FOUNDATION DESIGN

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TYPE OF BUILDING FOUNDATION

- Shallow Foundation
- Mat Foundation
- Deep Foundation

Shallow Foundation

Introduction

Bearing capacity analysis

Size of footings

Contact pressure

Total & differential settlement

Bearing capacity

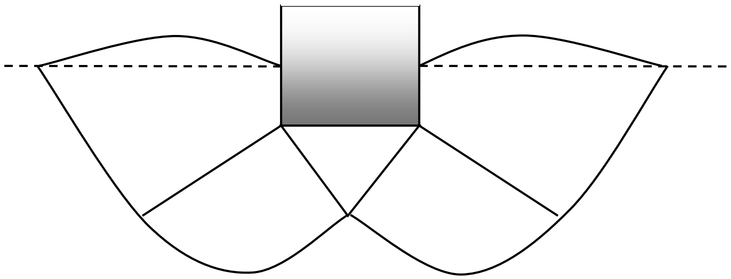
Bearing capacity is *the ability of the soil to support a foundation & structure*

Ultimate Bearing Capacity (q_u) is the load per unit area that will cause a shear failure of the soil immediately below foundation.

The allowable bearing capacity is the ultimate bearing capacity divided by a factor of safety.

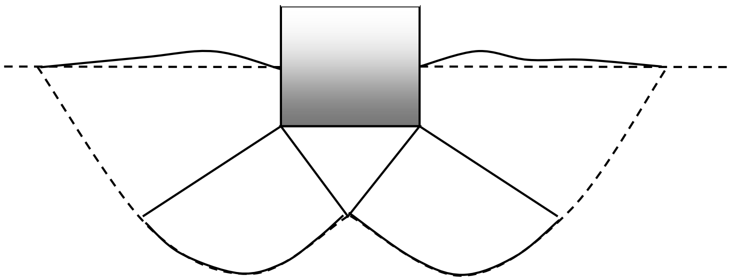
A factor of safety of 2.5 to 3.0 is commonly applied to the bearing capacity of the soil in order to avoid excessive settlement of the foundation.

Bearing capacity analysis



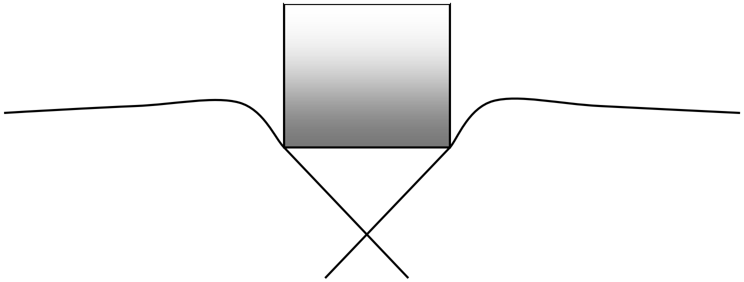
(a) General shear failure

Dense sand & stiff clay



(b) local shear failure

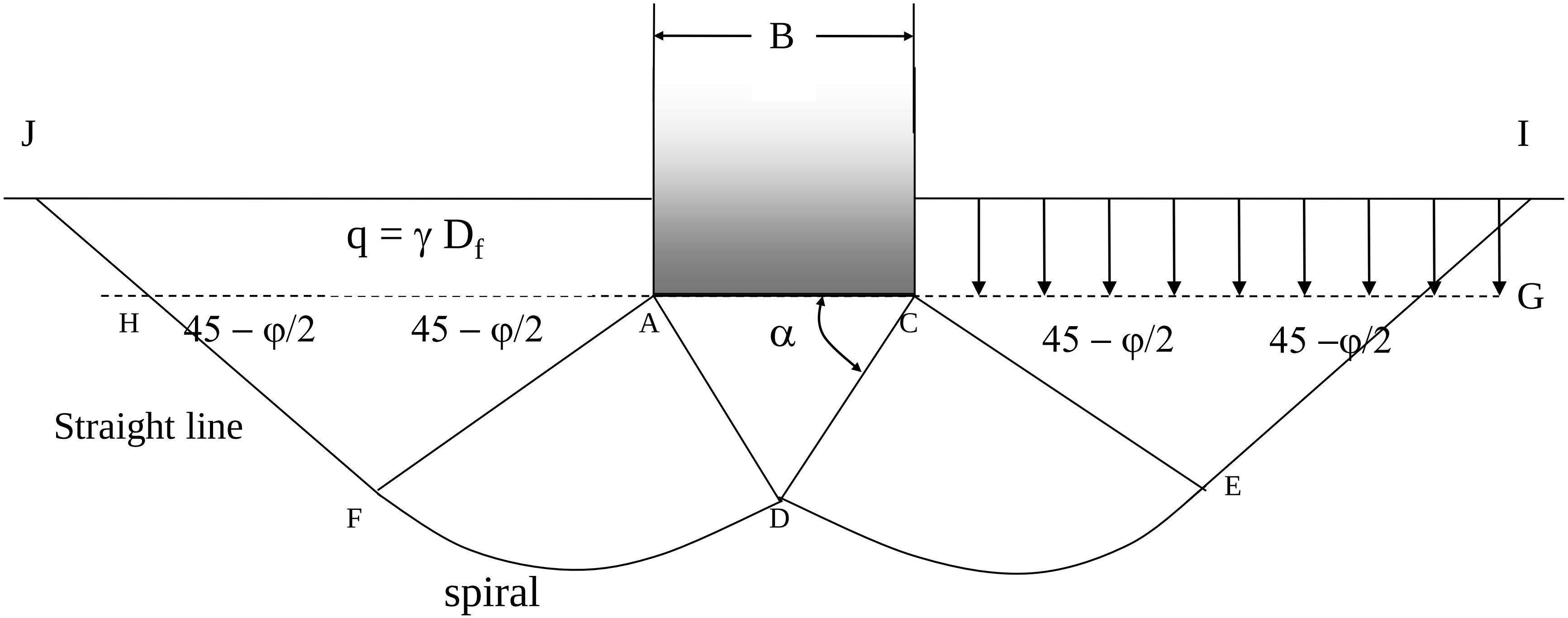
Loose sand & soft clay



(c) punching shear failure

Dense sand & stiff clay overlaying weak soil

General shear failure



Terzaghi Bearing Capacity Equation

For strip footing of width B.

$$q_u = c\,N_c + q\,N_q + ½\,\gamma\,B\,N_\gamma$$

For square foundation (B × B in size),

$$q_u = 1.3\,c\,N_c + q\,N_q + 0.4\,\gamma\,B\,N_\gamma$$

For circular foundation of diameter = *B*

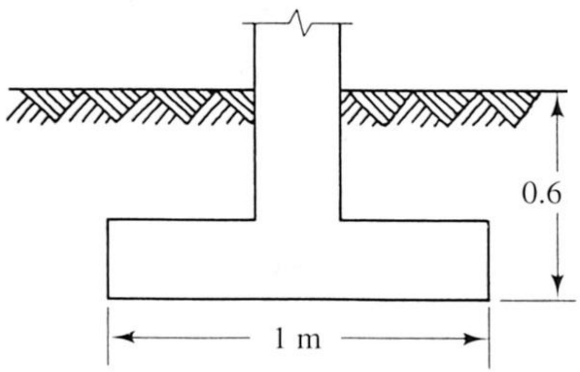
$$q_u = 1.3\,c\,N_c + q\,N_q + 0.3\,\gamma\,B\,N_\gamma$$

Example 1

A strip of footing 1 m wide is supported in a uniform deposit of stiff clay

Given the following:

- c_u = 140 kN/m^2
- γ = 20 kN/m^3
- D_f = 0.6m
- Groundwater was not encountered during subsurface exploration



Determine the q_{ult} & allowable wall load using a FS of 3

As the supporting stratum is stiff clay, a general shear condition is evident.

$$q_u = c\,N_c + q\,N_q + \tfrac{1}{2}\,\gamma\,B\,N_\gamma$$

For undrained condition $\phi_u = 0$,
Use Table 3.1 for Terzaghi formula $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$ (see Textbook)

Thus $$q_u = 140 \times 5.7 + 20 \times 0.6 \times 1 = 810\ kPa$$

Allowable load $$q_a = q_u / FS = 810/3 = 270\ kPa$$

$$Q_a = q_a \times B = 270 \times 1 = 270\ kN/m$$

Example 2

A square footing (2.25 × 2.25 m) is placed at depth of 1.5 m in sand with shear strength parameters $c' = 0$ and $\phi' = 38^o$. The unit weight of the sand is 18 kN/m^3. Determine the allowable bearing capacity and allowable load of the foundation using Terzaghi's bearing capacity equation for a factor of safety 3.

<u>Solution:</u>

For a square footing on sand

$$q_u = 1.3\, c\, N_c + q\, N_q + 0.4\, \gamma\, B\, N_\gamma$$

for $\phi' = 38^o$ by using Table 3.1, we get $N_q = 61.5$ and $N_\gamma = 82.3$

$q_u = (18 \times 1.5 \times 61.5) + (0.4 \times 18 \times 2.25 \times 82.3)$

$$q_u = 1661 + 1333 = 2994 \text{ kN/m}^2$$

Allowable bearing capacity

$$q_{all} = \frac{q_u}{FS} = \frac{2994}{3} = 998\ kN/m^2$$

Allowable load Q_{all} =

$$q_{all} B^2 = 998(2.25)^2 = 5052 \text{ kN}$$

Local shear failure

For foundation that exhibit local shear failure mode in soils, Terzaghi suggested to use

$$c' = \frac{2}{3}c$$

$$\phi' = \arctan(\frac{2}{3}\tan\phi)$$

Meyerhoff bearing capacity formula

Research showed that failure line assumed by Terzaghi in developing his formula is representative, but the angle formed by the failure line with horizontal is not equal to ϕ but more like $(45+\phi/2)$.

Based on these findings, Meyerhoff proposed a bearing capacity equation similar to Terzaghi equation, with different bearing capacity factors.

$$q_u = s_c\, d_c\, i_c\, c\, N_c + s_q\, d_q\, i_q\, q\, N_q + s_\gamma d_\gamma\, i_\gamma\, \tfrac{1}{2}\, B\, \gamma\, N_\gamma$$

where: s = shape factor, d =depth factor, i = inclination factor

for BC factors, see Table 3.2 (Textbook)

Shape and Depth Factors

Formula for shape factors given by De Beer (1970) are

$$Sc = 1 + \left(\frac{B'}{L'}\right)\left(\frac{N_q}{N_c}\right)$$

$$Sq = 1 + \left(\frac{B'}{L'}\right)\tan\phi$$

$$S\gamma = 1 - 0.4\left(\frac{B'}{L'}\right)$$

Formulas for depth factors proposed by Hansen (1970) are

For $D_f/B \leq 1$

$$d_c = 1 + 0.4$$

$$d_q = 1 + 2 \tan\phi(1 - \sin\phi)^2$$

$$d_\gamma = 1$$

For $D_f/ B > 1$

$$d_c = 1 + 0.4 \tan^{-1} D_f/B$$

$$d_q = 1 + 2 \tan\phi(1 - \sin\phi)^2 \tan^{-1} D_f/B$$

$$d_\gamma = 1$$

Inclination Factors

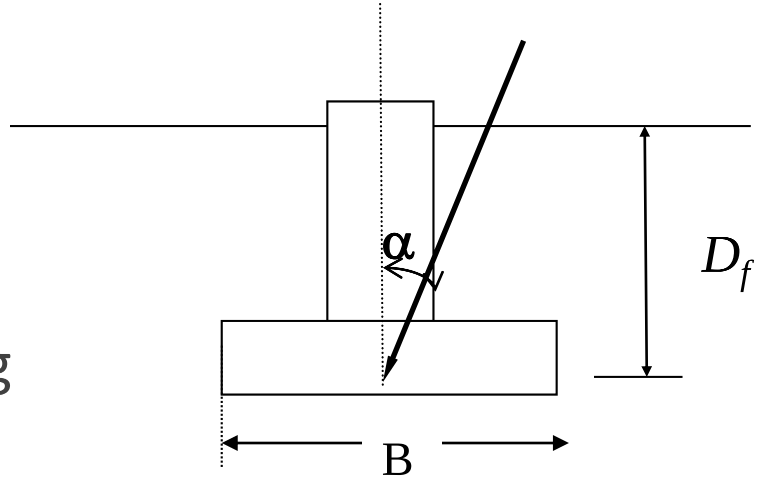
Footing may be subjected to inclined load and this condition leads to a reduction in bearing capacity if the direction of loading is similar to the direction of the major principal plane. The effect of load inclination is proposed by Meyerhoff (1963) and Hanna and Meyerhoff (1981)

$i_c = i_q = (1 - \alpha/90)^2$

$i_\gamma = (1 - \alpha/\varphi)^2$

where α is the angle of loading

with vertical axis



Example 3

A square footing (1.5 m by 1.5 m) is subjected to an inclined load as shown

Determine the FOS against bearing capacity failure

Soil is cohesive with c = 90 kPa; γ = 20.40 kN/m^3

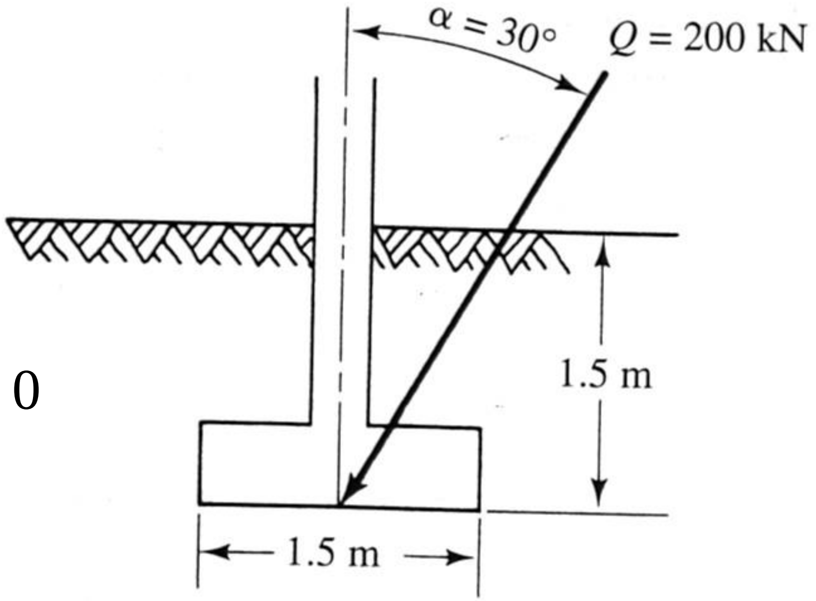
For a square footing,

$$q_{ult} = 1.3cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma$$

$$\gamma_1 = \gamma_2 = 20.40 \text{ kN/m}^3$$

$$D_f = 1.5 \text{ m}, \ B = 1.5 \text{ m}$$

$$for\ \phi = 0,\ N_c = 5.14,\ N_q = 1.0,\ N_\gamma = 0$$



For $\alpha = 30^{o}$

$i_c = i_q = 0.44$ and $i_\gamma = 0$

$$q_{ult} = 1.3(0.44)(90)(5.14) + (20.40)(0.44)(1.5)(1.0) + 0.4(1)(20.40)(1.5)(0)$$

$$q_{ult} = 290.31 \text{ kN/m}^2$$

$$Q_v = Q\cos 30^{o} = 200(\cos 30^{o}) = 173 \text{ kN}$$

$$FOS = \frac{Q_{ult}}{Q_v} = \frac{290.31(1.5 \times 1.5)}{173} = 3.78$$

Eccentric Load

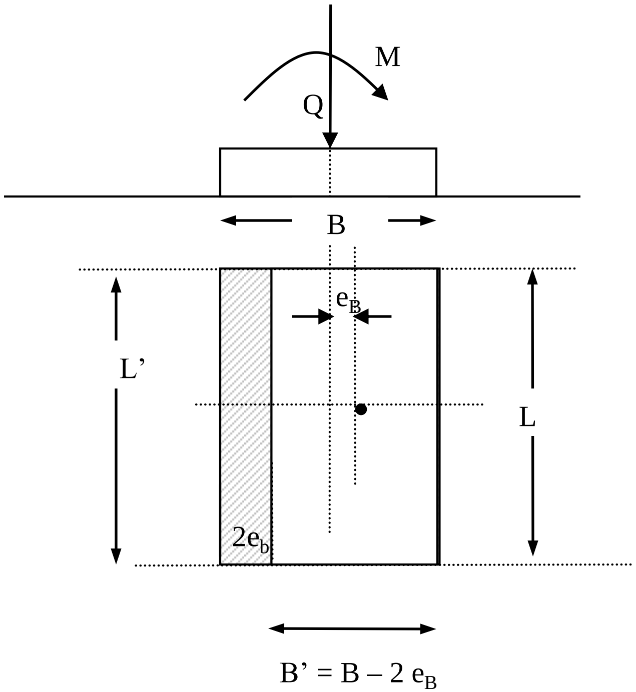
Some footing carry eccentric or moment load derived from the eccentricity of column or by horizontal load acting at some distance above the foundation base.

These loads may be permanent such as that presents on retaining wall or temporary such as wind or seismic load.

Meyerhoff suggested that the bearing capacity of foundation carrying an eccentric load should be evaluated by assuming that the load is uniformly distributed over an effective area of foundation base.

