Pile Foundation Design

Contents

- - - - - - - - - - - - - -

٦.١ Bearing capacity of pile groups

٦.١.١ Pile group in cohesive soil

٦.١.٢ Pile groups in non-cohesive soil

٦.١.٣ Pile groups in sand

Chapter ٧ Pile Spacing and Pile Arrangement

Chapter ٨ Pile Installation Methods

٨.١ Introduction

٨.٢ Pile driving methods (displacement piles)

٨.٢.١ Drop hammers

٨.٢.٢ Diesel hammers

٨.٢.٣ Pile driving by vibrating

٨.٣ Boring methods (non-displacement piles)

٨.٣.١ Continuous Flight Auger (CFA)

٨.٣.٢ Underreaming

٨.٣.٣ C.H.P

Chapter ٩ Load Tests on Piles

٩.١ Introduction

٩.١.١ CRP (constant rate of penetration)

٩.١.٢ MLT, the maintained increment load test

Chapter ١٠ Limit State Design

١٠.١ Geotechnical category GC ١

١٠.٢ Geotechnical category GC ٢

١٠.٣ Geotechnical category GC ٣

١٠.٣.١ Conditions classified as in Eurocode ٧

١٠.٤ The partial factors $\gamma_m, \gamma_n, \gamma_{Rd}$

LOAD DISTRIBUTION

To a great extent the design and calculation (load analysis) of pile foundations is carried out using computer software. For some special cases, calculations can be carried out using the following methods?...For a simple understanding of the method, let us assume that the following conditions are satisfied:

The pile is rigid

The pile is pinned at the top and at the bottom

Each pile receives the load only vertically (i.e. axially applied);

The force P acting on the pile is proportional to the displacement U due to compression

$$\therefore P = k.U \qquad\qquad ??????????????????\text{٣.١}$$

Since P = E.A

$$E.A = k.U \qquad\qquad ?????????????????\text{٣.٢}$$

$$k = \frac{E \cdot A}{U} \qquad\qquad ?????????????????\text{٣.٣}$$

where:

P = vertical load component

k = material constant

U = displacement

E = elastic module of pile material
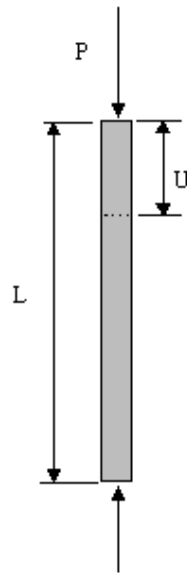
A = cross-sectional area of pile

Figure ٣-١ load on single pile

The length L should not necessarily be equal to the actual length of the pile. In a group of piles, If all piles are of the same material, have same cross-sectional area and equal length L , then the value of k is the same for all piles in the group.

Let us assume that the vertical load on the pile group results in vertical, lateral and torsion movements. Further, let us assume that for each pile in the group, these movements are small and are caused by the component of the vertical load experienced by the pile. The formulae in the forthcoming sections which are used in the calculation of pile loads, are based on these assumptions.

٣.١ Pile foundations: vertical piles only

Here the pile cap is causing the vertical compression U, whose magnitude is equal for all members of the group. If Q (the vertical force acting on the pile group) is applied at the neutral axis of the pile group, then the force on a single pile will be as follows :

$$P_V = \frac{Q}{n}$$

???????????????????٣.٤

where:

P_v = vertical component of the load on any pile from the resultant load Q

n = number of vertical piles in the group (see fig ٣.٤)

Q = total vertical load on pile group

If the same group of piles are subjected to an eccentric load Q which is causing rotation around axis z (see fig ٣.١); then for the pile *i* at distance r_{xi} from axis z:

$$U_i = r_{xi} \cdot \tan\vartheta \approx U_i = r_{xi} \cdot \vartheta \Rightarrow P_i = k \cdot r_{xi} \cdot \vartheta$$

???????????????????٣.٥

θ is a small angle ∴ tanθ ≈ θ see figure ٣.٤)

P_i = force (load on a single pile i

U_i = displacement caused by the eccentric force (load) Q

r_{xi} = distance between pile and neutral axis of pile group;

r_{xi} positive measured the same direction as e and negative when in the opposite direction.

e = distance between point of intersection of resultant of vertical and horizontal loading with underside of pile (see figure ٣.٨)

The sum of all the forces acting on the piles should be zero ⇔

$$\Leftrightarrow \sum P_i = \sum k \cdot r_{xi} \cdot \vartheta = k \cdot \vartheta \sum r_{xi} = 0 \qquad\qquad ????????????????٣.٦$$

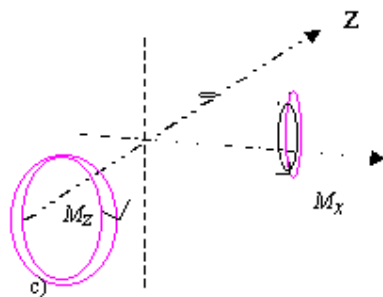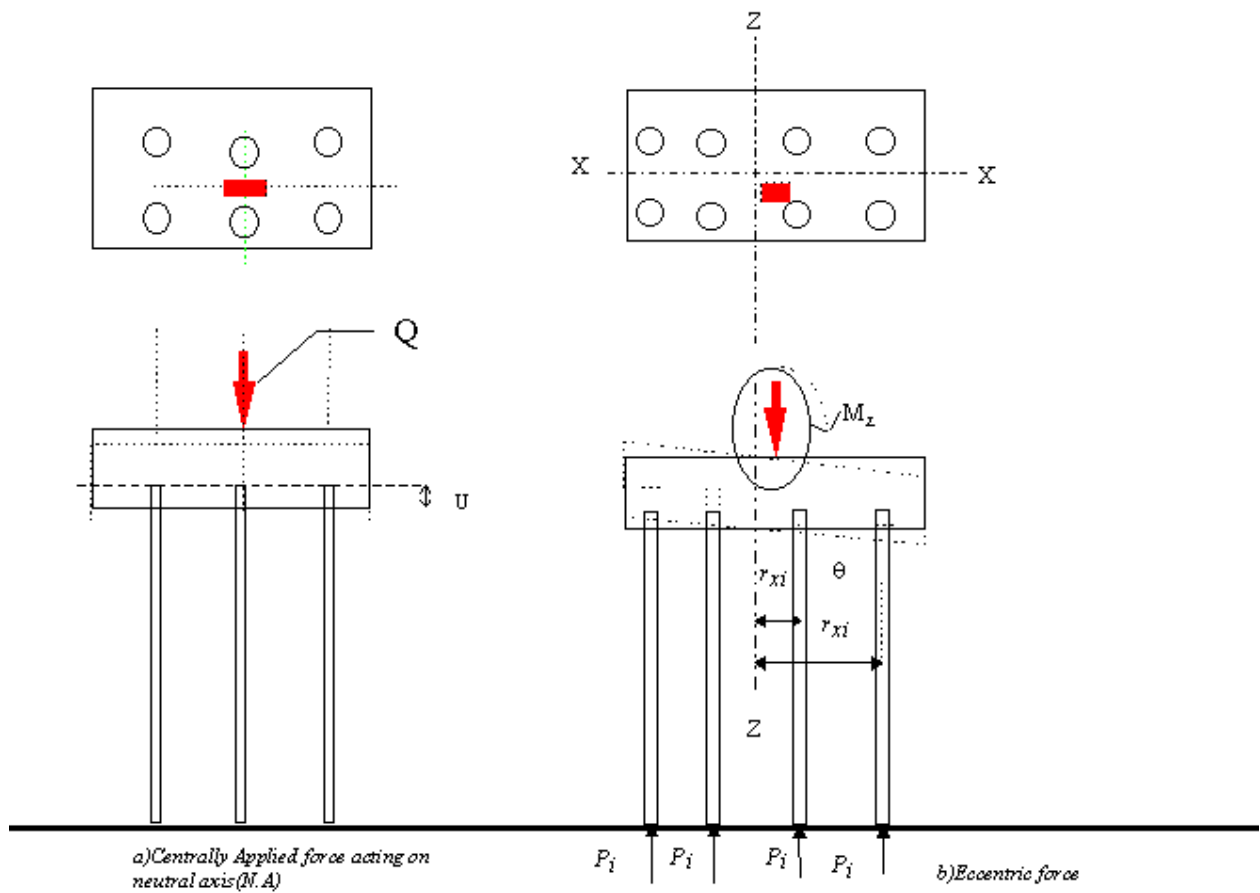a)Centrally Applied force acting on neutral axis (N.A)

b)Eccentric force

Figure ٣-٢ Moment

If we assume that the forces on the piles are causing a moment M about axis z-z then the sum of moments about axis z-z should be zero (see figure $\Upsilon.\Upsilon$ a& b)

$$M_{zi} = P_i \cdot r_{zi} = k \cdot r_{zi} \cdot \delta r_{zi} = k \cdot \delta r^2_{zi} \Rightarrow$$
$$\Rightarrow M_z = \sum M_z = k \cdot \delta \sum r^2_{zi}$$

?????????$\Upsilon.\vee$

from e.q. $\Upsilon.\Upsilon$ we see that
$$k \cdot \delta = \frac{P_i}{r_{mi}} \Rightarrow \left(\frac{P_i}{r_{zi}} \cdot \sum r^2_{zi} = \sum M_{zi} \right)$$

$$M_z = \sum M_z$$

$$\therefore \quad P_i = \frac{\left(M_{zi} \cdot r_{zi} \right)}{r^2_{zi}}$$

??????????????????????..$\Upsilon.\wedge$

applying the same principle, in the x direction we get equivalent equation. If we assume that the moment M_x and M_z generated by the force Q are acting on a group of pile, then the sum of forces acting on a single pile will be as follows:

$$P_i = \left(\frac{Q}{n} \right) \pm \left[\left(\frac{M_x \cdot r_{zi}}{\sum r^2_{zi}} \right) + \left(\frac{M_z \cdot r_{xi}}{\sum r^2_{xi}} \right) \right]$$

??????????????????$\Upsilon.\P$

if we dividing each term by the cross-sectional area of the pile, A, we can establish the working stream σ :

$$\frac{P_i}{A} = \left(\frac{Q}{A n} \right) \pm \left[\left(\frac{M_x \cdot r_{zi}}{A \cdot \sum r^2_{zi}} \right) + \left(\frac{M_z \cdot r_{xi}}{A \sum r^2_{xi}} \right) \right]$$

$$\sigma = \left(\frac{Q}{A} \right) \pm \left[\left(\frac{M_x \cdot r_{zi}}{I_z} \right) + \left(\frac{M_z \cdot r_{xi}}{I_x} \right) \right]$$

Example $\Upsilon.\Upsilon$

As shown in figure $\Upsilon.\Upsilon$, A group of Vertical piles are subjected to an eccentric force Q, magnitude of $\Upsilon\Upsilon\cdot\cdot$kN. Determine the maximum and the minimum forces on the piles. Q is located $\cdot.\Upsilon$ m from the x-axis and $\cdot.\Upsilon\circ$ m from the z-axis.
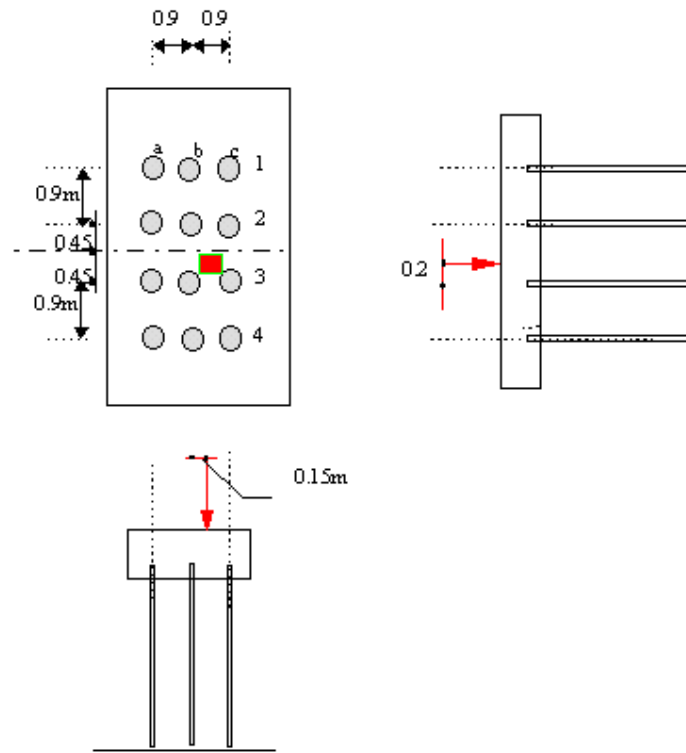
Figure ٣-٣ Worked example

Solution

١. Calculate Moment generated by the eccentric force

$M_x = ٢٦٠٠ (٠.٢) = ٥٢٠$ *KN*

$M_z = ٢٦٠٠ (٠.١٥) = ٣٩٠$ *KN*

٢.Calculate vertical load per pile: $\dfrac{Q}{n} = ٢٦٠٠/١٢ = ٢١٧$ *kN*

$$P_i = \left(\frac{Q}{n}\right) \pm \left[\left(\frac{M_x \cdot r_{zi}}{\sum r_{zi}^2}\right) + \left(\frac{M_z \cdot r_{xi}}{\sum r_{xi}^2}\right)\right]$$

PILE	DIST. r_{xi} m	r^2_{xi} m²	r_{zi} m	r^2_{zi} m²	M_X kN	M_Z kNm
a١,٤	٠.٩	٠.٨١*٢	١.٣٥	١.٨٢٣*٢	٥٢٠	٣٩٠
a٢,٣	٠.٩	٠.٨١*٢	٠.٤٥	٠.٢٠٣*٢	٥٢٠	٣٩٠
c١,٤	٠.٩	٠.٨١*٢	١.٣٥	١.٨٢٣*٢	٥٢٠	٣٩٠
c٢,٣	٠.٩	٠.٨١*٢	٠.٤٥	٠.٢٠٣*٢	٥٢٠	٣٩٠

b١,٤	٠.٩	٠	١.٣٥	١.٨٢٣*٢	٥٢٠	٣٩٠
b٢,٣	٠.٩	٠	٠.٤٥	٠.٢٠٣*٢	٥٢٠	٣٩٠
Σ		٦.٤٨		١٢.١٥٣		

PILE	Q/n k N	$M_x r_{zi}/\sum r^{\mathsf{r}}_{zi}$ (٥٢٠٠ r_{zi})/١٢.١٥٣	$M_z r_{xi}/\sum r^{\mathsf{r}}_{xi}$ (٣٩٠* r_{xi})/ ٦.٤٨	P_i k N
a١	٢١٧	٥٨	٥٤	٢١٧-٥٨-٥٤ = ١٠٥** Minimum load ١٠٥ KN, carried by pile a١
a٢		١٩	٥٤	٢١٧-١٩-٥٤ = ١٤٤
a٣		١٩	٥٤	٢١٧+١٩-٥٤ = ١٨٢
a٤		٥٨	٥٤	٢١٧+٥٨-٥٤ = ٢٢١
b١		٥٨	٠	٢١٧-٥٨-٠٠ = ١٥٧
b٢		١٩	٠	٢١٧-١٩-٠٠ = ١٥٧
b٣		١٩	٠	٢١٧+١٩-٠٠ = ٢٣٦
b٤		٥٨	٠	٢١٧+٥٨-٠٠ = ٢٧٥
c١		٥٨	٥٤	٢١٧+٥٨-٥٤ = ٢٢١
c٢		١٩	٥٤	٢١٧-١٩+٥٤ = ٢٥٢
c٣		١٩	٥٤	٢١٧+١٩+٥٤=٢٩٠
c٤		٥٨	٥٤	٢١٧+٥٨+٥٤ = ٣٢٩*** Maximum load ٣٢٩ KN, carried by pile c٤

