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SECTION-A (SOIL MECHANICS)

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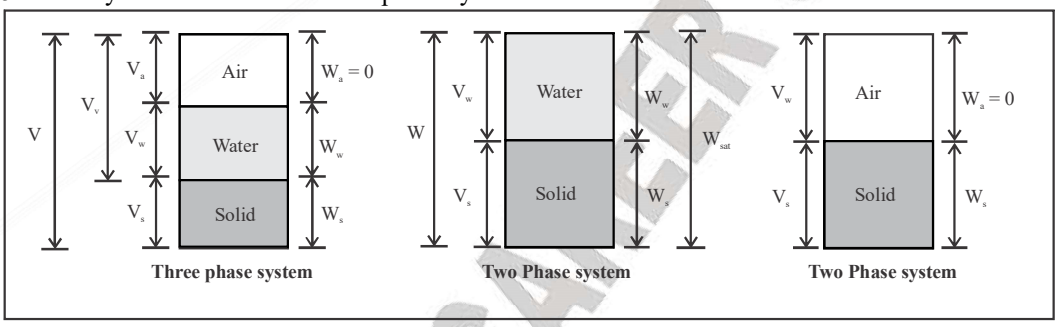
SECTION-A

SOIL MECHANICS

CHAPTER - 1
SOIL WATER RELATIONSHIP

1.1 SOIL WATER RELATIONSHIP

1. A soil mass consists of solid particles which form a porous structure. The voids in the soil mass may be filled with air, with water air and partly with water. The three constituents are blended together to form a complex material. However, for convenience, all the solid particles are segregated and placed in the lower layers of the three phase diagram. Likewise, water and air particles are placed separately, as shown. The 3-phase diagram is also known as Block diagram.
2. It may be noted that the three constituents cannot be actually segregated, as shown. A 3-phase diagram is an artifice used for easy understanding and convenience in calculation.
3. Soil can be either two phase or three phase composition.
4. Fully saturated soil and fully dry soil are two phase system.
5. Partially saturated soils are three phase system.



Where,
V_a is Volume of Air
V_w is Volume of water
V_s is Volume of solid
V is Total Volume of Soil mass
V_v is Volume of Voids i.e. sum of volume of air and volume of water.
W_a is Weight of air i.e. equals to zero.
W_w is Weight of water
W_s is Weight of Soil solids
W is Weight of Soil mass
W_{sat} is Saturated weight of soil mass

1.2 IMPORTANT DEFINITIONS

1.2.1 Water Content (w)

$$w = \frac{W_w}{W_s}; w \geq 0$$

1. Water content or moisture content of a soil mass is defined as the ratio of weight of water to the weight of solids (dry weight) of the soil mass $w = \frac{W_w}{W_s} \times 100$

CHAPTER - 2
SOIL COMPACTION

2.1 INTRODUCTION

1. Compaction of soil is the process of increasing the unit wt of soil by forcing the soil solids into a dense state and reducing the air voids.
2. Compaction leads to increase in shear strength and helps improve the stability and bearing capacity of soil. It also reduces the compressibility and permeability of soil.
3. This is achieved by applying static or dynamic loads to the soil.
4. Compaction is measured quantitatively-in terms of dry unit wt (γ_d) of the soil.
5. Difference between compaction and consolidation are as tabulated below.

Compaction	Consolidation
1. Instantaneous phenomenon	1. Time dependent Phenomenon
2. Soil always partially saturated air dry	2. Soil is completely Saturated
3. Densification due to reduction in the volume of air voids at a given water content	3. Volume reduction is due to expulsion of pore water from voids.
4. Specific compaction techniques are used	4. Consolidation occurs on account of a static load placed on the soil.

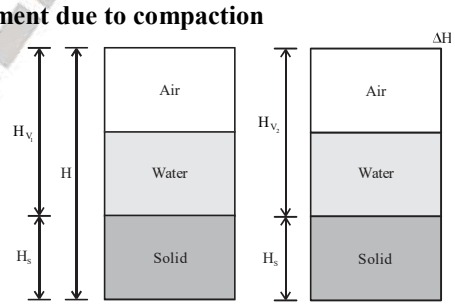
2.1.1 Why do we need to compact soil

1. Max shear strength occurs at min. void ratio.
2. Large air voids if left, may lead to compaction under working loads causing settlement of the structure during service or may get filled with water which reduces the shear strength.
3. Increase in water content is also accompanied by swelling and loss of shear strength with time.

