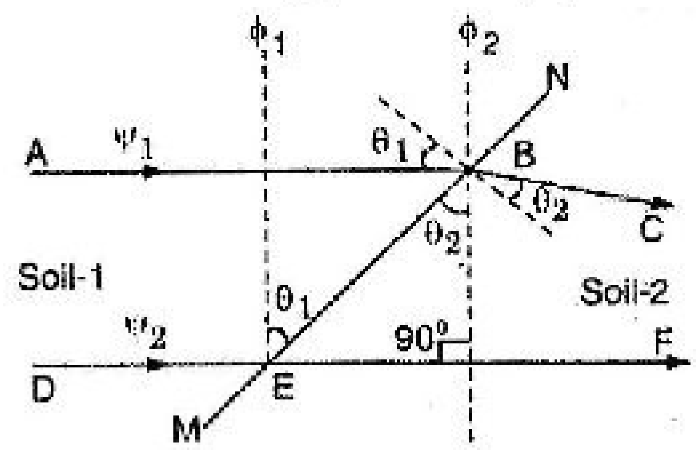
Let two flow lines ψ_1 (AB) and ψ_2 (DE) get deflected at the interface MN of two soils in the direction BC and EF respectively. k_1, k_2 be their permeability coefficients.

ϕ_1 and ϕ_2 be two equipotential lines having Δh as drop of potential. The equipotential lines also get deflected at the interface of the soils.

Let θ_1 and θ_2 be the angles that the flow line ψ_1 make with the normal to the interface before and after deflection respectively.

Now, the rate of flow Δq between the flow channel bounded by flow lines ψ_1 and ψ_2 is

$$\Delta q = k_1 i_1 (\text{EK}) = k_2 i_2 (\text{BP})$$



where $$i_1 = \frac{\Delta h}{\text{KB}}$$

and $$i_2 = \frac{\Delta h}{\text{EP}}$$

$$\therefore \quad \Delta q = k_1 \frac{\Delta h}{\text{KB}} \text{EK}$$

$$= k_2 \frac{\Delta h}{\text{EP}} \cdot \text{BP}$$

or $$\frac{k_1}{k_2} = \frac{\Delta h \cdot \text{BP}}{\text{EP}} \times \frac{\text{KB}}{\Delta h \cdot \text{EK}}$$

$$= \frac{\text{BP}}{\text{EP}} \cdot \frac{\text{KB}}{\text{EK}}$$

But $$\frac{\text{KB}}{\text{EK}} = \tan\theta_1$$

and $$\frac{\text{EP}}{\text{BP}} = \tan\theta_2$$

$$\therefore \quad \frac{\tan\theta_1}{\tan\theta_2} = \frac{k_1}{k_2}$$

Seepage through Soils of different permeabilities

When water seeps from a soil of permeability k_1 to a soil of permeability k_2, the principle of the square flow net is no longer valid. If we consider a flow net in which the head drops across each square boundary, Δh, is a constant then the flow through each figure is given by the expression

$$\Delta q = k \, \Delta h \frac{b}{l}$$

If Δq is to remain the same when k is varied, then b/l must also vary.

Consider the case of two soils with

$$k_1 = \frac{k_2}{3}$$

Then, $$\Delta q_1 = k_1 \, \Delta h \frac{b_i}{l_i}$$

and $$\Delta q_2 = k_2 \, \Delta h \frac{b_2}{l_2} = 3 k_1 \, \Delta h \frac{b_2}{l_2}$$

i.e. $$\frac{b_i}{l_i} = 3 \frac{b_2}{l_2}$$

If N_f be the number of flow channels present in the flow net, then the total quantity of seepage is given by,

$$q = k \times N_f \times \Delta h.$$

Again, if H be the initial difference of head and N_d be the number of equal head drops, then

$$\Delta h = \frac{\text{H}}{N_d}$$

If the portion of the flow net in the soil permeability k_1 is square, then

$$\frac{b_2}{l_2} = \frac{1}{3} \quad \text{or} \quad \frac{b_2}{l_2} = \frac{k_1}{k_2}$$

If k_x and k_y be the permeabilities in the x and y directions, then equivalent permeability

$$k = \sqrt{k_x k_y}$$

$$q = k.\text{H} \times \frac{N_f}{N_d}$$

Example. *What is the value of critical gradient for the soil, having its specific gravity equal to 2.68 and void ratio equal to 0.67 ?*

Solution : Critical hydraulic gradient,

$$i_c = \frac{S_s - 1}{1+e} = \frac{2.68-1}{1+0.87}$$

$$= \frac{1.68}{1.87} = \mathbf{0.90}$$

Example. *The specific gravity of the particles of a sand is 2.65. The porosity of the sand is a loose state is 0.52, and in a dense state is 0.37. What is the values of the critical hydraulic gradient in these two states respectively ?*

Solution : We know,

$$n = \frac{e}{1+e}$$

If sand is in loose state, then

$$n = 0.52$$

$$\therefore \quad 0.52 = \frac{e}{1+e}$$

or $$e = 1.08$$

Now, critical hydraulic gradient,

$$i_c = \frac{S_s - 1}{1+e} = \frac{2.65-1}{1+1.08} = \mathbf{0.79}$$

If sand is in dense state, then

$$n = 0.37$$

$$\therefore \quad 0.37 = \frac{e}{1+e},$$

or $$e = 0.59$$

Now, $$i_c = \frac{S_s - 1}{1+e} = \frac{2.65-1}{1+0.59} = \mathbf{1.04}$$

Example. *What is the value of critical gradient for a loose deposit of sand having void ratio of 0.67 and specific gravity of 2.67 ?*