Example ٣.٢

A pile trestle shown on figure ٣-٣ consists of four vertical piles surmounted by a ١.٢m thick pile cap. It carries a horizontal load applied to the surface of the cap of ٤٠٠kN. The only vertical load exerted on the pile group is the weight of the pile cup. Determine the loads on the piles.
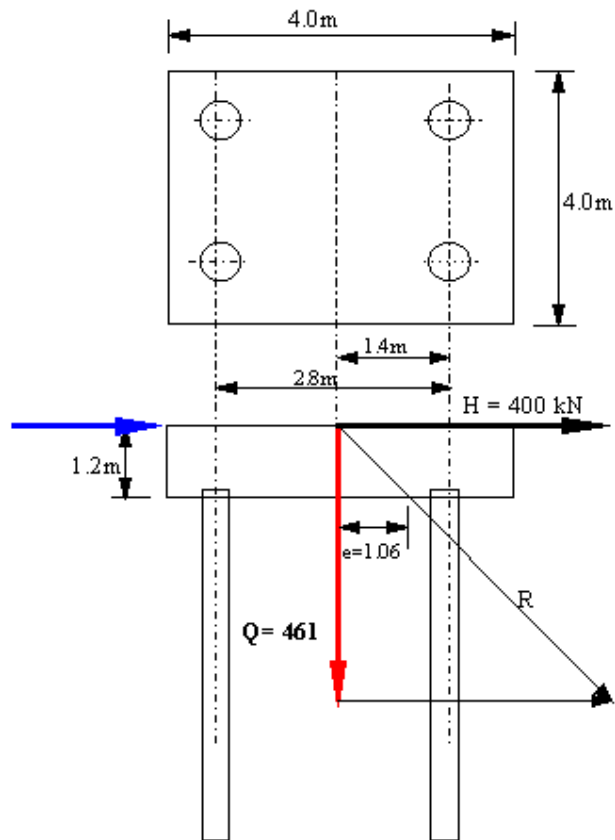
Figure ٣-٤ Example ٣.٢

Solution:

١. Determine the magnitude of the vertical force: For a pile cape ٤.٠٠٠m square, weight of pile cap is:

$٤ \times ٤ \times ١٢ \times ٤ = ٤٦١ kN$ ⇒ vertical load = ٤٦١kN

٢. Determine the location of the N.A. for the vertical piles:

$$3×0 +3×28 = 4·\overline{X} \rightarrow \overline{X} = \frac{2×28}{4} = 1.4m$$

٣. resultant of vertical load and horizontal load cuts the underside of the pile cup at a point ١.٠٦m from N.A. pile group. This can be achieved graphically. E.g. On a millimetre paper, in scale, draw the pile cup. Taking the top of the pile cup draw the vertical component downward as shown in figure ٢-٣ then taking the tip of the vertical component as reference point draw the horizontal component perpendicular to the vertical component. By joining the two components establish the resultant force R. Measure the distance from the N.A to the cutting point of R at the underside of the pile cup.

٤. Using the following formula, calculate the load on each pile:

$$P_i = \frac{Q}{n} + \frac{Q \, e \cdot \overline{X}}{\sum x^2} = \frac{461}{4} \pm \frac{461 \; 1.06 \cdot 1.4}{4 \cdot (14)^2} = 202 \, kN$$

=٢٠٢kN max and ٢٨kN minimum

٣.٢ Pile foundations: vertical and raking piles

To resist lateral forces on the pile group, it is common practice to use vertical piles combined with raking piles (see figure٣-٥) The example below illustrates how the total applied load is distributed between the piles and how the forces acting on each pile are calculated.
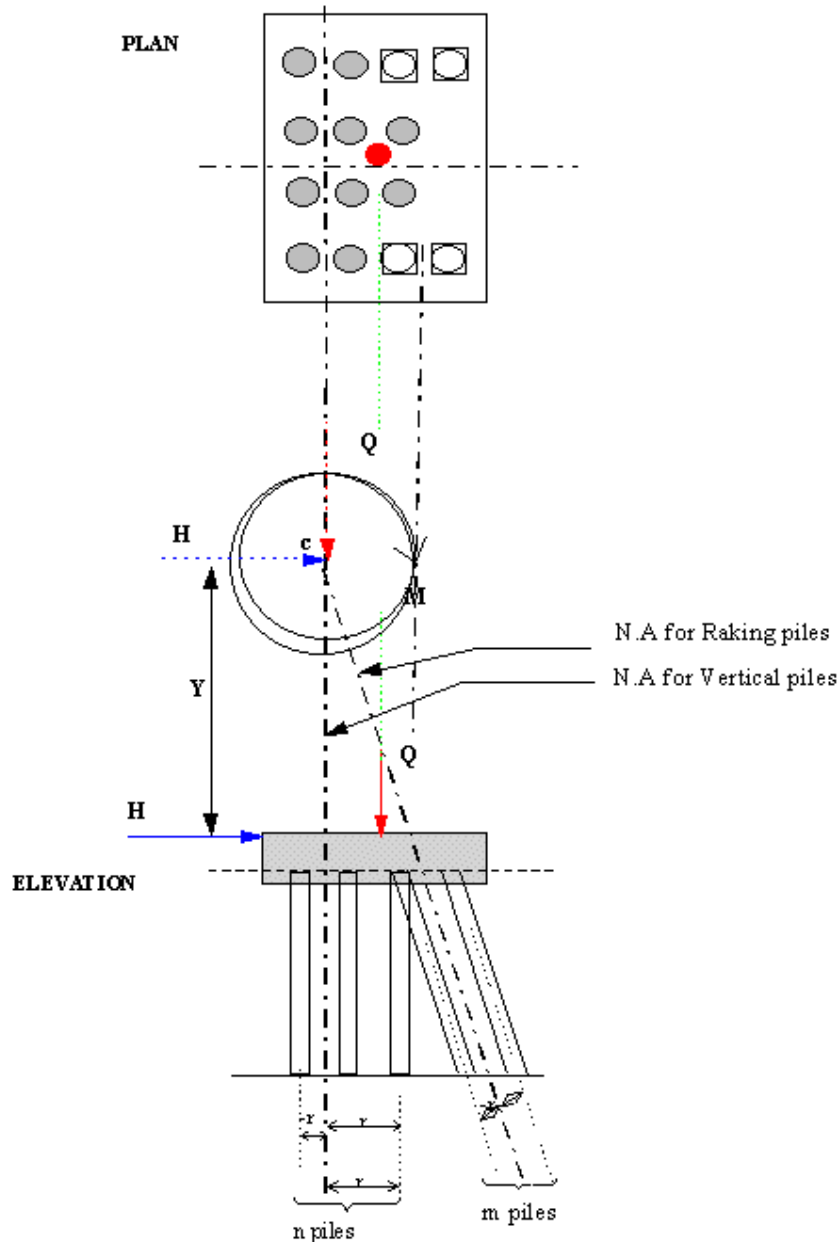
Figure ٣-٥ Load distribution for combined vertical and raking piles

To derive the formulae used in design, we first go through the following procedures:-

١. Decide the location of the N.A of the vertical and the raking piles in plan position. (see example below).
٢. Draw both N.A till they cross each other at point **c,** this is done in Elevation and move the forces **Q, H& M** to point **c** (see fig.٣.٥ elevation).
٣. Let us assume that the forces **Q** &**M** cause lateral and torsional movements at point c.
٤. Point **c** is where the moment M is zero. **Y** is the moment arm (see fig. ٣.٥)

Figure ٣.٦ shows that the resultant load R (in this case Q) is only affecting the vertical piles.
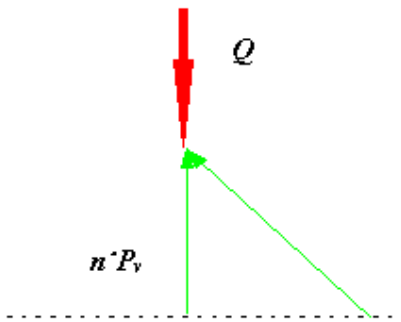
Figure ٣-٦

n = number of vertical piles

m = number of raking piles

$$P_{v=} \frac{Q}{n}$$

As shown in figure ٣.٦ the lateral force, H, is kept in equilibrium by the vertical and the raking piles.

$\Sigma H = \cdot: H - m \cdot P_r \cdot \sin\alpha = \cdot$

$\Sigma V = \cdot: m \cdot P_r \cdot \text{cosine}\alpha - n \cdot P_v = \cdot$

where:

$P_r = H/(m \sin\alpha)$

$P_v = H/(n \tan\alpha)$

Figure ٣-٧

NB : The horizontal force, H, imposes a torsional force on the vertical piles.

Sum of forces on a single pile = **Q + H + M**

*as a result of **Q**: $P_{vi} = Q/n$*

*as a result of **H**: $P_{vi} = - H/(n \tan\alpha)$*

*as a result of **H**: $P_{ri} = + H/(m \sin\alpha)$*

*as a result of moment **M**:*
$$P_i = \frac{M \cdot r_i}{\sum r_i^2}$$

r_i measured perpendicular to the N.A of both the vertical and raking piles

Example ٣.٣

Figure ٣.٧ shows a pile group of vertical and raking piles subjected to vertical load Q = ٣٠٠٠ kN and lateral load H = ٢٥٠ kN. Determine the forces acting on each pile. The raking piles lie at an angle of ٤:١.

Solution:

First we determine the location of the neutral axis, N.A, of both the vertical piles and the raking piles. From figure ٣.٧ we see that the number of vertical piles = ٨ and the number of raking piles = ٤

١. N.A for the vertical piles is determined as follows:

Here we assume ¢ through piles a١, a٢, a٣, a٤ as a reference point and start measuring in the positive direction of the X axis, where it is denoted on figure ٣.١٠ as **X-X**
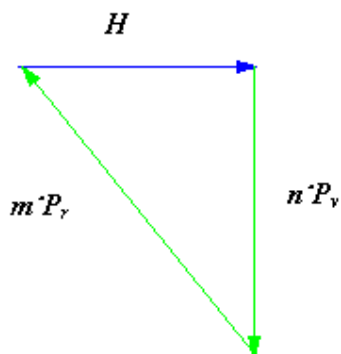
$(٤) \cdot ٠ \, m + (٢) \cdot ١ m + (٢)٢ \, m = n \cdot n \cdot e_o , \Rightarrow n \cdot e_o = \dfrac{6}{8} = ٠.٧٥ \, m$

∴ The neutral axis for the vertical piles is located at ٠.٧٥ m from the ¢ line of pile a١, a٢, a٣, a٤. $\Rightarrow (١.٠ - ٠.٧٥)m = ٠.٢٥m$ $\Rightarrow X = ٠.٢٥$ m, the distance to the vertical load Q.

where:

$n \cdot e_o = ٨ \cdot e_O$ and the numbers ٤, ٢, ٢ are number of piles in the same axis

٢. N.A for the raking piles:

Here we can assume that the ¢ for the raking piles b١ and b٤ as a reference line and calculate the location of the neutral axis for the raking piles as follows:

$(٢) \cdot ٠ \, m + (٢)١ m = (m)e_١$

where: $(m)e_١ = ٤ \cdot e_١$, ٤ is the total number of raking piles.

∴ $٤ \cdot e_١ \Rightarrow e_١ = \dfrac{2}{4} = ٠.٥ \, m \Leftrightarrow$ the location of neutral axis of raking piles at a distance of ($٠.٢٥ + ٠.٥$) m = ٠.٧٥m from e_o or from the N.A Of the vertical piles.

٣. Draw both neutral axis till they cross each other at point c. (see figure ٣.٩) and establish the lever arm distance, **Y**, so that we can calculate the moment M, about **C**.

Pile inclination ٤:١ ⇒ Y = (٠.٧٥)٤ - ٠.٦ = ٢.٤m

where ٠.٧٥ m is the location of N.A of raking piles from e_o or from the N.A Of the vertical piles.

∴ $\Sigma M = ٠$ ⇒ Q(X) - H(Y) = ٣٠٠٠(٠.٢٥) - ٢٥٠(٢.٤) = ١٥٠kNm

٤. Establish the angle α and calculate sin, cos, and tangent of the angle α

The inclination ٤:١ ⇒ α = ١٤.٠٤°

tanα = ٠.٢٥

sinα = ٠.٢٤

cos.α = ٠.٩٧

$\cos^2\alpha$ = ٠.٩٤

Figure ٣-٩ Example ٣.٣
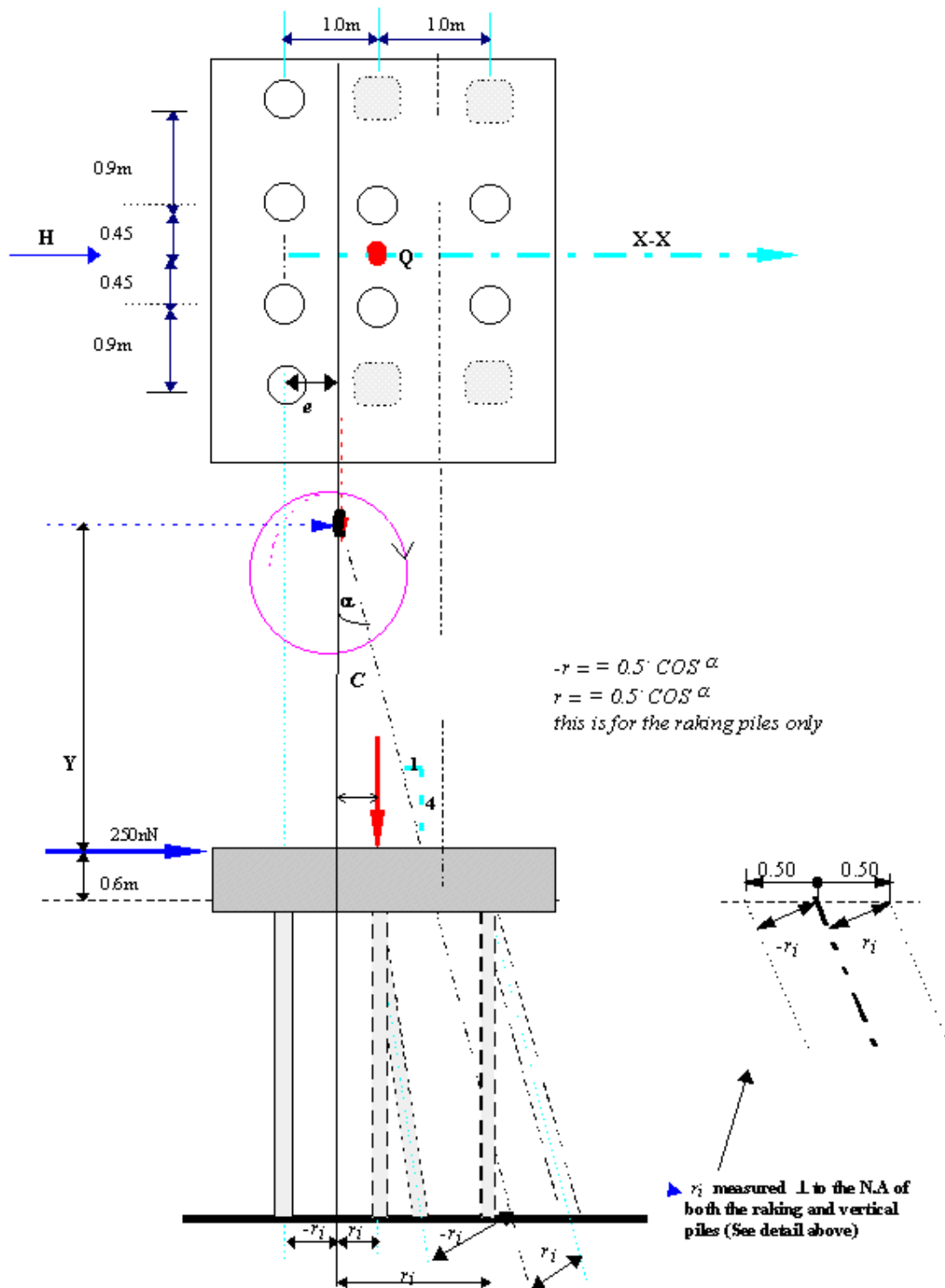
٥. Calculate the forces acting on each pile:

$$P_v = \frac{Q}{n} = \frac{3000}{8} = 375kN$$

$$P_{vi} = -\frac{H}{n \cdot \tan \alpha} = \frac{250}{4(0.25)} = -125kN$$

$$P_{ri} = +\frac{H}{m \cdot \sin \alpha} = \frac{250}{4(0.24)} = +260kN$$

$$\sum r_i^2 = 4(0.75)^2 + 2(0.25)^2 + 2(1.25)^2 + 4(0.5 \cdot 0.97)^2 = 6.44 m^2$$

$$\frac{M \cdot r_i}{\sum r_i^2} = \frac{150 \cdot r_i}{6.44} = 23.29 \cdot r_i$$

