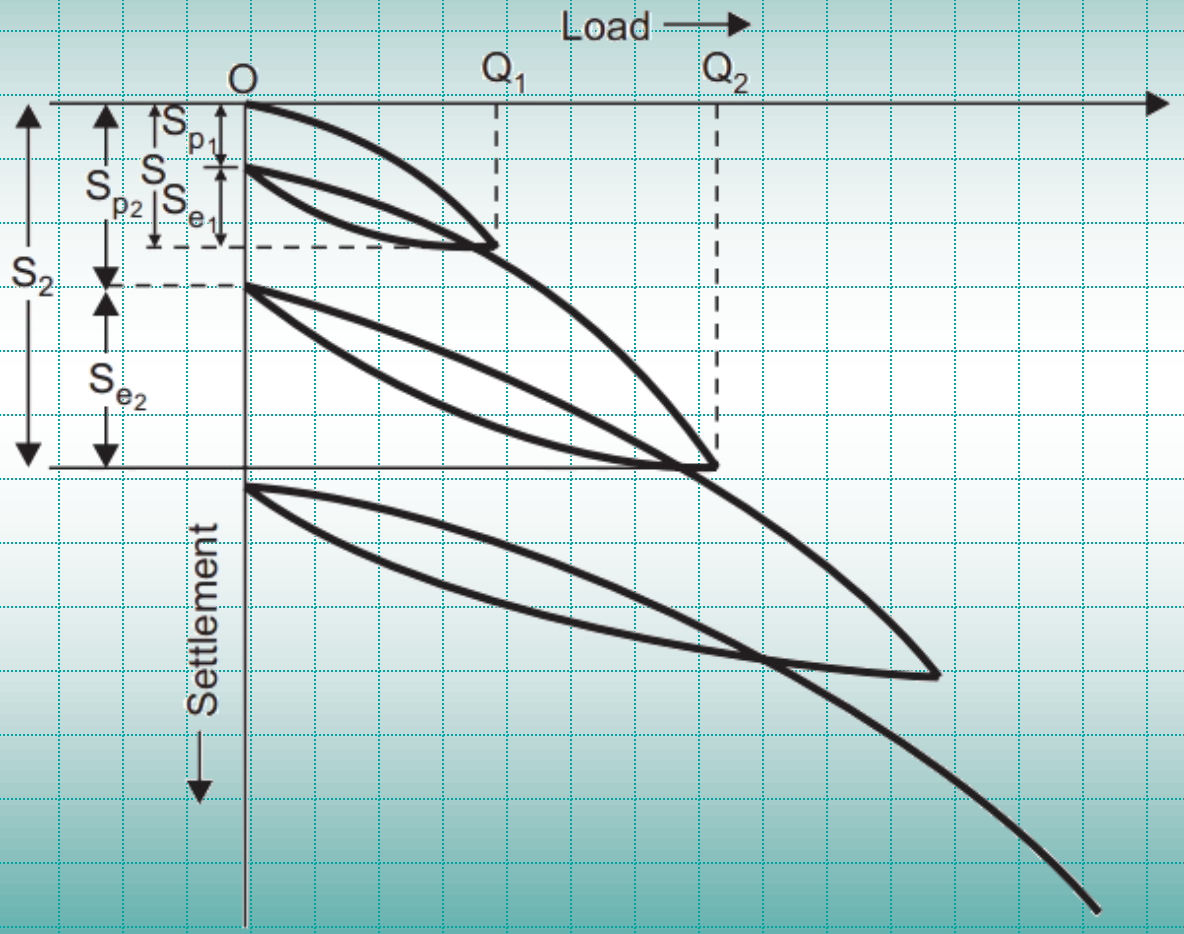
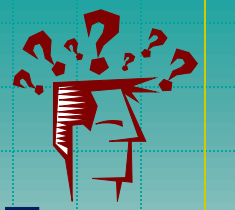


Fig. 33: Cyclic load test on a pile-load vs. settlement.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

Where,

S_e = elastic compression of the pile and soil at the base,

S_p = plastic compression of the soil at the base

S_{es} = elastic compression of the soil at the base, and

ΔL = elastic compression of the pile

(This is based on the assumption that plastic compression of the pile is negligible)

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

The elastic compression of the pile, ΔL may be obtained from the equation

$$\Delta L = \frac{(Q - Q_f/2)}{AE}$$

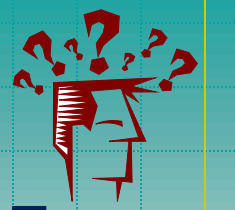
Where, Q = load on the pile,

Q_f = frictional resistance component, and

L, A, E = length, area of section and modulus of elasticity of the pile.

The total settlement may easily be separated into the elastic and plastic components by removal of the load and observation of the net settlement.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

The elastic settlement is got by deducting the net settlement after removal of load from the total value of the settlement under the load. The elastic settlement of the soil at the base is obtained by subtracting the elastic settlement of the pile from the total elastic component of the settlement.

The separation of the applied load at any load level into the point-bearing and frictional components is based on an experimental finding of Van Wheele (1957). Until the load reaches a certain value, the point-bearing component will be zero.