Solution : Critical hydraulic gradient,

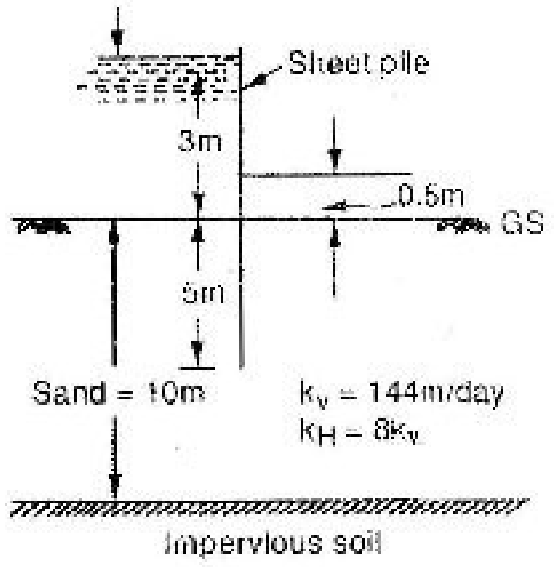
$$i_c = \frac{S_s - 1}{1+e} = \frac{2.67-1}{1+0.67} = \mathbf{1}$$

Example. *A sand deposit 10 m thick overlies an impervious soil. A vertical sheet pile penetrates half way into the sand deposit. The water level on one side of the wall is 3.0 m and on the other side is 0.5 m above the ground level. The sand stratum has a vertical permeability of 1.44 m per day, and the horizontal permeability equals 8 times the permeability in the vertical direction. A flow net construction reveals that there are 12 flow channels and 26 potential drops. The seepage flow per day, will be per meter length of sheet pile.*

Solution : $$k = \sqrt{k_H \cdot k_v}$$

$$= \sqrt{8 \cdot k_V \cdot k_V} = \sqrt{8}\, k_V$$

$$= \sqrt{8} \times 1.44 = 4.07 \text{ m/day}$$



Now, Seepage, $$q = K \cdot H_L \frac{N_f}{N_d}$$

where, k = 4.07 m/day

H_L = 3 – 0.5 = 2.5 m

N_f = 12;

N_d = 26

$$\therefore \quad q = 4.07 \times 2.5 \times \frac{12}{26} \text{ m}^3\text{/day}$$

= 4.7 cubic m per day per m length of sheet pile.

Example. *The soil at the toe of a dam is fully saturated and has a water content of 32%, and specific gravity of soil grains is 2.65. For safety measures against piping, the exit gradient is restricted to 17% of the critical hydraulic gradient. Find the permissible exit gradient.*

Solution : Given, w = 0.32 and S_s = 2.65

Critical gradient, $$i_c = \frac{\gamma_{sub}}{\gamma_w} = \frac{S_s - 1}{1+e}$$

But for a saturated soil,

$$e = w \cdot S_s = 0.32 \times 2.65 = 0.85$$

$$\therefore \quad i_c = \frac{2.65-1}{1+0.85} = \frac{1.65}{1.85} = 0.89$$

Permissible exit gradient = 17% critical gradient

= 0.17 × 0.89 = 0.15.

PILE AND WELL FOUNDATION

Pile Foundation. When the subsoil immediately below the foundation is very weak and incapable of holding load from the structure, then the load of structure has to be transmitted to hard strata at deep depths through piles of wells or caissons. Piles generally obtain support from a combination of friction along the surface of pile shaft and from end bearing at the bottom of the shaft.

Bearing Capacity of Piles. The maximum load which can be carried by a pile and at which the pile continues to sink without further increase of load, is called *ultimate bearing capacity* or *ultimate bearing resistance of the pile*.

The safe load which can be carried by a pile is determined on the basis of

(*i*) utimate bearing resistance divided by a suitable factor of safety.

(*ii*) permissible settlement.

(*iii*) overall stability of the pile foundation, called bearing capacity of the pile.

Classification of Piles

Generally piles are classified, based on their function, method of installation and material used

I. Based on the function

(*a*) End bearing piles : When the load is transferred to hard strata through water or soft soils, the pile is known as end *bearing piles* or *point bearing piles*.

(*b*) Friction piles : When the load is transferred by means of friction (skin friction) along the surface of the soil, the pile is known as *friction pile*.

(*c*) Compaction piles : To compact loose granular soils and to increase the bearing capacity, compaction piles are used.

(*d*) Tension piles : Piles used to anchor down the structure subjected to uplift force is known as *tension piles*.

(*e*) **Anchor piles :** Anchor piles are used to provide anchorage against horizontal pull from sheet pile walls or other pulling forces.

(*f*) **Batter piles :** Batter piles are used to resist horizontal force or inclined force due to mooring or berthing of ships.

(*g*) **Fender piles and dolphins :** They protect water front structures against the impact from ships or other floating objects.

II. Based on Method of Installation

(*a*) Precast driven piles

(*b*) Driven cast-in-situ piles

(*c*) Bored cast-in-situ piles

III. Based on Material used

(*a*) Concrete piles

(*i*) pre cast piles

(*ii*) cast-in-situ piles

(*iii*) pre-tested concrete piles

(*b*) Timber piles

(*c*) Steel piles

(*d*) Composite piles

(*i*) concrete and timber piles

(*ii*) concrete and steel piles

Pile Capacity

The capacity of individual pile can be estimated by static formula, by dynamic formula, by pile load test or by penetration test.

1. **Static Formula.** This formula is used to estimate the pile capacity on the basis of strength properties of soil.

The ultimate load capacity of piles can be estimated by calculating the resistance derived from the bearing and friction component of the total pile capacity.

Ultimate pile capacity,

$$Q = Q_e + Q_f$$

where, Q_e = end-bearing capacity

Q_f = side friction capacity

The end bearing component of pile can be estimated by using formula similar to that for estimating ultimate bearing capacity of shallow foundation.

$$Q_e = \text{Area}\,[CN_c + \gamma\, D\,(N_q - 1) + 0.4\,\gamma\, BN_\gamma] \text{ for square piles}$$

$$Q_e = \text{Area}\,[CN_c + \gamma\, D\,(N_q - 1) + 0.6\,\gamma\, BN_\gamma] \text{ for circular piles}$$

where C = cohesion

γ = unit weight

D = depth of pile

B = breadth of pile and N_c,

N_q, N_γ = bearing capacity factors.