Raking piles

r_i measured perpendicular to the neutral axis →

Figure ٣-١٠

b⊠ ,b⊠ , $r_i = -٠.٥٠(٠.٩٧) = -٠.٤٨٥$ m

c⊠ ,c⊠ , $r_i = ٠.٥٠(٠.٩٧) = ٠.٤٨٥$
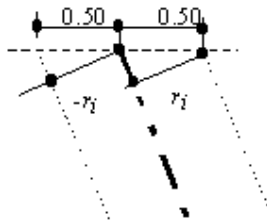
Vertical Piles

r_i measured perpendicular to the neutral axis

b⊠ ,b⊠ , $r_i = ٠.٢٥$ c٢,c٣, $r_i = ١.٢٥$m

a ⊠ , a ⊠ , a ⊠ , a ⊠ , $r_i = -٠.٧٥$m

PILE (k N)	a	b٢, b٣	c٢, c٣	b١, b٤	c١, c٤
r_i	-٠.٧٥ m	٠.٢٥ m	١.٢٥ m	٠.٤٨٥ m	٠.٤٨٥ m
Q (kN)	٣٧٥	٣٧٥	٣٧٥	٠	٠
H (kN)	-١٢٥	-١٢٥	-١٢٥	٢٦٠	٢٦٠
M (kN)	$٢٣.٢٩(-٠.٧٥) =$ -١٧.٤٧	$٢٣.٢٩(٠.٢٥) =$ ٥.٨٢	$٢٣.٢٩(١.٢٥) =$ ٢٩.١١	$٢٣.٢٩(٠.٤٨٥) =$ -١١.٣	$٢٣.٢٩(٠.٤٨٥) =$ ١١.٣٠
Σ force per pile Q + H + M	٢٣٣kN*	٢٥٦kN	٢٧٩kN*	٢٤٩ kN	٢٧kN

*As we can see the maximum load ٢٧٩kN will be carried by pile c١ and the minimum load ٢٣٣kN is carried by piles in row a١

٣.٣ Symmetrically arranged vertical and raking piles

Just as we did for the previous cases, we first decide the location of the neutral axis for both the vertical and raking piles.

Extend the two lines till they intersect each other at point c and move the forces **Q & H** to point **C**. (see fig.١١)

In the case of symmetrically arranged piles, the vertical pile I is subjected to compression stress by the vertical component P_v and the raking pile P_r is subjected to tension (see figure ٣.١١ - ١٢)

Figure ٣-١٢   Figure ٣-١٣

$P_v = k\ (U)$

$p_r = k\ (U\ cos.\alpha\ ) = P_v\ cos.\alpha$

$\Sigma V = \cdot \Rightarrow Q - n\cdot P_v - m \cdot P_r\ cos.\alpha = \cdot$
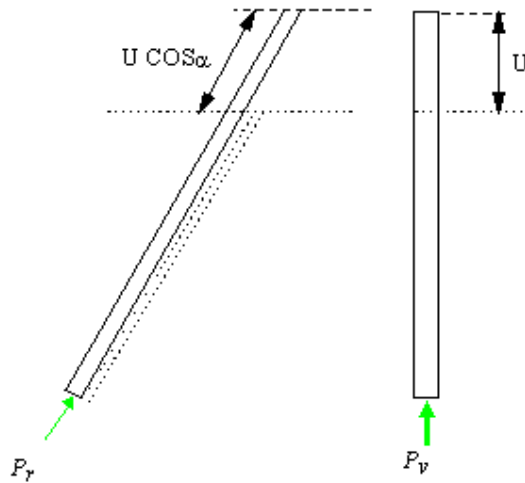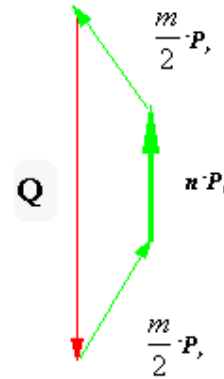
$P_r = P_v\ cos.\alpha \Rightarrow P_v = \dfrac{Q}{n + m\cdot cos.^2\ \alpha}$

The symmetrical arrangement of the raking piles keeps the lateral force, H, in equilibrium and it?s effect on the vertical piles is ignored.

With reference to figure ٣.١٣ Horizontal projection of forces yield the following formulae.

$\Sigma H = \cdot \Rightarrow$

$H - \dfrac{m}{2}\cdot P_r \cdot sin\ \alpha - \dfrac{m}{2}\cdot P_r \cdot sin\ \alpha = 0$

Figure ٣-١٤

NB the lateral force H imposes torsional stress on half of the raking piles.

Example٣.٤

Symmetrically arranged piles:

Determine the force on the piles shown in figure ٣.١٥. The inclination on the raking piles is ٥:١, the vertical load, Q =٣٦٠٠ kN the horizontal load, H =٢٠٠ kN and is located ٠.٦ m from pile cutting level.
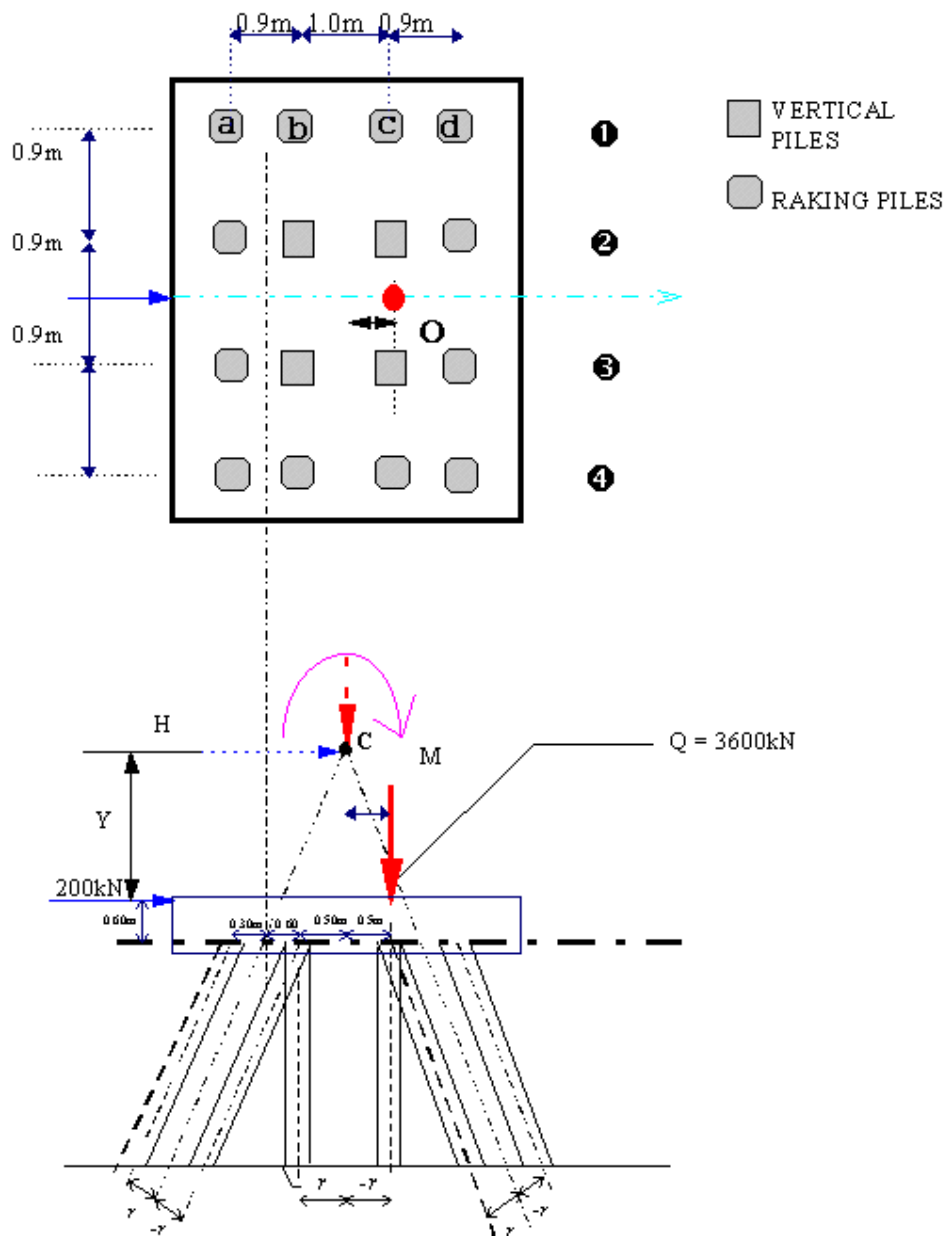
Figure ٣-١٥ Example ٣.٤

Solution

١. NA for the raking piles : $٤× (٠)+٢× (٠.٩) = ٦e \Rightarrow e_r = ٠.٣$ m

٢. NA for the vertical piles: $٢× (٠)+٢× (١) = ٤e \Rightarrow e_v = ٠.٥$ m

٣. Establish moment arm Y

Inclination $٥:١ \Rightarrow Y = ٥× (٠.٦+٠.٥) -٠.٦ = ٤.٩$ m

$\therefore M = Q (X) - H(Y) = ٣٦٠٠(٠.٢) - ٢٠٠(٤.٩) = -٢٦٠$ kNm

٤. Establish the angle α and the perpendicular distance *r,* of the piles from the neutral axis.

slope $٥:١ \Rightarrow \alpha = ١١.٣°$

$\sin\alpha = ٠.١٩٦$

$\cos \alpha = ٠.٩٨$

$\cos^{٢}\alpha = ٠.٩٦$

$\tan\alpha = ٠.٢٠$

<u>Raking piles</u>

For raking piles laying on axis a×,

$-r_i = ٠.٣× (\cos \alpha)$

$+r_i = ٠.٦× (\cos \alpha)$

$r_i^2 = (٠.٣^٢× \cos^{٢}\alpha)$

$\Sigma r_i^٢ = (٠.٣^٢ × ٠.٩٦) × ٤ = ٠.٣٤٦$ m²

For raking piles laying on axis b× and b× ,

$r_i^2 = (٠.٦^٢ × \cos^{٢}\alpha)$

$r_i^2 = (٠.٦^٢ × ٠.٩٦)× ٢_{(two piles)} = ١.٠٣٧$ m²

$\Sigma r_i^2 = (٠.٣٤٦+١.٠٣٧) × ٢ = ٢.٠٧$ m²

<u>Vertical piles</u>

$r_i = ± ٠.٥ m$

vertical piles laying on axis b and c

$\Sigma r_i^2 (٠.٥^٢× ٢ + ٠.٥^٢× ٢) = ١.٠$ m²

$\Sigma r_i^2 =$ vertical and raking piles $= ٢.٠٧ + ١.٠ = ٣.٠٧$ m²

٥. Calculate load distribution on individual piles:

$$Q \Rightarrow P_V = \frac{Q}{n + m \cdot \cos^2 \alpha} = \frac{3600}{4 + 12 \cdot 0.96} = 232 kN$$

$$P_r = P_{V} \cos\alpha = ٢٣٢ \angle ٠.٩٨ = ٢٢٧ kN$$

$$H \Rightarrow P_r = \pm \frac{H}{m \cdot \sin \alpha} = \frac{200}{12 \cdot 0.196} = 85 kN$$

$$M \Rightarrow -\frac{260 \cdot r_i}{3.07} = 85 \cdot r_i$$

PILE	a_r	b_r	b_v	c_v	c_r	d_r
Q (kN)	٢٢٧	٢٢٧	٢٣٢	٢٣٢	٢٢٧	٢٢٧
H (kN)	-٨٥	-٨٥	٠	٠	٨٥	٨٥
$M = -\frac{260 \cdot r_i}{3.07} = 85 \cdot r_i$	-٨٥ ∠ (-٠.٣∠٠.٩٨) = ٢٥	-٥٠.***	٤٣**	-٤٣**	٥٠.***	-٢٥
Σ force on P_i (kN)	١٦٧	٩١	٢٧٥	١٨٩	٣١٣	٢٨٧

$a_r = -٨٥ ∠ (-٠.٣ ∠ ٠.٩٨) = ٢٥$

$b_r = -٨٥ ∠ (٠.٦ ∠ ٠.٩٨) = -٥٠.٠$ ***

$b_v ٢, b_v ٣ = -٨٥ ∠ (-٠.٥) = ٤٢.٥$ **

$c_v ٢, c_v ٣ = -٨٥ ∠ (٠.٥) = -٤٢.٥$

where:

a_r, b_r, b_v, c_v, c_r, d_r represent raking and vertical piles on respective axis.

٣.٣.١ *Example on installation error*

Until now we have been calculating theoretical force distribution on piles. However during installation of piles slight changes in position do occur and piles may miss their designed locations. The following example compares theoretical and the actual load distribution as a result of misalignment after pile installation.
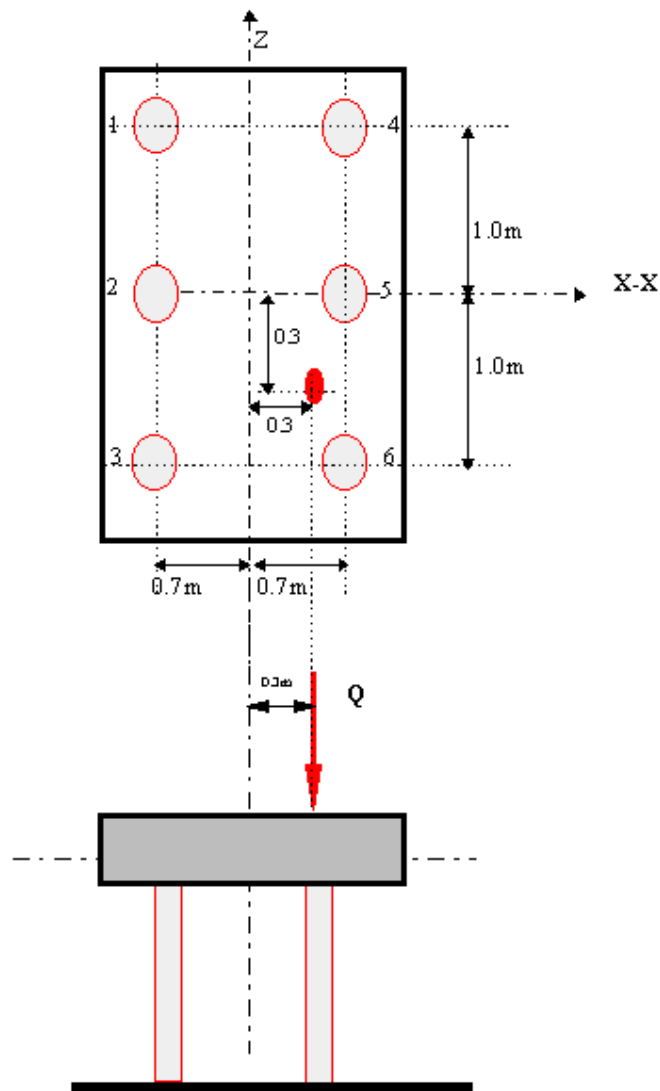
Figure ٣-١٦ Example on installation error