Bearing Capacity of Pile Foundation

Pile Load Test: Axial Compression: Cyclic Load Test

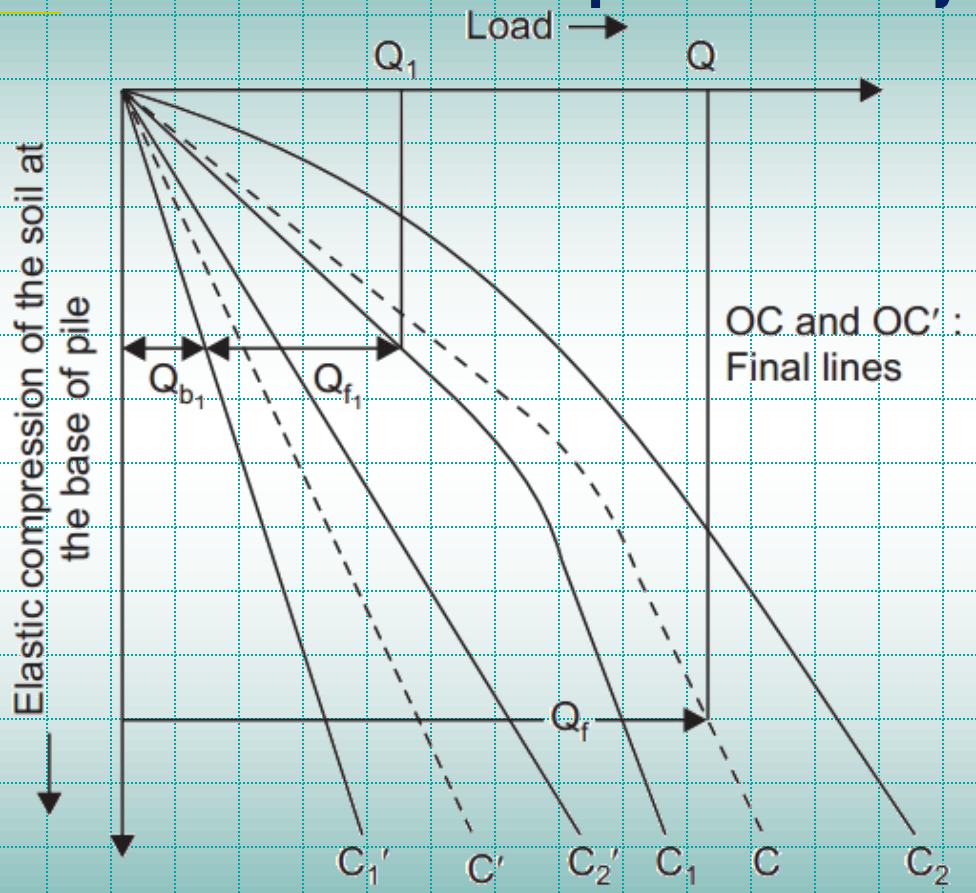


Fig. 34 Separation of point-bearing and skin-friction resistances from cyclic load test data.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

With increase in load, both the friction and point-bearing components go on increasing. The frictional component attains a maximum value at a particular load level and thereafter the point-load component goes on increasing with further increase in load. Van Wheele found that the point-load component increases linearly with the elastic compression of the soil at the base and that the straight line showing this linear relationship is parallel to the straight line portion of the curve between the load on the pile and elastic compression of the soil.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

Since the right-hand side of Eq. 16.30 contains Q_f which is not known to start with, a sort of a 'trial and error' procedure is employed to determine Q_f and Q_b (frictional and point bearing components) corresponding to any pile loadQ. The procedure is illustrated in Fig. 34.

1. Since Q_f is not known to start with, ΔL is assumed to be zero. Then the elastic compression of the soil at the base is equal to the total elastic recovery of pile head. A curve OC_1 is drawn between the pile load and the elastic compression of the soil, calculated in this manner.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

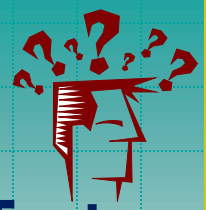
2. A line OC_1' is drawn from the origin O parallel to the straight portion of the curve OC_1. This line is supposed to divide the pile load Q into the components Q_b and Q_f

3. For different loads Q_1, Q_2 ..., components Q_{f1}, Q_{f2} are determined, as shown in Fig. 34.

4. The values of ΔL corresponding to different values of Q_f are computed from Eq. 16.30.

5. The elastic compressions of the soil are obtained by deducting these values of ΔL from the corresponding values of elastic recovery of the pile head.

Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

6. A modified curve OC_2 is now drawn between pile load and elastic compression of soil.

7. Through the origin, a line OC_2' is drawn parallel to the straight line portion of OC_2.

8. Steps 3 – 7 are repeated to get the next modified curve between the pile load and the elastic compression of the soil.

9. The procedure is repeated until a curve which gives sufficiently accurate values of Q_b and Q_f is obtained. It has been found from experience that the third curve gives the desired degree of accuracy.

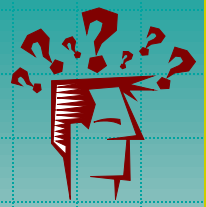
Bearing Capacity of Pile Foundation



Pile Load Test: Axial Compression: Cyclic Load Test

A factors of safety 2 and 2.5 on the ultimate values of skin friction resistance and point resistance, respectively is recommended. Hence, the allowable load on the pile may be obtained by adding $Q_f/2.0$and $Q_b/2.5$, where Q_f and Q_b correspond to the values corresponding to a load causing a total settlement of one-tenth of the pile diameter. It should be obvious that the settlement required to cause ultimate point resistance is greater than that required to cause ultimate skin resistance.

Bearing Capacity of Pile Foundation



Pile Load Test: Uplift Capacity

A typical arrangement for uplift loading tests is shown in Fig. 31. The arrangement involving jacking at the centre is preferred because an even load can be applied to the test pile. The arrangement of applying load at one end of the beam is not recommended because of risk of instability.

Reaction piles should be placed at least three test pile diameters, or a minimum of 2 m, from the centre of the test pile. Where the spacing is less than this, corrections for possible pile interaction should be made. Alternatively, an O-cell installed at the base of pile can also be used in an uplift test.