The friction component of pile capacity can be estimated by considering the adhesion and friction between the soil and the pile. For clayey soils the frictional resistance is given by the equation.

$$Q_f = \pi\, dl\, \alpha\; C_u$$

where d = diameter of pile,

L = length of pile,

α = adhesion factor,

C_u = undrained cohesion

For *sandy soils* the friction component of pile resistance depends on the coefficient, of friction between the soil and the pile and the horizontal intergranular stress adjacent to the pile surface. The friction component of the pile resistance for sandy soil,

$$Q_f = \Sigma\pi\, d_i\, l_i\, K\, \bar{\sigma}_{vi} \tan\delta$$

where K = lateral effective earth presure coefficient

$\bar{\sigma}_{vi}$ = average effective vertical inter-granular stress over increment

d_i = diameter of pile over increment i

l_1 = length of pile

δ = friction angle of the soil on the pile.

2. **Dynamic Formula.** The load capacity of pile often estimated from the resistance of the pile to penetration during driving. Energy input from the pile rammer is equal to the energy used to drive the pile plus energy losses. The most commonly used formulae are given below.

(*a*) Engineering News Formula

(*i*) Drop hammer :

$$\text{Pile capacity, } Q_u = \frac{WH}{6(S+0.25)}$$

(*ii*) Single acting steam hammer :

$$Q_u = \frac{WH}{6(S+0.25)}$$

(*iii*) Double acting steam hammer :

$$Q_u = \frac{(W+ap)H}{6(S+0.25)}$$

where W = weight of hammer falling through a height in kg

H = height of fall of hammer or length of stroke of piston in cm

P = mean effective steam pressure in kg/cm^2

a = effective area of piston in cm^2

S = Finat set per blow usually taken as average penetration for last 5 blows of drop hammer or 20 blows of a steam hammer

3. Hiley's Formula. Indian Standards Institution has recommended the use of Hiley's formula

Ultimate load on the pile,

$$Q_f = \frac{\eta_h WH \eta_b}{S + \frac{1}{2}C}$$

where W = weight in kilogram

H = height of hammer fall in cm

S = average penetration per blow in cm

C = effective elastic compression

$= C_1 + C_2 C_3$

C_1, C_2, C_3 = temporary elastic compression of Dolly and Pecking, the pile and the soil respectively

η_h = efficiency of hammer 65% for double acting steam hammer and 100% for drop hammer released by trigger

η_b = efficiency of hammer blow i.e, ratio of energy after impact to the striking energy of the ram

$$= \frac{W + e^2P}{W + P}, \text{ if } W > eP$$

$$= \frac{W + e^2P}{W + P} - \left[\frac{W - eP}{W + p}\right]^2 \text{ if } W < eP$$

where P = weight of the pile, anvil, helmet and the follower, and

e = coefficient of restitution as under

For the timber piles, $e = 0$

Double acting hammer for steel and RCC piles without driving cap or helmet but with proper packing on top

$$e = 0.5$$

Note : *The above equations are only applicable to friction piles. For end bearing piles, the value of P is reduced to half.*

Pile Load Test

The most reliable method of determining the capacity of individual pile is by pile load test. A test pile is installed using the procedure proposed for construction and it is loaded to a near failure condition.

After the pile has been installed, the load test is performed by applying an initial load to the top of the pile in increments. This is usually carried out by joining the top of pile against a dead load that has been positioned over the pile or by tension piles that have been driven adjacent to test pile. Friction piles driven into saturated clay should be tested only after sometime to allow the remoulded clay to regain strength.

The maximum load on test pile should not less than 2½ times the estimated design load and on a working pile it should not less than 1½ times. The safe load of the pile is assessed from the results of load versus settlement behaviour and by taking the smallest of the following.

(*i*) Two third of the final load which gives a net settlement value of 6 mm.

(*ii*) One half to two thirds of the load which gives a net settlement value of 6 mm.

(*iii*) Half of the final load at which the total settlement equal to one tenth of the pile diameter.

Penetration Test

Mayerhof proposed the following equations to compute the bearing capacity for displacement piles using SPT and cone resistance values

Skin resistance, $q_f = 2N$ kN/m² (SPT)

where N – SPT value

Skin resistance, $q_f = \frac{q_c}{2}$ kN/m²,

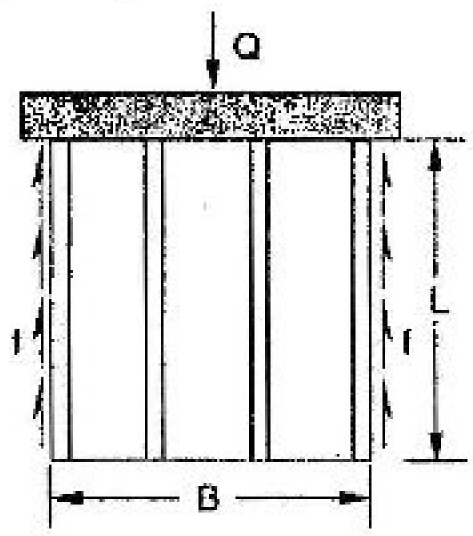
where q_c = cone resistance

$Q = q_e A_p + q_f A_s$

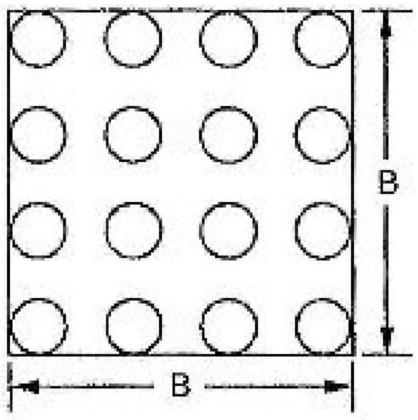
where, A_p = area of pile tip, and

A_s = area of contact surface of pile shaft

Capacity of pile groups. Piles are driven generally in groups in regular pattern to support the structural loads. The structural load applied to the pile that distributes the load to individual piles. If piles are spaced sufficient distance apart, then the capacity of pile group is the sum of the individual capacities of piles. However if the spacing between the piles is too close, the zones of stress around the pile will overlap and the ultimate load of group is less than the sum of the individual pile capacities.