Before installation (theoretical position) see fig.٣-١٦

$Q = ٥٠٠$ kN $\Rightarrow M_X = ٥٠٠ × ٠.٣ = ١٥٠$

$\Rightarrow M_z = ٥٠٠ × ٠ = ٠$

$Q/n = ٥٠٠/٦ = ٨٣.٣$ kN

$P_i = Q/n \pm (M_z \cdot r_{xi})/ \sum r^٢_{xi}$

$\sum r^٢_{xi} = ٠.٧^٢ × ٣ = ٠.٧^٢ × ٣ = ٢.٩٤$ m^٢

$\therefore P_i = ٨٣.٣ - (١٥٠/٢.٩٤) × r_{xi}$

$P_{١,٢,٣} = ٨٣.٣ - (١٥٠/٢.٩٤) × ٠.٧ = ٤٧.٦$ kN

$P_{٤,٥,٦} = ٨٣.٣ + (١٥٠/٢.٩٤) × ٠.٧ = ١١٩$ kN

After installation

Displacement of piles in the X-X direction measured, left edge of pile cap as reference point (see figure ٣.١٧)
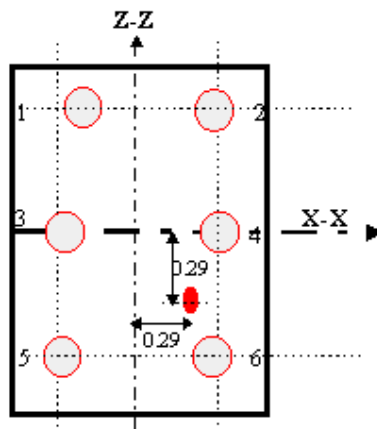
Figure ٣-١٧ piles after installation

∴ The new neutral axis (N A) for the pile group:

$(٠.٥+٠.٦+٠.٤+٢.٠+٢.١+١.٧) ⁄ ١ = ٦ ⁄ e ⟹ e = ١.٢٢$ m

The new position of Q = ٠.٢٩ m

$∴ M = ٥٠٠ ⁄ (٠.٢٩) = ١٤٥$ kNm

Measured perpendicular to the new N.A, pile distance, r_i, of each pile:

$r_{i١} = ١.٢٢-٠.٥ = ٠.٧٢$

$r_{i٢} = ١.٢٢-٢.٠ = -٠.٧٩$

$r_{i٣} = ١.٢٢-٠.٦ = ٠.٦٢$

$r_{i٤} = ١.٢٢-٢.١ = -٠.٨٨$

$r_{i٥} = ١.٢٢-٠.٤ = ٠.٨٢$

$r_{i٦} = ١.٢٢-١.٧ = -٠.٤٩$

$∴ \sum r_{xi}^٢ = ٠.٧٢^٢ + ٠.٧٩^٢ + ٠.٦٢^٢ + ٠.٨٨^٢ + ٠.٨٢^٢ + ٠.٤٩^٢ = ٣.٢$ m

$$⟹ \frac{M_x · r_{xi}}{\sum r_{xi}^٢} = \frac{145 · r_{xi}}{3.2} = 45.3 · r_{xi}$$

pile	Q/N (kN)	٤٥.٣ (r_{xi})	sum of forces on each pile $P_i = Q/n ±(M_z · r_{xi})/ \sum r_{xi}^٢$
١	٨٣.٣	٤٥.٣· (-٠.٧٢)	٥١
٢		٤٥.٣· (٠.٧٩)	٤٩

٣		٤٥.٣. (-٠.٦٢)	٥٥
٤		٤٥.٣. (٠.٨٨)	١٢٣
٥		٤٥.٣. (-٠.٨٢)	٤٧
٦		٤٥.٣. (٠.٤٩)	١٠٥

Chapter 4

LOAD ON SINGLE PILE

٤.١ Introduction

In this section, considering pile/soil interaction, we learn to calculate the bearing capacity of single piles subjected to compressive axial load. During pile design, the following factors should be taken into consideration:

- pile material compression and tension capacity

- deformation area of pile, bending moment capacity

- condition of the pile at the top and the end of the pile

- eccentricity of the load applied on the pile

- soil characteristics

- ground water level ..etc.

Nevertheless, calculation method that can satisfy all of these conditions will be complicated and difficult to carry out manually, instead two widely used simplified methods are presented. These two methods are refereed as geotechnical and dynamic methods. This section too has worked examples showing the application of the formulae used in predicting the bearing capacity of piles made of different types of materials.

Learning outcome

When students complete this section, they will be able to

- understand the theoretical back ground of the formulae used in pile design
- carry out calculation and be able to predict design bearing capacity of single piles
- appreciate results calculated by means of different formulae

٤.٢ The behaviour of piles under load

Piles are designed that calculations and prediction of carrying capacity is based on the application of ultimate axial load in the particular soil conditions at the site at relatively short time after installation.

This ultimate load capacity can be determined by either:

- the use of empirical formula to predict capacity from soil properties determined by testing, or
- load test on piles at the site

Fig.٤-١, When pile is subjected to gradually increasing compressive load in maintained load stages, initially the pile-soil system behaves in a linear-elastic manner up to point A on the settlement-load diagram and if the load is realised at any stage up to this point the pile head rebound to its original level. When the load is increase beyond point A[?] there is yielding at, or close to, the pile-soil interface and slippage occurs until point B[?] is reached, when the maximum skin friction on the pile shaft will have been mobilised. If the load is realised at this stage the pile head will rebound to point C[?] , the amount of permanent settlement being the distance OC. When the stage of full mobilisation of the base resistance is reached (point D), the pile plunges downwards with out any farther increase of load, or small increases in load producing large settlements.

[?] No end-bearing is mobilised up to this point. The whole of the load is carried by the skin friction on the pile shaft *see figure ٤-١ I)*

[?] The pile shaft is carrying its maximum skin friction and the pile toe will be carrying some load

[?] At this point there is no further increase in the load transferred in skin friction but the base load will have reached its maximum value.

I)

Rc

D

B

Reloading

unloading

A

Initial loading

C

0

Pile head vertical settlement (δ)

II)

R_a

Shaft

R_b

Base resistance

Pile head vertical settlement (δ)
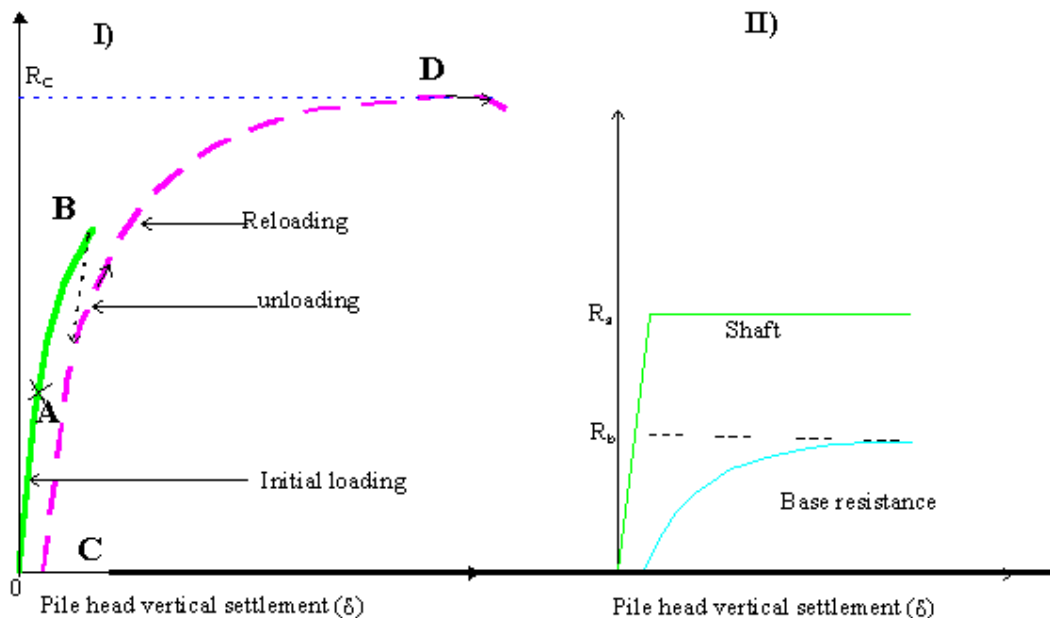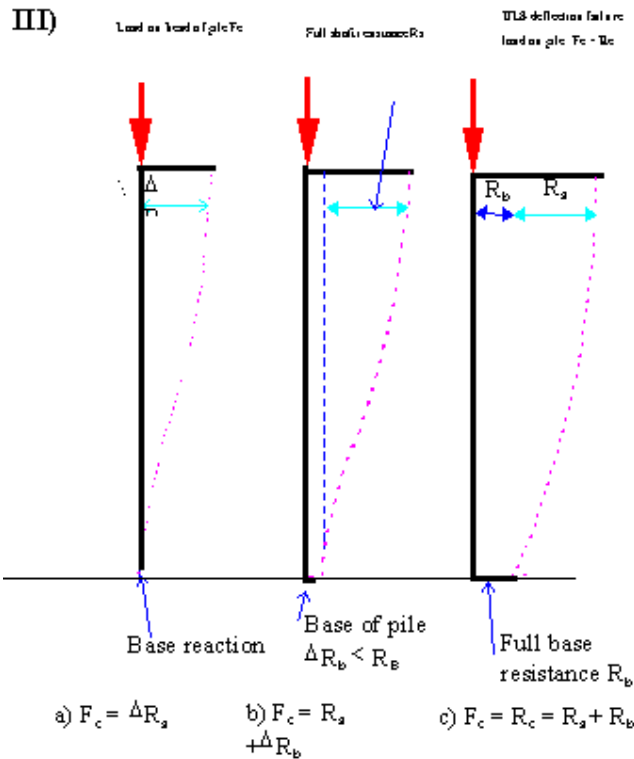
Figure -١ axial compression of pile

٤.٣ Geotechnical design methods

In order to separate their behavioural responses to applied pile load, soils are classified as either granular/noncohesive or clays/cohesive. The generic formulae used to predict soil resistance to pile load include empirical modifying factors which can be adjusted according to previous engineering experience of the influence on the accuracy of predictions of changes in soil type and other factors such as the time delay before load testing.

(Fig ٤-١II) the load settlement response is composed of two separate components, the linear elastic shaft friction R_s and non-linear base resistance R_b. The concept of the separate evaluation of shaft friction and base resistance forms the bases of "static or soil mechanics" calculation of pile carrying capacity. The basic equations to be used for this are written as:

$Q = Q_b + Q_s - W_p$ or

$R_c = R_b + R_s - W_p$

$R_t = R_s + W_p$

Where: $Q = R_c =$ the ultimate compression resistance of the pile

$Q_b = R_b =$ base resistance

$Q_s = R_s =$ shaft resistance

$W_p =$ weight of the pile

$R_t =$ tensile resistance of pile

In terms of soil mechanics theory, the ultimate skin friction on the pile shaft is related to the horizontal effective stress acting on the shaft and the effective remoulded angle of friction between the pile and the clay and the ultimate shaft resistance R_s can be evaluated by integration of the pile-soil shear strength τ_a over the surface area of the shaft:

$\tau_a = C_a + \sigma_n \cdot tan\phi_a$

Where: $\sigma_n = K_s \cdot \sigma_v$ *(refer geotechnical notes)*

$\therefore \tau_a = C_a + K_S \cdot \sigma_v \cdot tan\phi_a$

$$R_s = \int_0^L p\,\tau_a \cdot dz = \int_0^L p(C_a + K_a\,\sigma_v \tan\phi_a)dz$$
and

where: p = pile perimeter

L = pile length

ϕ *= angle of friction between pile and soil*

K_s *= coefficient of lateral pressure*

the ultimate bearing capacity, R_b, of the base is evaluated from the bearing capacity theory:

$$R_{bu} = A_b(C \cdot N_c + \sigma_{vb} \cdot N_q + 0.5 \cdot \gamma d N_r)$$

A_b *= area of pile base*

C *= undrained strength of soil at base of pile*

N_C *= bearing capacity factor*

$$\therefore R_u = R_s + R_b - (W_p - W_s) \Leftrightarrow \int_0^L p(C_a + K_s \cdot \sigma_v \cdot tan\phi_a)dz + A_b(C \cdot N_c + \sigma_{vb} \cdot N_q + 0.5\,\gamma d \quad \text{............}$$
$$\text{4.1}$$

Nevertheless, in practise, for a given pile at a given site, the undrained shear strength C_a varies considerably with many factors, including, pile type, soil type, and methods of installations.

Ideally, C_a should be determined from a pile-load test, but since this is not always possible, C_a is correlated with the undrained cohesion C_u by empirical adhesion factor α so that the general expression in e.q. (4-1) could be simplified to the following expression:

$$R = R_s + R_b - (W_p - W_s) \Leftrightarrow \int p(C_a + K_s \cdot \sigma_v \cdot tan\phi_a)dz + A_b(C \cdot N_c + \sigma_{vb} \cdot N_q + 0.5\,\gamma d N_r$$
$$\text{...}$$
$$\text{4.2}$$

$$\alpha \bar{C}_s\,A_s + C_b \cdot N_c \cdot A_b - (W_p - W_s)$$

Where: W_s = weight of soil replaced by the pile

$\bar{C}$ =average value of shear strength over the whole shaft length

٤.٣.١ The undrained load capacity (total stress approach)

For piles in clay, the undrained load capacity is generally taken to be the critical value unless the clay is highly over consolidated. If the undrained or short-term ultimate load capacity is to be computed, the soil parameters C, θ ,α , γ should be appropriate to undrained conditions and σ_v and σ_{vb} should be the total stresses. If the clay is saturated[?] , the undrained angle of friction ϕ_u is zero, and ϕ_a (angle of friction between pile and soil) may also be taken as zero. In addition, $N_q =$ ١, $N\gamma =$ ١, so that the eq in(٤-١) reduces to:

$$\int_0^L pC_a dz + A_b(C_u N_c + \sigma_{vb}) - (W_p - W_s) \qquad \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \dots \text{٤.٣}$$

Where: N_c, N_q, $N\gamma$,= bearing capacity factors and are functions of the internal angle of friction ϕ of the soil, the relative compressibility of the soil and the pile geometry.

٤.٣.٢ Drained load capacity (effective stress approach)

For piles installed in stiff, over consolidated clays, the drained load capacity is taken as design criterion. If the simplified assumption is made that the drained pile-soil adhesion C?$_a$ is zero and that the term in eq (٤-١)…involving N_c, $N\gamma$ ignoring the drained ultimate bearing capacity of the pile may be expressed as :

$$R_u - \int_0^L p(K_s \cdot \sigma'_v \cdot \tan \phi_a)dz + A_b \sigma'_{vb} \cdot N_q - (W_p - W_s) \qquad \dots \dots \dots \dots \dots \dots \dots \dots \dots \text{٤.٤}$$

Where: s ? $_v$, and s ? $_{vb}$ = effective vertical stress at depth z respective at pile base

f ? $_a$,= effective angle of friction between pile/soil and implied can be taken as f ? ,

N_q which is dependant up on the values of f ? may be taken to be the same as for piles in sand, and can be decided using table ١٠-٥ & ١٠-٦

٤.٣.٣ Pile in sand

If the pile soil adhesion C_a and term N_c are taken as zero in e.q (٤-١)… and the terms ٠.٥γ d $N\gamma$ is neglected as being small in relation to the term involving $N\gamma$, the ultimate load capacity of a single pile in sand may be expressed as follows:

$$R_u = \int_0^L F_w \cdot p(K_s \cdot \sigma'_v \cdot \tan \phi_a)dz + A_b \sigma'_{vb} \cdot N_q - (W_p - W_s) \qquad \dots \dots \dots \dots \dots \dots \dots \dots \text{٤.٥}$$

Where: s ? $_v$, and s ? $_{vb}$ = effective vertical stress at depth z respective at pile base

F_w = correction factor for tapered pile (= ١ for uniform diameter)

٤.٤ Dynamic approach

Most frequently used method of estimating the load capacity of driven piles is to use a driving formula or dynamic formula. All such formulae relate ultimate load capacity to pile set (the vertical movement per blow of the driving hammer) and assume that the driving resistance is equal to the load capacity

to the pile under static loading they are based on an idealised representation of the action of the hammer on the pile in the last stage of its embedment.