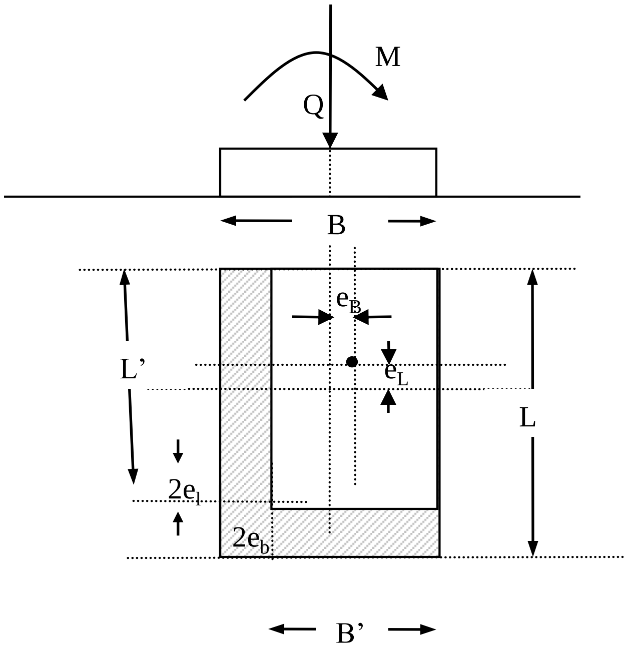
Eccentric Load

One way eccentricity



$$B' = B - 2\,e_B$$
$$L' = L$$

Two way eccentricity



$$B' = B - 2\,e_B$$
$$L' = L - 2\,e_L$$

Contact Pressure for eccentric load

$$q = \frac{Q}{A} \pm \frac{M_x\, y}{I_x} \pm \frac{M_y\, x}{I_y}$$

$$q = \frac{Q}{A}\left(1 \pm \frac{6\, e_B}{B} \pm \frac{6\, e_L}{L}\right)$$

Example 4

A 1.5 m by 1.5 m footing is loca

1.2 m below the ground surfac

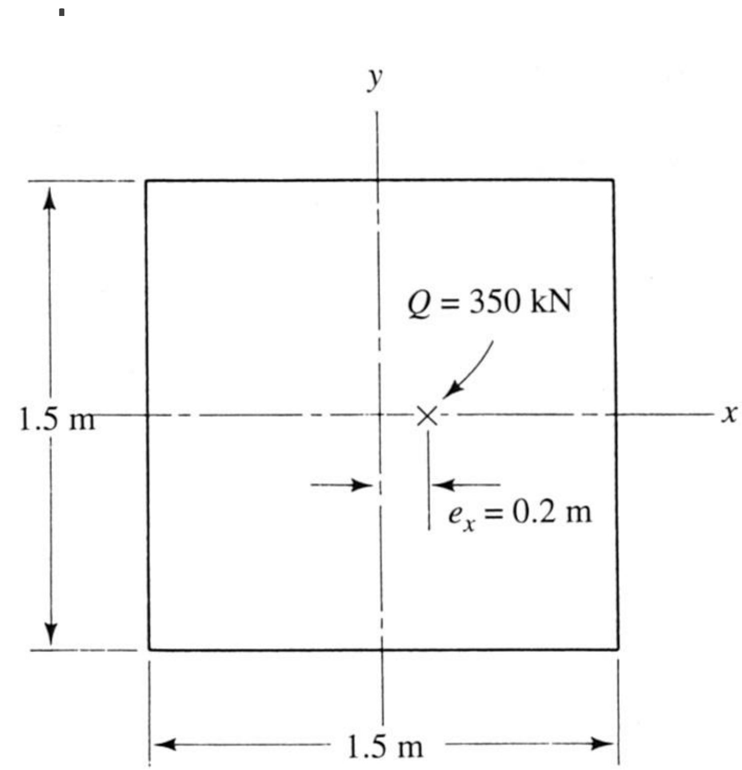
The footing is subjected to an

eccentric load of 350 kN

The subsoil consists of a thick

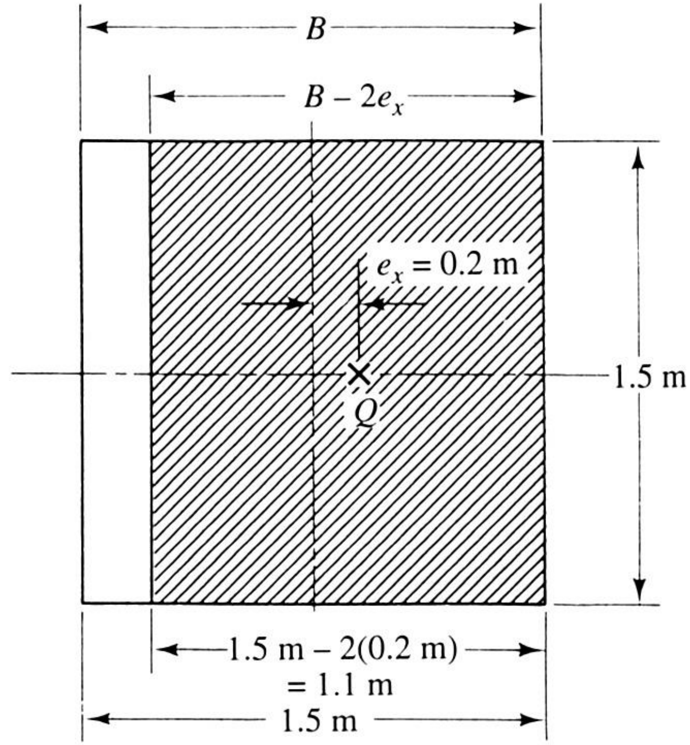
deposit of cohesive soil with

- c_u = 100 kN/m^2
- γ = 20.40 kN/m^3



GWT is at a great depth, & its effect can be ignored

Determine the FOS against bearing capacity failure by concept of useful width



From the figure, the useful width $=1.1$ m

$$q_{ult} = 1.3cN_c + \gamma_1 D_f N_q + 0.4\gamma_2 B N_\gamma$$

For $\phi = 0$; $N_c = 5.14$, $N_q = 1.0$, $N_\gamma = 0$

$\gamma_1 = \gamma_2 = 20.40\ \text{kN/m}^3$, $B = 1.1$ m

$$q_{ult} = (1.3)(100)(5.14) + (20.40)(1.2)(1.0) + (0.4)(20.40)(1.1)(0)$$

$$q_{ult} = 641.3\ \text{kN/m}^2$$

$$FS = \frac{641.3}{\left(\dfrac{350}{1.1\times1.5}\right)} = 3.02$$

Gross vs Net pressure

The results of bearing capacity calculation usually is presented in the form of gross allowable load which is the actual pressure may be placed on the soil. The net pressure is the gross pressure less the effective weight of soil removed for construction of the foundation i.e :

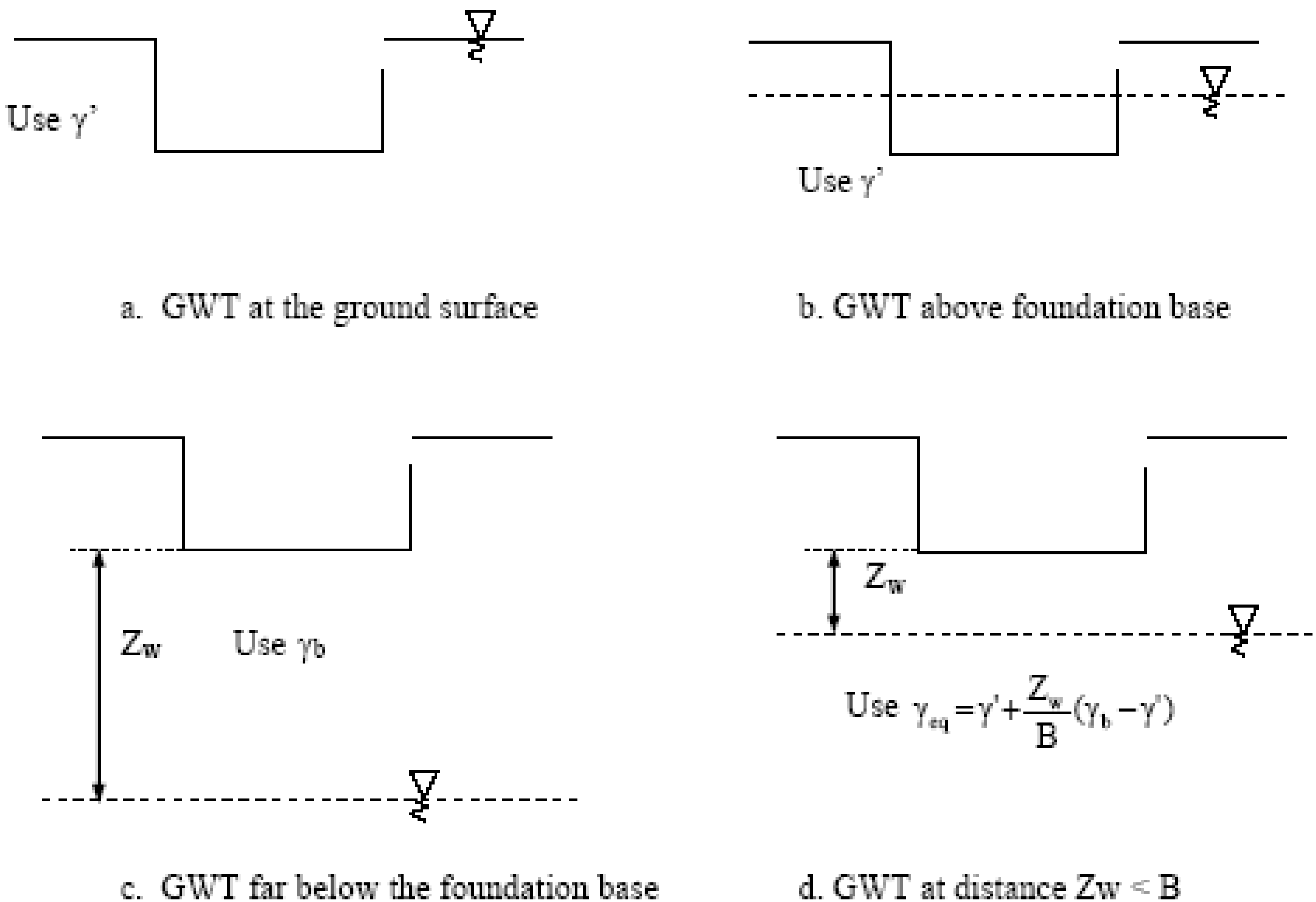
$$q_a = q_d - \gamma D_f$$

The factor of safety with respect to shear failure is defined in terms of the net ultimate or design bearing capacity (q_d) where :

$$FS = \frac{q_u}{q_a} = \frac{q_u}{q_d - \gamma D_f}$$

However, in the case of shallow footing, there is no significant difference in the factor of safety obtained in terms of net or gross pressure. The consideration of net pressure is very important for the case of design of mat or raft foundation.

Groundwater Table position



Example 5

A circular raft foundation of diameter 15 m is founded at a depth of 4.5 m. The groundwater level is located at depth of 2.5 m below the ground level. The soil is a deep layer of a lightly over-consolidated clay with shear strength properties of $c' = 10$ kPa and $\phi' = 24°$, unit weight γ_b=16.8 kN/m^3 and γ_{sat} = 18 kN/m^3 (use γ_w = 9.8 kN/m^3).

(a) Determine the ultimate bearing capacity of the soil

(b) Calculate the design load for the foundation considering the position of the base of foundation and factor of safety 3.0.

Use Terzaghi formula for circular footing

$q_u = 1.3\, c\, N_c + \gamma D_f N_q + 0.3\, B\, \gamma N_\gamma$

For $\phi' = 24^o$

$N_c = 23.4 \quad N_q = 11.4 \quad N_\gamma = 7.9$ (Table 3.1)

$q_u = (1.3\times10\times23.4 + (16.8\times2.5+(18-9.8)\times2)\times11.4 + 0.3\times15\times(18-9.8)\times7.9$

$= 304 + 665 + 291$

$q_u = 1260\ \text{kN/m}^2$

$q_d = q_a + \gamma D_f$

$(1260/3) + (16.8\times2.5+(18-9.8)\times2)$

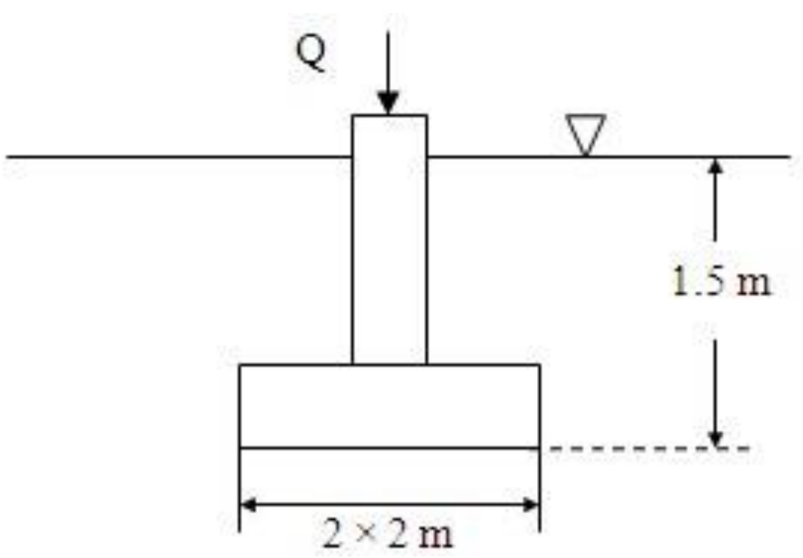
$= 420 + 58.4 = 478.4$

$Q_d = q_d \times A = 478.4\times(176.7)$

$Q_d = 84497$ kN = 84.5 MN.

Example 6

A 2 × 2 m square footing is located 1.5 m below the ground surface. The groundwater table coincide the ground surface. The properties of the subsoil are: $\phi = 20^o$, c = 15 kPa, γ_{sat} = 16.5 kN/m^3. Estimate the allowable bearing capacity of the footing if local shear failure is expected to occur. Use factor of safety 3



The bearing capacity for local shear failure condition

$$q_u = 1.3\,c'N_c + \gamma D_f N_q + 0.4\,\gamma B N_\gamma$$

$c' = 2/3\ c = 2/3\ 15 = 10$ kPa,

$\tan\phi' = 2/3 \tan\phi \rightarrow \phi' = 13.6°$

For $\phi' = 13°$, based on Table 3.1,

we get $N_c = 11.4$, $N_q = 3.6$, $N_\gamma = 1.6$

Because ground water table is at the ground surface, then use γ' for the second and the third term in bearing capacity equation:

Assume $\gamma_w = 9.8$ kN/m^3 → $\gamma' = \gamma_b - \gamma_w = 16.5 - 9.8 = 6.7$ kN/m^3

$$q_u = (1.3\times10\times11.4) + (6.7\times1.5\times3.6) + (0.4\times6.7\times2\times1.6)$$

$$q_u = 148.2 + 36.1 + 21.4 = 205.7 \text{ kPa}$$

$$q_{all} = \frac{q_u}{FS} = \frac{205.7}{3} = 68.5 \text{ kPa}$$

$Q_{all} = q_{all} \times$ area of footing $= 68.5 \times 2 \times 2 = 274$ kN.

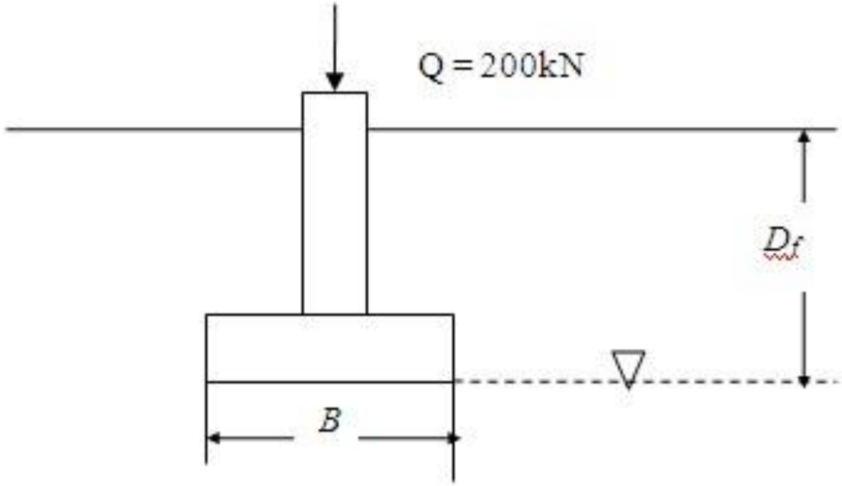
Example 7

A strip footing is designed to carry a load of 200 kN/m^1. The soil supporting the foundation has a shear strength parameters of $c' = 24$ kN/m^2 and $\phi' = 0$.

The saturated unit weight of the soil is $\gamma = 19$ kN/m^3 while the unit weight of the soil above water table is 17.5 kN/m^3. The water table is at 1.5 m below the soil surface.

(a) Determine the depth and the width of the strip footing,

(b) Using your design value for B and D_f, evaluate the safe bearing capacity of the foundation soil based on Terzaghi's method with a factor of safety of 3.0.



(a) Since the depth of ground water table is 1.5 m, then for practical reason the base of the foundation should be above the ground water table. We place the foundation slightly above the water table and do the calculation for D_f = 1.5 m.

Thus, using Terzaghi equation for strip footing

For $\phi' = 0$; $N_c = 5.7, N_q = 1, N_\gamma = 0,$

$$q_u = c\,N_c + \gamma D_f N_q$$

$$q_u = 24 \times 5.7 + 17.5 \times 1.5 \times 1$$

$$q_u = 163 \text{ kPa}$$

For strip footing

$$Q_{all} = \frac{q_u\, B}{FS} \qquad B = \frac{Q_{all}\, FS}{q_u} = \frac{200 \times 3}{163} = 3.68 \text{ m}$$

(b) For design, we use $D_f = 1.5$ m, $B = 4$ m, and $FS = 3$.

$$Q_{all} = \frac{q_u\, B}{FS} = \frac{163 \times 4}{3} = 217 \text{ kN/m} > 200 \rightarrow \text{OK}$$

Empirical Bearing Capacity

There are occasions when we need to estimate the bearing capacity of shallow foundation quickly whereas we do not have the results of laboratory testing for shear strength parameters. Some formulas have been developed for this problem based on the results of field testing, i.e.: standard penetration test (SPT), cone penetration test (CPT), and plate load test. These empirical formulas were developed based on settlement limitation.

BC Based on SPT Value

Estimation of bearing capacity for shallow foundation based on SPT test results developed by Meyerhoff for settlement of 25 mm:

For B < 1.2 m,
$$q_a = 20N\left(1 + 0.33\frac{D_f}{B}\right)$$

For B > 1.2 m,
$$q_a = 12.5N\left(\frac{B + 0.33}{B}\right)^2\left(1 + 0.33\frac{D_f}{B}\right)$$

q_a is the allowable load in kPa, N is average SPT number in blows/305mm obtained between depth 0.5 B above the footing base, and depth 2 B below the footing base

For settlement (s_j) is not equal to 25 mm,
$$q_a' = \frac{s_j}{25}q_a$$

where q_a' is the bearing capacity for settlement of s_j.