Group action is evaluated by considering the piles to fail as a unit around the perimeter of the group. Capacity in end bearing pile is evaluated by considering the area enclosed by the perimeters of piles as the area of footing located at a depth corresponding to the elevation of pile tips. The friction component of pile support is evaluated by considering the friction that can be mobilised around the perimeter of the pile group over the length of the piles.



Ultimate capacity, $Q = q_o B^2 + 4.\ BL\, f$ (square)

where q_o = ultimate bearing pressure of footing of area B^2

B = size of pile group

L = length of pile,

and f = shear resistance

Efficiency of pile group

It depends upon the following factors :

(*i*) Spacing of piles

(*ii*) Total number of piles in a row and number of rows in a group, and

(*iii*) Characteristic of pile (material, diameter and length)

Negative Skin Friction

When the piles are driven through compressible soils before consolidation is complete or the site has newly placed fill the compressible soil will move downward relative to the pile. The downward movement of soil develop skin friction between the pile and surrounding soil and it is termed negative skin friction. Negative skin friction can be developed from lowering of water level in compressible soils such as soft clay, peat, mud and soft soil and also due to increase in stress by some means (e.g. filling).

To compute negative skin friction on group of piles, the minimum value from the following equation should be used.

(*i*) The negative skin friction

It is the sum of individual piles,

$$Q_n = nF_n$$

where n = number of piles in a group, and

F_n = negative skin friction on each pile

$= SpL$ for cohesive soils

$= \frac{1}{2}L^2 pK_o\, \gamma f$ for granular soils

p = perimeter of the pile,

and L = depth of fill

S = αc (α reduction factor, and c cohesion)

K_o = earth pressure coefficient

$= (1 - \sin\phi)$

γ = unit weight of fill

f = coefficient of friction

$= \tan\phi$

ϕ = angle of friction between pile and soil

(*ii*) Block skin resistance

When the piles are placed close to each other, the negative skin resistance may act effectively on block perimeter of pile group.

$$Q_n = SLp + \gamma LA$$

where S = shear resistance of soil

L = depth of fill

p = perimeter of pile group

γ = unit weight of soil

A = area of pile group enclosed in perimeter p

Well Foundations. A typical section of a well foundation is shown in the figure below.

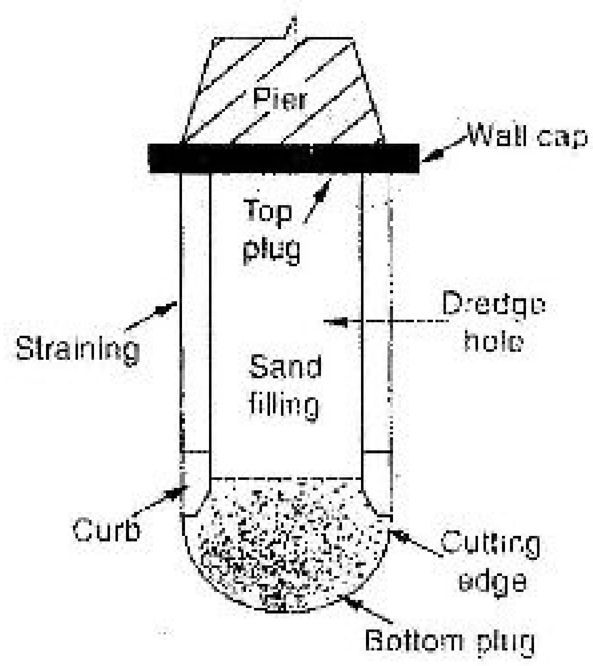


Fig. A well foundation

Main components of a well foundation are

1. Well cap
2. Top plug
3. Steining
4. Bottom plug
5. Curb
6. Cutting edge

Depth of well foundation

It is based on the following two criterion.

(*i*) The well should be taken below the scour depth called the grip length.

(*ii*) The well should be sunk to sufficient depth so as to satisfy the requirement of bearing capacity.

The *grip length* is calculated from the scour depth. The depth of scour can be determined using Lacey's formula

$$R_L = 1.35\left(\frac{q^2}{f}\right)^{1/3}$$

where q = discharge in cumecs per linear metre of waterway

f = Lacey's silt factor $\left(1.67\sqrt{d_{mm}}\right)$

Maximum depth of scour,

$$R = 2\,R_L$$

$$\text{Scour level} = \text{H.F.L.} - R_L$$

where H.F.L. = Highest flood level

$$\text{Grip length} = \frac{1}{3}R_L\,(\text{IRC code})$$

$$= 1\frac{1}{2}R_L\,(\text{Railway Practice})$$

$\therefore$ Minimum depth of foundation

$$= 1\frac{1}{3}R_L \text{ below H.F.L. (IRC code)}$$

$$= 1\frac{1}{2}R_L \text{ (Railway Practice)}$$

Minimum depth of embedment below the scour level should not be less than 2.0 m for piers and abutments with arches and 1.2 m for piers and abutments supporting other types of structures.

Bearing Capacity.