Usually, pile-driving formulae are used either to establish a safe working load✍ or to determine the driving requirements for a required working load.

The working load is usually determined by applying a suitable safety factor to the ultimate load calculated by the formula. However, the use of dynamic formula is highly criticised in some pile-design literatures. Dynamic methods do not take into account the physical characteristics of the soil. This can lead to dangerous miss-interpretation of the results of dynamic formula calculation since they represent conditions at the time of driving. They do not take in to account the soil conditions which affect the long- term carrying capacity, reconsolidation, negative skin friction and group effects.

✍ *specified load acting on the head of the pile*

Pile Foundation Design

Chapter 5

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SINGLE PILE DESIGN

٥.١ End bearing piles

If a pile is installed in a soil with low bearing capacity but resting on soil beneath with high bearing capacity, most of the load is carried by the end bearing.

In some cases where piles are driven in to the ground using hammer, pile capacity can be estimated by calculating the transfer of potential energy into dynamic energy . When the hammer is lifted and thrown down, with some energy lose while driving the pile, potential energy is transferred into dynamic energy. In the final stage of the pile's embedment,On the bases of rate of settlement, it is able to calculate the design capacity of the pile.

For standard pile driving hammers and some standard piles with load capacity $(F_{Rsp,})$, the working load for the pile can be determined using the relationship between bearing capacity of the pile, the design load capacity of the pile described by: $F_{Rsp} ? \gamma_n$ /φοντ> F_{Sd} and table ٥-١

where: F_{Sd} = design load for end baring.

The data is valid only if at the final stage, rate of settlement is ١٠ mm per ten blow. And pile length not more than ٢٠ m and geo-category ٢ . for piles with length ٢٠ - ٣٠ m respective ٣٠ - ٥٠ m the bearing capacity should be reduced by ١٠ res. ٢٥%.

☐

Table ٥-١ Baring capacity of piles installed by hammering

hammer	DROP HAMMER (released by trigger)			drop hammer (activated by rope and friction winch		
		cross-sectional area of pile			cross-sectional area of pile	
	fall height	0.0055_m^{2}	0.0073_m^{2}	fall height	0.0055_m^{2}	0.0073_m^{2}
٣ TON	٠.٣	٤٢٠ kN	٤٥٠ kN	٠.٤	٣٩٠ kN	٤٢٠ kN
	٠.٤	٤٩٠	٥٢٠	٠.٥	٤٥٠	٤٨٠
	٠.٥	٥٦٠	٥٩٠	٠.٦	٥٢٠	٥٤٠
٤ TON	٠.٣	٤٧٠	٥١٠	٠.٤	٤٤٠	٤٨٠
	٠.٤	٥٤٠	٥٩٠	٠.٥	٥١٠	٥٥٠
	٠.٥	٦١٠	٦٨٠	٠.٦	٥٥٠	٦١٠
٥ TON	٠.٣	٥٨٠	٦٤٠	٠.٤	٥٥٠	٦٠٠
	٠.٤	٦٧٠	٧٤٠	٠.٥	٦١٠	٦٦٠
	٠.٥	٧٦٠	٨٤٠	٠.٦	٦٧٠	٧٣٠

Example ٥.١

A concrete pile with length ٢٦ m and cross-sectional area (٢٣٥) /φοντ> (٢٣٥) is subjected to a vertical loading of ٣٩٠ kN (ultimate) load. Determine appropriate condition to halt hammering. Type of hammer Drop hammer activated by rope and friction winch. Class ٢, GC ٢, pile length ٢٠ m

☐

solution:

$\Phi_{P\sigma\pi}? \gamma_v. \Phi_{\sigma\delta}$

$\gamma_v = 1.1$ (ταβλε 10–3)

vertical load ٣٩٠ kN /φοντ> $\Phi_{P\sigma\pi}?\square$ $(1.1)390/0.9**=477κN$

Pile cross-sectional area /φοντ> $0.235^{2} = 0.0055$ m² /φοντ>

type of hammer: Drop hammer activated by rope and friction winch /φοντ>

☐

***For piles ٢٠m - ٣٠m length, the bearing capacity should be reduced by ١٠%

∴ Table value (table ٥-١): Hammer weight = ٤ ton /φοντ> fall height ٠.٤٥m (interpolation)

Hammer weight = ٣ ton /φοντ> fall height ٠.٥٤ m

٤ ton hammer with fall height ٠.٤٥m is an appropriate choice.

☐

٥.٢ Friction piles

Load on piles that are driven into friction material, for the most part the weight is carried by friction between the soil and the pile shaft. However considerable additional support is obtained form the bottom part.

In designing piles driven into friction material, the following formulas can be used

$$F_{Rd} = \left(\frac{1}{\gamma_{RD}}\right) \cdot \sum_i q_i \cdot c_{u\bar{a}} \cdot A_{mi} \qquad \dots\dots\dots\dots\dots\dots\dots\dots ٥.١$$

where: q_{ci} = consolidation resistance

*α can be decided using table ٤-١٠

A_b = end cross-sectional area of the pile

A_{mi} = shaft area of the pile in contact with the soil.

γ_{RD} should be ؟ ١.٥ for piles in friction material

q_{cs} = end resistance at the bottom of the pile within ٤
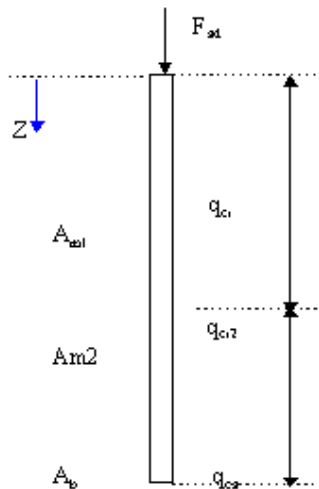 /font> πιλε διαμετερ φρομ τηε ενδ οφ τηε πιλε

Figure ٥-١ Friction Pile

Example ٥.٢

Pile length ٢٢ m, steel pile, friction pile, external diameter ١٠٠ mm, GC٢,

Determine the ultimate bearing capacity of the pile

solution:

q_c $Z?$ m (depth measured from ground level to bottom of pile)	MPa
٠m – ٥m	٥.٤
٥ – ١١	٦.٤
١١ – ١٨	٧.٠
١٨ – ٢٢	٧.٥
٢٢ m	٨.٠

The values are slightly scattered then the usual while the rest of the condition is favourable.

γ_{Rd} = ١.٥ (the lowest value)

γ_n = ١.١

At the base where condition is unfavourable we get :

$$\gamma_m = 1.5 + \frac{0.4}{4} = 1.7 \Rightarrow 1.7(0.8) = 1.36$$

α_s = ٠.٥

α_m = ٠.٠٠٢٥

$$F_{Rd} = (\frac{1}{1.5})\left[\left(\frac{0.5 \cdot 8 \cdot \pi d^2}{(1.1)(1.7)4}\right) + \sum\left(\frac{(0.0025(5 \cdot 5.4 + 6 \cdot 6.4 + 7 \cdot 7.0 + 4 \cdot 7.5) \pi 0.1)}{(1.1 \cdot 1.36)}\right)\right] = 62 kN$$

Design bearing capacity of the pile is ٦٢ KN.

٥.٣ Cohesion piles

Piles installed in clay: The load is carried by cohesion between the soil and the pile shaft.
Bearing capacity of the pile can be calculated using the following formula for pile installed in clay.

$$F_{Rd} = (\frac{1}{\gamma_{RD}}) \cdot \sum_i \alpha_i \cdot c_{udi} \cdot A_{mi} \qquad \dots\dots\dots\dots\dots\dots\dots\dots\dots \; ٥.٢$$

Where:

a_i = adhesion factor for earth layer

c_{udci} = undrained shear strength of clay.

A_{mi} = area of pile shaft in contact with the soil.

The adhesion factor α is taken as ٠ for the firs three meters where it is expected hole room and fill material or week strata. For piles with constant cross-sectional area the value of α can be taken as ١.٠ and for piles with uniform cross-sectional growth the value of α can be taken as ١.٢ .
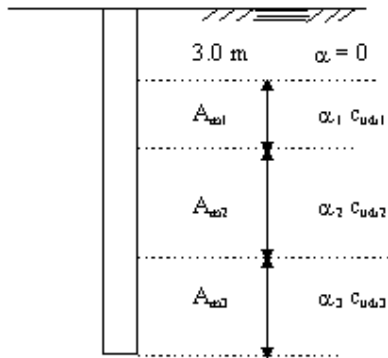
Figure ٥-٢ Cohesion Pile

Example ٥.٣

١٨ m wood pile is installed small end down in clay. Pile diameter is ١٢٥ mm at the end and ١٠ mm/m increase in diameter. The undrained shear strength of the soil, measured from the pile cut-off level is: ٠-٦ m = ١٢ kP ٦-١٢ m = ١٦ kPa ١٢-١٨ m = ١٩ kPa. Determine the ultimate load capacity of the pile. Pile cut-off level is ١.٥m from the ground level. γ_{Rd} = ١.٧

Figure ٥-٣ Example ٥-٣

solution

decide the values for α

α = ٠ for the first ٣.٠ meters

α = ١.٢ for the rest of the soil layer

divided the pile into ٣ parts (each ٦.٠ m in this case)

calculate Average diameter at the middle of each section:

Average diameter : **B**ottom (section) = ٠.١٢٥+٣.٠ /φovτ> (٠.٠١) = ٠.١٥

Middle (section) = ٠.١٥٥+٦ /φovτ> ٠.٠١ = ٠.٢١

Top (section) = ٠.٢١٥+(٣+٢.٢٥) /φovτ> (٠.٠١) = ٠.٢٦٨

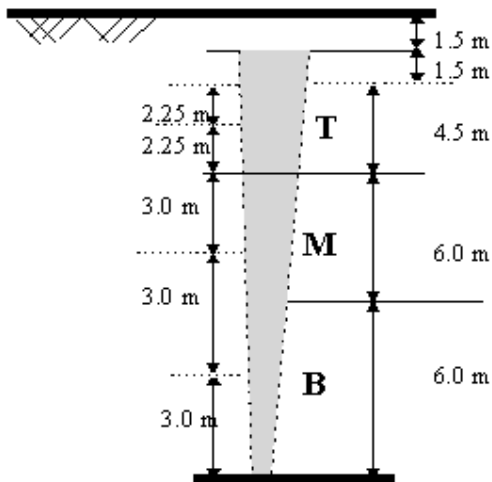$$F_{Rd} = \left(\frac{1}{\gamma_{RD}}\right) \cdot \sum_i a_i \cdot c_{ud} \cdot A_{sd} = \left(\frac{1}{17}\right) \cdot \sum_i 12 \cdot 12 \cdot 45(0.268) \cdot \pi + 16 \cdot 6(0.215) \cdot \pi + 19 \cdot 6(0.155) \cdot \pi = 117 kN$$

∴ Ultimate bearing capacity of the pile is ١١٧kN

☐

٥.٤ **Steel piles**

Because of the relative strength of steel, steel piles withstand driving pressure well and are usually very reliable end bearing members, although they are found in frequent use as friction piles as well. The comment type of steel piles have rolled **H**, **X** or circular cross-section(pipe piles). Pipe piles are normally, not necessarily filled with concrete after driving. Prior to driving the bottom end of the pipe pile usually is capped with a flat or a cone-shaped point welded to the pipe.

Strength, relative ease of splicing and sometimes economy are some of the advantages cited in the selection of steel piles.

The highest draw back of steel piles is corrosion. Corrosive agents such as salt, acid, moisture and oxygen are common enemies of steel. Because of the corrosive effect salt water has on steel, steel piles have restricted use for marine installations. If steel pile is supported by soil with shear strength greater than ٧kPa in its entire length then the design bearing capacity of the pile can be calculated using the following formulas. Use both of them and select the lowest value of the two:

☐

$$F_{Rd} = \frac{(0.44)\,\mu_m}{\gamma_n}\,(f_{yc} \cdot A) \qquad\qquad \dots\dots\dots\dots\dots\dots\dots\dots\dots \; ٥.٣$$

$$F_{Rd} - \frac{4 \cdot \mu_m}{\gamma_n}\sqrt{E_{sc} \cdot I \cdot C_{US}} \qquad\qquad \dots\dots\dots\dots\dots\dots\dots\dots\dots \; ٥.٤$$

Where:μ_m = correction factor

E_{SC} = elasticity module of steel

I = fibre moment

f_{yc} characteristic strength of steel

A = pile cross-sectional area

C_{uc} = characteristic undrained shear strength of the soil.

Example ٥.٤

Determine the design bearing capacity of a Steel pile of external diameter ١٠٠ mm, thickness of ١٠ mm. Treated against corrosion. pile. Consider failure in the pile material. C_c of the soil is ١٨ kPa, favourable condition. S٢

Steel BS ٢١٧٢

solution :

$\gamma_n = ١.١$

$\mu_m = ٠.٩$

$E_{sc} = ٢١٠$ Gpa

for BS ٢١٧٢ $f_{yc} = ٣٢٠$ MPa

$$F_{Rd} = \frac{(0.44)\,\mu_m}{\gamma_n}(f_{yc} \cdot A) = \frac{(0.44)0.9}{(1.1)4}(320 \cdot \pi(0.1^2 - 0.08^2)) = 0.326\,MN$$

$$F_{Rd} = \frac{4 \cdot \mu_m}{\gamma_n}\sqrt{E_{sc} \cdot I \cdot C_{us}} = \frac{4 \cdot 0.9}{1.1}\sqrt{210 \cdot 10^3 \cdot (\frac{\pi}{64})(0.1^4 - 0.08^4)(0.018)} = 0.343\,MN$$

The first formula gives us lower value, therefore, the design bearing capacity of the pile is ٠.٣ MN

If we consider corrosion of ١mm/year /φovτ>

$$F_{Rd} = \frac{(0.44)\,\mu_m}{\gamma_n}(f_{yc} \cdot A) = \frac{(0.44)0.9}{(1.1)4}(320 \cdot \pi((0.1^2 - 0.001) - 0.08^2)) = 0.306\,MN$$

٥.٥ Concrete piles

Relatively, in comparable circumstances, concrete piles have much more resistance against corrosive elements that can rust steel piles or to the effects that causes decay of wood piles, furthermore concrete is available in most parts of the world than steel.

Concrete piles may be pre-cast or cast-in place. They may be are reinforced, pre-stressed or plain.

٥.٥.١ Pre-cast concrete piles

These are piles which are formed, cast to specified lengths and shapes and cured at pre casting stations before driven in to the ground. Depending up on project type and specification, their shape and length are regulated at the prefab site. Usually they came in square, octagonal or circular cross-section. The diameter and the length of the piles are mostly governed by handling stresses. In most cases they are limited to less than ٢٥ m in length and ٠.٥ m in diameter. Some times it is required to cut off and splice to adjust for different length. Where part of pile is above ground level, the pile may serve as column.

If a concrete pile is supported by soil with undrained shear strength greater than ٧ MPa in its entire length, the following formula can be used in determining the bearing capacity of the pile :

☐

$$F_{Rd} = \mu_m \cdot N_u \qquad\qquad \dots\dots\dots\dots\dots\dots\dots\dots ٥.٥$$

$$F_{Rd} = 2.4(\mu_m) \cdot \frac{\sqrt{E_{sc} \cdot I_c \cdot C_{uc}}}{\chi_n} \qquad\qquad \dots\dots\dots\dots\dots\dots\dots\dots ٥.٦$$

Where: N_u = bearing capacity of the pile, designed as concrete column

E_{sc} = characteristic elasticity module of concrete

I_c = fibre moment of the concrete cross-section ignoring the reinforcement

C_{uc} = characteristic undrained shear strength of the soil in the loose part of the soil within a layer of ٤.٠ m

Example ٥.٥

Concrete pile (٠.٢٣٥) /φοντ> (٠.٢٣٥) cross-section installed in clay with characteristic undrained shear strength of ١٢ kPa. In favourable condition. C٥٠. Determine design load of the pile. Consider failure in the material.