2.1.2 The various advantages of compaction are

1. Settlement can be reduced or prevented.
2. Soil strength increases and stability can be improved.
3. Load carrying capacity of the pavement subgrade can be improved.
4. Undesirable volume changes (by frost action, swelling shrinkage) may be controlled.

2.1.3 General formula for settlement due to compaction



$H_{V1} = e_0 H_s$;

Where e_0 is initial void ratio

$H_{v_2} = e_f H_s$;

CHAPTER - 3
PRINCIPLE OF EFFECTIVE STRESS AND PERMEABILITY

3.1 TOTAL STRESS, PORE WATER PRESSURE AND EFFECTIVE STRESS

3.1.1 Total stress

Total stress (σ) on a plane with in a soil mass is the force per unit area of soil mass transmitted in normal direction across a plane.

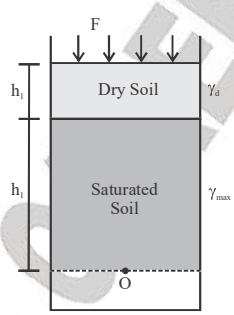
$\therefore$ Total stress (σ) = $\dfrac{P}{A}$

Where, P is Force on plane X –X for weight above plane X – X

A is Area of cross section of soil mass

When there is imposed load on soil mass as shown below, the total stress value at point O is given by

Total stress at 'O' = $\dfrac{F + \gamma_d \cdot A \cdot h_1 + \gamma_{sat} \cdot Ah_2}{A_3}$

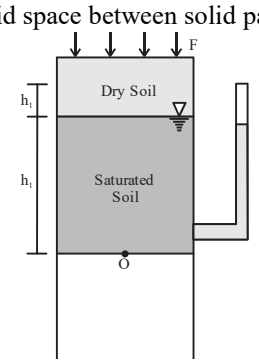


$$\sigma = \frac{F}{A} + \gamma_d h_1 + \gamma_{sat} h_2$$

Total stress is a physical parameter which can be measured by suitable arrangement, such as by **pressure cell**.

3.1.2 Pore Water Pressure (u)

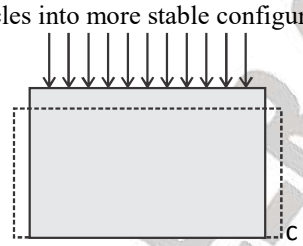
1. It is the Pressures of water filling void space between solid particles.



CHAPTER - 4
CONSOLIDATION

4.1 INTRODUCTION

1. When a soil mass is subjected to a compressive force, like all other materials, its volume decrease.
2. The property of the soil due to which a decrease in volume occurs under compressive forces is known as the compressibility of soil.
3. The decrease in volume of soil, under stress is because of
(i) Compression and Expulsion of Pore-Air.
(ii) Compression and Expulsion of Pore Water.
(iii) Gradual readjustment of clay particles into more stable configuration.



4. Reduction in volume of soil mass results in change of lateral and vertical dimensions of soil mass.
5. As soil being infinitely large in the lateral direction, hence the change in dimension in this direction is considered to be negligible, but there is significant change in vertical direction which is termed as settlement of soil.
6. In other words settlement of soil is the gradual sinking of the structure due to compression of the soil below.
7. Total settlement of soil is expressed as three components.

$$S_t = S_{immidtate} + S_{1^0\ consolidation} + S_{2^0\ consolidation}$$

4.1.1 Immediate settlement

1. If the soil is initially partially saturated, expulsion of air as well as compression of pore air may take place with the application of external loads which is called Initial Compression. It is a. immediate phenomenon.

2. After the initial compression, soil reaches into fully saturated state, further reduction in volume occurs due to expulsion of pore water i.e. water present in the soils.

3. Immediate settlement can also occur if significant lateral strain takes place. This is due to deformation of soil under undrained condition. This immediate settlement can be calculated from elastic theory.