According to Terzaghi and Peck, the ultimate bearing capacity can be determined from the equation.

$$Q_f = Q_p + Q_s$$

where

$$Q_s = 2\,\pi\,R\,f_s\,D_f$$

$$Q_p = \pi\,R^2\left(1.2\,CN_c + \gamma D_f N_q + 0.5\,\gamma\,RN_\gamma\right)$$

C = cohesion acting along the surface

f_s = average skin friction

R = radius of the well

D_f = depth of well

N_c, N_q, N_q = bearing capacity factors.

Example. *A single acting steam hammer weighing 2500 kg and falling through a height of 1.2 m drives a pile to an average penetration of 0.85 cm under the last few blows. Find the allowable load on the pile.*

Solution : According to Engineering News formula, the load carrying capacity of the pile is given by

$$Q_a = \frac{WH}{6(S+0.25)}$$

$$= \frac{2500 \times 120}{6(0.85+0.25)}$$

$$= \mathbf{45454.55\ kg}$$

Example. *A wooden pile is driven by a drop hammer weighing 1000 kg and having a free fall of 3 mm. The final set is 1.2 cm. Determine allowable load, using the Engineering News formula.*

Solution : Given, W = 1000 kg,

$$H = 3 \text{ m} = 300 \text{ cm},$$

$$S = 1.2 \text{ cm}$$

According to Engineering News formula, the allowable load is given by

$$Q_a = \frac{WH}{6(S+2.5)}$$

$$= \frac{1000 \times 300}{6(1.2+2.5)} = \mathbf{13513.15\ kg}$$

Example. *A single acting steam hammer weighing 2400 kg and falling through a height of 1.0 m drives a pile to an average penetration of 0.75 cm under the last few blows. Determine allowable load for the pile.*

Solution :

Given, W = 2400 kg;

H = 100 cm;

S = 0.75 cm

According to Engineering News formula

$$Q = \frac{WH}{6(S+C)}$$

$$= \frac{2400 \times 100}{6(0.75+0.25)}$$

$$= \mathbf{40,000\ kg}$$

As single acting hammer is used,

$$C = 0.25$$

Example. *A pile of 0.25 m × 0.25 m cross-sectional area is penetrated to soft soil having C = 0.75 kg/cm² for a length of 15 metres and finally rests on hard soil. Find its load carrying capacity by skin friction.*

Solution :

Load carrying capacity of the pile (by skin friction),

$= \pi\,d_a\,L \times C_a$ — for circular piles

$= 4\,d_a\,L \times C_a$ — for square piles

where d_o = side of a square pile and diameter of a circular pile

$C_a = m \times c$

m = 0.4 for still clay and 1.0 for soft clay

$$\therefore \quad Q = 4d_a\,LC_a$$

$$= 4 \times 0.25 \times 15 \times 1 \times \frac{0.75 \times 100 \times 100}{1000}$$

= **112.5 tones**

Hence, load carrying capacity of the pile by skin friction = **112.5 tonnes**

EXERCISE-I

1. The effect or cohesion on a soil is to

(*a*) reduce both the active earth pressure intensity and passive earth pressure intensity

(*b*) increase both the active earth pressure intensity and passive earth pressure intensity

(*c*) reduce the active earth pressure intensity but to increase the passive earth pressure intensity

(*d*) increase the active earth pressure intensity but to reduce the passive earth pressure intensity

2. The radius of friction circle or ϕ-circle in friction circle method is

(*a*) r (*b*) $r \sin \phi$

(*c*) $r \cos \phi$ (*d*) $r \tan \phi$

where r is the radius of failure arc.

3. A shallow foundation is defined as a foundation which

(*a*) has low bearing capacity

(*b*) has a depth of embedment less than its width

(*c*) is resting on the ground surface

(*d*) causes less settlement

4. For an anisotropic soil, permeabilities in x and y directions are K_x and K_y respectively in a two dimensional flow. The effective permeability K_{eq} for the soil is given by :

(*a*) $K_x + K_y$ (*b*) K_x/K_y

(*c*) $\left(K_x^2 + K_y^2\right)^{1/2}$ (*d*) $(K_x K_y)^{1/2}$

5. The description of 'sandy ilty clay' signifies that

(*a*) the soil contains unequal proportions of the three constituents, in the order sand > silt > clay

(*b*) the soil contains equal proportions of sand, silt and clay

(*c*) the soil contains unequal proportions of sand, silt and clay.

(*d*) the soil contains unequal proportions of the three constituents such that clay > silt > sand.

6. A soil having particles of nearly the same size is known as

(*a*) well graded (*b*) uniformly graded

(*c*) poorly graded (*d*) gap graded

7. The soils most susceptible to liquefaction are

(*a*) saturated dense sands

(*b*) saturated fine and medium sands of uniform particle size

(*c*) saturated clays of uniform size

(*d*) saturated gravels and cobbles

8. Skempton's pore pressure coefficient B for saturated soil is

(*a*) 1 (*b*) zero

(*c*) between 0 and 1 (*d*) greater than 1

9. The slope of the e-log p curve for a soil mass gives

(*a*) coefficient of permeability, k

(*b*) coefficient of consolidation, C_v

(*c*) compression index, C_c

(*d*) coefficient of volume compressibility, m.