Bearing Capacity of Pile Foundation

Pile Load Test: Uplift Capacity

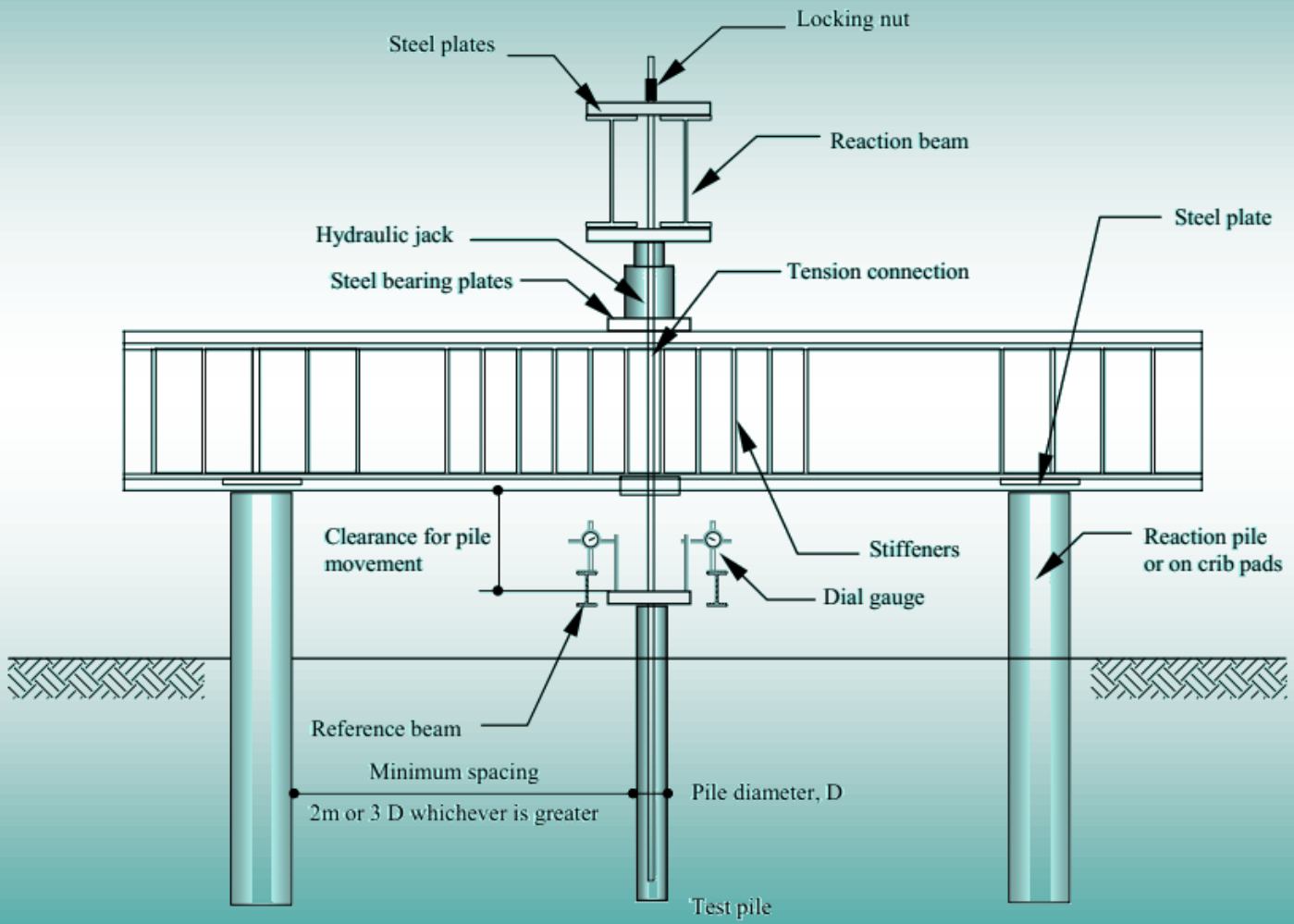


Fig. 31: Pile Load Test Arrangements an Uplift Test (Tomlinson, 1994)

Bearing Capacity of Pile Foundation



Pile Load Test: Lateral Load Capacity

In a lateral loading test, two piles or pile groups may be jacked against each other (Fig. 32a). It is recommended that the centre spacing of the piles should preferably be a minimum of ten pile diameters (CGS, 1992).

Alternative reaction systems including a 'deadman' or weighted platform are also shown in Figs. 32(b) and 32(c).