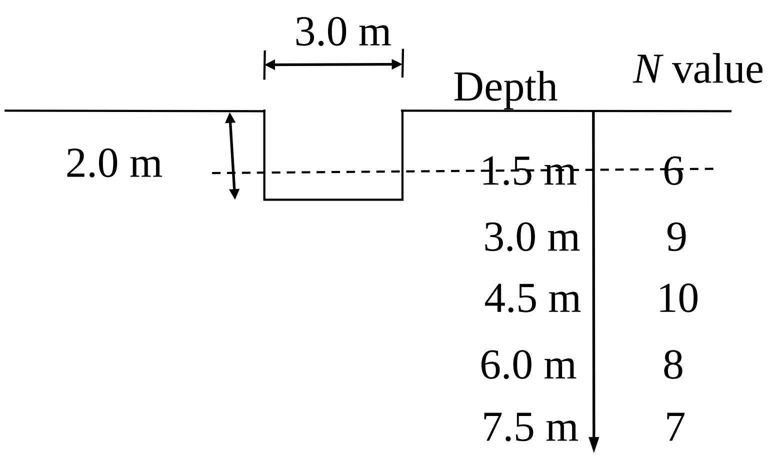
Example 8

A footing of width 3 m is placed at 2 m below ground surface. The corrected N values obtained from SPT test are shown in Figure. What is the allowable bearing capacity of the footing for the maximum settlement (a) 25 mm (b) 12.5 mm

Solution

The width of foundation B = 3 m, thus the N value to consider are from depth (0.5 B above the foundation base to 2B below the base) of 0.5 m to 8 m from the ground surface.

$$N_{ave} = \frac{6+9+10+8+7}{5} = 8$$



For $B > 1.2$ m

$$q_a = 12.5\,N\left(\frac{B+0.33}{B}\right)^2\left(1+0.33\,\frac{D_f}{B}\right)$$

$$q_a = 12.5\times 8\left(\frac{3+0.33}{3}\right)^2\left(1+0.33\times\frac{2}{3}\right)$$

$q_a = 150$ kPa

For $B = 3$ m, $Q_a = q_a \times B = 150 \times 3 = 450$ kN

If the allowable settlement is 12.5 mm, then

$$q_a' = \frac{S}{25}\,q_a = \frac{12.5}{25}\,150.6 = 75\text{ kPa}$$

For $B = 3$ m, $Q_a = q_a \times B = 75 \times 3 = 225$ kN

BC Based on CPT Value

Estimasi daya dukung pondasi tapak shallow foundation based on CPT dapat dilakukan dengan menggunakan persamaan L'Herminier yaitu

$$q_a = \frac{q_c}{10}$$

q_a is the allowable load in the same unit as q_c ,

q_c is average cone resistance obtained between depth 0.5 B above the footing base, and depth 2 B below the footing base

BC Based on CPT Value

Dengan rumus Schmertmann, 1978 (Dalam buku Bowles) Berasing capacity of foundation in (kPa)

Untuk pondasi di atas tanah pasir:

Pondasi lajur $q_u = 28 – 0.0052\ (300- q_c)^{1.5}$

Pondasi tapak $q_u = 48 – 0.009\ (300- q_c)^{1.5}$

Untuk pondasi di atas tanah lempung:

Pondasi lajur $q_u = 2 + 0.28q_c$

Pondasi tapak $q_u = 5 + 0.34q_c$

q_u is the ultimate load in the same unit as q_c ,

q_c is average cone resistance obtained between depth 0.5 B above the footing base, and depth 1.1 B below the footing base in kg/cm^2

Gunakan Faktor keamanan 3 untuk mendapatkan q_a atau $q_a = \frac{q_u}{3}$

Example 9

Estimate the bearing capacity of shallow foundation of width 3m if the average cone resistance obtained between the depth of 0.5 B above foundation base and 2 B below foundation base is 15 kg/cm^2. Tanah dawah pondasi merupakan tanah lempung.

Solution

Use L'Herminier formula:

$$q_a = \frac{q_c}{10} = \frac{15}{10} = 1.5 \text{ kg/cm}^2$$

The capacity of the foundation of 3 x 3 m = q_a x B x B = 150 kPa x 9 = 1350 kN

Use Schmertmann Formula

$$q_u = 5 + 0.34q_c = 5 + 0.34 \text{ x } 15 = 515 \text{ kPa}$$

$$q_a = \frac{q_u}{3} = 171 \text{ kPa}$$

The capacity of the foundation of 3 x 3 m = q_a x B x B = 171 kPa x 9 = 1539 kN

Plate Load Test

The most reliable method of obtaining the ultimate bearing capacity of a foundation is to perform a plate load test.

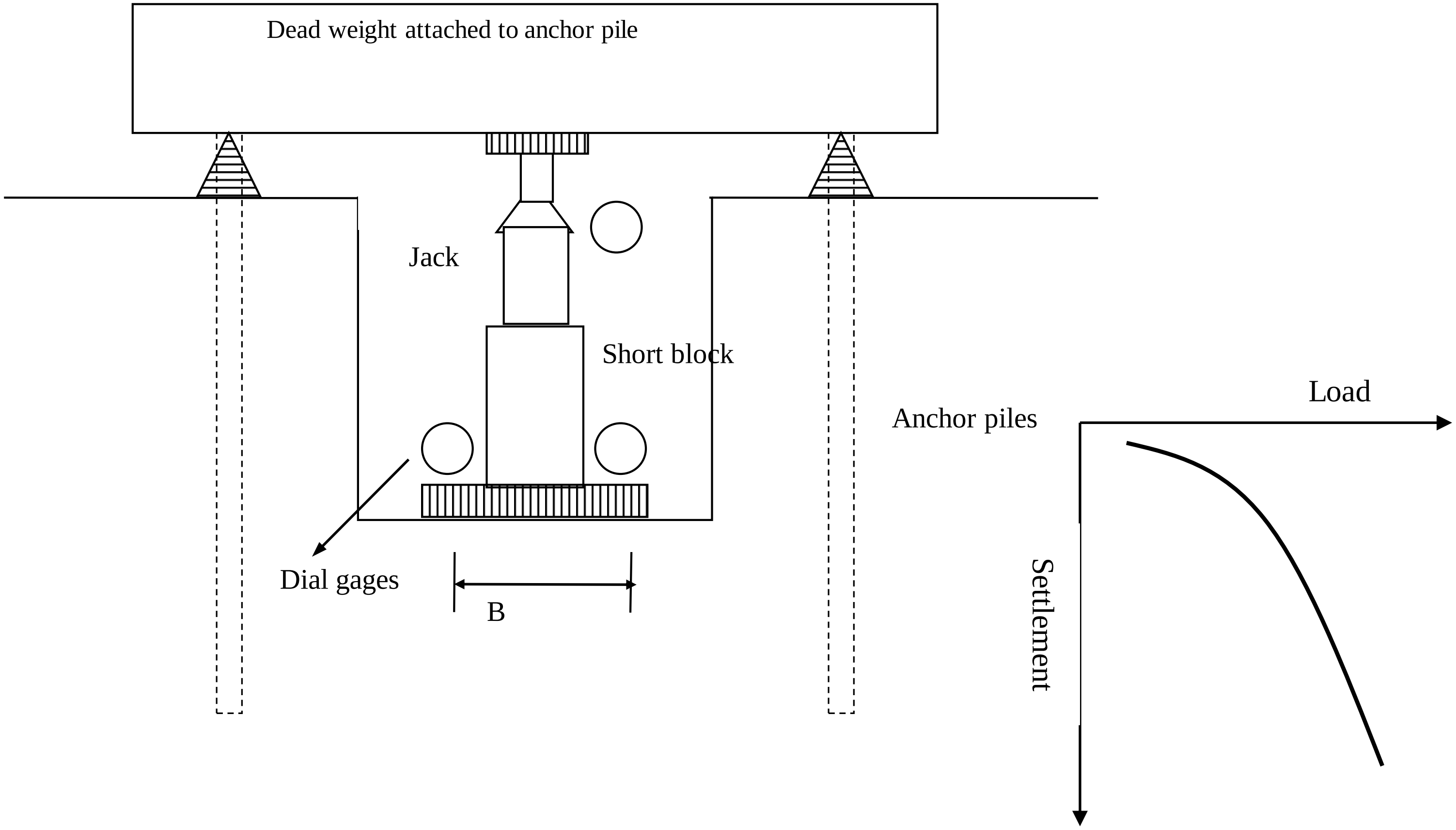
The common practice is to load a small square plate of size 300 × 300 mm or 600 × 600 mm.

The procedure of the test is standardized as ASTM D 1194.

The ultimate bearing capacity is defined as the load applied at 25 mm settlement.

$$q_a = q_{plate}\left(\frac{B_{foundation}}{B_{plate}}\right)$$

Plate Load Test



Example 10

Plate load test done (600 × 600 mm) at depth of 1.5 m below the ground surface.

(a) Plot the load versus displacement from the test and estimate the ultimate load for 25 mm settlement.

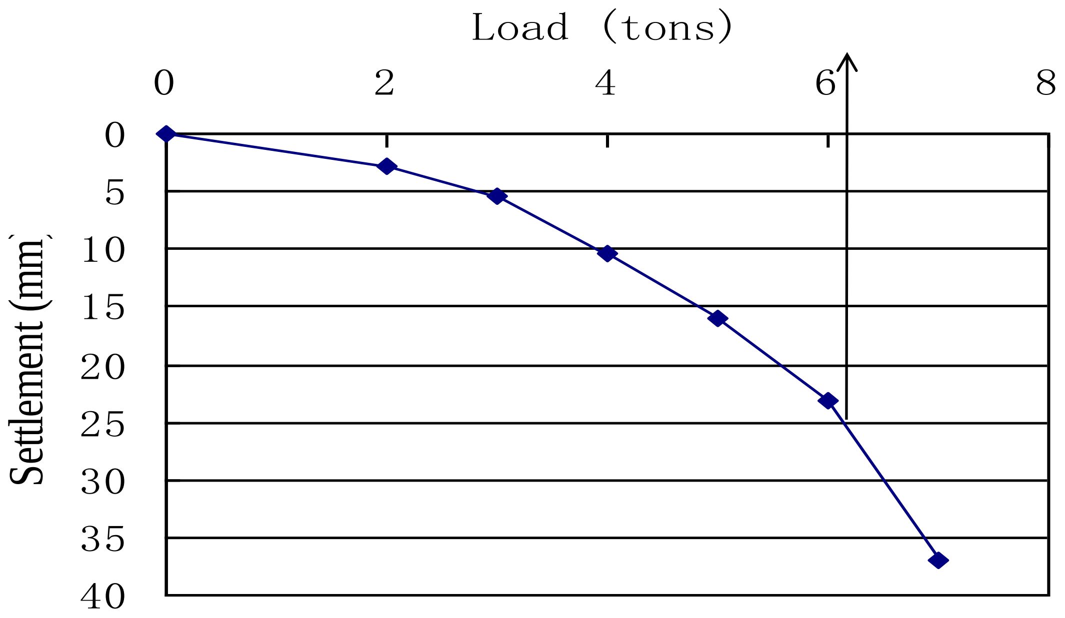
(b) Calculate the bearing capacity of a shallow foundation of width 3 m embedded at 1.5 m below the ground surface.

(c) If the properties of the soil is as follows $\gamma_b = 17.3$ kN/m^3; $\phi' = 25^o$, and $c = 10$ kPa; calculate the bearing capacity of the footing according to Terzaghi.

Compare (b) and (c)

Load (tons)	0	2	3	4	5	6	7
displacement(mm)	0	2.85	5.38	10.31	16.03	23.16	36.98

(a)



From the plot, the load that cause 25 mm settlement is 6.2 tons = 62 kN

(b) $q_{plate} = \dfrac{\text{Load}}{\text{Area of plate}} = \dfrac{62}{0.6 \times 0.6} =$ 172.2 kPa

$$q_a = q_{plate}\left(\frac{B_{foundation}}{B_{plate}}\right)$$

$$q_a = 172.22\left(\frac{3\,\text{m}}{0.6\,\text{m}}\right) = 861\text{kPa}$$

(c) Using Terzaghi equation for the given data

Footing dimension, B = 3 m, D_f = 1.5 m

Soil Properties: γ_b = 17.3 kN/m^3; ϕ' = 25^o, and c' = 10 kPa

For ϕ' = 25^o,

use Table 3.1 → N_c = 25.1 N_q = 12.7 N_γ = 9.2

$q_u = 1.3\, c\, N_c + \gamma D_f N_q + 0.4\, \gamma B N_\gamma$

$q_u = (1.3 \times 10 \times 25.1) + (17.3 \times 1.5 \times 12.7) + (0.4 \times 17.3 \times 3 \times 9.2)$

$q_u = 326 + 330 + 191$

$q_u = 847$ kPa

Total & differential settlement

Footing may also fail as a result of excessive settlement

Thus, after the size of the footing has been determined by bearing capacity analysis, footing settlement should be calculated & the design revised if calculated settlement is excessive

Calculation of settlement is covered in the

Topik: Compressibility & Settlement

Pile Foundation

Pile Foundation

Pile foundation is used in cases where the soil upon which a structure is to be built is of such poor quality that a shallow foundation would subject to bearing failure/excessive settlement

They are differentiated from footing foundations in that the ratio of the depth of the foundation to the size of the pile is greater than four.

It works by transferring load to greater depth where the firmer soil is

Piles are needed when designing foundation of transmission tower, offshore platforms or basement mats subjected to uplifting force.

Pile should extend to stable soil layer when the foundation soil is susceptible to swelling or collapse.

Piles are required to support bridge abutments to avoid scouring at the foundation base.

Piles are used extensively to resist both vertical and lateral loads from retaining structures and tall buildings, as well as harbor and offshore structures.

Pile Foundation

Pile may be categorized based on some characteristics such as:

- material forming the pile,
- transverse and longitudinal sections,
- installation method
- load transmission.

Types of piles

By Material Type; Allowable Load & Length

Type of pile	Allowable load (kN)	Maximum length (m)
Timber	150 – 300	15 – 20
Steel H or pipe	300 – 600	Unlimited
Steel pipe, concrete filled	400 – 600	30 – 38
Precast concrete	300 – 500	15 – 20
Cast in situ concrete	300 – 500	15 – 22.5
Cast insitu concrete – bulb type	300 – 500	Up to 30 m
Composite	200 – 300	Up to 45 m

Types of piles

By Material forming the pile

Timber piles cannot withstand hard-driving stress, therefore; the pile capacity is usually limited. Timber pile is highly durable when embedded in saturated soil but deteriorate easily when subjected to change in moisture.

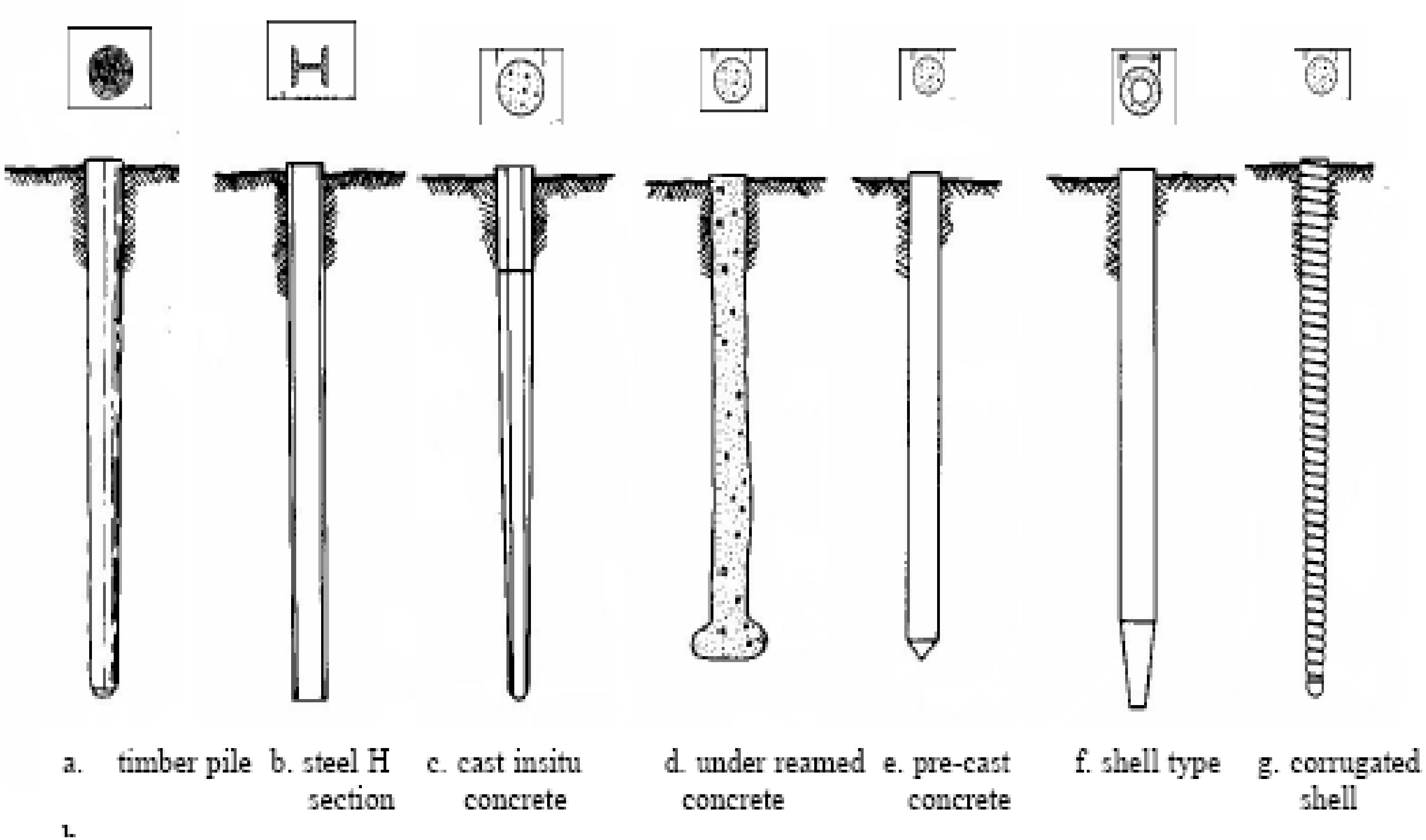
Steel piles are selected when load is high, but they may be subjected to corrosion. Usually H or O section. Pipe piles are often filled with concrete after driving . Steel pile may withstand hard driving condition.