10. If r = frequency ratio, D = damping ratio, then magnification factor in a damped forced vibration is equal to :

(*a*) $\dfrac{1}{\sqrt{\left(1-r^2\right)^2 + 4r^2D^2}}$ (*b*) $\dfrac{1}{\sqrt{\left(1-r^2\right)^2 + 4\,Dr}}$

(*c*) $\dfrac{1}{\sqrt{\left(1-r^2\right)^2 - 4r^2\,D^2}}$ (*d*) $\dfrac{1}{\sqrt{\left(1+r^2\right)^2 - 4r^2\,D^2}}$

11. Degree of freedom of a block type machine foundation is

(*a*) 2 (*b*) 3

(*c*) 4 (*d*) 6

12. Given that damping ratio = 0.10 and damping coefficient = 225 kN sec/m. Then the critical damping coefficient in kN sec/m will be

(*a*) 22.5 (*b*) 225

(*c*) 2250 (*d*) 22500

13. In case of free over damped vibration,

(*a*) $C = \sqrt{k.m}$ (*b*) $C = 2\sqrt{k.m}$

(*c*) $C > 2\sqrt{k.m}$ (*d*) $C < 2\sqrt{k.m}$

14. The correct increasing order of the surface areas of the given soils is

(*a*) silt, sand, colloids, clay

(*b*) sand, silt, colloids, clay

(*c*) sand, silt, clay, colloids

(*d*) clay, silt, sand, colloids

15. In a standard Proctor compaction, the water content (w) and maximum dry density (γ_{dmax}) are related as :

(*a*) γ_{dmax} is linearly proportional to w

(*b*) w is inversely proportional to γ_{dmax}

(*c*) γ_{dmax} corresponds to a unique value of w

(*d*) γ_{dmax} corresponds to $w = (w_p + w_L)/2$ where w_p and w_L are respectively plastic and liquid limits.

16. Using Mohr's diagram, the relation between major principal stress, σ_1 and minor principal stress σ_3, and shear parameters c and ϕ is given by

$$\sigma_1 = \sigma_3 N_\phi + 2c\sqrt{N_\phi}$$

where N_ϕ is equal to

(*a*) $\frac{\sin\phi}{(1+\sin\phi)}$ (*b*) $\frac{\sin\phi}{(1-\sin\phi)}$

(*c*) $\frac{(1-\sin\phi)}{(1+\sin\phi)}$ (*d*) $\frac{(1+\sin\phi)}{(1-\sin\phi)}$

17. The total settlement of a compressible soil stratum 2 m deep and having a coefficient of volume compressibility of 0.02 cm^2 /kg under a pressure increment of 2 kg/cm^2 will be

(*a*) 2 cm (*b*) 4 cm

(*c*) 8 cm (*d*) 10 cm

18. Given that for a single degree of freedom system,

k = stiffness coefficient

m = mass of machine and foundation, critical damping is best defined by the expression

(*a*) $2\sqrt{km}$ (*b*) $2k\sqrt{m}$

(*c*) $2\pi k\sqrt{\frac{1}{m}}$ (*d*) $\frac{1}{2\pi}\sqrt{\frac{k}{m}}$

19. Which one of the following statements provides the best argument that direct shear tests are not suited for determining shear parameters of a clay soil?

(*a*) Failure plane is not the weakest plane.

(*b*) Pore pressure developed cannot be measured.

(*c*) Satisfactory strain levels cannot be maintained

(*d*) Adequate consolidation cannot be ensured.

20. Two specimens of clay A and B are tested in a consolidated apparatus.

If $(m_v)_A = 3.6 \times 10^{-4}$ m^2/kN and
$(m_v)_B = 18 \times 10^{-4}$ m^2/kN,
$(C_v)_A = 3.8 \times 10^{-4}$ cm^2/s,
$(C_v)_B = 1.9 \times 10^{-4}$ cm^2/s,

then the ratio k_A/k_B is equal to

(*a*) 0.0625 (*b*) 0.25

(*c*) 1.0 (*d*) 4.0

21. Terzaghi's equation of ultimate bearing capacity for a strip footing may be used for square footing resting on pure clay soil with the correction factor

(*a*) 0.4 (*b*) 0.6

(*c*) 1.2 (*d*) 1.3

22. A building is supported on shallow foundation in sand at 1 m below ground level. The water table is at 5 m below the ground surface. For which one of the following foundations will the net bearing capacity of the soil be a maximum?

(*a*) 2 *m* wide strip footing

(*b*) 2 *m* × 2 *m* square footing

(*c*) 2 *m* diameter circular footing

(*d*) 4 *m* × 1 *m* rectangular footing

23. The determination of ultimate bearing capacity on an eccentrically loaded square footing depends upon the concept of useful

(*a*) square (*b*) width

(*c*) triangle (*d*) circle

24. The upstream slope of an earth dam under steady seepage condition is

(*a*) equipotential line (*b*) phreatic line

(*c*) flow-line (*d*) seepage line

25. A cohesionless soil having an angle of shearing resistance of ϕ is standing at a slope angle of i. The factor of safety of the slope is

(*a*) $\frac{\tan i}{\tan\phi}$ (*b*) $\tan i - \tan\phi$

(*c*) $\frac{\tan\phi}{\tan i}$ (*d*) $\tan\phi - \tan i$

26. Match List I with List II and select the correct answer using the codes given below the lists :

List I (*Cause*)

A. Water present in the soil above water table

B. Upward seepage flow

C. Downward seepage flow

D. Fluctuation of water level above ground level

List II (*Effect*)

1. Increase in effective stress
2. No change in effective stress
3. Water is in a state of tension
4. Decrease in effective stress

Codes :

	A	B	C	D
(*a*)	3	4	1	2
(*b*)	3	2	1	4
(*c*)	2	3	1	4
(*d*)	1	4	3	2

27. In consolidation testing, curve fitting method is used to determine

(*a*) compression index

(*b*) swelling index

(*c*) coefficient of consolidation

(*d*) time factor

28. Westergaard's analysis for stress distribution beneath loaded areas is applicable to

(*a*) sandy soils (*b*) clayey soils
(*c*) stratified soils (*d*) silty soils

29. A square footing is to be proportioned on a cohesionless soil with an average N value of 40. The allowable bearing pressure of this footing will be governed by

(*a*) general shear failure (*b*) local shear failure
(*c*) progressive failure (*d*) settlement criteria

30. According to Skempton's formula for a surface footing of square shape, the net ultimate bearing capacity on a purely cohesive soil of cohesion c is