Bearing Capacity of Pile Foundation

Pile Load Test: Lateral Load Capacity

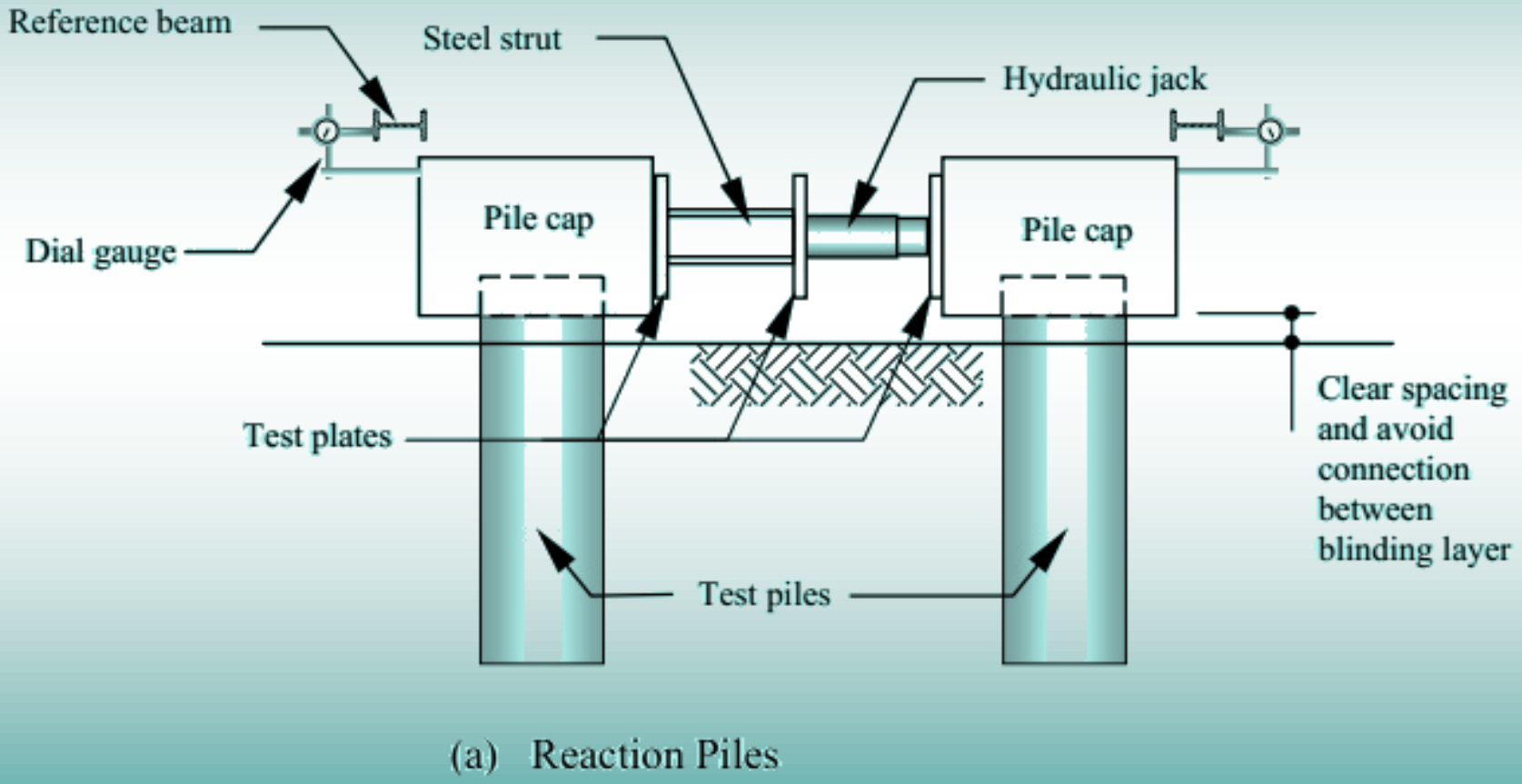
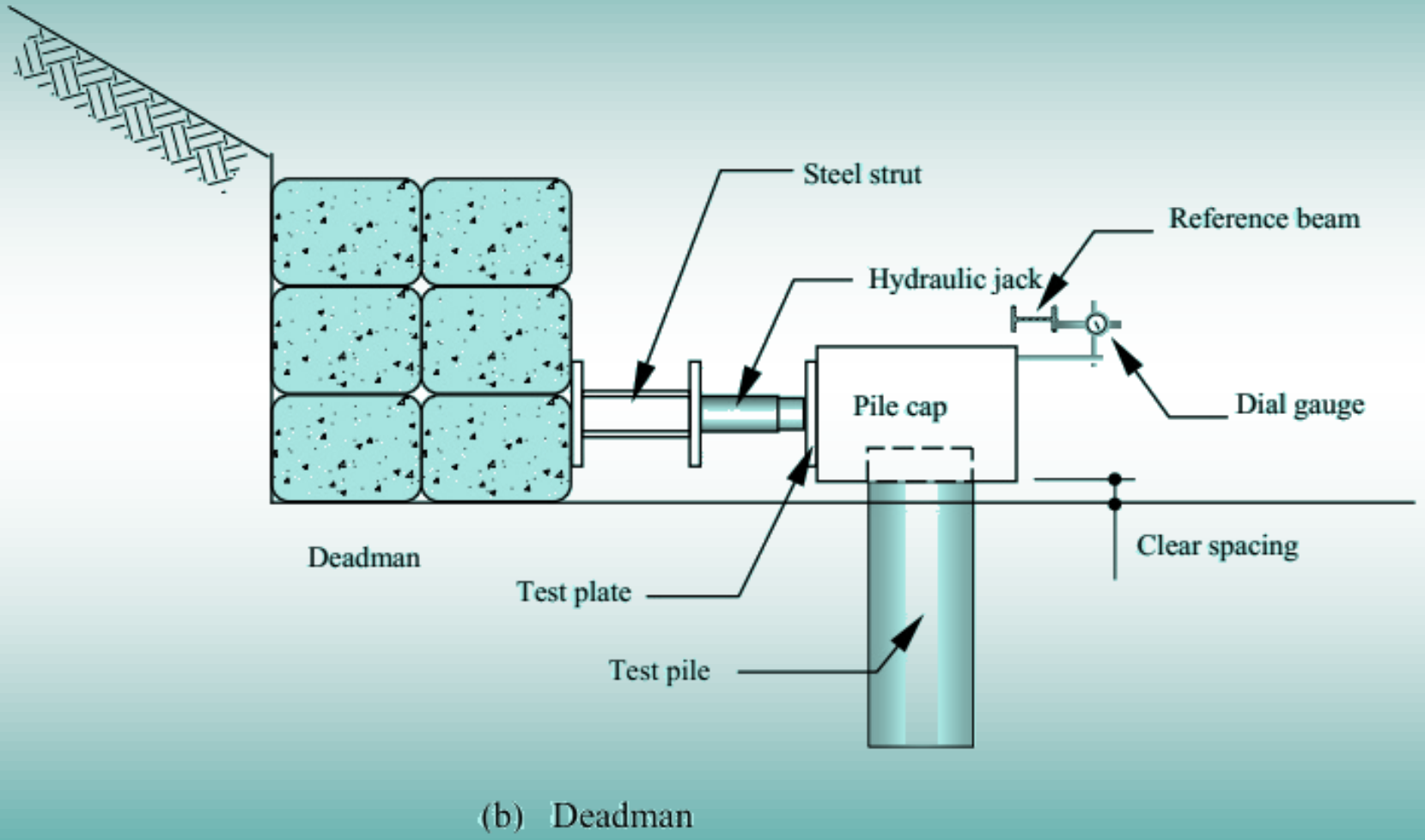


Fig. 32(a): Typical Arrangement of a Lateral Loading Test.