4.1.2 Primary Consolidation

1. Primary-consolidation occurs due to expulsion of excess pore water pressure generated due to increase total stress. It is a time dependent phenomenon.

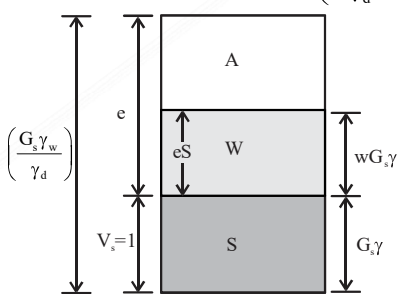
WORKBOOK

Example 1. Prove that $S = \dfrac{w}{\dfrac{\gamma_w}{\gamma_t}(1+w) - \dfrac{1}{G_s}}$.

Solution.

Proof We have $S = \left(\dfrac{V_w}{V_V}\right)$

$$= \left(\frac{V_w}{V - V_S}\right) \qquad = \frac{\left(\dfrac{wG_s\gamma_w}{\gamma_w}\right)}{\left(\dfrac{G_s\gamma_w}{\gamma_d} - 1\right)}$$



$$\gamma_t = \frac{wG_s}{G_s\left[\dfrac{\gamma_w}{y_d} - \dfrac{1.0}{G_s}\right]} = \frac{w}{\gamma_w(1+w) - 1}G_s$$

$$\left[\because \gamma_d = \frac{\gamma_t}{1+w}\right]$$

$$\therefore S = \frac{w}{\dfrac{\gamma_w}{\gamma_t}(1+w) - \dfrac{1}{G_s}}$$

Example 2. A sampler with a volume of 45 cm^3 is filled with a soil sample When the soil is poured into a graduated cylinder, it displaces 25 cm^3 water. What is the porosity and void ratio of the soil.

Solution.

Here, Total volume of soil V = 45 cm^3

As we drop the soil in water, the solid particles will displace water.

Hence, volume of soil $V_s = 25cm^3$

$$V_v = V - V_s = 45 - 25 = 20$$

$$e = \frac{V_v}{V_a} = \frac{20}{25} = 0.80$$

$$n = \frac{V_v}{V} = \frac{20}{45} = 0.444$$

Example 3. The void-ratio and specific gravity of a sample of clay are 0.73 and 2.7 respectively. If the voids are 92% saturated, find the bulk density, the dry density and the water content.

What would be the water content for complete saturation, the void ratio remaining the same?

Solution.

e = 0.73, G = 2.7, S = 92%

S.e = w.G.

$$w = \frac{0.92 \times 0.73}{2.7} \qquad w = 0.248$$

Bulk density γ_t

$$= \left(\frac{G + Se}{1+e}\right)\gamma_w = \left(\frac{2.7 + 0.92 \times 0.73}{1 + 0.73}\right) \times 9.81$$

$$= 19.118 kN/m^3$$

Dry density

$$\gamma_d = \frac{\gamma_t}{1+w} = \frac{19.118}{1+0.248} = 15.318 kN/m^3$$

Water content for full saturation at same at same void ratio.

$\Rightarrow$S.e. = w.G

$$\Rightarrow \frac{1 \times 0.73}{2.7} = w \Rightarrow w = 0.$$

Example 4. In a Proctor compaction test, the soil specimen of one of the observation had a bulk density of 19 kN/m^3 with a moisture content of 15%. Find,

(a) Degree of saturation of the specimen if $G_S = 2.7$

GATE QUESTIONS

1. The percent reduction in the bearing capacity of a strip footing resting on sand under flooding condition (water level at the base of the footing) when compared to the situation where the water level is at a depth much greater than the width of footing, is approximately

[GATE - 2018]

(a) 0 (b) 25
(c) 50 (d) 100

2. In a shrinkage limit test, the volume and mass of a dry soil pat are found to be 50 cm^3 and 88 g respectively. The specific gravity of the soil slides is 2.71 and the density of water is 1 g/cc. The shrinkage limit (in % up to two decimal places) is _____.