Pre-cast concrete pile is made of reinforced concrete which may be pre-stressed to provide high capacity. High strength concrete is to be used for pre-stressed piles.

Cast-in-situ concrete piles are created by filling a drilled hole with concrete. The hole can be cased or uncased. A bulb or expanded based can be formed by dropping a hammer on the fresh concrete to provided larger contact area at the base.

Types of piles

By Transverse and Longitudinal section



Types of Pile

By Installation Method

Driven/displacement pile

- totally preformed piles driven into the ground (displacement piles)

  e.g. timber piles, pre-cast reinforced concrete, pre-cast pre-stressed concrete, and post-tension concrete piles.
- driven cast in-place (small displacement) piles.

  e.g. shell and steel H sections.

Drilled/replacement piles or non-displacement

e.g. bored piles, micro piles, and flight auger piles.

Types of Piles

by load transmission

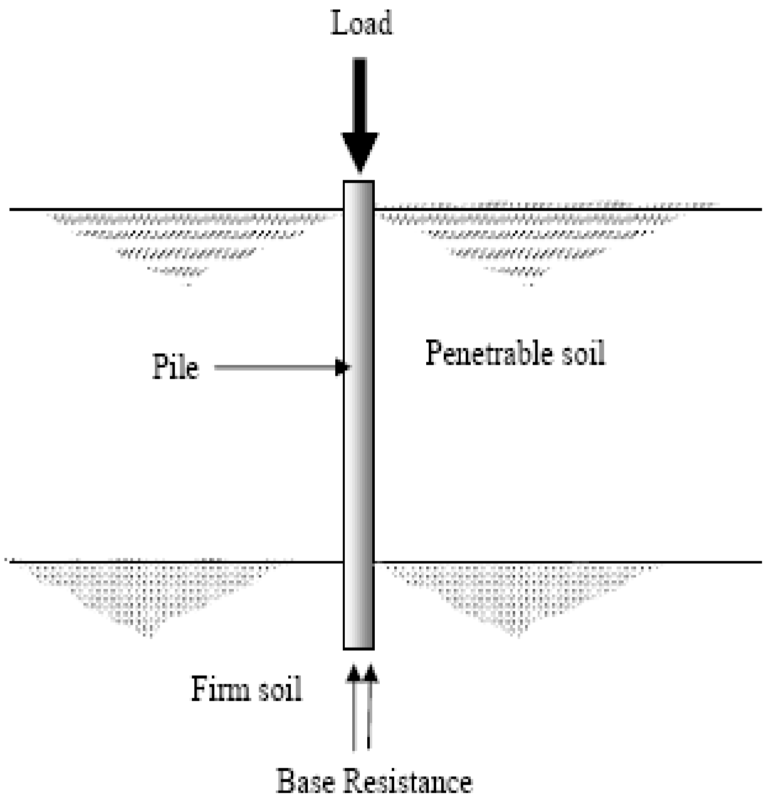


Figure 4.2 End bearing pile to support compression loa

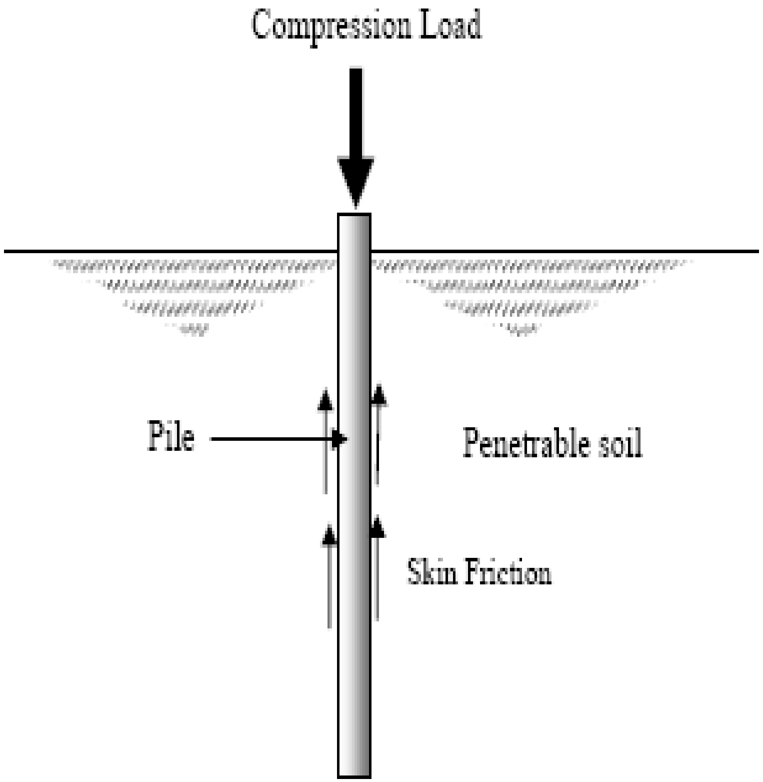


Figure 4.3 Friction piles to support compression load

End bearing piles transfer the load directly to the pile base which rests on a relatively firm soil such as rock, very dense sand or gravel and the base of the pile bears the load of the structure. The load of the structure is transmitted through the pile into this firm soil. *Examples of this type of pile are preformed timber pile and in-situ reinforced concrete pile.*

Friction piles transmit the load of the structure to the penetrable soil by means of skin friction or cohesion between the soil and the embedded surface of the pile. *It is more likely to predominate in clays and silts.*

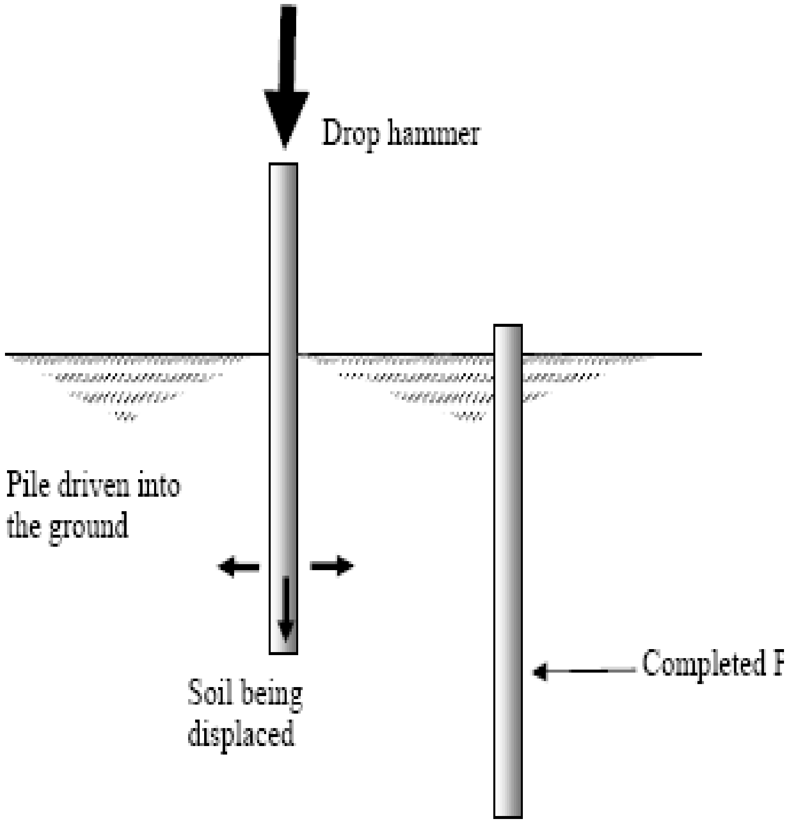
Selection & Design Criteria

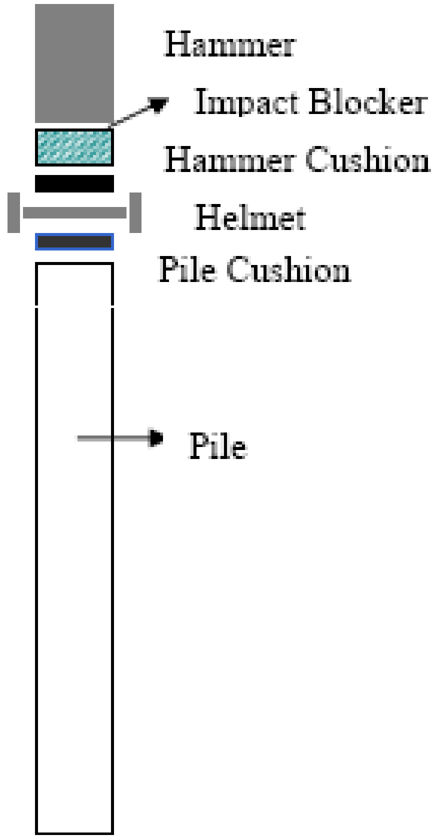
Selection of pile type should be based on some consideration e.g.:

- Topography: surface and drainage conditions
- Soil condition at site
- Type of structure and applied load
- Equipment and technical difficulties such as obstructions etc.
- Environmental condition such as adjacent structures, chemical conditions etc.

Pile Installation

DRIVEN PILES





Pile driving system

Construction of driven piles

- Construction of pile foundation consists of driving the piles & installing pile caps
- Most piles are driven by pile hammer, by alternately raising & dropping
- Several types of pile hammers are available
- Selection of a pile hammer for a specific job depends on a number of factors such as soil condition and pile material.

Pile Installation

DRILLED PILES

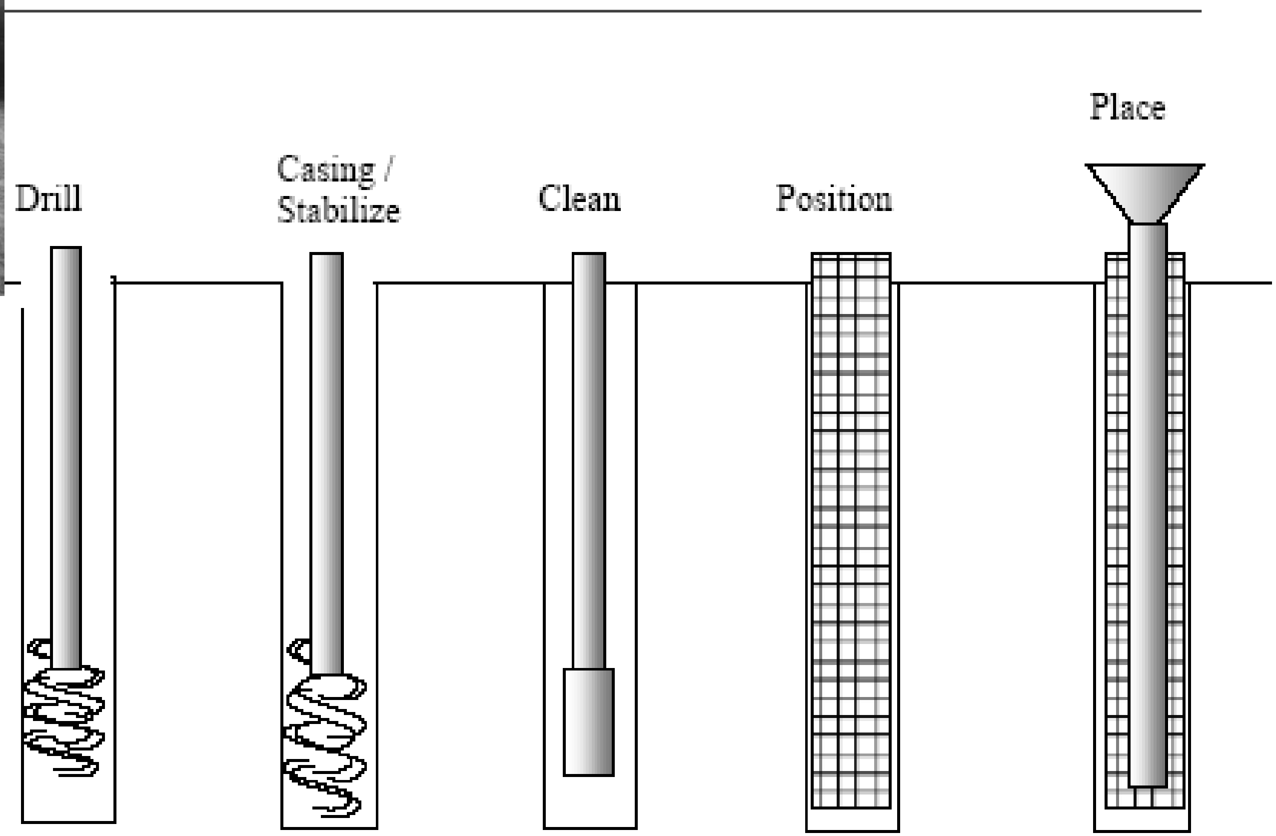


Figure 4.7 Drilling process for installation of bored pile

Construction of drilled shaft

Casing or slurry may be required when there is a potential of cave-in or if ground water table presents.

The base of bored pile can be enlarged to provide greater end bearing capacity of suitable strata and resistance to uplifting.

Construction of bored piles in deposits of dense sand and gravel is easier than driven piles, but this pile is also effective on soft ground or in situation where subsoil condition consists of different soil layers.

Bored pile is versatile in which the depth and diameter of pile can be easily varied.

Drilling equipment is relatively light and easy to use.

Drilling process does not cause excessive noise and ground vibrations

BEARING CAPACITY

Bearing Capacity of Piles

Capacity of pile depends on structural strength & supporting strength of soil

Soil strength

- Bearing Capacity
- Settlement

Structural strength

- size & shape
- type of material

Bearing Capacity of Piles

$$Q_u = Q_b + Q_s$$

$$Q_u = q_b A_b + f_s A_s$$

$$Q_u = q_b A_b + \int_o^{D_f} f_s \, p \, d_h$$

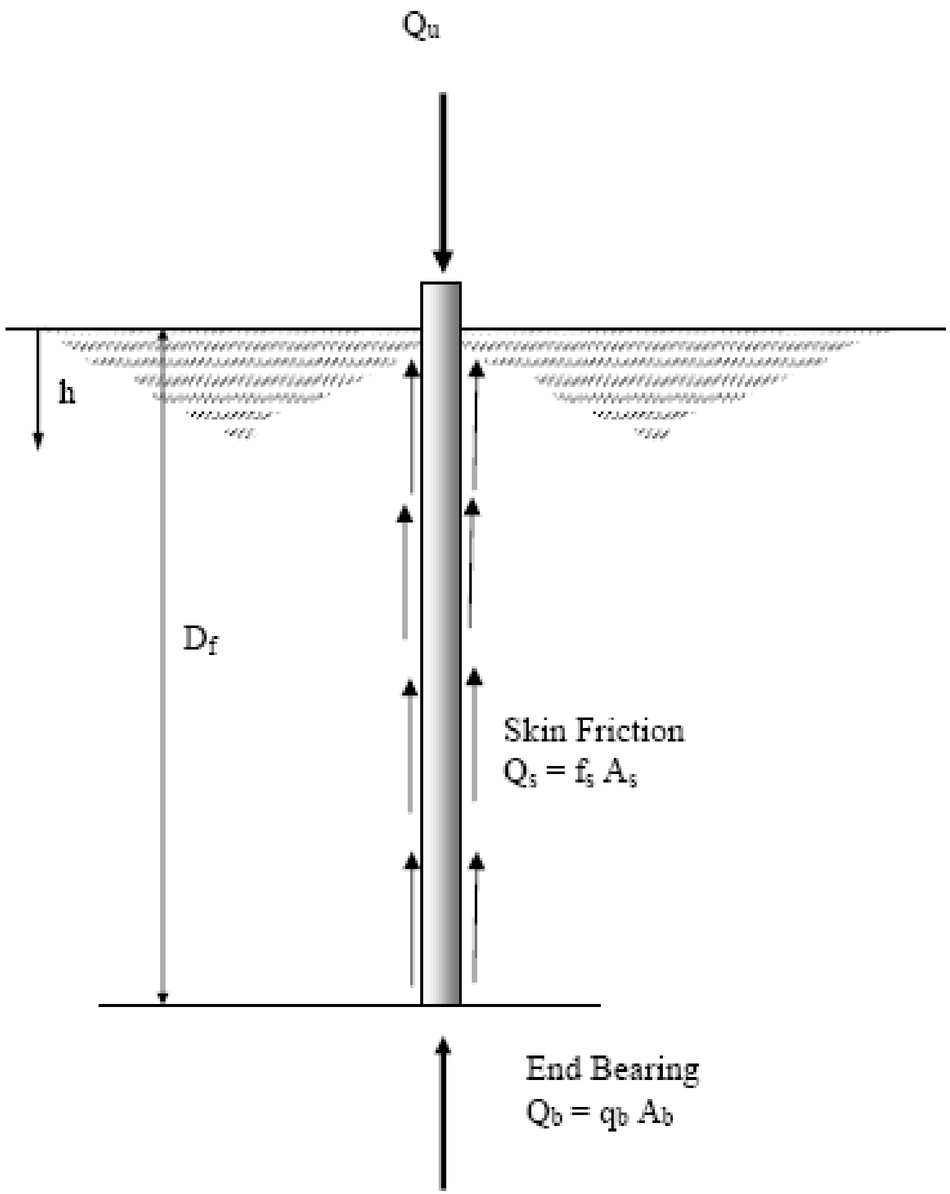


Figure 4.8 Load Transfer Mechanism and Bearing capacity of pile

End Bearing Capacity

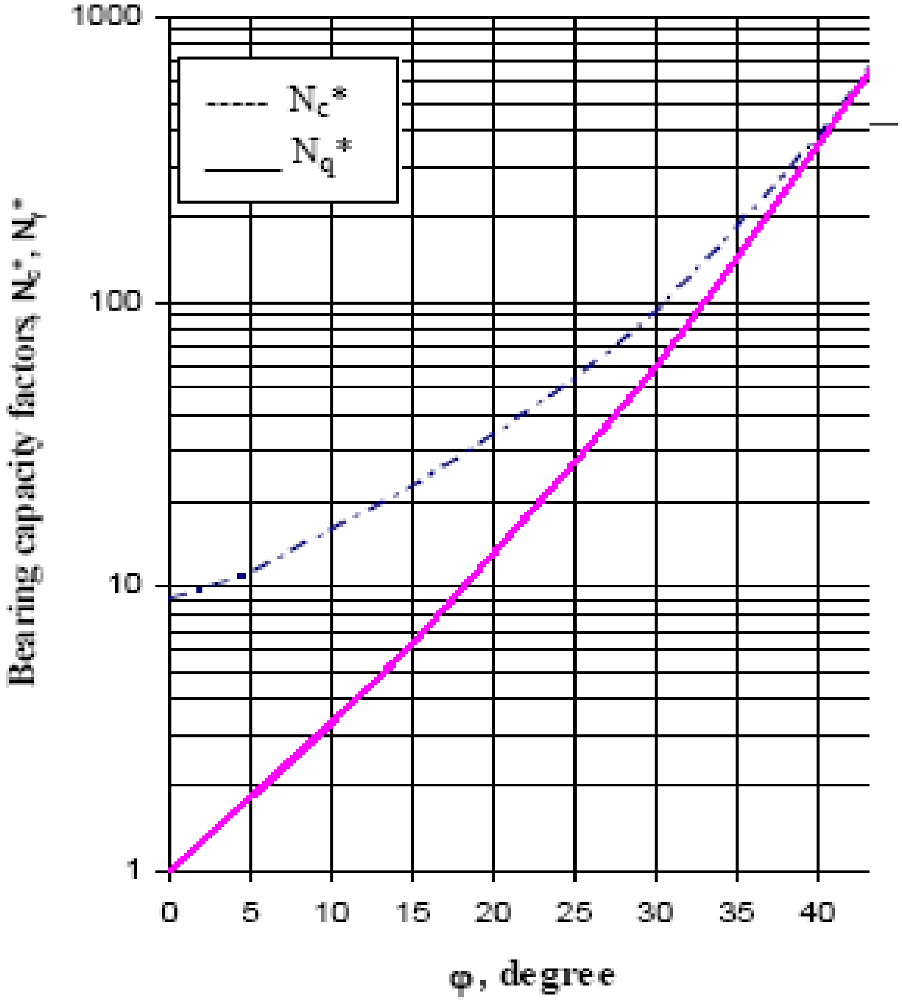
End bearing capacity of piles can be calculated based on Terzaghi BC equation:

$$q_b = c\,N_c + q\,N_q + ½\,\gamma\,B\,N_\gamma$$

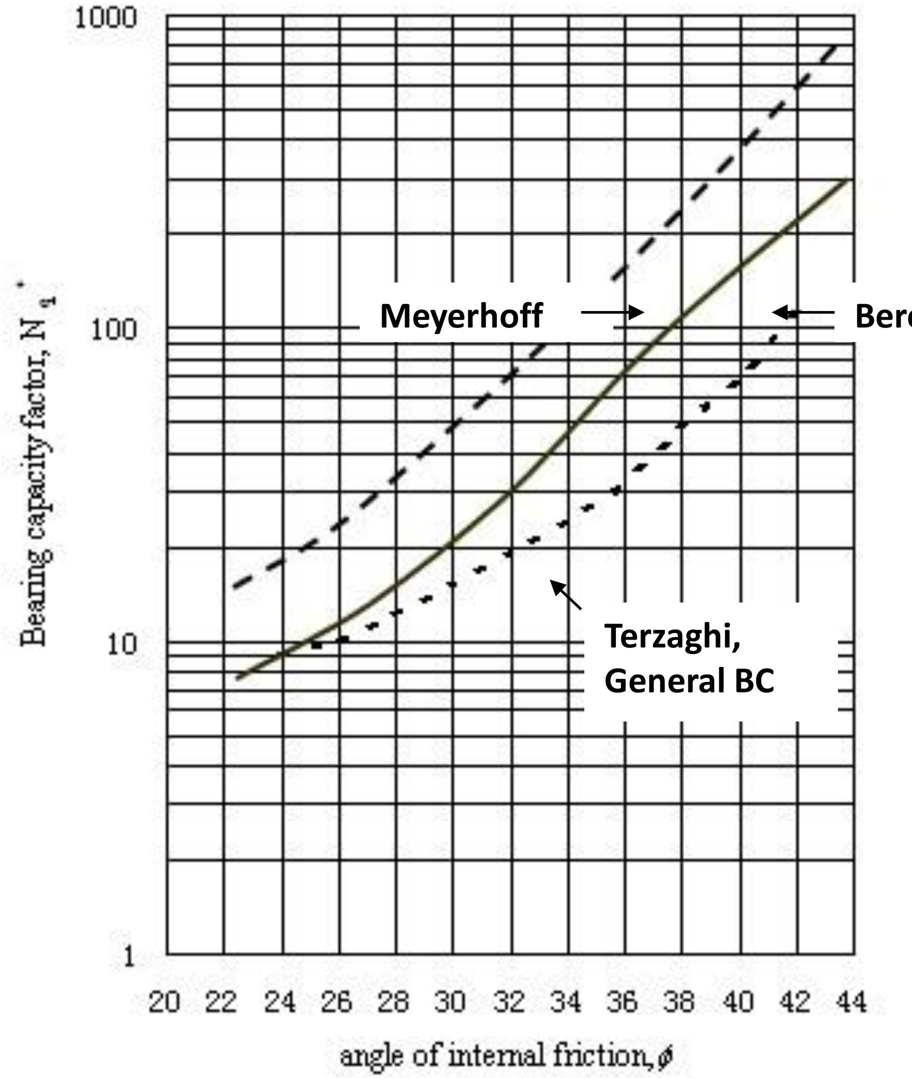
The N_γ term can be neglected because the pile dimension B is usually very small compared to the depth of pile embedment. Since the shape of pile is usually square or circular, and the pile is placed at a substantial depth, the N_c and N_q should be adjusted to shape and depth factors. Furthermore, adjustment for pile weight should be made.

$$q_b = c\,N_c^* + \sigma'_{vo}\,(N_q^* - 1)$$

End Bearing Capacity



Bearing Capacity Factors for End Bearing Capacity of Piles



Friction Bearing Capacity

Ultimate skin resistance is produced at small values of relative slip between the pile and the soil. The slip is progressing down the shaft with increasing load.

The amount of slip required to produce maximum skin resistance is on the order of 5 to 10mm. This is independent of pile diameter and embedment length, but solely depends of the soil properties.

On the other hand, the mobilization of the base resistance requires a settlement of the order 10 to 30% of the pile diameter.

There are three methods available for obtaining unit frictional resistance of pile. The α and λ methods are normally used for piles embedded in clay, while β method is commonly used for pile in sand.

Friction Bearing Capacity

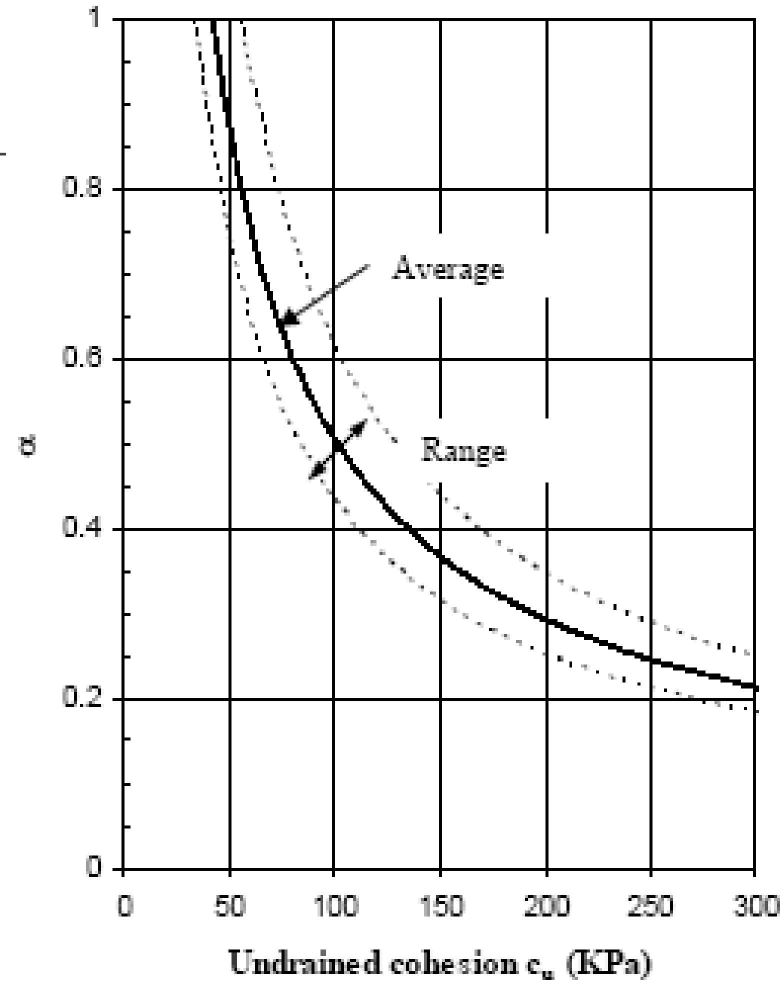
the α method

$$f_s = \alpha\, c + \sigma_{vo}'\, K \tan\delta$$

α = empirical adhesion factor,

c = is average cohesion for soil stratum of interest,

σ_{vo}' = the effective vertical stress at depth of interest, K is the coefficient of lateral earth pressure, and δ is the friction angle between the soil and the pile.



2 Coefficient α as a function of undrained shear strength (c_u).

Friction Bearing Capacity

the β method

For cohesionless soil $f_s = \beta \sigma_{vo}' = K \tan\delta\, \sigma_{vo}'$

σ_{vo}' = the effective vertical stress at depth of interest,

K = the coefficient of lateral earth pressure

δ= the friction angle between the soil and the pile.

Pile material	*K*		$\tan\delta$ (°)
	Loose sand	Dense sand	
Steel (corrugated)	0.5	1.0	Use tan φ of sand
Steel (rough, rusted)	0.5	1.0	0.4
Steel (smooth)	0.5	1.0	0.2
Timber	1.5	3.0	0.4
Concrete	1.0	2.0	0.45

BC of Piles in Cohesive soil

$Q_u = Q_b + Q_s$

$Q_b = N_c^* \, c_u \, A_b = 9 \, c_u \, A_b$

where c_u is the average cohesion in the vicinity of the pile base

$Q_s = \Sigma f_s \, A_s = \Sigma f_s \; p \, \Delta L = \Sigma \, \alpha \, c_u \, p \, \Delta L$

where p is the perimeter of the pile, ΔL is the incremental pile length, and $f_s = \alpha \, c_u$ and c_u is the average cohesion along the incremental length of pile

Notes on Piles driven in clay

Soft clay adjacent to piles may lose a large portion of their strength as a result of being disturbed by pile driving. The original clay's full strength is usually regained within a month after pile driving stops

In cases where the pile has to be loaded immediately after driving, the effect of decreased strength must be taken into account

Slender piles driven in soft clay have a tendency to buckle when loaded

Heavy steel, timber & concrete piles do not tend to buckle if embedded in the soil for their entire length

The ultimate structural load can be computed by:

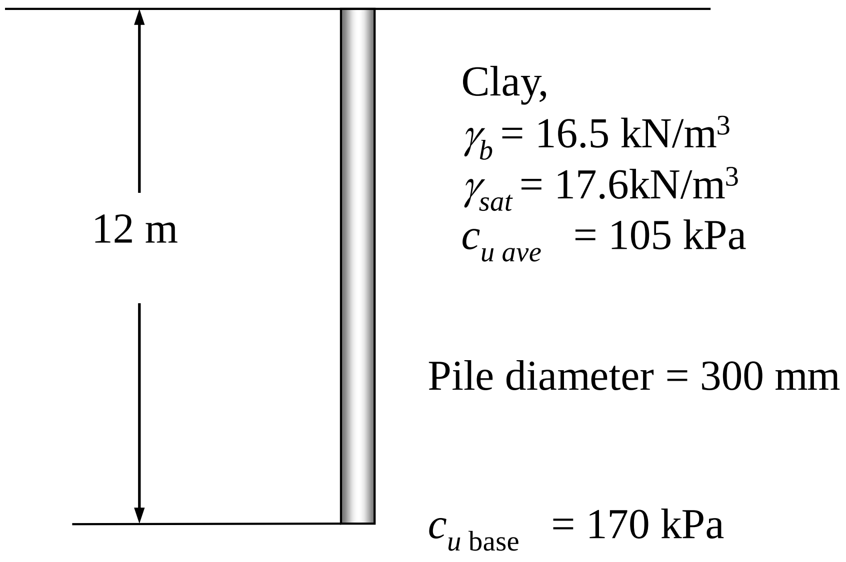
$$Q_u = \lambda \sqrt{cEI}$$

where λ is a ratio between 8 and 10, c is cohesion, E and I are modulus and moment of inertia of the pile

Example 1

A pile of 0.6 m in diameter is driven into clay layer as shown in Figure. If the adhesion coefficient α is 0.45.

Calculate the bearing capacity of the pile if it is embedded between depths of 1 m and 13 m



Ultimate bearing capacity of pile

$$Q_u = q_b A_b + f_s A_s$$

$$q_b = c_u N_c^* = 9 \times 170 = 1530$$

$$A_b = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times (0.6)^2 = \quad 0.2827 \text{ m}^2$$

embedded length of pile $L = 13 - 1 = 12$ m

$f_s = \alpha\ c_u = 0.45 \times 105 = 47.25$

$A_s = \pi d L = \pi \times 0.6 \times 12 = 22.62 \text{ m}^2$

$$\begin{aligned} Q_u &= q_b A_b + f_s A_s \\ &= (1530 \times 0.2827) + (47.25 \times 22.62) \\ &= 432 + 1068 \end{aligned}$$

$$Q_u = 1500 \text{ kN}$$

For FS = 3 $\qquad Q_{all} = \dfrac{Q_u}{3} = 500$ kN

BC of Piles in Cohesionless soil

$$Q_u = Q_b + Q_s$$

the end bearing capacity of pile in cohesionless soil is:

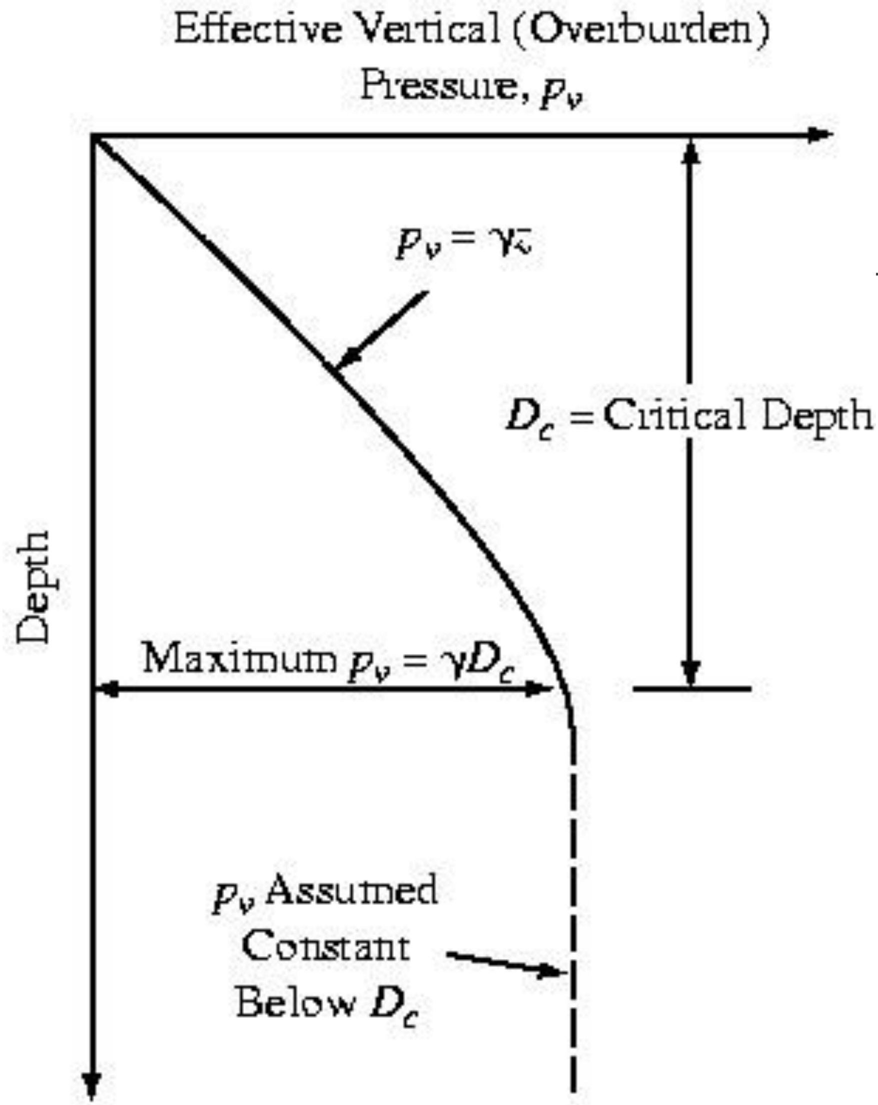
$$q_b = N_q{}^* \, \sigma_{vo}'$$

The shaft friction can be estimated using β method

$$Q_s = \Sigma f_s A_s = \Sigma f_s \; p \, \Delta L = \Sigma \; K\sigma_{vo}' \tan \delta \, p \, \Delta L$$

where p is the perimeter of the pile, ΔL is the incremental pile length, and

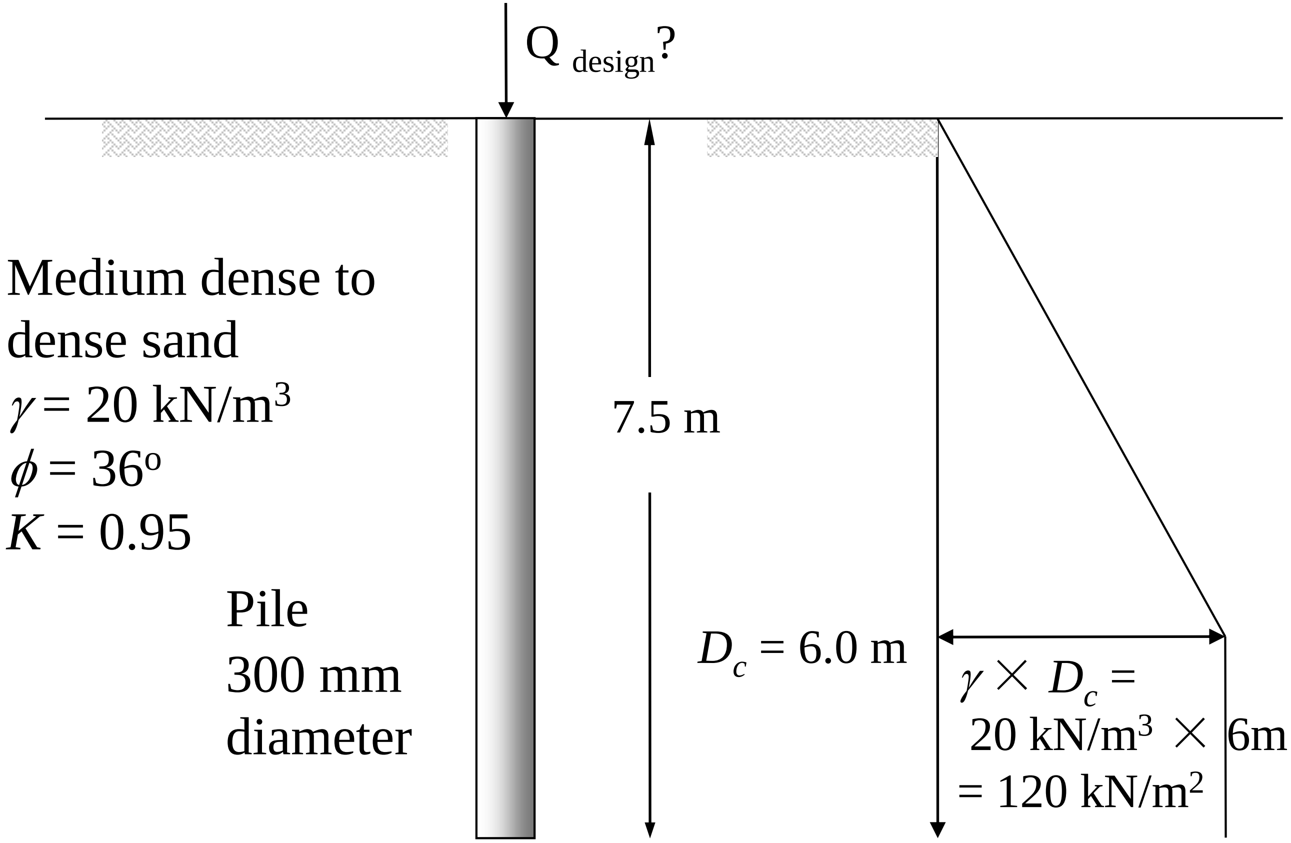
σ_{vo}' is the effective overburden pressure