Bearing Capacity of Pile Foundation

Pile Load Test: Lateral Load Capacity

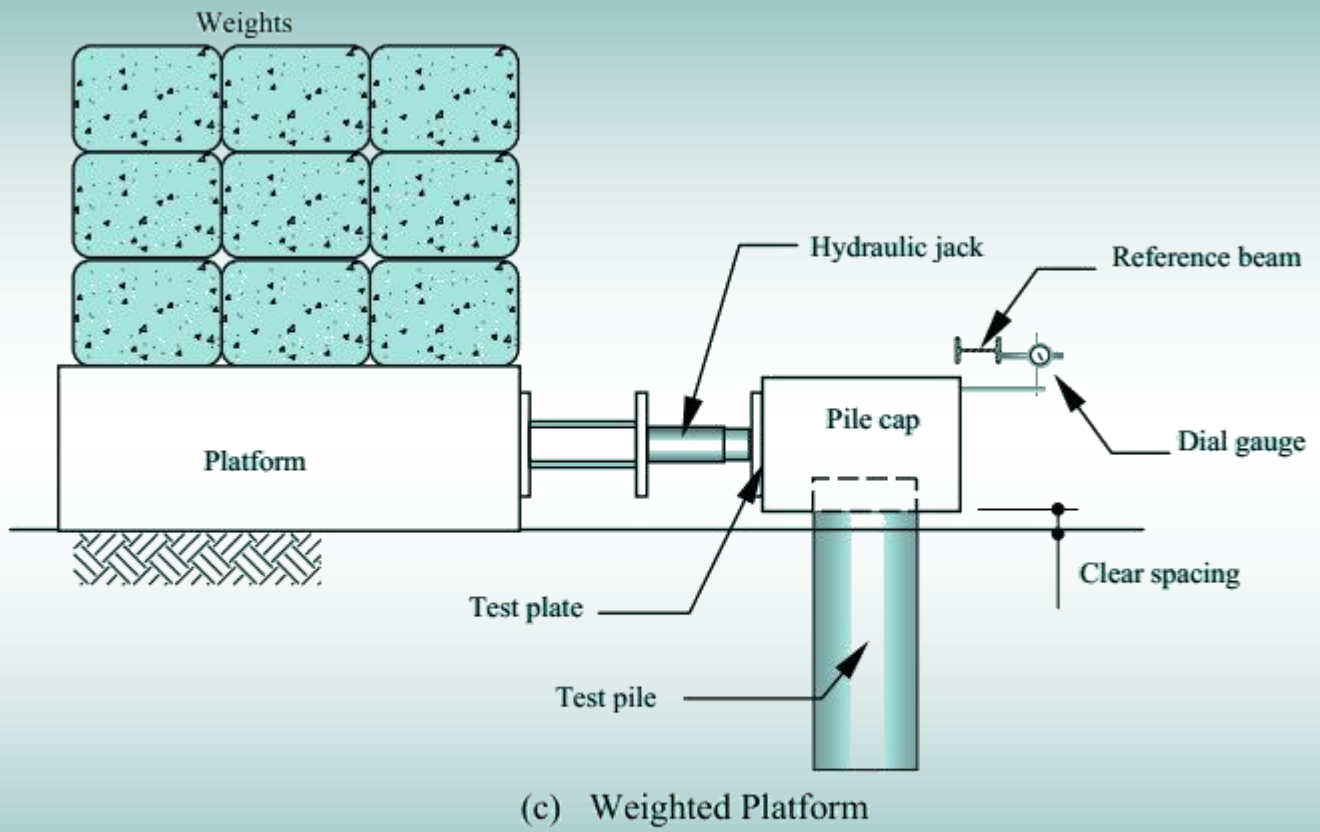


(b) Deadman

Fig. 32(b): Typical Arrangement of a Lateral Loading Test.

Bearing Capacity of Pile Foundation

Pile Load Test: Lateral Load Capacity



(c) Weighted Platform

Note : Load cells with appropriate plates can be inserted between test plate and hydraulic jack.

Fig. 32(c): Typical Arrangement of a Lateral Loading Test.

Bearing Capacity of Pile Foundation

Negative Skin Friction

In conditions where the soil can move down relative to the pile, instead of the pile going down because of the loading, the soil tends to hang on to the pile' and transfer its load also on to the pile, Fig. 33. Here the skin friction between soil and pile increases the load on the pile and hence this type of friction is called negative skin friction or downdrag.

Bearing Capacity of Pile Foundation

Negative Skin Friction

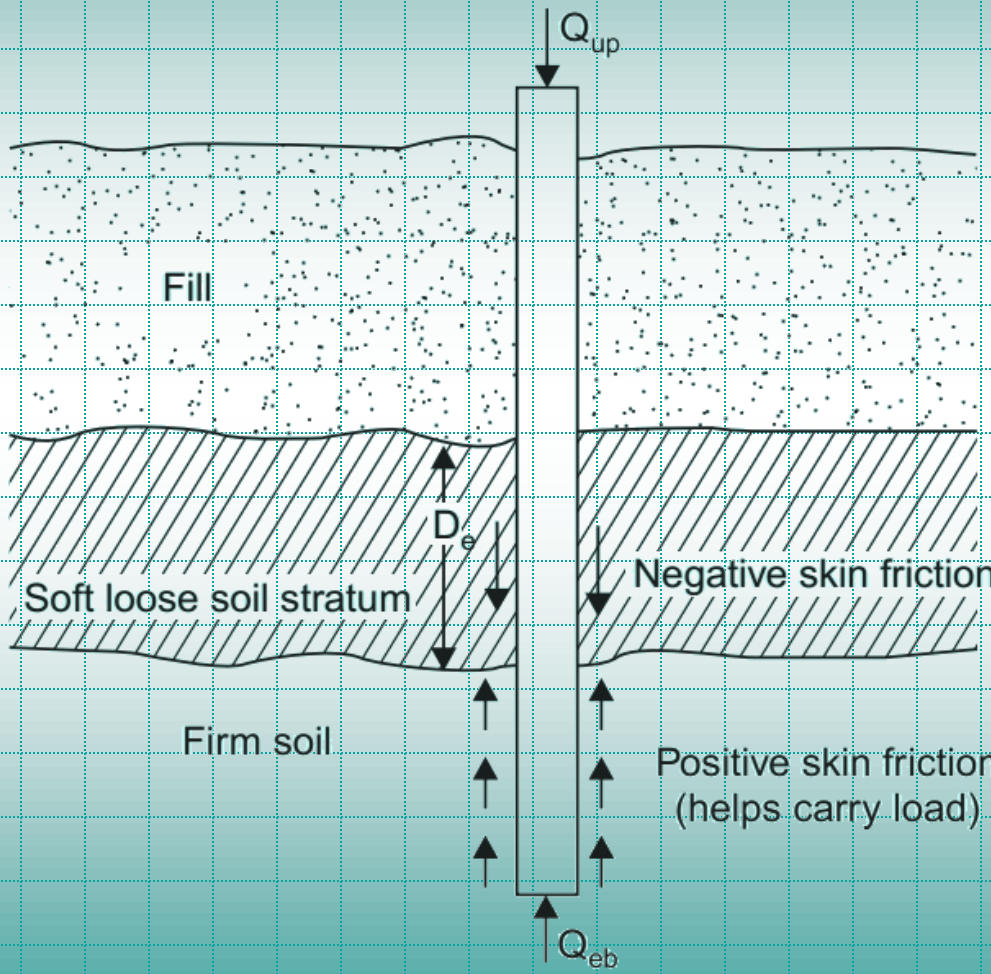


Fig. 33: Phenomenon of Negative Skin Friction.

Bearing Capacity of Pile Foundation



Conditions for Negative Skin Friction

Negative skin friction can develop under different general field conditions, such as the following:

1) Pile is installed in a fill that undergoes consolidation. The most common situation where large negative shaft resistance develops occurs when fill is placed over a compressible layer immediately prior to, or after piles are driven. This condition is shown in Fig. 34(a).