Solution:

$$F_{Rd} = \mu_m \cdot N_u$$

$$F_{Rd} = 2.4(\mu_m) \cdot \frac{\sqrt{E_{sc} \cdot I_c \cdot C_{uc}}}{\chi_n}$$
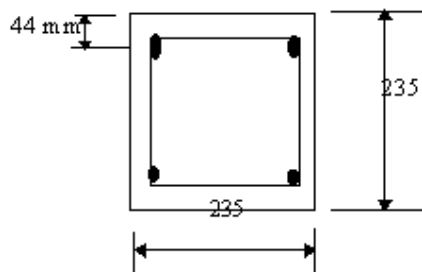
$$N_u = \frac{k_c A_c f_{cc}}{1 + k_p \cdot \omega^p}$$

$\varphi_{ef} = ١.٣$

$l_c/h = ٢٠$

$k_c = ٠.٦$, $k_\varphi = ٠.٢٤$, $k_s = ٠.٦٢$

$f_{cc} = ٣٥.٥ /(١.٥ \ /φovτ> ١.١) = ٢١.٥$ MPa

$f_{st} = ٤١٠/(١.١٥ \ /φovτ> ١.١) = ٣٢٤$ MPa

$$A_{s2} = A_s \frac{1-(2 \cdot t)/h}{0.7} , if \frac{t}{h} \geq 0.15, \frac{t}{h} = \frac{44}{235} = 0.187$$

$$A_{s2} = 804 \cdot 10^{-6} \frac{(1-(2 \cdot 0.187)}{0.7} = 719 \cdot 10^{-6} m^2$$

Effective reinforced area:

$$N_u = 0.69 \cdot (0.235)^2 \frac{21.5}{1+0.24 \cdot (1.3)} + (0.62) \cdot 719 \cdot 10^{-5} \cdot 324 = 0.769 = MN$$

☐

$F_{Rd} = \mu \ m \ _{/φovτ>} N_U$

$\mu \ m = ٠.٩ \ /φovτ> F_{Rd} = (٠.٩) \cdot ٠.٧٦٩ = ٠.٦٩٢$ MN

☐

Failure checking using the second formula:

$E_{cc} = ٣٤$ GPa

$$F_{Rd} = \frac{2.4 \cdot A_n \cdot}{\chi_n} \sqrt{Esc \cdot Ccu \cdot Ic} = \frac{2٤ \times 0.9}{1.1} \times \sqrt{34 \cdot 10^3 \cdot C.012 \cdot 0.234^4 /12} = J63 \ MN$$

The lowest value is ٠.٦٣٢ MN $_{/φovτ>}$ Design capacity $= ٠.٦٣$ M

٥.٦ Timber piles (wood piles)

Timber piles are frequently used as cohesion piles and for pilling under embankments. Essentially timber piles are made from tree trunks with the branches and bark removed. Normally wood piles are installed by driving. Typically the pile has a natural taper with top cross-section of twice or more than that of the bottom.

To avoid splitting in the wood, wood piles are sometimes driven with steel bands tied at the top or at the bottom end.

For wood piles installed in soil with undrained shear strength greater than ٧kPa the following formula can be used in predicting the bearing capacity of the pile:

$$F_{Rd} = \mu_n \cdot f_{Red} \cdot A \qquad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots ٥.٧$$

Where: f_{Red} = reduced strength of wood

A = cross-sectional area of the pile

If the wood is of sound timber, (e.g. pinewood or spruce wood with a diameter > ٠.١٣m), then f_{Red} (reduced strength) of the pile can be taken as ١١MPa.

Increase in load per section of pile is found to be proportional to the diameter of the pile and shear strength of the soil and can be decided using the following formula:

$$P_{mi} = \alpha \cdot A_{mi} \cdot C_{mi} \cdot \frac{1}{\gamma_{RD} \cdot \gamma_m \cdot \gamma_n} \qquad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\text{٥.٨}$$

where: $A_{m,}$ = area of pile at each ٣.٥ m section mid point of pile

C_m = shear strength at each ٣.٥m section mid point of pile

d_m = diameter of pile at each ٣.٥ m section mid point of pile

P_{mi} = pile load at the middle of each section

Example ٥.٦

Determine the design bearing capacity of a pile ١٢m pile driven in to clay with characteristic undrained shear strength ١٠KPa and ١.٠kPa increase per metre depth. Piling condition is assumed to be favourable and the safety class ٢. The pile is cut at ١.٥m below the ground level. Top diameter of the pile is ١٨٠mm and growth in diameter is ٩mm/m.
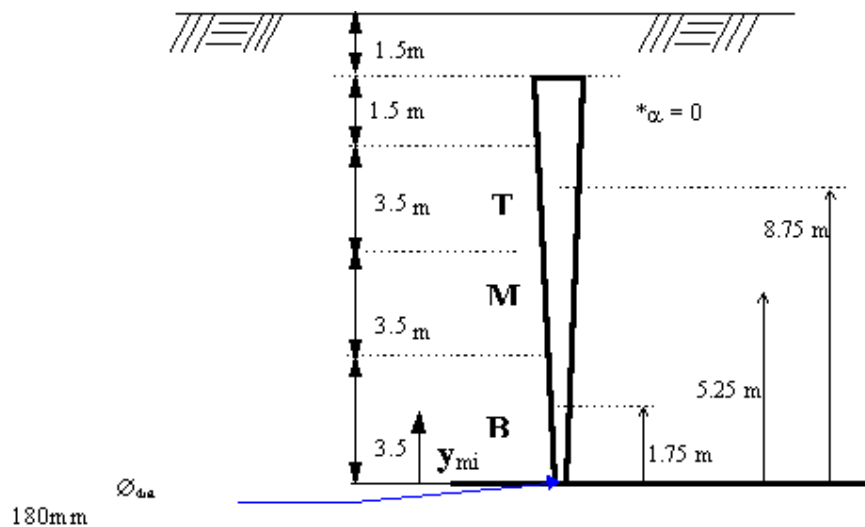

Figure ٥-٤ Example ٥.٦

*Often it is assumed that cohesive strength of the soil in the fires three meters is half the values at the bottom.

solution:

First decide which part of the pile is heavily loaded. To do so, divide the pile which is in contact with the soil in three parts or sections (see fig.٤.١) in this example the pile is divided into three ٣.٥m parts

Calculate and decide diameter of the pile at the mid point of each ٣.٥m section (٠.١٨٠+٠.٠٠٩(yᵢ) ; yᵢ growth per meter from the end point.

Calculate the shear strength of the soil at the mid point of each ٣.٥m section $C_{mi} = (٢٢ - ١(y_i))$. Shear strength at the end of the pile = (١٠MPa + ١MPa (١٢m))=٢٢ MPa

Decide the values of the partial coefficients $\gamma_{gd}, \gamma_m, \gamma_s$ from table **(١٠-١ - ١٠-٤)**

Part	y_{mi}(see fig. ٥.٤) m	$d_{mi}=$ (٠.١٨٠+٠.٠٠٩ /φovτ> ψₗ μ	$C_{mi} = (٢٢ - ١$ /φovτ> (ψ_i)	$P_{ei} = d_{ei} \cdot C_{ei} \frac{12 \cdot 35 \cdot \pi}{17 \cdot 11 \cdot 144}$
T(top) section	٨.٧٥	٠.٢٥٩	١٣.٣	١٦.٩
M(middle) section	٥.٢٥	٠.٢٢٧	١٦.٨	١٨.٧
B(bottom) section	١.٧٥	٠.١٩٦	٢٠.٣	١٩.٥

Pₜᵢ = pile load at the top of each section

Part	P_{ti} kN	y_{ti} m	$d_i = 0.180 + 0.009 \cdot y_i$ m	$\sigma_{ti} = \frac{P}{A} - \frac{4}{\pi} \cdot \frac{P_{ti}}{d_i^2} kN$
T(top)	٥٥.١	١٠.٥	٠.٢٧٥	٩٢٨ this part of the pile is highly loaded
M(middle)	٣٨.٢	٧.٠	٠.٢٤٣	٨٢٤
B(bottom)	١٩.٥	٣.٥	$d_d = 0.180 + 0.009 \cdot 3.5$ = 0.212	٥٥٢

$$\sigma_{ti} - \frac{P}{A} - \frac{4}{\pi} \cdot \frac{P_{ti}}{d_i^2} kN$$ = stress at the top of the pile

∴ The bearing capacity of the pile is ٥٥.١kN

Now using the equation in (٦-٧), we will check the pile for failure

f_{Red} = ١١MPa (see section ٥.٦)

$\mu_n = \cdot.9$

$\gamma_n = 1.1$

$\therefore \quad F_{kd} = \mu_n \cdot f_{Red} \cdot A = \dfrac{0.9}{1.1} \cdot 11 \cdot A$

In consideration of failure in the pile material, the pile can be loaded up to $9.\cdot$ MPa

In consideration of cohesion force, the pile can be loaded up to $\circ\circ$ MPa

the bearing capacity of the pile is therefore, $\circ\circ$ MPa

$\circ.\mathsf{1}.\mathsf{1}$ Simplified method of predicting the bearing capacity of timber piles

Consider the previous case and use the following formula :

$$F_{Rd} = \frac{1}{\gamma_{Rd}} \cdot a \cdot C_{ud} \cdot A \qquad \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots \circ.9$$

regarded the pile in its full length

calculate average diameter of the pile /ϕovτ> $\dfrac{0.18 + 0.275}{2} = 0.288m$

calculate average shear strength of the pile/ϕovτ> $C_c = \dfrac{22 + 11.5}{2} = 16.75 kPa$

r. decide the values of $\gamma_{Rd,\,\gamma m}$ and α (**table $1\cdot-1 - 1\cdot-\mathsf{\xi}$**) :

$\gamma_{Rd,} = 1.V$

$\gamma_m = 1.\Lambda$ /ϕovτ> $(\cdot.\Lambda) = 1.\mathsf{\xi\xi}$

$\alpha = 1.\mathsf{Y}$

$$F_{Rd} = \frac{1}{\gamma_{Rd}} \cdot 1.2 \cdot \frac{16.75}{(1.1) \cdot 144} 0.228 \cdot \pi \ 10.5 = 56.1 kN$$

$$\sigma = \frac{P}{A} = \frac{56.1 \cdot 10^3 \cdot 4}{\pi \ 0.18^2} = 2.2\ MPa$$

$\therefore$ the bearing capacity of the pile is $\circ\mathsf{1}$ kN

DESIGN OF PILE GROUP

Introduction

Group action in piled foundation: Most of pile foundations consists not of a single pile, but of a group of piles, which act in the double role of reinforcing the soil, and also of carrying the applied load down to deeper, stronger soil strata. Failure of the group may occur either by failure of the individual piles or as failure of the overall block of soil. The supporting capacity of a group of vertically loaded piles can, in many cases, be considerably less than the sum of the capacities the individual piles comprising the group. Grope action in piled foundation could result in failure or excessive settlement, even though loading tests made on a single pile have indicated satisfactory capacity. In all cases the elastic and consolidation settlements of the group are greater than those of single pile carrying the same working load as that on each pile within the group. This is because the zone of soil or rock which is stressed by the entire group extends to a much greater width and depth than the zone beneath the single pile *(fig. ٦-١)*
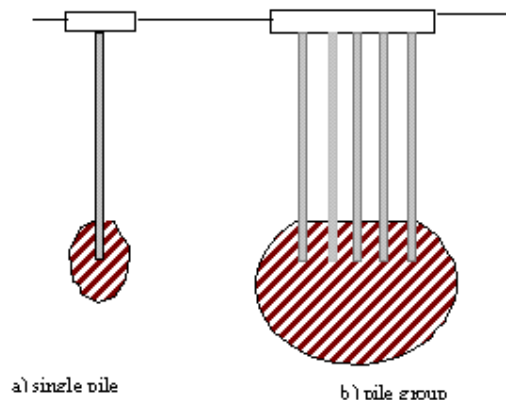
a) single pile   b) pile group

Figure ٦-١ Comparison of stressed zone beneath single pile and
pile group

Learning out come

When students complete this section, they will be able:

       o   to calculate and predict design bearing capacity of pile group in different soil types
       o   to appreciate the governing factors in design of group of piles
       o   to design pile groups with appropriate pile spacing

٦.١ Bearing capacity of pile groups

Pile groups driven into sand may provide reinforcement to the soil. In some cases, the shaft capacity of the pile driven into sand could increase by factor of ٢ or more.

But in the case of piles driven into sensitive clays, the effective stress increase in the surrounding

soil may be less for piles in a group than for individual piles. this will result in lower shaft capacities.

Figure ٦-٢ Under axial or lateral load, In a group, instead of failure of individual piles in the group, block failure (the group acting as a block) may arise.
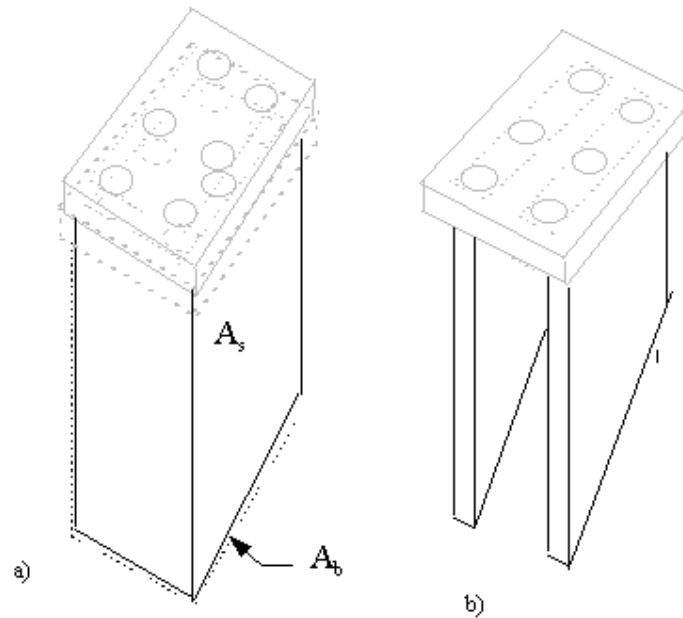

Figure ٦-٢ Block failure

In general ,the bearing capacity of pile group may be calculated in consideration to block failure in a similar way to that of single pile, by means of equation ٤-١ ,but hear As as the block surface area and Ab as the base area of the block or by rewriting the general equation we get:

$$R_u = 4 \cdot A_s \cdot \bar{c} \cdot a + C_b \cdot A_b \cdot N_c - (W_p - W_s)$$

.................................(٦.١)

where:

A_s, surface area of block

A_b = base area of block (see fig.٦-٣)

C_b, C_s= average cohesion of clay around the group and beneath the group.

N_c = bearing capacity factor. For depths relevant for piles, the appropriate value of N_c is ٩

W_p and W_s = weight of pile respective weight of soil

In examining the behaviour of pile groups it is necessary to consider the following elements:

- a free-standing group, in which the pile cap is not in contact with the underlying soil.
- a "piled foundation," in which the pile cap is in contact with the underlying soil.
- pile spacing
- independent calculations, showing bearing capacity of the block and bearing capacity of individual piles in

the group should be made.
- relate the ultimate load capacity of the block to the sum of load capacity of individual piles in the group (the ratio of block capacity to the sum of individual piles capacity) the higher the better.
- In the case of where the pile spacing in one direction is much greater than that in perpendicular direction, the capacity of the group failing as shown in Figure ٦-٢ b) should be assessed.

٦.١.١ Pile groups in cohesive soil

For pile groups in cohesive soil, the group bearing capacity as a block may be calculated by mans of e.q. ٤-٥ with appropriate N_c value.

٦.١.٢ Pile groups in non-cohesive soil

For pile groups in non-cohesive soil, the group bearing capacity as a block may be calculated by means of e.q. ٤-٧

٦.١.٣ Pile groups in sand

In the case of most pile groups installed in sand, the estimated capacity of the block will be well in excess of the sum of the individual pile capacities. As a conservative approach in design, the axial capacity of a pile group in sand is usually taken as the sum of individual pile capacities calculated using formulae in ٤-٨.