Dense sand: $D_c = 20d$
Loose sand: $D_c = 10d$
d = diameter or the least dimension of pile

Example 2

A concrete pile is to be driven into sand to a depth of 7.5 m as shown in Figure. No groundwater was encountered during site investigation. Estimate the pile axial capacity if K = 0.95. Use FS = 2



The ultimate bearing capacity of the pile

$$Q_u = A_b\, q_b + A_s\, f_s$$

For dense sand $\quad D_c$ = 20 pile diameter = 20 $\times$ 0.3 m = 6 m

At depth of 6 m, σ_{vo}' = 20 $\times$ 6 = 120 kN/m^2

Base resistance: $\quad Q_b = q_b\, A_b$

From Figure 4.10 $\quad \phi = 36^o, \quad N_q^* = 60$

$$q_b = N_q^*\, \sigma_{vo}' = 60 \times 120 = 7200 \text{ kN/m}^2$$

$$A_b = \frac{\pi}{4} d^2 = \frac{\pi}{4}(0.3)^2 = 0.073 \text{ m}^2$$

$$Q_b = 0.073 \times 7200 = 525 \text{ kN}$$

Friction Resistannce: $\quad Q_s = f_s A_s = K\, \sigma_{vo}' \tan\delta\; p\, L$

Area of pressure diagram $\quad \sigma_{vo}' L$ = ½ $\times$120$\times$ 6+120$\times$(7.5 – 6) = 540 kN/m

$$p = \pi d = \pi \times 0.3 = 0.942 \text{ m}^2$$

For concrete pile $\quad \tan\delta = 0.45$

$$Q_s = K\, \sigma_{vo}' \tan\delta\; p\, L = 0.95 \times 540 \times 0.45 \times 0.942 = 241 \text{ kPa}$$

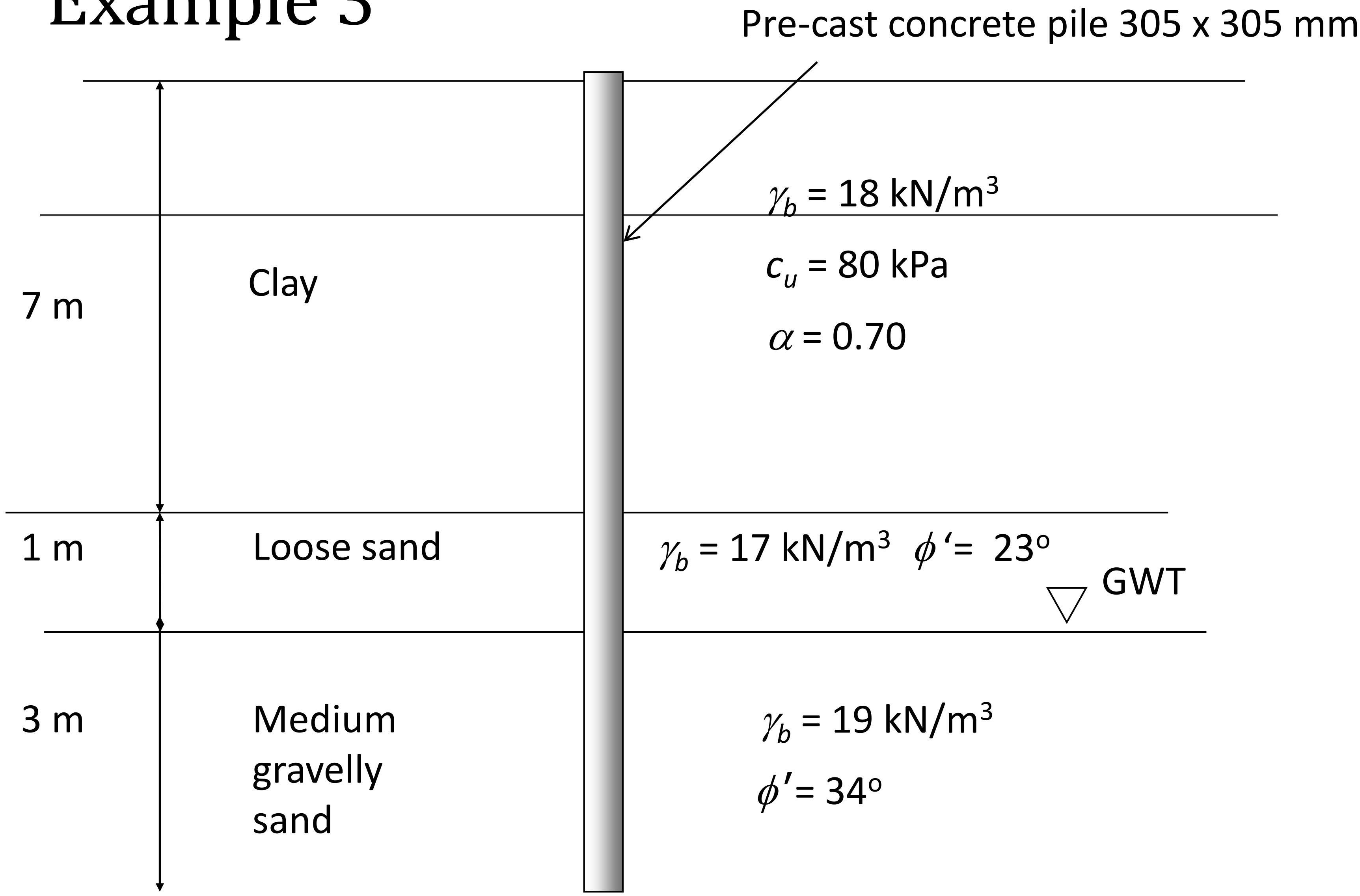
Q_u = 525 + 241 = 766 kN $\qquad$ *For FS* = 2 → $Q_{design} = \dfrac{766}{2}$ = 383 kN

Pile in clay and sand layer

Soils are not homogeneous in nature. There are cases where pile has to penetrate different layers of soil, some time of different types.

The solution is to treat individual layer for friction bearing AND to consider the end bearing of soil at pile tip

Example 3



Pre-cast concrete pile 305 x 305 mm
7 m
Clay
γ_b = 18 kN/m^3
c_u = 80 kPa
α = 0.70
1 m
Loose sand
γ_b = 17 kN/m^3 ϕ'= 23^o
GWT
3 m
Medium gravelly sand
γ_b = 19 kN/m^3
ϕ'= 34^o

Ultimate bearing capacity of pile $Q_u = q_b A_b + f_s A_s$

For pile embedded in medium gravelly sand

$$q_b = N_q^* \sigma_{vo}'$$

Take D_c = 20 pile diameter = 20 × 0.305 m = 6.1 m

At depth of 6.1 m, $\sigma_{vo}' = 18 \times 6.1 = 109.8$ kN/m^2

For $\phi = 34$, use Figure 4.10 $N_q^* = 40$

$q_b = N_q^* \sigma_{vo}' = 40 \times 109.8 = 4392$ kN/m^2

$A_b = 0.305 \times 0.305 = 0.093$ m^2

$$Q_b = q_b A_b = 4392 \times 0.093 = 408.5 \text{ kN}$$

Friction resistance: $Q_s = (f_{s1}\ L_1 + f_{s2} L_2 + f_{s3} L_3)\ p$

Clay layer: $f_{s1}\ L_1 = \alpha\, c_u\ L_1 = 0.7 \times 80 \times 7\text{m} = 392\ \text{kN/m}^2$

Loose sand: $\sigma_{vo}' = 18 \times 6.1 = 109.8\ \text{kN/m}^2$ (uniform)

$K = 1 - \sin 23^o = 0.61$

$\tan\delta = \tan (0.6 \times 23) = 0.25$

$$f_{s2}\ L_2 = K\ \sigma_{vo}' \tan\delta\, L_2 = 0.61 \times 109.8 \times 0.25 \times 1\text{m} = 16.75\ \text{kN/m}^2$$

Dense sand: $\sigma_{vo}' = 109.8\ \text{kN/m}^2$ (uniform)

$K = 1 - \sin 34^o = 0.441$

$\tan\delta = \tan (0.6 \times 34) = 0.372$

$$f_{s3}\ L_3 = K\ \sigma_{vo}' \tan\delta\, L_2 = 0.441 \times 109.8 \times 0.372 \times 3\ \text{m} = 54\ \text{kN/m}^2$$

$Q_s = (f_{s1}\ L_1 + f_{s2} L_2 + f_{s3} L_3)\ p$ $\qquad p = 4B = 4 \times 0.305 = 1.22\ \text{m}^2$

$Q_s = (392 + 16.75 + 54) \times 1.22 = 564.5\ kN$

$Q_u = 408.5 + 564.5 = 973\ \text{kN}$

$$FS = 3,\ Q_{all} = \frac{Q_u}{FS} = \frac{973}{3} = 32\ \text{kN}$$

Empirical Bearing Capacity

Pile capacity based on SPT values (Meyerhoff, 1976)

The end bearing capacity

q_b = 40 N' (D_f/B)≤ 400 N (kPa)   driven pile

q_b = (40/3)N'(D_f/B) ≤ 400 N (kPa) drilled pile

N = the corrected SPT N value near the pile base or within the range of 1 B above the tip and 2 B below the tip,

D_f = embedded length of pile, and B is the smallest dimension of the pile. Most piles have greater ratios, thus the upper limit nearly always control.

Empirical Bearing Capacity

Pile capacity based on SPT values

The friction bearing of the pile:

$f_s = 2\ N$ (kPa) large displacement piles

$f_s = N$ (kPa)   small displacement pile

N is the average SPT value along the embedded length of pile.

Note that these equations are applicable for piles embedded in cohesionless soils because the standard penetration test does not give reliable estimation of pile capacity in cohesive soil.

Example 4

An HP 310 steel pile is driven into medium dense sand at depth of 22 m. The smallest dimension of the pile cross-section is 308 mm. The corrected *N* value near the pile base is 45. Assume that friction resistance of the pile is to be neglected, calculate bearing capacity of the pile based on SPT value.

$$q_b = 40\,N\,\frac{D_f}{B} \le 400\,N \qquad q_b = 40 \times 45\left(\frac{22}{0.308}\right) = 125871 \text{ kN/m}^2$$

Limiting value $q_b = 400 \times 45 = 18000$ kN/m^2

Since $q_b > q_b$ limiting, then use q_b limiting

$Q_b = A_b\,q_b = 0.0955 \times 18000 = 1719$ kN

Friction resistance is to be neglected, then $Q_a = \dfrac{Q_b}{3} = \dfrac{1719}{3} = 573$ kN

Daya dukung tiang berdasarkan hasil sondir (Beggemann, 1965)

Untuk Tip resistance q_b berdasarkan Briaud and Milan (1991)

$$q_b = q_c \times k_c$$

dimana q_c adalah tahanan konus rata2 dari 1.5B di atas tip sampai 1.5 B dibawah tip;

sedangkan k_c untuk pondasi tiang di tanah kohesif adalah 0.6 sedangkan untuk tiang bore 0.375.

Nilai koreksi ini dimasukkan dalam FOS dimana untuk tiang pancang FOS = 3 sedang untuk tiang bore FOS = 5

Untuk friction resistance berdasarkan Nottingham & Schmertmann (1975) dimana untuk tanah kohesif

$$f_s = a \times T_f$$

dimana nilai a tergantung tipe tiang dan tipe sondir. Nilai a ini dimasukkan dalam FOS = 5.

Empirical Bearing Capacity

Pile capacity based on CPT values

Berdasarkan referensi dari Beggemenn di atas maka

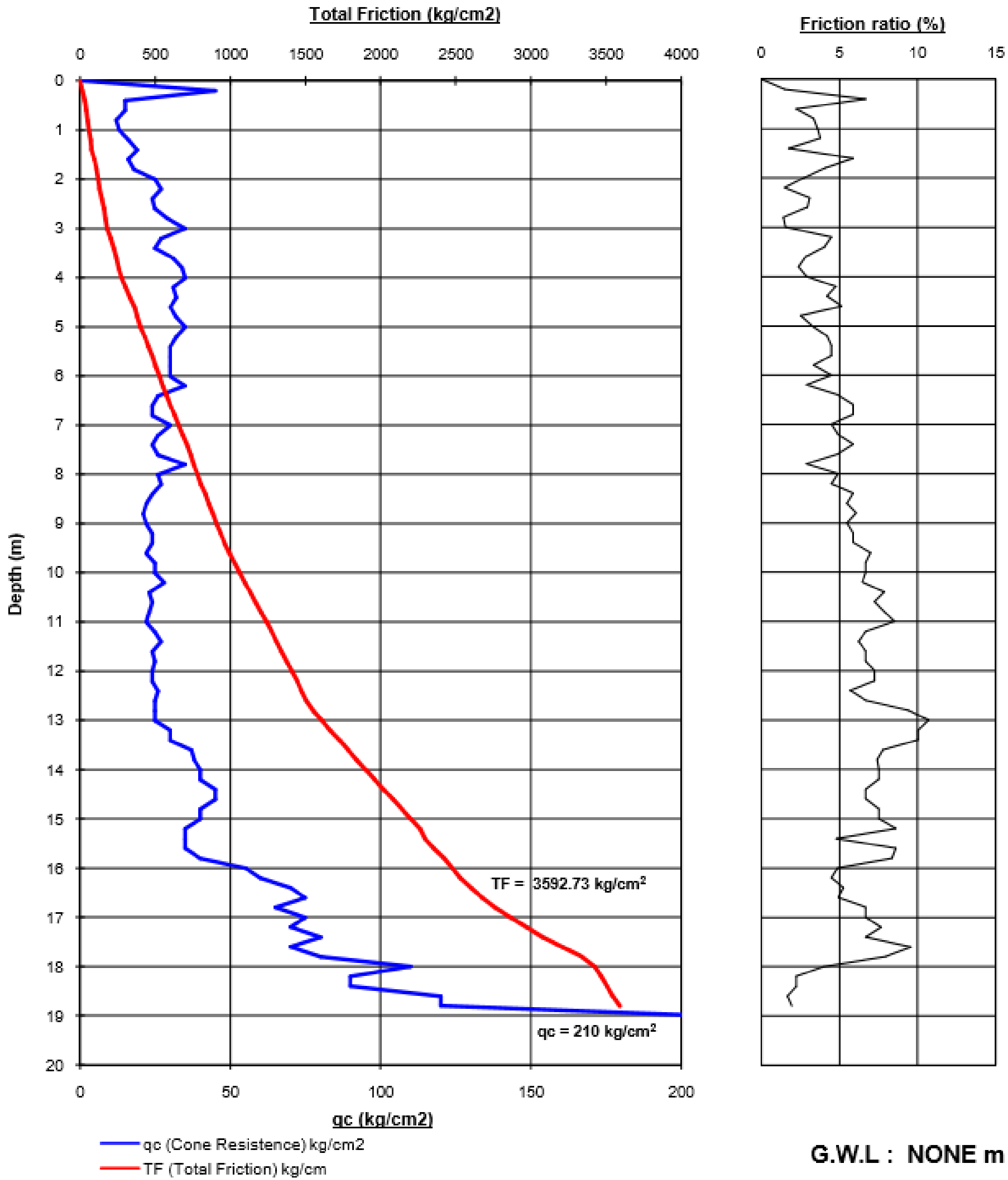
Untuk Tiang pancang

$$Q_{all} = \frac{q_c\, A_p}{3} + \frac{T_{fs}\, A_s}{5}$$

Untuk Tiang bor

$$Q_{all} = \frac{q_c\, A_p}{5}$$

Dimana Q_{all} = daya dukung yang diizinkan q_c = nilai konus ; A_p = Luas penampang tiang; T_{fs} = *JHP* = Jumlah hambatan lekat; A_s = Luas selimut atau keliling tiang


Total Friction (kg/cm2)
0
500
1000
1500
2000
2500
3000
3500
4000
Friction ratio (%)
0
5
10
15
Depth (m)
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
TF = 3592.73 kg/cm²
qc = 210 kg/cm²
0
50
100
150
200
qc (kg/cm2)
qc (Cone Resistence) kg/cm2
TF (Total Friction) kg/cm
G.W.L : NONE m

Example 5

Berdasarkan data sondir yang diberikan di atas, dan tiang pancang dengan diameter 300mm

a) Berapakan Panjang Tiang yang diperlukan?
b) Hitung kapasitas tiang pancang pada kedalaman tersebut?
c) Hitung kapasitas tiang pancang apabila hanya mencapai kedalaman 12 m.