[GATE - 2018]

3. The clay material, whose structural units are held together by potassium bond is

[GATE - 2018]

(a) Halloysite (b) Illite
(c) Kaolinite (d) Smectite

4. The laboratory tests on a soil sample yields the following results; natural moisture content = 18% liquid limit = 60%, plastic limit = 25%, percentage of clay sized fraction = 25%. The liquidity index and activity (as per the expression proposed by Skempton) of the soil, respectively, are

[GATE - 2017]

(a) –0.2 and 1.4 (b) 0.2 and 1.4
(c) –1.2 and 0.714 (d) 1.2 and 0.714

5. The porosity (n) and the degree of saturation (S) of a soil sample are 0.7 and 40%, respectively. In a 100 m^3 volume of the soil, the volume (expressed in m^3) of air is _____

[GATE - 2016]

6. A fine grained soil is found to be plastic in the water content range of 26-48%. As per Indian Standard Classification System, the soil is classified as

[GATE - 2016]

(a) CL (b) CH
(c) CL-ML (d) CI

7. A 588 cm^3 volume of moist sand weighs 1010 gm. Its dry weight is 918 gm and specific gravity of solids, G is 2.67. Assuming density of water as 1 gm/cm^3, the void ratio is _____.

[GATE - 2015]

8. If the water content of a fully saturated soil mass is 100%, the void ratio of the sample is

[GATE - 2015]

(a) Less than specific gravity of soil
(b) Equal to specific gravity of soil
(c) Greater than specific gravity of soil
(d) Independent of specific gravity of soil

9. An earth embankment is to be constructed with compacted cohesion less soil. The volume of the embankment is 5000 m^3 and the target dry unit weight is 16.2 kN/m^3. Three nearby sites (see figure below) have been indentified from where the required soil can be transported to the construction site. The void ratios (e) of different sites are shown in the figure. Assume the specific gravity of soil to be 2.7 for all three sites. If the cost of transportation per km is twice the cost of excavation per m^3 of borrow pits, which site would you choose as the most economic solution? (Use unit weight of water = 10 kN/m^3)

[GATE - 2015]

SOLUTIONS

Sol. 1. (c)
Visual examination should establish the colour, grain size, grain shapes of the coarse grained part of soil.
Dilatancy test is one of the test used in field to identify fine grained soil. In this test, a wet pat of soil is taken and shaken vigorously in the palm. Silt exhibits quick response and water appears on surface, where as clay shows no or slow response.

Sol. 2. (c)
n = 0.3
$G_s = 2.6$
w = 4.94%

$$e = \frac{n}{1-n} = \frac{0.3}{1-0.3} = \frac{3}{7}$$

So, $es = wG_s$

$$\frac{3}{7} \times s = \frac{4.94}{100} \times 2.6$$

$\Rightarrow S = 0.299$
$S \approx 30\%$

Sol. 3. (b)
w = 0.38
$G_s = 2.65$
S = 1
$es = wG_S$
$1.e = 0.38 \times 2.65 \Rightarrow e = 1.007$

$$\gamma_{sat} = \frac{(G_s + Se)\gamma_w}{1+e} = \left(\frac{2.65 + 1 \times 0.007}{1 + 1.007}\right) \times 9.81$$

$= 17.88\ kN/m^3$

Sol. 4. (d)
$V_1 = (1 + e_1)V_s$
$V_2 = (1 + e_2)V_s$

$$\frac{V_2}{V_1} = \frac{1+e_2}{1+e_1}$$

$$V_2 = \frac{1.7}{2.2} \times 30$$

$$V_2 = 23.18 \simeq 23.2 m^3$$

Sol. 5. (b)
$\gamma_d = 18\ kN/m^3$
w = 0.16
$G_s = 2.65$

$$\gamma_d = \frac{G_s \gamma_w}{1+e}$$

$$\Rightarrow 18 = \frac{2.65 \times 9.81}{1+e}$$

$\Rightarrow e = 0.444$
$e \times s = wG_s$
$\Rightarrow 0.444 \times s = 0.16 \times 2.65$
$\Rightarrow s = 0.9547$
s = 95.5%

Sol. 6. (d)
Shear strength of soils at liquid limit is approximately 2.7 kN/m2
The volume of soil do not change, when subjected to drying at water content below shrinkage limit.
Plastic limit is always lower than the liquid limits for any type of soil.