(*a*) 1.4 *c* (*b*) 6.0 *c*
(*c*) 7.4 *c* (*d*) 9.0 *c*

31. Undisturbed soil samples are required for conducting

(*a*) hydrometer test (*b*) shrinkage limit test
(*c*) consolidation test (*d*) specific gravity test

32. Soil pressure distribution below a rigid footing on the surface of a cohesive soil is

(*a*) maximum at the centre and minimum at edges
(*b*) minimum at the centre and maximum at edges
(*c*) uniform throughout
(*d*) maximum at one end and minimum at the other end

33. Compression index on a soil helps to determine

(*a*) total time required for consolidation
(*b*) time required for 50 percent consolidation
(*c*) total settlement of clay layer
(*d*) pro-consolidation pressure of clay

34. According to Bousinesq's theory, the vertical stress at a point in a semi-infinite soil mass depends upon

(*a*) point load, coordinates of the point and modulus of elasticity of soil
(*b*) point load, coordinates of the point, modulus, of elasticity of soil and its Poisson's ratio
(*c*) point load and coordinates of the point
(*d*) point load, coordinates of the point, modulus of elasticity of soil and its density

35. By using sieve analysis, the particle site distribution curve has been plotted for a particular soil. The coefficient of curvature C_c is given by

(*a*) $\dfrac{D_{30}}{D_{30} \times D_{10}}$ (*b*) $\dfrac{\sqrt{D_{30}}}{D_{60} \times D_{10}}$

(*c*) $\dfrac{D_{30}}{\sqrt{D_{60} \times D_{10}}}$ (*d*) $\dfrac{D_{30}^2}{D_{60} \times D_{10}}$

36. Consider the following factors pertaining to flow through soil :

1. Hydraulic gradient
2. Grain size
3. Void ratio
4. Cross-sectional area of the sample

Of these, the factors affecting permeability include

(*a*) 1 and 4 (*b*) 2 and 3
(*c*) 1, 2 and (*d*) 2, 3 and 4

37. For sampling saturated sands and other soft and wet soils satisfactorily, the most suitable soil sampler is

(*a*) Open drive thin-walled tube sampler
(*b*) standard split-spoon sampler
(*c*) stationary piston sampler
(*d*) rotary sampler

38. A vane shear test on a soil sample gives moment of total resistance M. The shear stress failure, 'S' being more or less uniform at top, bottom and surface of cylinder of soil, is given by (where H = height of vane, D = Diameter of vane)

(*a*) $S = \dfrac{2M}{\pi D^2 H}$ (*b*) $S = \dfrac{2M}{\pi D^2 (H + D)}$

(*c*) $S = \dfrac{2M}{\pi D^2 \left(H + \dfrac{D}{3}\right)}$ (*d*) $S = \dfrac{2M}{\pi DH}$

39. If the settlement of a single pile in sand is denoted by S and that of a group of N identical piles (each pile carrying the same load) by Sg, then the ratio Sg/S will

(*a*) be equal to 1 irrespective of width of the group
(*b*) be equal to N irrespective of width of the group
(*c*) decrease as the width of the group increases
(*d*) increase as the width of the group increases

40. Match List I with List II and select the correct answer using the codes given below the lists :

List I (*Field test*)

A. Vane shear test
B. Standard penetration test
C. Static cone penetration test
D. Pressure meter test

List II (*Useful for*)

1. End bearing and skin friction resistance
2. In-situ stress-strain characteristics
3. Soft clay
4. Sandy deposits

Codes ;

	A	B	C	D
(*a*)	4	2	1	3
(*b*)	3	4	1	2
(*c*)	4	3	2	1
(*d*)	3	4	2	1

41. Consider the following statements regarding settlement of foundations:

1. Differential settlement of foundation leads to structural damage to the superstructure
2. In non-cohesive soils, the major components of settlement is due to consolidation
3. Lowering of ground water table contributes to settlement of foundations.

Of these statements

(*a*) 1 and 2 are correct (*b*) 1 and 3 are correct
(*c*) 2 and 3 are correct (*d*) 1, 2 and 3 are correct

42. Boring method is to be chosen depending upon the type of exploratory strata. In this context, match List I with List II and select the correct aswer using the codes given below the lists :

List I

A. Auger boring
B. Wash boring
C. Percussion drilling
D. Rotary drilling

List II

1. Partly saturated sands, silts and medium to stiff cohesive soils
2. All types of soils and rocks except in stony or porous soils and fissured rocks
3. Practically all types of soils except hard and cemented soil or rock
4. All types of soils and rocks. Difficult in loose sands and soft sticky clays.

Codes :

	A	B	C	D
(*a*)	1	4	3	2
(*b*)	1	3	4	2
(*c*)	2	4	3	1
(*d*)	2	3	4	1

43. Consider the following statements: Clays which exhibit high activity

1. contain montmorittonite.
2. contain kaolinite,
3. have a high silt content.
4. have a high plasticity index.
5. have a low plasticity index.

Of these statements

(*a*) 1, 3 and 5 are correct (*b*) 2, 3 and 5 are correct
(*c*) 2 and 4 are correct (*d*) 1 and 4 are correct

44. A sample of clay and a sample of sand have the same specific gravity and void ratio. Their permeabilities would differ because

(*a*) their porosities would be different
(*b*) their degrees of saturation would be different
(*c*) their densities would be different
(*d*) the size ranges of their voids would be different

45. Consider the following limitations:

1. Can be performed only on purely cohesionless soils
2. Plane of failure is predetermined
3. There is virtually no control on drainage
4. Non-uniform distribution of stresses
5. Principle stresses in the sample cannot be determined.