JAWABAN:

a) Lihat kedalaman dimana q_c = 150 kg/cm^2 tercapai. Didapat : 19 m

b) Kapasitas Tiang pancang bila mencapai 19 m dan tidak ada pergeseran tiang pancang setelah mencapai kedalamn tersebut

$$Q_{all} = \frac{150 \times 70{,}7}{3} = 35.5 \text{ kN}$$

c) Pada kedalaman 12 m qc cukup kecil sehinga ada kemungkinan terjadi pergeseran yang diperlukan untuk memobilisasi friction, maka:

$$Q_{all} = \frac{25 \ \times 70{,}7}{3} + \frac{1400\ 94{,}2}{5} = 270 \text{ kN}$$

KESIMPULAN: daya dukung akibat geseran cukup significant; oleh karena itu perlu dipastikan adanya pergeseran tiang setelah pemancangan untuk memobilisasi kekuatan gesekan antara tiang dan tanah

Pile-driving formula

The ultimate resistance of driven piles may be predicted based on the amount of energy delivered to the pile by the hammer and the resulting penetration of the pile. The greater the resistance to drive the pile, the greater the capacity of the pile is to carry the load.

The net kinetic energy is equal to the work done during penetration equal to the soil resistance.

$$W\,h - E_L = R\,s$$

W = the weight of hammer, h = the height of falling hammer,

R = the soil resistance,

s = set, that is the average depth of penetration during the last blow count. http://www.youtube.com/watch?v=diryeIdK378

Pile-driving formula

Pile Driving Formulae: Engineering News Record (ENR), US Navy, Gates, Danish, Eytelwein, etc.

The most widely used dynamic formula in Malaysia is Hiley formula proposed in 1930.

$$R = eW_h\,h\,\frac{\left[\dfrac{W_h + W_p\,\eta^2}{W_h + W_p}\right]}{\left(s + \dfrac{c}{2}\right)}$$

W_h, *h, s* and *c* are defined previously**,** *e* = the efficiency factor of the hammer, used to take into account energy losses during hammer drop,

W_p = the weight of the pile, η = the coefficient of restitution which takes into account the energy loss through cushion and pile cap.

Factors of safety of the order of 2 to 3 were suggested when using this formulae.

Pile load tests

An adequate number of piles load test is required in order to verify design capacity of piles

Number of test depends on: the extend of the area, total # of piles, results of site investigation

To conduct pile test, test piles are driven at locations where soil conditions are known & relatively poor

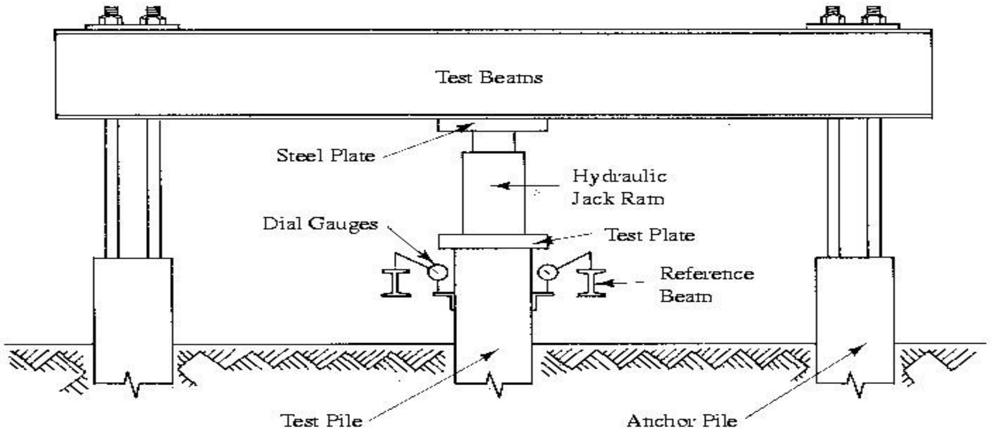
Both the method of driving for the test and actual piles should be the same

The total load on the test pile should be 200% of the proposed design load

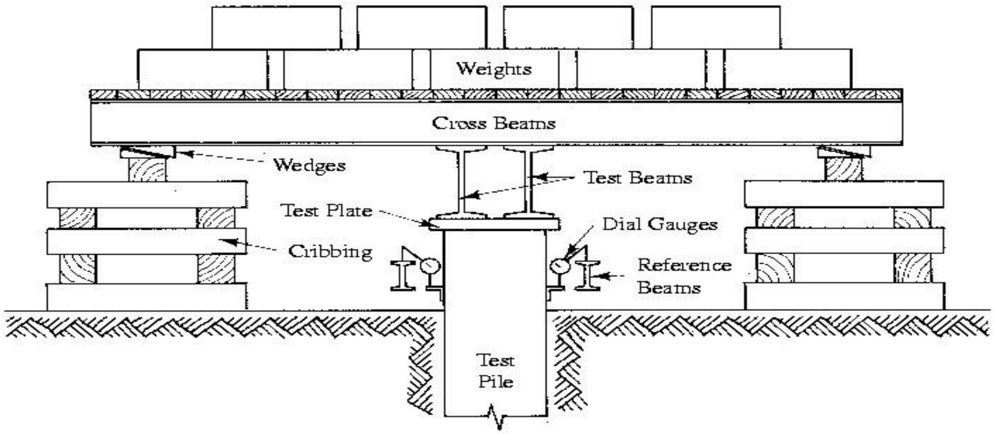
Pile load tests

Standard:
- BS 8004
- ASTM D1143-81

Type of Test
- Maintain Load Test
- Constant Rate of
  Penetration



(a)



(b)

Static Loading Test

http://www.youtube.com/watch?v=UPVuWBjAw2M&feature=related

Load-settlement graph

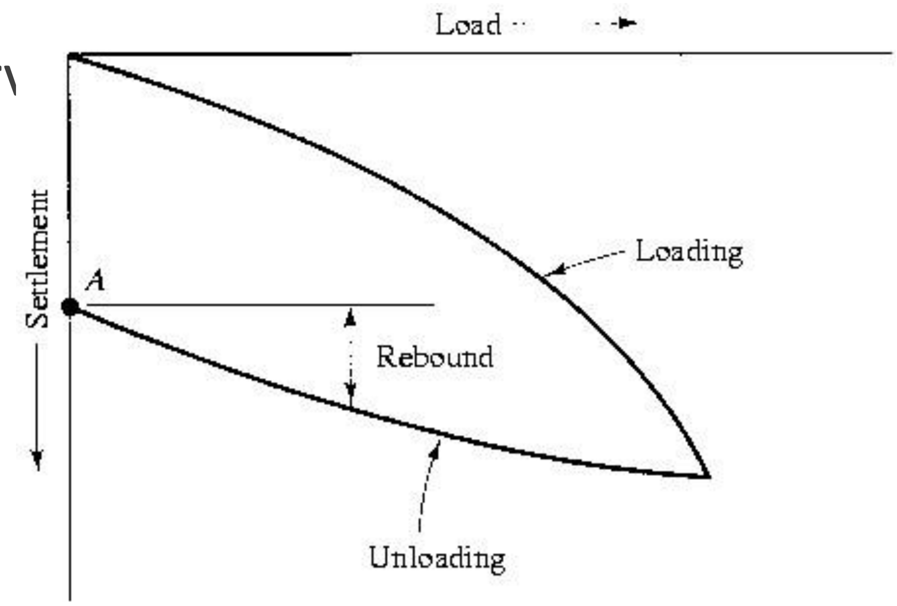
Ordinates along the loading curve gives gross settlement

Subtracting the final settlement upon unloading (point A) from ordinates along the unloading curv gives the rebound

Net settlement =

gross settlement – rebound

The allowable pile load is usually determined by the local code of practice



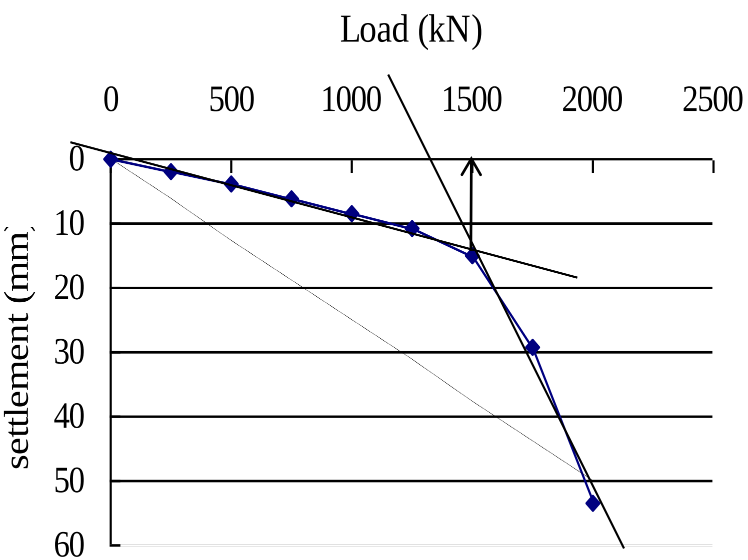
Example 6

Given load-settlement data from a full-scale load test on a 400 mm square, 17 m long concrete pile.

Determine the allowable load for this pile by the application of the Factor of Safety of 2 to the ultimate load determined by the intersection of the initial and final tangents to a curve fitted to the plotted results of pile load test.

Determine the pile capacity based on a set-up a criteria that the allowable pile load is taken as one half of load that produces a net settlement of not more than 0.025 mm/kN.

Load (kN)	Settlement (mm)
250	1.8
500	3.9
750	6.2
1000	8.3
1250	10.8
1500	15.1
1750	29.3
2000	53.3



Both tangent lines intersect at a load of 1600 kN, thus for FS = 2,

$$Q_{all} = \frac{Q_u}{2} = \frac{1600}{2} = 800 \text{ kN}$$

Draw a line corresponding to settlement of 0.025 mm/kN. The initial curve produces settlement of less than 0.025 mm/kN while the final curve produces settlement more than the stated amount. Hence, according to the criteria, the ultimate load is = 1500 kN and the allowable load on the pile is

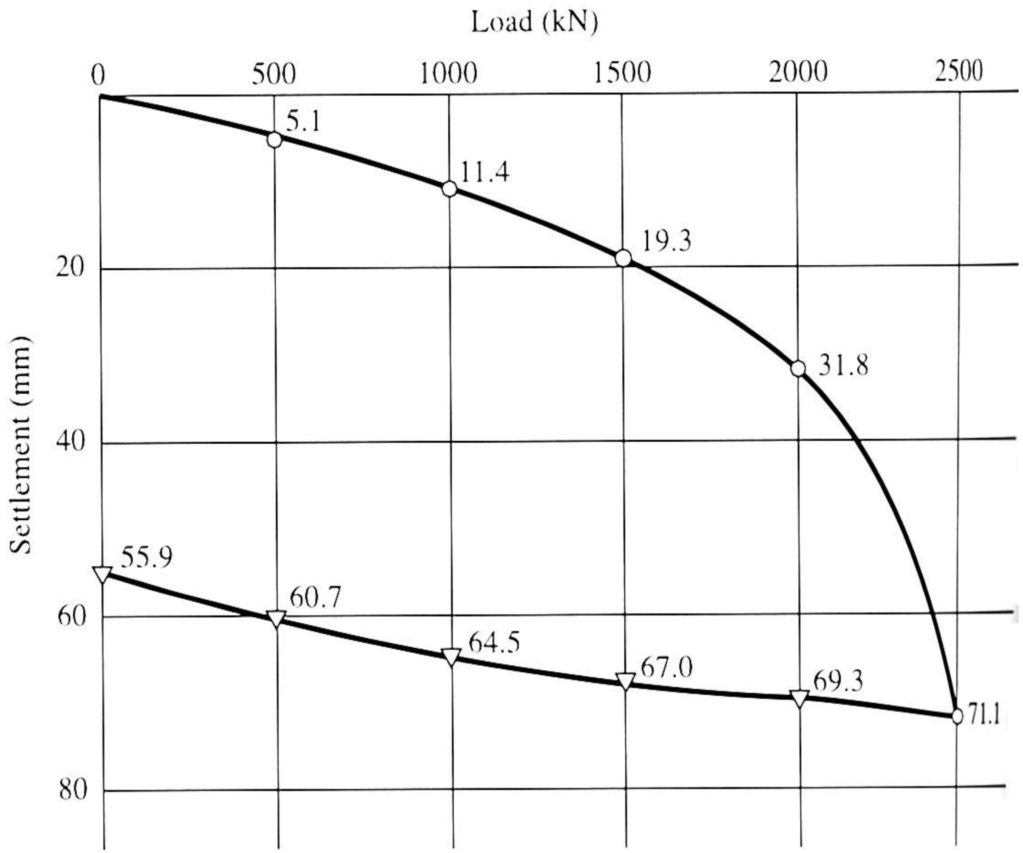
$$Q_{all} = \frac{Q_u}{2} = \frac{1500}{2} = 750 \text{ kN}$$

Example 7

A 300 mm diameter pipe pile with a length of 15 m was subjected to a pile load test

The local building code states that the allowable pile load is taken as one-half of that load that produces a **net** settlement of not more than 0.25 mm/kN but in no case more than 19mm

Determine the allowable pile load



Solution

Test load (kN)	Gross settlement (mm)	Rebound (mm)	Net settlement (mm)	Max allowable (mm)
500	5.1	4.8	0.03	< 12.5
1000	11.4	8.6	2.80	< 25
1500	19.3	11.2	8.10	< 38
2000	31.8	13.5	18.30	< 50
2500	71.1	15.2	55.90	< 62.5

Problems related to Pile

Uplift / Tension Resistance of Piles

The uplift force in piles is resisted by friction and the weight of pile itself.

$$P_u = f_s A_s + W_p$$

Additional uplift resistance may be obtained by under-ream or enlarged base of piles.

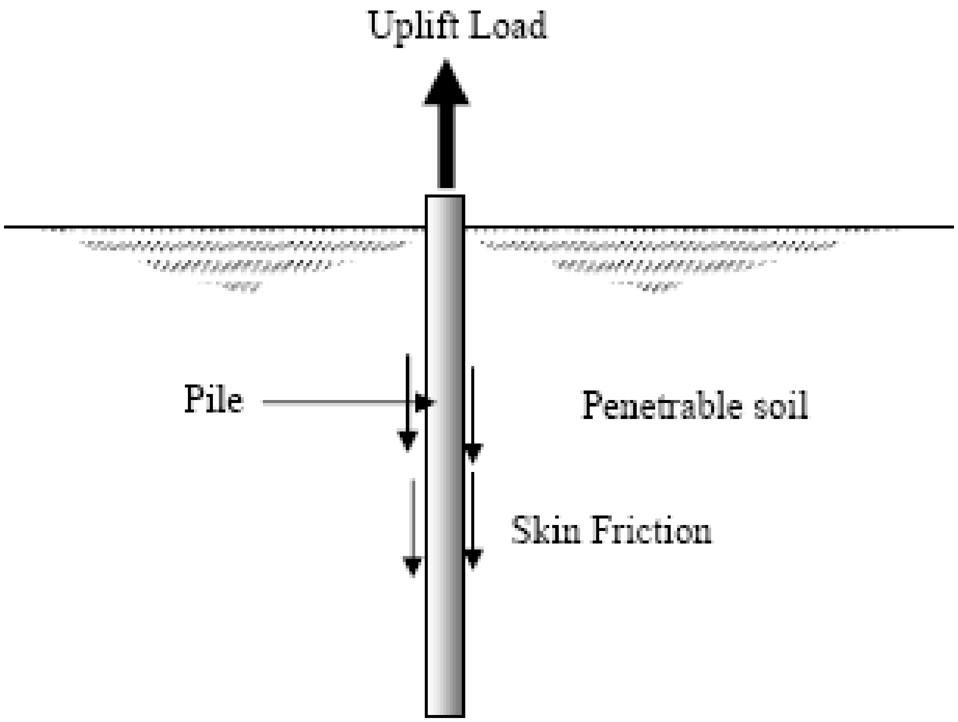
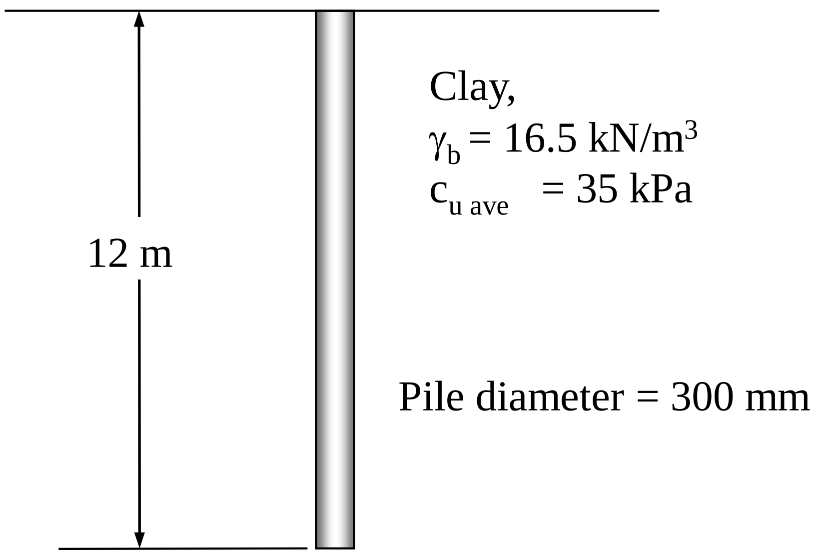


Figure 4.15 Friction piles to support an uplift load

Example 8

A 300 mm diameter concrete pile is driven at a site as shown in Figure. The embedded length of the pile is 12 m. Determine the uplift/tension resistance of the pile if the average c_u along the embedment length is 35 kPa and coefficient $\alpha = 0.9$. Assume the unit weight of pile is 24.5 kN/m^3 and use factor of safety = 4.