2) Negative shaft resistance can also develop whenever the effective overburden pressure is increased on a compressible layer through which a pile is driven due to lowering of the ground water table as illustrated in Fig. 34(b), for example. Lowering or variation of ground water leads to significant settlement of soil strata around the pile.

Bearing Capacity of Pile Foundation



Conditions for Negative Skin Friction

3) Soils that are disturbed or remoulded thoroughly during the pile installation.

4) Piles installed in soft clay with surcharge loading.

5) Soils that are disturbed or remoulded thoroughly during the pile installation.

6) Piles driven through a strata of soft clay into firmer soils and thus the soft clay tends to settle relative to the pile.

7) Piles in a clay stratum that undergoes shrinkage settlement.

Bearing Capacity of Pile Foundation

Conditions for Negative Skin Friction

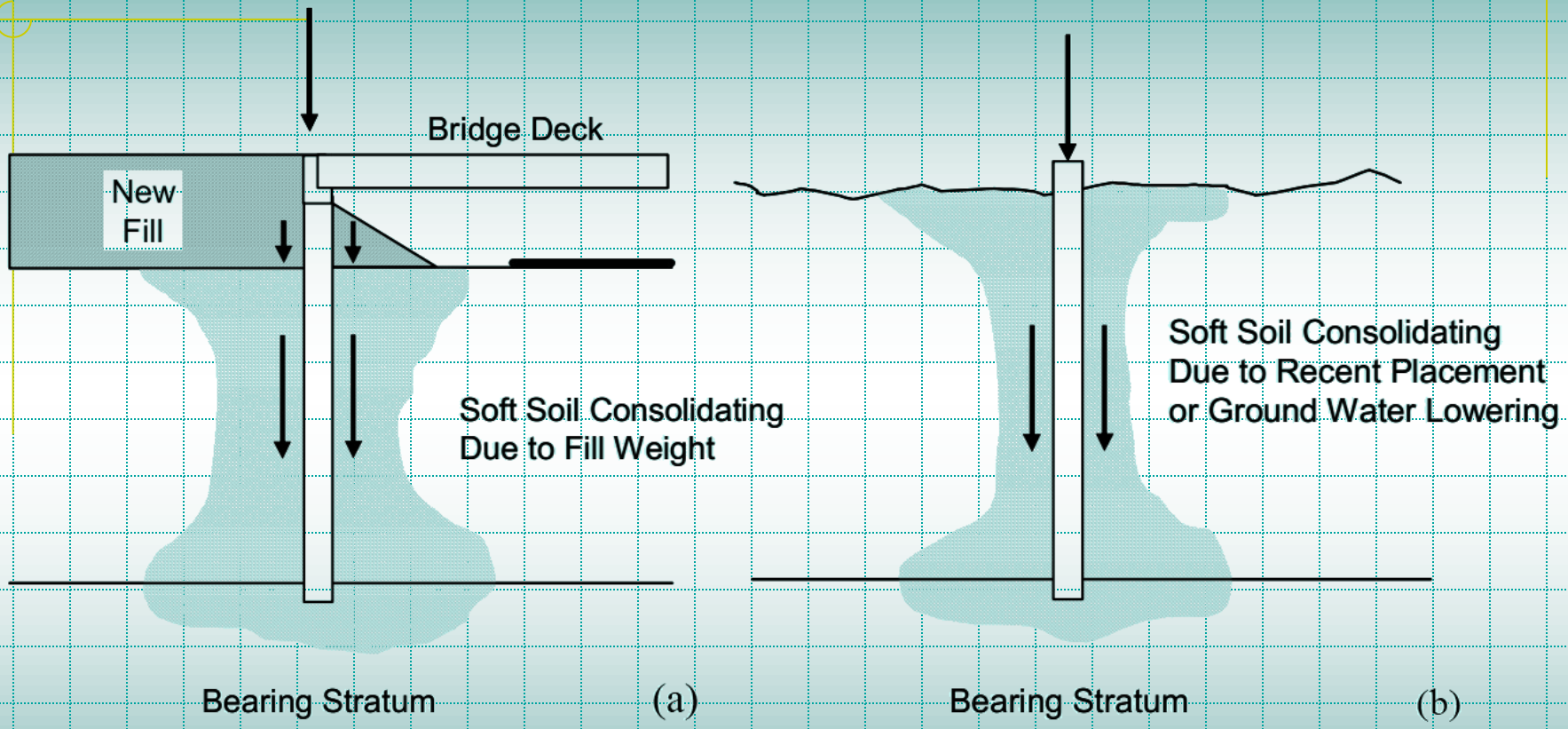


Fig. 34: Negative skin friction or downdrag; (a) Common downdrag situation due to fill weight; (b) Common downdrag situation due to ground water lowering.

Bearing Capacity of Pile Foundation



Other Conditions for Negative Skin Friction

Briaud and Tucker (1993) presented the following criteria for identifying when negative shaft resistance may occur. If any one of these criteria is met, negative shaft resistance should be considered in the design. The criteria are:

1) The total settlement of the ground surface will be larger than 100 mm (4 in).

2) The settlement of the ground surface after the piles are driven will be larger than 10 mm (0.4 in).

Bearing Capacity of Pile Foundation



Other Conditions for Negative Skin Friction

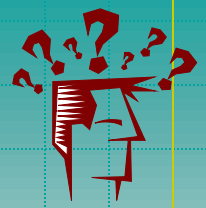
3) The height of the embankment or fill to be placed on the ground surface exceeds 2 m (6.5 ft).

4) The thickness of the soft compressible layer is larger than 10 m (33 ft).

5) The water table will be lowered by more than 4 m (13 ft).

6) The piles will be longer than 25 m (82 ft).

Bearing Capacity of Pile Foundation



Negative Skin Friction: Analysis

A shallow, soft layer loaded by an embankment, for example, settles more than the pile, which may rest in a strong deeper layer. In this instance the soil drags the pile down during the soil settlement. Because the pile does not move downward as much as the soil, at the ground surface it looks like the pile is coming out of the ground. So, when it is seen piles coming slowly out of the ground, it could be downdrag due to negative skin friction.

Bearing Capacity of Pile Foundation

Negative Skin Friction: Analysis

This downdrag load extends to the point where the settlement of the soil becomes equal to the settlement of the pile, Fig. 35; that point is called the neutral point (NP). In the case of no downdrag, the load in the pile decreases from the top down to the pile point. In the case of downdrag, the load increases to the NP and then decreases to the pile point. In extreme cases the NP is at the pile point. The evolution of the loads during and after construction is shown in Fig. 36 for the case of downdrag and no downdrag.