The limitation inherent in direct shear test include :

(*a*) 1, 2 and 3 (*b*) 2, 3 and 4
(*c*) 3, 4 and 5 (*d*) 1, 2 and 5

46. Consider the following assumptions :

1. Failure occurs on a plane surface.
2. Wall is smooth but not necessarily vertical.
3. Failure wedge is a rigid body.

Coulomb's theory of earth pressure is based on assumptions

(*a*) 1, 2 and 3 (*b*) 1 and 2
(*c*) 1 and 3 (*d*) 2 and 3

47. In a saturated clay layer undergoing consolidation with single drainage at its top, the pore water pressure would be the maximum at its

(*a*) top
(*b*) middle
(*c*) bottom
(*d*) top as well as the bottom

48. Consider the following field tests :

1. Vertical pile load test
2. Cyclic pile load test
3. Lateral pile load test
4. Instrumented test pile

While estimating the load carrying capacity of a pile, the tests that can be used for separating the skin resistance from point resistance, would include

(*a*) 1 and 3 (*b*) 1 and 4
(*c*) 2 and 3 (*d*) 2 and 4

49. The foundation soil under the toe of a dam has a void ratio '*e*'. The specific gravity of the soil solid is G, Factor of safety against piping is to be taken as 2.5. The maximum permissible upward exit gradient is given by

(*a*) $i = 2.5\left[\frac{G-1}{1+e}\right]$ (*b*) $i = 2.5\left[\frac{1-e}{G-1}\right]$

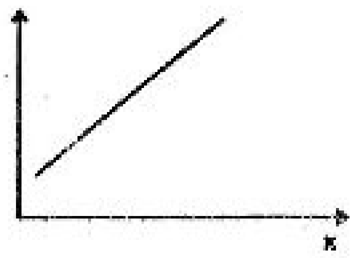
(*c*) $i = 0.4\left[\frac{1+e}{G-1}\right]$ (*d*) $i = 0.4\left[\frac{G-1}{1+e}\right]$

50. Which one of the following parameters can be used to estimate the angle of internal friction of a sandy soil?

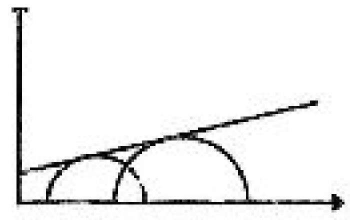
(*a*) particle size
(*b*) roughness of particle
(*c*) particle size distribution
(*d*) density index

51. Which one of the following diagrams below correctly illustrates the Mohr 's stress conditions of unconfined shear test on cohesive soil? (x-axis Normal stress; y-axis Shear stress)

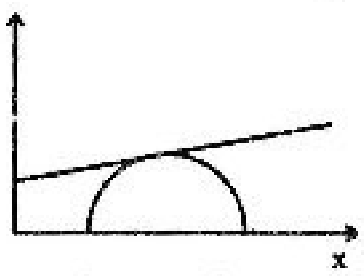
(*a*)



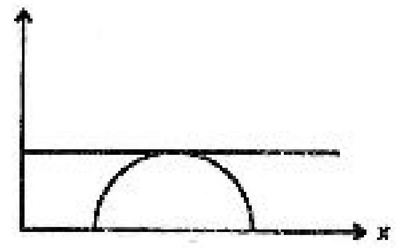
(*b*)



(*c*)



(*d*)



52. A cantilever sheet pile derives its stability from

(*a*) lateral resistance of soil

(*b*) self-weight

(*c*) the deadman

(*d*) the anchor rod

53. Deflection of a sheet pile in a braced cut

(*a*) increases from top to bottom

(*b*) decreases from top to bottom

(*c*) increases from top and then decreases

(*d*) decreases from top and then increases

54. The time '*t*' required for attaining a certain degree of consolidation of a clay layer is proportional to

(*a*) H^2 and C_V (*b*) H^2 and $\frac{1}{C_V}$

(*c*) $\frac{1}{H^2}$ and C_V (*d*) $\frac{1}{H^2}$ and $\frac{1}{C_V}$

55. Match List I with List II and select the correct answer using the codes given below the lists :

List I

A. Elastic settlement

B. Primary consolidation

C. Secondary consolidation

D. Creep

List II

1. Constant effective stress with change in volume of soil
2. Dissipation of excess pore water pressure
3. Occurs within a short period
4. Compression and rearrangement of particles

Codes :

	A	B	C	D
(*a*)	3	2	1	4
(*b*)	4	3	1	2
(*c*)	3	2	4	1
(*d*)	4	3	2	1

56. Given that for an over consolidated clay soil deposit, the pressure under which the deposit has been fully consolidated in the past is 125 kN/m^2 and the present overburden pressure is 75 kN/m^2, the over consolidation ratio of the soil deposit is

(*a*) $\frac{75}{125}$ (*b*) $\frac{50}{75}$

(*c*) $\frac{125}{75}$ (*d*) $\frac{200}{75}$

57. Match List I with List II and select the correct answer using the codes given below the lists :

List I

A. Stress distribution due to point load in homogeneous isotropic medium

B. Stress distribution due to point load in an anisotopic medium

C. Influence chart for stress distribution in a rectangular area

D. Influence chart for stress distribution in irregularly shaped areas

List II

1. Stein Brenner
2. Newmark
3. Boussinesq
4. Westergaard

Codes :

	A	B	C	D
(*a*)	4	3	2	1
(*b*)	3	4	2	1
(*c*)	3	4	1	2
(*d*)	4	3	1	2

58. The static cone penetration test and a standard penetration test are performed on a soil at a certain depth. The value of static cone penetration test is 8 MPa and the N value is 20. The soil met with at that depth is

(*a*) sandy silt

(*b*) clay-silt mixture

(*c*) sand and gravel mixture

(*d*) medium dence sand

59. Match List I with List II and select the correct answer using the codes given below the Lists :

List I

A. Optimum moisture content

B. Vibratory rollers

C. Zero air void line

List II

1. Compaction of cohesive soil
2. Compaction of granular soil
3. Maximum dry density
4. Relative density
5. 100% saturation