$P_u = f_s A_s + W_p$

$f_s = \alpha c_u = 0.9 \times 35 = 31.5$ kPa

$A_s = \pi d L = \pi \times 0.3 \times 12 = 11.31$ kPa

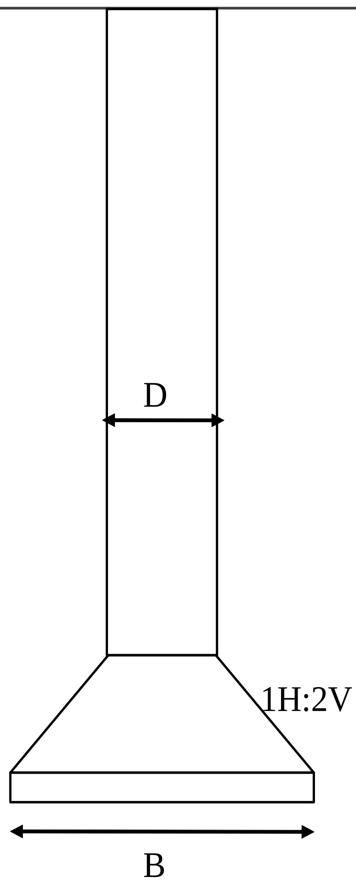
The weight of pile (diameter 0.6 m, length 13 m)

$$W_p = \frac{\pi}{4} d^2 \times L \times \gamma_c = \frac{\pi}{4}(0.6)^2 \times 12 \times 24.5 = 83 \text{ kN}$$

$P_u = f_s A_s + W_p = 31.5 \times 11.31 + 83 = 440$ kN

For a factor of safety 4, $P_{all} = P_u/4 = 110$ kN.

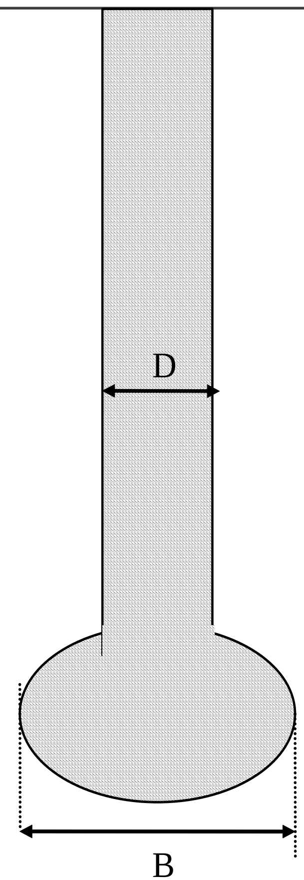
Under-reamed pile



Typical dimension
$D \geq 600$ mm
$B \leq 3D$

Non contributing part for computation of friction resistance:

Top 1.5 m,
Bottom 1D at the stem
Enlarged part

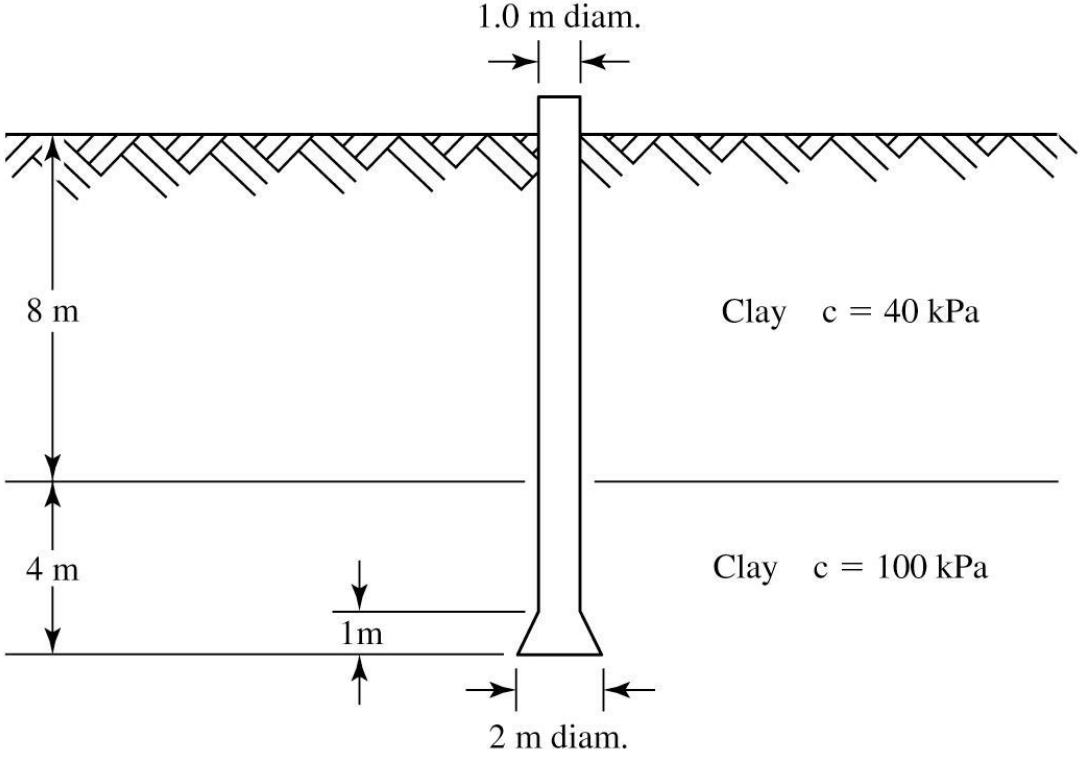


Example 9

A 1 m diameter drilled shaft is constructed in clay with a 2.00m base

The excavation is drilled dry

Determine the max allowable axial design load on the foundation



Solution

$$Q_{ultimate} = f_s \cdot A_{surface} + q_b \cdot A_{base}$$

$$f_s = \alpha_z c_{uz}$$

For $0 < \text{z} < 8, c_u/p_a = (40)/(100) = 0.4 < 1.5, \;\; \alpha_z = 0.55$

For $8 < \text{z} < 12, c_u/p_a = 1.0 < 1.5, \;\; \alpha_z = 0.55$

$$f_{shaft1} = (0.55)(40) = 22 \text{ kN/m}^2$$

$$f_{shaft2} = (0.55)(100) = 55 \text{ kN/m}^2$$

$$A_{shaft1} = (\pi \times 1)(8 - 1.5) = 20.4 \text{ m}^2$$

$$A_{shaft2} = (\pi \times 1)(4 - 1 - 1) = 6.28 \text{ m}^2$$

Since $z = 12 \text{ m} > 3 \times \text{diameter}$,

$$q_b = N_c c_u \qquad c_u = 100 > 96 \text{ kPa}, \;\; N_c = 9$$

$$q_b = (9)(100) = 900 \text{ kN/m}^2$$

$$A_{base} = (\pi)(2)^2/4 = 3.14 \text{ m}^2$$

$$Q_{ultimate} = (22)(20.4) + (55)(6.28) + (900)(3.14)$$

$$= 3620 \text{ kN}$$

$$Q_{allowable} = Q_{ultimate}/2.5 = (3620)/2.5 = 1448 \text{ kN}$$

Example 10

An under–reamed bored pile is to be installed in a stiff clay deposit. The diameter of the pile shaft is 1.05 m while the diameter of the under-reamed base is 3.00 m. The base of pile cap is at 2.5 m below ground surface while the base of the pile is at 22 m. The height of the under-reamed base is 2 m. If the adhesion factor α along the pile is 0.4, determine the allowable load on the pile to ensure

(a) an overall allowable load (Q_{all}) for a factor of safety 2, and

(b) the allowable load (Q_{all}) for a factor of safety 3 under the base while the shaft resistance is fully mobilized.

Depth (m)	c_u (kN/m^2)
2	40
4	60
6	110
8	80
10	150
12	175
14	110
16	165
18	170
20	165
22	220

The ultimate load is $Q_u = q_b A_b + f_s A_s$

At base level (22m) c_u = 220 m

$$q_b = c_u N_c^* = 220 \times 9 = 1980 \text{ kN/m}^2$$

$$A_b = \frac{\pi}{4} \times 3^2 = 7.06 \text{ m}^2$$

For calculation of skin friction, the shaft at 1.5 m below pile cap and over the length from base to 2*B* above the top of the under – ream should be disregarded. Thus the shaft friction act from depth of (2.5+1.5) m from top to (22–2–(2 × 1.05)) m from below or from 4 m to 17.9 m.
The average c_u between depth of 4 – 17.9 m is:

$$c_{u\,ave} = \frac{60+110+80+150+75+110+165+170}{8} = 130 \text{ kN/m}^2.$$

$$f_s = \alpha\ c_u = 0.4 \times 130 = 52 \text{ kN/m}^2$$

$$A_s = \pi \times 1.05 \times 13.9 = = 45.83 \text{ m}^2$$

$$Q_u = q_b A_b + f_s A_s$$
$$= 1980 \times 7.06 + 52 \times 45.83$$
$$= 13996 + 2384$$
$$Q_u = 16380 \text{ kN}$$

For an overall factor of safety 2 $\quad Q_{all} = \dfrac{Q_u}{2} = \dfrac{16380}{2} = 8190 \text{ kN}$

For a factor of safety 3 for base resistance while shaft friction is fully mobilized

$$Q_{all} = \frac{Q_\mathrm{b}}{3} + Q_s = \frac{13996}{3} + 2384 = 704 \text{ kN}$$

Note that the load calculated here is inclusive of the weight of pile

Problems related to Pile

Negative skin friction (Down drag)

Skin friction that causes down drag is known as negative skin friction

This happens when the soil adjacent to the pile settles more than the pile itself

It may be caused by consolidation or lowering of GWT

Hence, its magnitude should be determined and subtracted from the pile's load carrying ability

Problems related to Pile

Negative skin friction (Down drag)

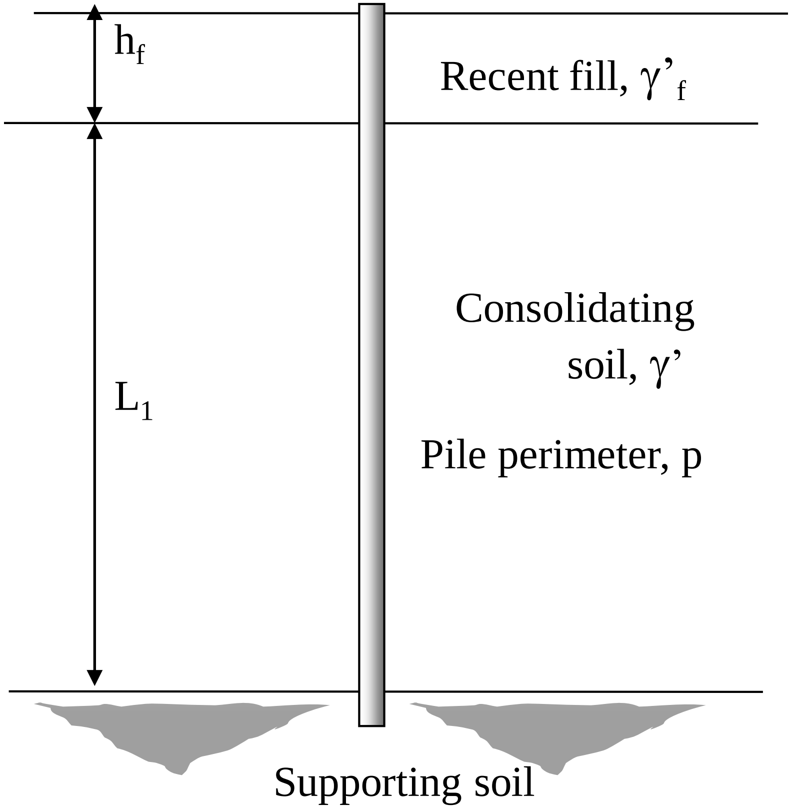
The negative skin friction (Q_n) can be calculated based on the effective overburden stress distribution along the pile:

$$Q_n = p\,K\,L_1 \tan\delta\,(\gamma_f' h_f + ½\ \gamma' L_1)$$

where K is lateral earth pressure coefficient, and δ is the wall friction angle.

If there is no fill above the clay layer:

$$Q_n = ½\ \gamma' p\,K \tan\delta\ L_1^2$$



Example 11

A pile is driven into a saturated clay layer of 12 m thick. The cross-section of the pile is circular with diameter 305 mm. The unit weight of the clay is 16 kN/m^3 and the shear strength is given by $\phi = 32^o$, while the wall friction angle is 0.6ϕ.

Determine the negative skin friction along the clay layer.

$Q_n = ½\, p\, K\, \gamma_1\, L_1^{\,2} \tan \delta$

$= ½\, \pi \times 0.305 \times (1 - \sin \phi) \times (16\text{-}9.8) \times (12)^2 \times \tan (0.6\phi)$

$Q_n = 78.89\ kN$

Example 12

A pile is driven into a layer of clay overlain by a 2 m thick sand fill.

The unit weight of the fill γ_f is 16.5 kN/m^3 and ϕ = 34^o.

The saturated unit weight of the clay layer is 17.2 kN/m^3.

The length of the pile is 20 m while the pile diameter is 0.305 m.

Ground water level is at the surface of clay layer.

Determine the negative skin friction

Length of pile = 20 m,

H_f = 2 m, then L_1 = 20 – 2 = 18 m

$Q_n = p\, K \tan \delta (\gamma_f' H_{f} + ½\ \gamma' L_1)\, L_1$

$= (\pi \times 0.305) \times (1 - \sin\phi') \times \tan (0.6 \times 34) \times (16.5\ 2 + ½(17.2 - 9.8) \times 18) \times 18$

= 2.83 (33 + 66.6)

= 281.8 kN

Pile groups & spacing of piles

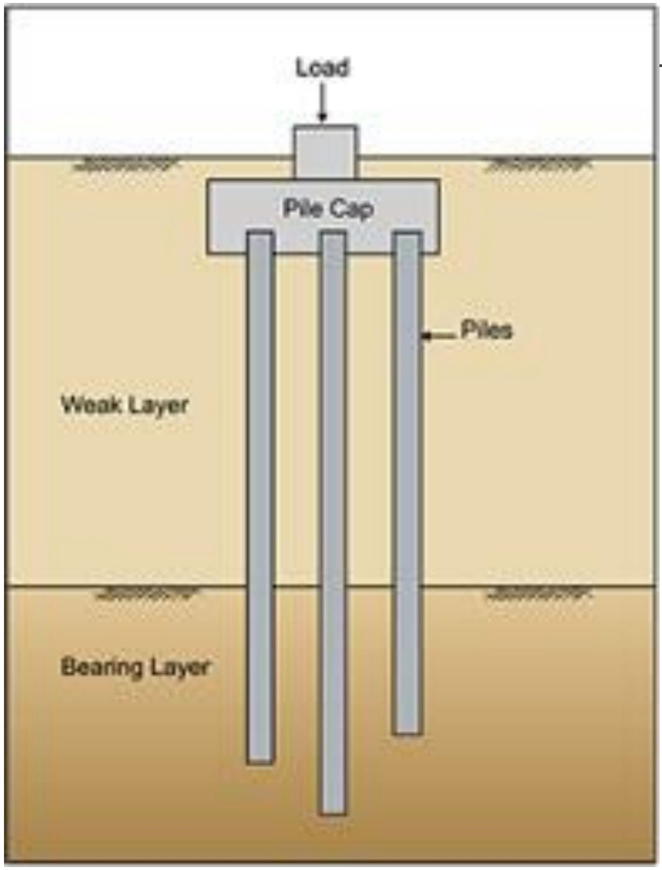
Piles are almost always arranged in groups of three or more and is commonly tied together by a pile cap

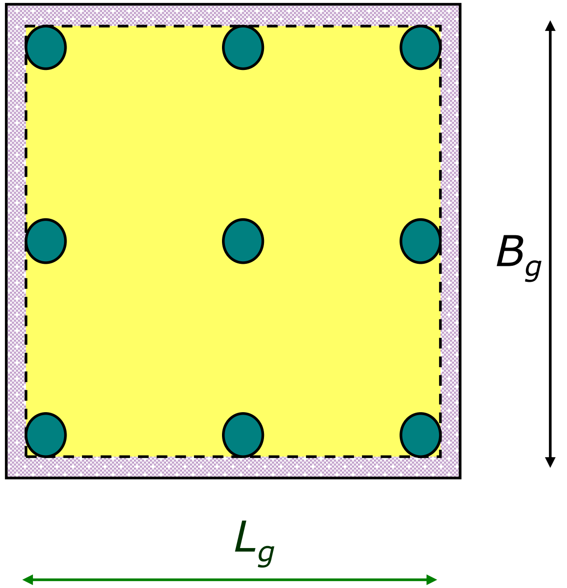
If 2 piles are driven close together, soil stresses caused by the piles tend to overlap, thus the bearing capacity of the pile group will be less than the sum of the individual capacities

If the piles are far enough that the stresses do not overlap, the bearing capacity of the pile group is not reduced significantly from the sum of the individual capacities

Hence, min allowable pile spacing is specified in building code to maximize pile group capacities and to reduce the size of the pile cap

Pile groups & spacing of piles





$$L_g = (m\text{-}1)d + D$$

$$B_g = (n\text{-}1)d + D$$

Usual spacing (c/c) = *2-8 d*; Optimum spacing = *3d*

Pile group in sand

For driven pile, s > 3d

$$Q_{ug} = N\,Q_u$$

For drilled pile, s > 3d

$$Q_{ug} = {}^{2}\!/_{3} \text{ to } {}^{3}\!/_{4}\; N\,Q_u$$

G*RF* = Group Reduction Factor

Converse-Labarre Equation

$$E_g = 1 - \theta\,\frac{(n-1)m + (m-1)n}{90mn}$$

θ = arctan *d/s* (in degrees)

n = number of piles in row

m = number of rows of piles

d = diameter of piles

s = spacing of piles, c/c

Pile group in clay

For group pile in clay, we need to do two types of analysis:

1. if s < 3d, and the piles dos not reach supporting soil (friction piles)

Use equivalent raft concept (L_g x B_g) and depth of D_f, then

$$Q_{ug} = 1.3\, c_{u\,(base)}\, N_c\, L_g\, B_g + \alpha\, c_{u(ave)}\, 2\,(L_g + B_g)\, D_f$$

N_c = bearing capacity factor from shallow foundation (for ϕ_u = 0, then, N_c = 5.14)

Pile group in clay

2: if s > 3d

Assume each pile acts as a single pile

$$Q_{ug} = GRF\ Q_{u\ 1pile}\ (mxn)$$

GRF, use Converse-Labarre equation,

The efficiency varies

from 0.7 for s = 3d to 1 fro s = 8d

References

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Thanks for your attention