Bearing Capacity of Pile Foundation

Negative Skin Friction: Analysis

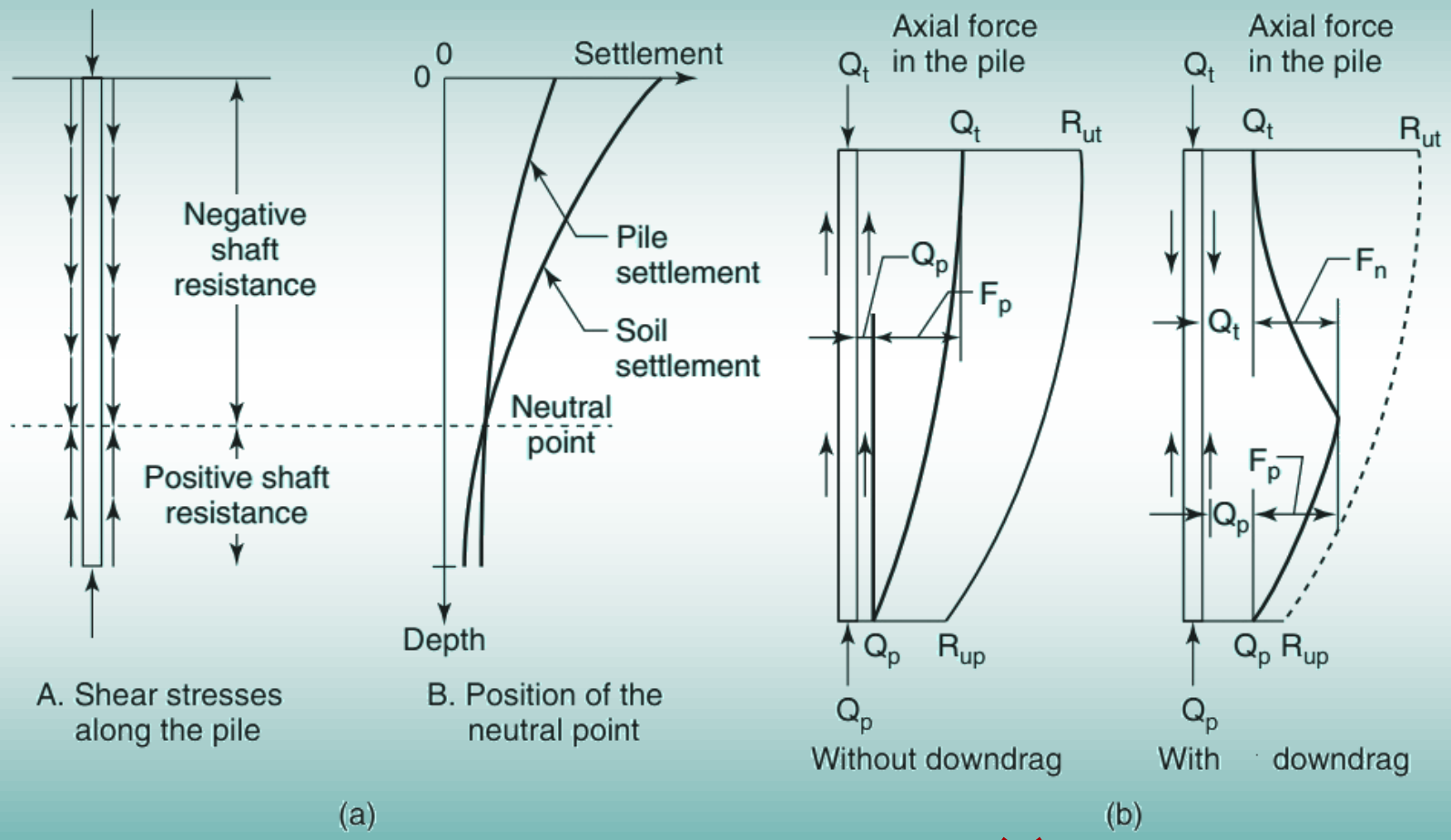


Fig. 35: Settlement Profile and Load Distribution; (a) Settlement Profile; (b) Load Distribution.