Worked Example ٦-١

Calculate the bearing capacity and group efficiency of pile foundation installed in uniform clay of bulk unit weight, γ of ٢٠kN/m³ and undrained shear strength of C_u of ٥٠kN/m². The foundation consists of ٢٥ piles each ١٨m long ,٠.٤m in diameter and weight ٦٠kN. The weight of the pile cap is ٦٠٠kN and founded ١m below the ground level. The adhesion factor α for the soil/pile interface has a value of ٠.٨
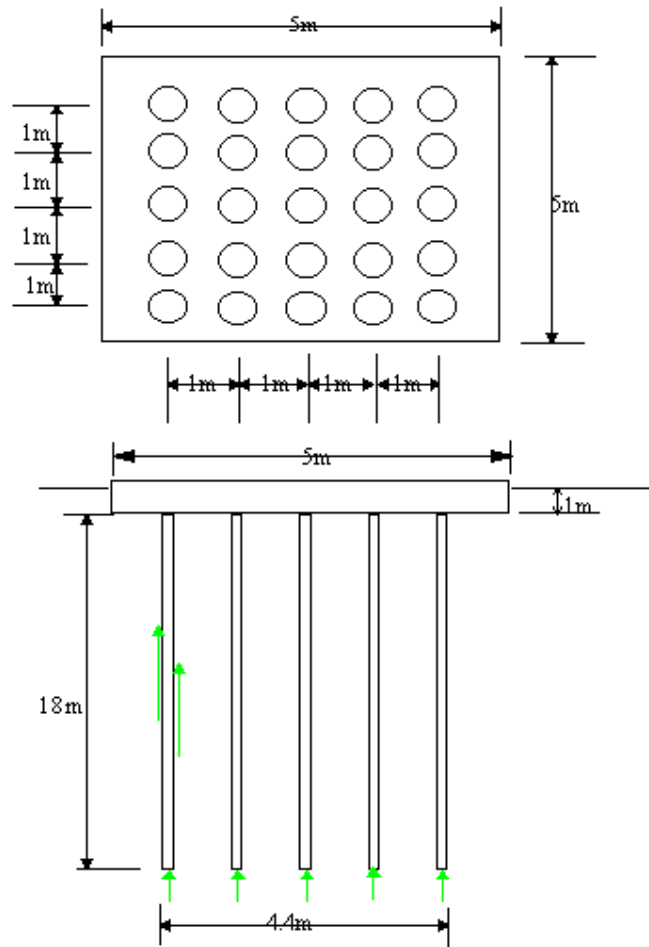
Figure ٦-٣ Worked Example ٦-١

SOLUTION

Calculate single pile bearing capacity:

$R_s = \alpha$ /φοντ> C_s /φοντ> $A_s = ٠.٨$ /φοντ> ٥٠ /φοντ> ٠ /φοντ> ١٨ /φοντ> π /φοντ> (٠.٤) = <u>٩٠٤kN</u> ?

☐

$R_b = N_c$ /φοντ> C_b /φοντ> $A_b = ٩$ /φοντ> ٥٠ /φοντ> π /φοντ> $(٠.٢)^٢ = ٥٦.٦kN$

∴ $R_{ci} = R_{si} + R_{bi} = ٩٠٤ + ٥٦.٦ = ٩٦٠$

$(W_p + W_{cap}) - W_s = (٦٠? ٢٥+(٦٠٠-٢٠? ٥٠.٠? ٥٠.٠? ١.٠)) - (٢٠? ١٨$ /φοντ> π /φοντ> $(٠.٢)^٢$ /φοντ> ٢٥ = <u>٤٦٩kN</u> ??

☐

☐

∴ total load capacity of ٢٥ piles = $R_{uc٢٥}$ = $(R_{ci} = R_{si} + R_{bi})$ /φοντ> ٢٥ - {$(W_p + W_{cap}) - W_s$} = ٩٦٠ /φοντ> ٢٥ - ٤٦٩ = <u>٢٣٥٣١kN</u>

calculate block load capacity :

= ٤ ? (١٨? ٤.٤? ٥٠ /φοντ> ٠.٨)+ ٥٠ /φοντ> ٤.٤ ? ٤.٤ /φοντ> ٩ = ٢٥٦٥٠kN

Pile Foundation Design

Chapter 7

Pile spacing and pile arrangement

In certain types of soil, specially in sensitive clays, the capacity of individual piles within the a closely spaced group may be lower than for equivalent isolated pile. However, because of its insignificant effect, this may be ignored in design. Instead the main worry has been that the block capacity of the group may be less than the sum of the individual piles capacities. As a thumb rule, if spacing is more than ٢ - ٣ pile diameter, then block failure is most unlikely.

It is vital importance that pile group in friction and cohesive soil arranged that even distribution of load in greater area is achieved.

Large concentration of piles under the centre of the pile cap should be avoided. This could lead to load concentration resulting in local settlement and failure in the pile cap. Varying length of piles in the same pile group may have similar effect.

For pile load up to ٣٠٠kN, the minimum distance to the pile cap should be ١٠٠ mm

for load higher than ٣٠٠kN, this distance should be more than ١٥٠ mm.

In general, the following formula may be used in pile spacing:

End-bearing and friction piles: S = ٢.٥ /φovτ> (δ) + 0.02 . Λ ٧.١
Cohesion piles: S = ٣.٥ /φovτ> (δ) + 0.02   /φovτ> Λ ٧.٢

where:

d = assumed pile diameter

L = assumed pile length

S = pile centre to centre distance (spacing)

Example ٧-١

A retaining wall imposing a weight of ١٢٠kN/m including self-weight of the pile cap is to be constructed on pile foundation in clay. Timber piles of ٢٥٠mm in diameter and each ١٤m long with bearing capacity of ٩٠kN/st has been proposed. Asses suitable pile spacing and pile arrangement.

Solution:

١. recommended minimum pile spacing:

S = ٣.٥ /φovτ> (d) + ٠.٠٢   /φovτ> L = ٣.٥   /φovτ> (٠.٢٥) + ٠.٠٢   /φovτ> ١٤ = <u>١.١٦ m</u>[?]

٢. try arranging the piles into two rows:

Top diameter = 250 mm

vertical load = ١٢٠ kN/M

single pile load capacity = ٩٠ kN/st

$$\therefore \frac{pile}{meter} = \frac{120}{90} = st/m$$
$$= ١.٣٣m$$

spacing in the two rows /φovτ>

$$\frac{1}{1.33} \cdot 2 = 1.50m$$

B = pile cap width

1.50m
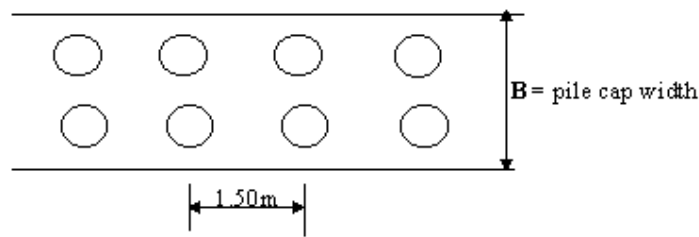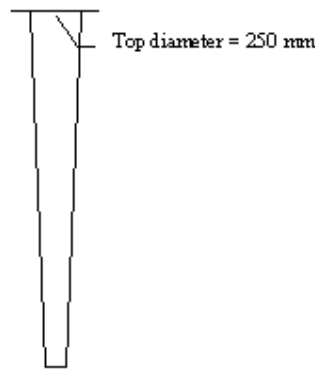
minimum distance to the edge of the pile = ٠.١m /φovτ> **B** = ٢ /φovτ> ٠.١ + ٠.٢٥ + ١.١٠ = <u>١.٥٥m</u>

---

here because of the descending nature of the pile diameter a lesser value can be taken , say ١.١٠m

PILE INSTALATION METHODS

٨.١ Introduction

The installation process and method of installations are equally important factors as of the design process of pile foundations. In this section we will discuss the two main types of pile installation methods; installation by pile hammer and boring by mechanical auger.

In order to avoid damages to the piles, during design, installation Methods and installation equipment should be carefully selected.

If installation is to be carried out using pile-hammer, then the following factors should be taken in to consideration:

- the size and the weight of the pile
- the driving resistance which has to be overcome to achieve the design penetration
- the available space and head room on the site
- the availability of cranes and
- the noise restrictions which may be in force in the locality.

٨.٢ Pile driving methods (displacement piles)

Methods of pile driving can be categorised as follows:

١. Dropping weight
٢. Explosion
٣. Vibration
٤. Jacking (restricted to micro-pilling)
٥. Jetting

٨.٢.١ Drop hammers

A hammer with approximately the weight of the pile is raised a suitable height in a guide and released to strike the pile head. This is a simple form of hammer used in conjunction with light frames and test piling, where it may be uneconomical to bring a steam boiler or compressor on to a site to drive very limited number of piles.

There are two main types of drop hammers:

- Single-acting steam or compressed-air hammers
- Double-acting pile hammers

1. Single-acting steam or compressed-air comprise a massive weight in the form of a cylinder (see fig.٨-١). Steam or compressed air admitted to the cylinder raises it up the fixed piston rod. At the top of the stroke, or at a lesser height which can be controlled by the operator, the steam is cut off and the cylinder falls freely on the pile helmet.

2. Double-acting pile hammers can be driven by steam or compressed air. A pilling frame is not required with this type of hammer which can be attached to the top of the pile by leg-guides, the pile being guided by a timber framework. When used with a pile frame, back guides are bolted to the hammer to engage with leaders, and only short leg-guides are used to prevent the hammer from moving relatively to the top of the pile. Double-acting hammers are used mainly for sheet pile driving.

235mm square Driven Precast Piling

Figure ٨-١ Pile driving using hammer
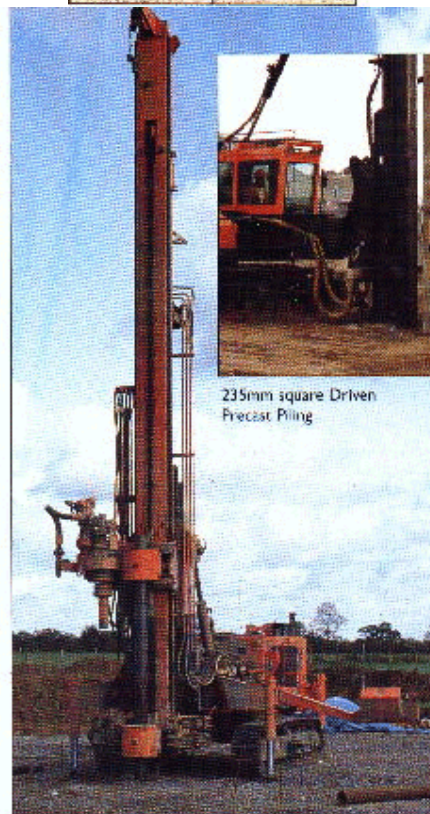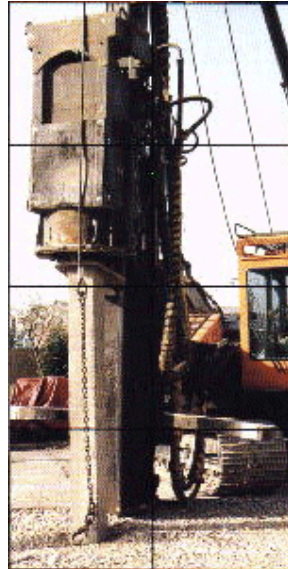
٨.٢.٢ *Diesel hammers*

Also classified as single and double-acting, in operation, the diesel hammer employs a ram which is raised by explosion at the base of a cylinder. Alternatively, in the case of double-acting diesel hammer, a vacuum is created in a separate annular chamber as the ram moves upward, and assists in the return of the ram, almost doubling the output of the hammer over the single-acting type. In favourable ground conditions, the diesel hammer provide an efficient pile driving capacity, but they are not effective for all types of ground.

٨.٢.٣ *Pile driving by vibrating*

Vibratory hammers are usually electrically powered or hydraulically powered and consists of contra-rotating eccentric masses within a housing attaching to the pile head. The amplitude of the vibration is sufficient to break down the skin friction on the sides of the pile. Vibratory methods are best suited to sandy or gravelly soil.

Jetting: to aid the penetration of piles in to sand or sandy gravel, water jetting may be employed. However, the method has very limited effect in firm to stiff clays or any soil containing much coarse gravel, cobbles, or boulders.

٨.٣ Boring methods (non-displacement piles)

٨.٣.١ *Continuous Flight Auger (CFA)*

An equipment comprises of a mobile base carrier fitted with a hollow-stemmed flight auger which is rotated into the ground to required depth of pilling. To form the pile, concrete is placed through the flight auger as it is withdrawn from the ground. The auger is fitted with protective cap on the outlet at the base of the central tube and is rotated into the ground by the top mounted rotary hydraulic motor which runs on a carrier attached to the mast. On reaching the required depth, highly workable concrete is pumped through the hollow stem of the auger, and under the pressure of the concrete the protective cap is detached. While rotating the auger in the same direction as during the boring stage, the spoil is expelled vertically as the auger is withdrawn and the pile is formed by filling with concrete. In this process, it is important that rotation of the auger and flow of concrete is matched that collapse of sides of the hole above concrete on lower flight of auger is avoided. This may lead to voids in filled with soil in concrete.

The method is especially effective on soft ground and enables to install a variety of bored piles of various diameters that are able to penetrate a multitude of soil conditions. Still, for successful operation of rotary auger the soil must be reasonably free of tree roots, cobbles, and boulders, and it must be self-supporting.

During operation little soil is brought upwards by the auger that lateral stresses is maintained in the soil and voiding or excessive loosening of the soil minimise. However, if the rotation of the auger and the advance of the auger is not matched, resulting in removal of soil during drilling-possibly leading to collapse of the side of the hole.
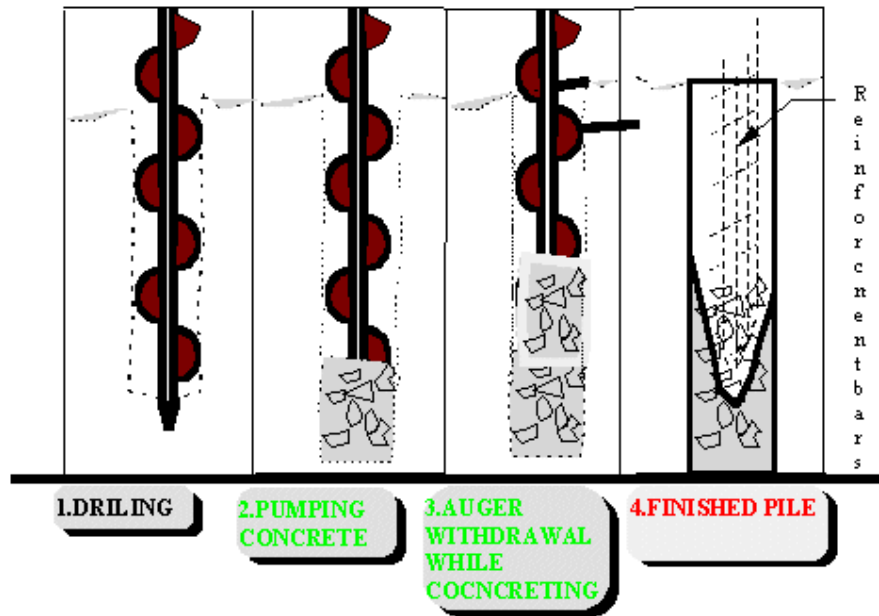
Figure ٨-٢ CFA Process

٨.٣.٢ Underreaming

A special feature of auger bored piles which is sometimes used to enable to exploit the bearing capacity of suitable strata by providing an enlarged base. The soil has to be capable of standing open unsupported to employ this technique. Stiff and to hard clays, such as the London clay, are ideal. In its closed position, the underreaming tool is fitted inside the straight section of a pile shaft, and then expanded at the bottom of the pile to produce the underream shown in fig. ٨-٣.Normally, after installation and before concrete is casted, a man carrying cage is lowered and the shaft and the underream of the pile is inspected.