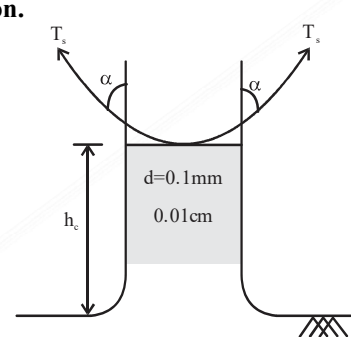
Sol. 7. (b)
Mass of soil + paraffin = 460 g
Mass of paraffin = 9g
Mass of soil = 451 g
Volume of soil + volume of paraffin = 300 cc
Volume of soil +
Volume of soil = 290 cc
Dry density of soil
e = 0.704

Sol. 8. (d)
(i) Illite is the mineral which has plasticity index and activity of intermediate level while montmorillonite is the mineral which is largely responsible for swelling and shrinkage behavior

WORKBOOK

Example 1. A capillary glass tube of 0.1 mm internal diameter is immersed vertically in a beaker full of water. Assume the tube to be perfectly clean and wet, determine the height of the capillary rise of water in the tube when the room t_{temp} is 20°C. Given at 20°C unit weight of water = 0.9980 gm/cc and surface tension = 72.8 dyne/cm.

Solution.



For equilibrium

$$(T_s \cos\alpha)\times \pi d = \left(\frac{\pi}{4} d^2 h_c \times \rho_w\right) g$$

$$\Rightarrow h_c = \frac{4T_s \cos\alpha}{(d\rho_w g)}$$

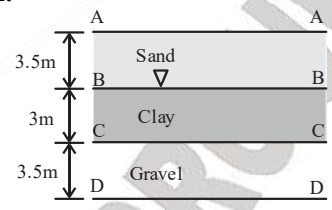
Assuming the tube to be perfectly clean and wet, cos α = 1

$$= \frac{4\times 72.8}{(0.998\times 0.01\times 981)}$$

[h_c = 29.74 cm]

Example 2. A soil profile consists of a surface layer of sand 3.5 m thick (ρ = 1.65 Mg/m^3), intermediate layer of caly 3 m thick (ρ = 1925 Mg/m^3) and the bottom layer of gravel 3.5 m thick (ρ = 1.925 Mg/m^3). The water table is at the upper surface of the clay layer. Determine the effective pressure at various level after placement of a surcharge load of 58.86kN/m^2 to the ground surface.

Solution.



At section A – A q = 58.86 kN/m^2, u = 0

σ = q = 58.86 kN/m^2

$$\bar{\sigma} = (\sigma - u) = (58.86 - 0) = 58.86 \text{kN/m}^2$$

At Section B – B

q = 58.86kN/m^2, ρ = 1.65 Mg/m^3 = 1.65 kg/m^3

σ = (q + γh) = (58.86 + 1.65 × 9.81 × 3.5)

= 115.51 kN/m^2

u = 0

$$\therefore \bar{\sigma} = (\sigma - u) = (115.51 - 0) = 115.51 \text{kN/m}^2$$

At Section C – C – q = 58.86 kN/m^2

ρ = 1.95 Mg/m^3 = 1.95 kg/m^3

σ = (q + γ_1 h_1 + γ_2 h_2)

= (58.86 + 1.65 × 9.81 × 3.5 + 1.95 × 3 × 9.81)

= 172.90 kN/m^2

u = hγ_w

= (3 × 9.81) = 29.43 kN/m^2

$$\therefore \bar{\sigma} = (\sigma - u) = (172.90 - 29.43)$$

= 143.47kN/m^2

At Section D – D q = 58.86 kN/m^2

ρ = 1.925M$_g$/m^3 = 1.925 kg/m

∴ σ = (q + γ_1 h_1 + γ_2 h_2 + $\gamma_3 h_3$)

= (58.86 + 1.65 × 9.81 × 3.5 +1.95 × 3 × 9.81) + 1.925 × 3.5 × 9.81)

= 238.99 kN/m^2

u = (3 + 3.5)γ_w = (6.5 × 9.81) = 63.765 kN/m^2

∴ $\bar{\sigma}$ (σ - u) = (238.99 – 63.765)

= 175.22 kN/m^2

CHAPTER - 5
SHEAR STRENGTH OF SOIL

5.1 INTRODUCTION

1. Shear strength is the capacity to resist shear stress.
2. If the value of shear stress on any plane or a surface at any point equals or exceeds the shear strength value, failure will occur in the soil mass because of the movement of a portion of soil mass along that particular plane or surface and soil is said to have failed in shear.
3. Thus shear strength is a very important property of soil which keeps it in a stable equilibrium, under type of loading which produces shear stress.
4. Shear strength of soil governs bearing capacity of soil, stability of slopes, Earth pressure retaining structure.