Bearing Capacity of Pile Foundation

Negative Skin Friction: Analysis

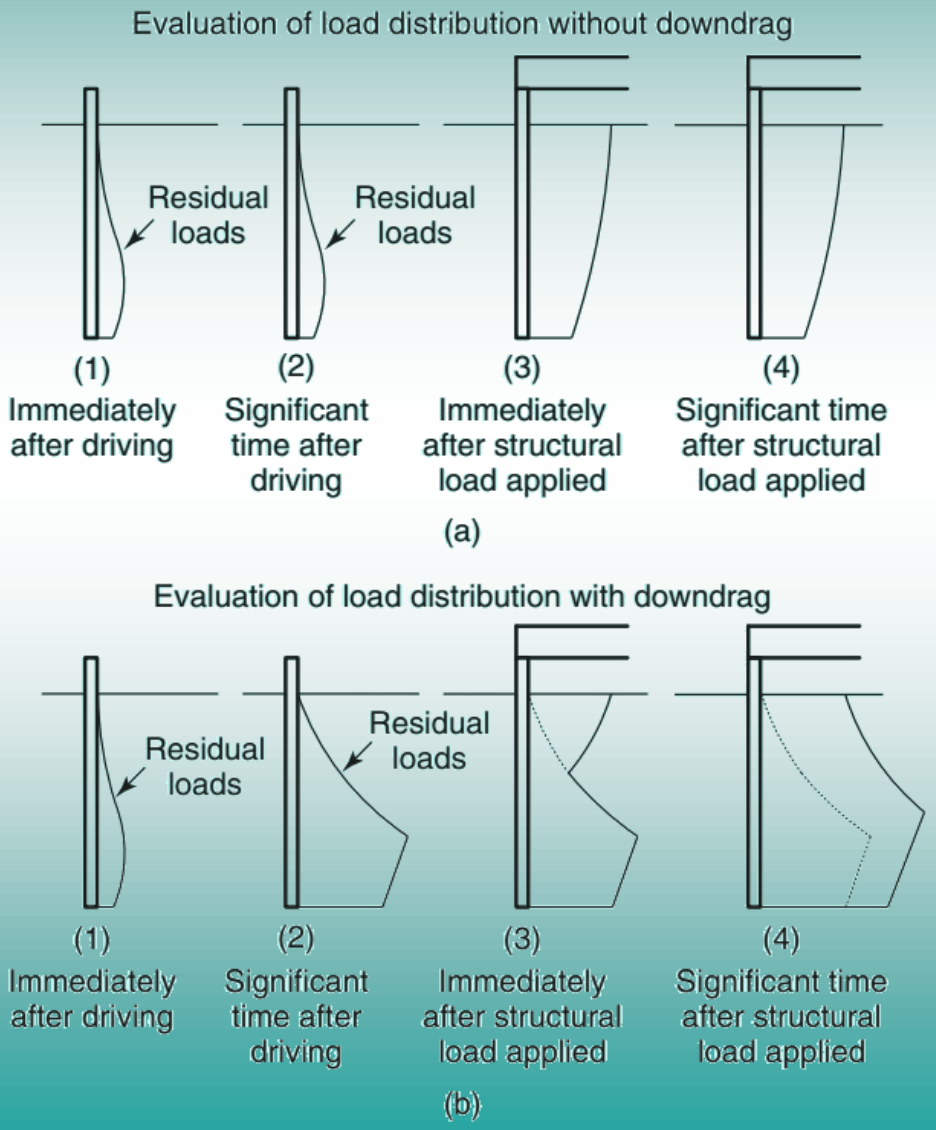


Fig. 36: Evolution of the Loads during and after Construction.

Bearing Capacity of Pile Foundation

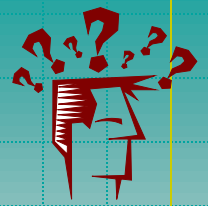


Negative Skin Friction: Analysis

As with other foundation design problems, the two limit states must be satisfied: serviceability limit state (allowable settlement) and ultimate limit state (ultimate capacity).

What is special about downdrag cases is that the serviceability limit state (allowable settlement criterion) controls the design much more often than does the case of no downdrag. Indeed, downdrag induces larger settlement of the pile and settlement calculations must be performed with this in mind. Also, one must check that the load at the NP, which is most often the highest load in the pile, will not crush the pile material.

Bearing Capacity of Pile Foundation



Negative Skin Friction: Analysis

Two main equations are the basis for finding the location of the neutral point. Vertical equilibrium gives the following expression:

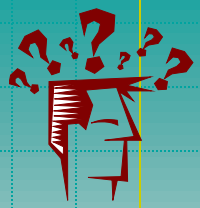
$$Q_t + F_n = F_p + Q_p$$

Where Q_t is the top load on the pile, F_n is the downdrag or negative friction load, F_p is the positive friction load, and Q_p is the point load (Figure 35). Compatibility of movement at the NP gives:

$$w_{NP(soil)} = w_{NP(pile)}$$

Where $w_{NP(pile)}$ is the pile movement at NP and $w_{NP(soil)}$ is the soil movement at NP. These two equations are used together with an iteration procedure to find the depth of NP.

Bearing Capacity of Pile Foundation



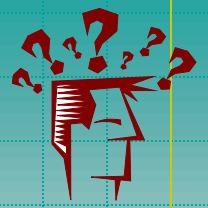
Negative Skin Friction: Estimation

Even a small relative movement between pile and soil around the pile (of the order of 10 mm), mobilizes full negative skin friction. Following two conditions that can occur in the field.

Case I. The pile may rest on a hard stratum without any possible downward movement of the pile but the soil around it settles. Here full negative friction acts.

Case 2. The pile rests on relatively compressible strata so that the pile settles to the same extent with the surrounding soil. If the settlement of the pile is large, the soil around the pile will have to pull the pile up in 'positive friction. Hence, judgement must be used when estimating these forces.

Bearing Capacity of Pile Foundation



Negative Skin Friction: Estimation

Method 1: Maximum negative skin friction = (cohesion) x (area of pile shaft)

$$f_{s(neg)} = cA_p$$

Where, A_p is the perimeter area of the pile.

Method 2: For negative friction developed by consolidation of soil with piles, the following expression is applicable.

$$f_{s(neg)} = \beta\sigma'_v$$

Where, β = the reduction factor. Meyerhof recommends values depending on length of piles, 0.3, 0.2, 0.2 for length 15, 40, 60 metres of pile lengths, respectively.

Bearing Capacity of Pile Foundation



Negative Skin Friction: Mitigation

In situations where the negative shaft resistance on piles is large and a reduction in the pile design load is impractical, negative shaft resistance forces can be handled or reduced by using one or more of the following techniques:

1) **Reduce Soil Settlement:** Preconsolidation of compressible soils can be achieved by preloading and consolidating the soils prior to pile installation. This approach is often used for bridge foundations in fill sections.

2) **Use Lightweight Fill Material:** Construct structural fills using lightweight fill material to reduce the downdrag loads. Lightweight fill materials often used, depending upon regional availability, include geofoam, foamed concrete, wood chips, blast furnace slag, and expanded shales.

Bearing Capacity of Pile Foundation



Negative Skin Friction: Mitigation

3) Use a Friction Reducer: Bitumen coating and plastic wrap are two methods commonly used to reduce the friction at the pile-soil interface. It has been proved to reduce skin friction as much as 90 per cent of the theoretical value. **Bitumen coatings should only be applied to the portion of the pile which will be embedded in the negative shaft resistance zone.**

One of the major problems with bitumen coatings is protecting the coating during pile installation, especially when driving through coarse soils. An inexpensive solution to this problem is to weld an over-sized collar around the pile where the bitumen ends.

Bearing Capacity of Pile Foundation



Negative Skin Friction: Mtigation

5) **Drive the Piles Inside a Casing:** In the top negative friction height, the space between pile and casing is filled with a viscous material and the casing is withdrawn after installing the pile.

6) **Use a Smaller Diameter of Shaft as Compared to the Base:** This is only possible for bearing piles.

7) **Increase Allowable Pile Stress:** In piles where the allowable pile material strength has not been fully utilized, the pile design stress can be increased to offset the negative shaft resistance load. Increased structural capacity can also be obtained by using higher strength pile materials, or in the case of pipe piles, by using an increased wall thickness. Foundation settlement at the increased loading should be computed and checked against the foundation performance criteria.