Figure ٨ -٣ a)hydraulic rotary drilling equipment b) C.F.A, c)undrreaming tool open position

٨.٣.٣ C.H.D.P

Figure ٨-٤, Continuous helical displacement piles: a short, hollow tapered steel former complete with a larger diameter helical flange, the bullet head is fixed to a hallow drill pipe which is con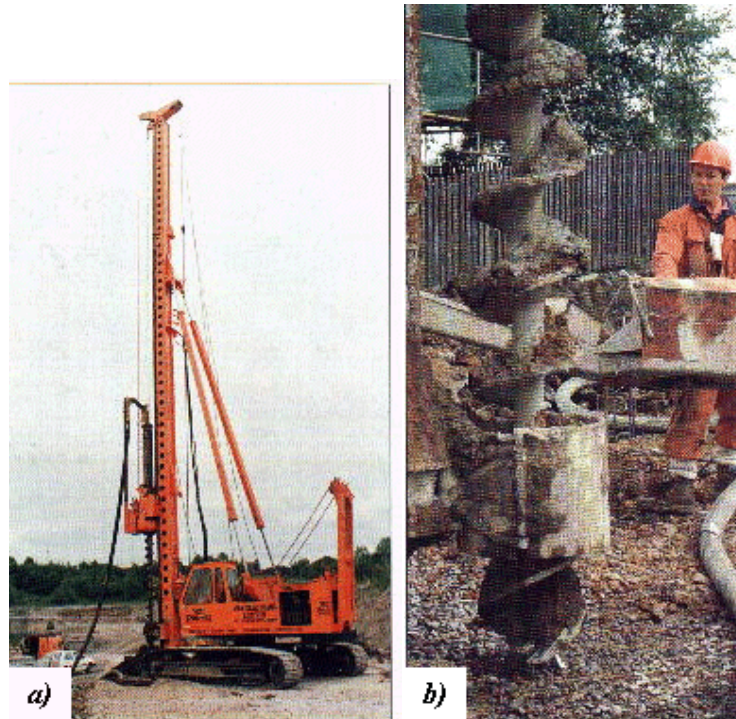nected to a high torque rotary head running up and down the mast of a special rig. A hollow cylindrical steel shaft sealed at the lower end by a one-way valve and fitted with triangular steel fins is pressed into the ground by a hydraulic ram. There are no vibrations.

Displaced soil is compacted in front and around the shaft. Once it reaches the a suitably resistant stratum the shaft is rotated. The triangular fins either side of its leading edge carve out a conical base cavity. At the same time concrete is pumped down the centre of the shat and through the one-way valve. Rotation of the fins is calculated so that as soil is pushed away from the pile base it is simultaneously replaced by in-flowing concrete. Rates of push, rotation and concrete injection are all controlled by an onboard computer. Torque on the shaft is also measured by the computer. When torque levels reach a constant low value the base in formed. The inventors claim that the system can install a\ typical pile in ١٢ minute. A typical ٦m long pile with an ٨٠٠mm diameter base and ٣٥٠mm shaft founded on moderately dense gravel beneath soft overlaying soils can achieve an ultimate capacity of over ٢٠٠t. The pile is suitable for embankments, hard standing supports and floor slabs, where you have a soft silty layer over a gravel strata.
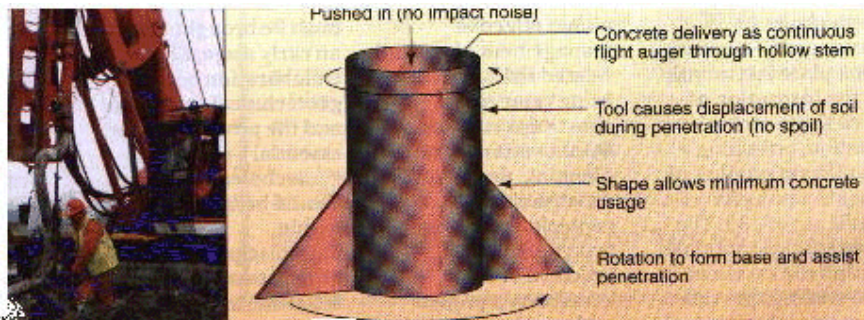

Figure ٨ -٤ C.H.D.P.

Pile Foundation Design

Chapter 9

LOAD TEST ON PILES

٩.١ Introduction

Pile load test are usually carried out that one or some of the following reasons are fulfilled:

- To obtain back-figured soil data that will enable other piles to be designed.
- To confirm pile lengths and hence contract costs before the client is committed to over all job costs.
- To counter-check results from geotechnical and pile driving formulae
- To determine the load-settlement behaviour of a pile, especially in the region of the anticipated working load that the data can be used in prediction of group settlement.
- To verify structural soundness of the pile.

Test loading: There are four types of test loading:

- compression test
- uplift test
- lateral-load test
- torsion-load test

the most common types of test loading procedures are Constant rate of penetration (CRP) test and the maintained load test (MLT).

٩.١.١ CRP (constant rate of penetration)

In the CRP (constant rate of penetration) method, test pile is jacked into the soil, the load being adjusted to give constant rate of downward movement to the pile. This is maintained until point of failure is reached.

Failure of the pile is defined in to two ways that as the load at which the pile continues to move downward without further increase in load, or according to the BS, *the load which the penetration reaches a value equal to one-tenth of the diameter of the pile at the base.*

Fig.٩-٢, In the cases of where compression tests are being carried out, the following methods are usually employed to apply the load or downward force on the pile:

A platform is constructed on the head of the pile on which a mass of heavy material, termed "kentledge" is placed. Or a bridge, carried on temporary supports, is constructed over the test pile and loaded with kentledge. The ram of a hydraulic jack, placed on the pile head, bears on a cross-head beneath the bridge beams, so that a total reaction equal to the weight of the bridge and its load may be obtained.

٩.١.٢ *MLT, the maintained increment load test*

Fig.٩-١, the maintained increment load test, kentledge or adjacent tension piles or soil anchors are used to provide a reaction for the test load applied by jacking(s) placed over the pile being tested. The load is increased in definite steps, and is sustained at each level of loading until all settlements has either stop or does not exceed a specified amount of in a certain given period of time.
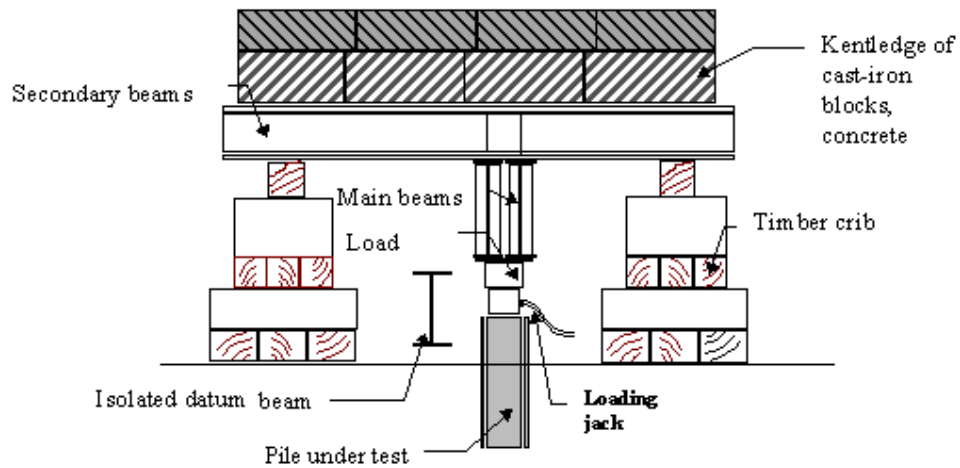

Figure ٩-١ test load arrangement using kentledge

Figure ٩-٢ test being carried out

Pile Foundation Design

Chapter 10

Limit State Design

Introduction

Traditionally, design resistance of foundations has been evaluated on an allowable stress basis that piles were designed with ultimate axial capacity between ٢ and ٣ times than working load. However structural design is now using a limit state design (LSD) bases whereby partial factors are applied to various elements of the design according to the reliability with which the parameters are known or can be calculated. LSD approach is the base of all the Eurocodes, including that for foundations design. It is believed that Limit state design has many benefits for the economic design of piling. The eurocode approach is particularly rigorous, and this guide adopts the partial factors presented in the codes.

> See *Elements of Soil Mechanics* (٨th Edition) to learn how to design piles (and other geotechnical structures) to Eurocode ٧. Full details are here….

Eurocode ٧ divides investigation, design and implementation of geoconstructions into three categories. It is a requirement of the code that project must be supervised at all stages by personnel with geotechnical knowledge.

In order to establish minimum requirements for the extent and quality of geotechnical investigation, deign and construction three geotechnical categories defined. These are: Geotechnical Category ١, ٢, ٣.

١٠.١ Goetechnical category ١, GC ١

this category includes small and relative simple structures:

-for which is impossible to ensure that the fundamental requirements will be satisfied on the basis of experience and qualitative geotecnical investigation;

-with negligible risk for property and life.

Geotechnical Category ١ procedures will be only be sufficient in ground conditions which are known from comparable experience to be sufficiently straight-forward that routine methods may be used for foundation design and construction. Qualitative geotechnical investigations

١٠.٢ Geotechnical Category, GC ٢

This category includes conventional types of structures and foundations with no abnormal risks or unusual or exceptionally difficult ground or loading conditions. Structures in Geotechnical category ٢ require quantitative geotechnical data and analysis to ensure that the fundamental requirements will be satisfied, but routine procedures for field and laboratory testing and for design and execution may be used. Qualified engineer with relevant experience must be involved.

١٠.٣ Geotechnical Category, GC ٣

This category includes structures or parts of structures which do not fall within the limits of Geotechnical Categories ١ and ٢.

The following are examples of structures or parts of structures complying with geotechnical category ٢:

conventional type of :

- spread foundations;
- raft foundations;

- piled foundations;
- walls and other structures retaining for supporting soil or water;
- excavations;
- bridge piers and abutments;
- embankment and earthworks;
- ground anchors and other tie-back systems;
- tunnels in hard, non-fractured rock and not subjected to special water tightness or other requirement.

Geotechnical Category ٣ includes very large or unusual structure. Structures involving abnormal risks or unusual or exceptionally difficult ground or loading conditions and highly seismic areas. Qualified geotechnical engineer must be involved.

The following factors must be considered in arriving at a classification of a structure or part of a structure:

- Nature and size of the structure
- Local conditions, e.g. traffic, utilities, hydrology, subsidence, etc.
- Ground and groundwater conditions
- Regional seismicity…..

١٠.٣.١ *Conditions classified as in Eurocode ٧*

In the code, conditions are classified as favourable or unfavourable.

Favourable conditions are as such:

+ if experience shows that the material posses limited spreading characteristic

+ if large scale investigation was carried out and test results are reliable

+ the existence of well documented investigation carried out using reliable methods which can give reproducible results

+ if additional tests, investigations and supervisions are recommend

+ high certainty in defining test results

+ failure is plastic

-

Unfavourable conditions are as such:

-- if experience shows that the material posses spreading characteres

-- if test results shows large spreading than the normal conditions

-- if the extent of investigation is limited

-- limited experience and methods lucking reproducibility

-- where there is no recommendation for additional test, investigations and supervision

-- uncertainty in analysing test results

-- if failure is brittle

Eurocode ٧ refers to foundation loadings as action. The se can be permanent as In the case of weights of structures and installations, or variable as imposed loading, or wind and snow loads. They can be accidental, e.g. vehicle impact or explosions.

Actions can vary spatially, e.g. self-weights are fixed (fixed actions), but imposed loads can vary in position (free actions). The duration of actions affections affects the response of the ground. It may cause strengthening such as the gain in strength of a clay by long-term loading, or weakening as in the case of excavation slopes in clay over the medium or long term. To allow for this Eurocode ٧ introduces a classification related to the soil response and refers to transient actions (e.g. wind loads), short-term actions (e.g. construction loading) and long-term actions. In order to allow for uncertainties in the calculation of he magnitude of actions or combinations of actions and their duration and spatial distribution, Euorcode requires the design values of actions F_d to be used for the geotechnical design either to be assessed directly or to be derived from characteristic values F_k :

$$F_d = F_k \times \gamma_F$$

١٠.٤ The partial factors γ_{m}, γ_{n}, γ_{Rd}

The partial factor γ_{m}**:** this factor is applied as a safety factor that the characteristic values of the material is divided by this factor. (m = material index) and covers :

- unfavourable deviation from the material product property
- inaccuracies in the conversion factors: and
- uncertainties in the geometric properties and the resistance model.

In ultimate limit state, depending upon a given conditions, for Geotechnical Category ٢, the values of the γ_{m} may be decided using table ١٠-١& ١٠-٢.

The partial co-efficient γ_{n}: in order to ensure stability and adequate strength in the structure and in the ground, in the code, cases A, B, and C have been introduced. Values of γ_{n} is given in table ١٠-٣

Partial co-efficient γ_{Rd}: this co-efficient is applied in consideration of deviation between test results and future construction. Values of the γ_{n} should be between ١.٤ - ١.٨

Table ١٠-١ partial factors on material properties for conventional design situations for ultimate limit states

Material property	Partial factor γ_{m}
tanφ	١.١- ١.٢٥
modules	١.٢ - ١.٨
other properties	١.٦ - ٢.٠

Table ١٠-٢ partial factors on material properties for conventional design situations for service limit state

Material property	Partial factor γ_{m}
modules	١.٢ - ١.٨

| other properties | ١.٦ - ٢.٠ |

Normally the design values, d , Ed, tanφ , can be decided using the following formulae:

$$f_d = f_k/(\gamma_n \quad_{/\phi ov\tau > \gamma \mu})$$

$$E_d = E_k/(\gamma_n \quad_{/\phi ov\tau > \gamma \mu})$$

$$\tan\phi_d = \tan\phi_k/(\gamma_n \quad_{/\phi ov\tau > \gamma \mu})$$

Where:

f = reaction force

ϕ = *internal angle of friction*

E = elastic module

Table ١٠-٣ partial factor γ_n

Class	γ_n
A	١.٠
B	١.١
C	١.٢

Table ١٠-٤ adhesion factor α

pile	α_b	α_s
Concrete piles	٠.٥	٠.٠٠٥
Steel piles	٠.٥	٠.٠٠٢
timber piles (wood piles)	٠.٥	٠.٠٠٩

The table is used for $q_{c\,?}$ ١٠ Mp$_a$

Table ١٠-٥ Bearing factors Nγ , N$_q$, N$_c$

ϕ_d	Nγ	N$_c$	N$_q$
٢٥	٦.٤٨	٢٠.٧	١٠.٧
٢٦	٧.٦٤	٢٢.٢	١١.٨
٢٧	٨.٩٩	٢٣.٩	١٣.٢
٢٨	١٠.٦	٢٥.٨	١٤.٧
٢٩	١٢.٥	٢٧.٩	١٦.٤

٣٠	١٤.٧	٣٠.١	١٨.٤
٣١	١٧.٤	٣٢.٧	٢٠.٦
٣٢	٢٠.٦	٣٥.٥	٢٣.٢
٣٣	٢٤.٤	٣٨.٩	٢٦.١
٣٤	٢٩.٠	٤٢.٢	٢٩.٤
٣٥	٣٤.٤	٤٦.١	٣٣.٣
٣٦	٤١.٩	٥٠.٦	٣٧.٧
٣٧	٤٩.١	٥٥.٦	٤٢.٩
٣٨	٥٨.٩	٦١.٣	٤٨.٩
٣٩	٧٠.٩	٦٧.٩	٥٦.٠
٤٠	٨٥.٦	٧٥.٣	٦٤.٢
٤١	١٠٤	٨٣.٩	٧٣.٩
٤٢	١٢٦	٩٣.٧	٨٥.٤
٤٣	١٥٤	١٠٥	٩٩.٠
٤٤	١٩٠	١١٨	١١٥
٤٥	٢٣٤	١٣٤	١٣٥