5.2 MECHANISM OF SHEAR RESISTANCE (SHEAR STRENGTH)

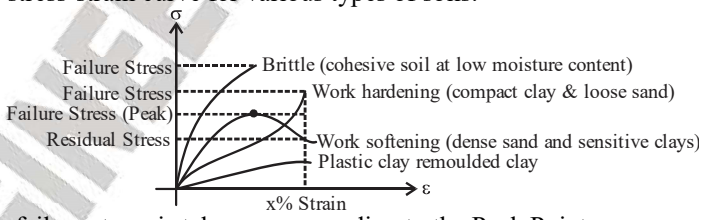
1. Shear strength is the resistance to shear deformation. It is categorized into two broad Categories.
(i) Frictional strength
(ii) Cohesive strength

2. Frictional strength takes into account the particle to particle friction and also the interlocking between particles.

3. Cohesive strength takes into account
(i) True cohesion .between particles.
(ii) Apparent cohesion between particles.

4. To calculate the shear strength of soil and take preventive measures such that soil mass does not fail in shear, we have to define state of failure upon stressing to certain:
(i) At what stress the failure will occur: By knowing the stress at failure, we can design the system such that, the failure stress does not generate
(ii) On what plane failure will occur: By knowing the orientation of potential failure plane we can take suitable strengthening measure to prevent failure on that plane.

5. To define the stress at which failure will occur, we use **stress-strain curve.** The following figure-shows the stress-strain curve for various types of soils.



6. For brittle soil, failure stress is taken corresponding to the Peak Point.

7. For work softening material, failure is taken at peak point.

8. For work hardening material or for plastic clay, failure stress shall be defined at some % of strain.

CHAPTER - 6
EARTH PRESSURE AND RETAINING WALLS

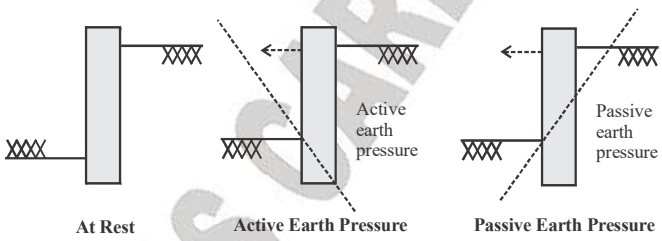
6.1 INTRODUCTION

1. Soil mass is stable when the slope of the surface of the soil mass is flatter than the safe slope. But at some places the space is limited, it is not possible to provide flat slope and the soil is to be retained at a slope steeper than the safe one.
2. Therefore to retain this soil mass in a stable state a retaining structure is provided to provide the lateral support to the soil mass.
3. In the design of these retaining structure it becomes imperative to know the magnitude and line of action of Earth pressure, where earth pressure is the lateral force exerted by the soil on any structure retaining that soil
4. The magnitude of the lateral earth pressure depends upon a number of factors such as the mode of movement of wall, the flexibility of the wall, the properties of the soil, and drainage conditions

6.2 TYPES OF LATERAL EARTH PRESSURE

Lateral earth pressure can be divided into 3 categories, depending upon the movement of retaining wall with respect to bask fill soil.

1. Earth Pressure **At Rest** – wall does not move at all
2. **Active** Earth Pressure – wall moves away from the backfill soil.
3. **Passive** Earth Pressure – wall moves towards the backfill soil.



6.2.1 Earth Pressure at Rest

1. A soil element in its natural state at any depth z below the ground surface is not subjected to any strain the element in this condition is known as at rest condition.
2. It is possible to evaluate earth pressure at rest using the theory of elasticity.
3. For analysis of earth pressure at rest, consider a soil mass element at depth z below the ground surface and following assumptions are made.

6.2.1.1 *Assumptions*

1. Soil mass is homogenous, isotropic and semi infinite.
2. Elastic modulus, E and Poission's ratio, μ is constant throughout the depth.
3. For Plane strain condition we can write ε_x as.

CHAPTER - 7
STABILITY OF SLOPES

7.1 INTRODUCTION

1. A slope in a soil mass is encountered when the elevation of the ground surface gradually charger from a lower level
to a higher one. Such a slope may be either natural (in hilly region) or man-made (in artificially constructed embankment or excavations).

2. The soil mass bounded by a slope has a tendency to slide down. The principal factor causing such a sliding failure is the self-weight of the soil. However, the failure may be aggravated due to seepage of water or seismic forces. Every man-made slope has to be properly designed to ascertain the safety of the slope against sliding failure.

3. Various methods are available for analyzing the stability of slopes. Generally these methods are based on the following assumptions:
(i) Any slope stability problem is a two-dimensional one.
(ii) The shear parameters of the soil are constant along any possible slip surface.

4. In problems involving seepage of water, the flow net can be constructed and the seepage forces can be determined.

7.2 STABILITY OF INFINITE SLOPES

1. In Fig. (a), X-X represents an infinite slope which is inclined to the horizontal at an angle β. On any plane YY (YY| |XX) at a depth z below the ground level the soil properties and the overburden pressure are constant; Hence, failure may occur along a plane parallel to the slope at some depth. The conditions for such a failure may be analysed by considering the equilibrium of the soil prism ABCD of width b.

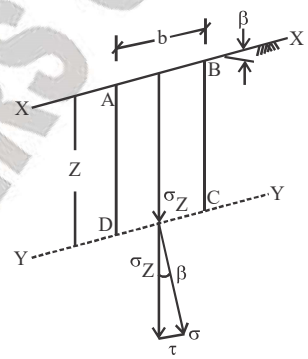


Fig (a)

Considering unit thickness, volume of the prism, V = z b cosβ
And, weight of the prism, W = γz b cosβ
Vertical stress on YY due to self-weight,

$$\sigma_z = \frac{W}{b} = \gamma z \cos\beta \qquad \ldots(i)$$

GATE QUESTIONS

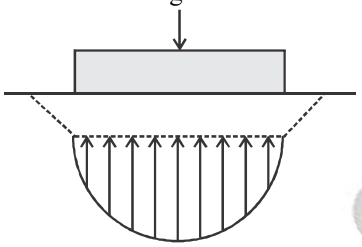
1. The width of a square footing and the diameter of a circular footing are equal. If both the footing are placed on the surface of sandy soil, the ratio of the ultimate bearing capacity of circular footing to that of square footing will be

[GATE - 2018]

(a) $\frac{4}{3}$ (b) 1

(c) $\frac{3}{4}$ (d) $\frac{2}{3}$

2. The contact pressure and settlement distribution for a footing are shown in the figure

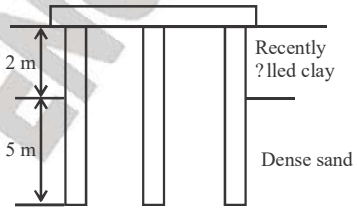


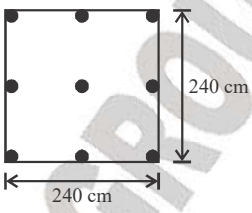
The figure corresponds to a

[GATE - 2018]

(a) Rigid footing on granular soil
(b) Flexible footing on granular soil
(c) Flexible footing on saturated clay
(d) Rigid footing on cohesive soil

3. A group of nine piles in a 3 × 3 square pattern is embedded in a soil strata comprising dense sand underlying recently filled clay layer, as shown in the figure. The perimeter of an individual pile is 126 cm. The size of pile group is 240 cm × 240 cm. The recently filled clay has undrained shear strength of 15 kPa and unit weight of 16 kN/m^3.





The negative frictional load (in kN, up to two decimal places) acting on the pile group is _____.

[GATE - 2018]

4. The old concert hall was demolished because of fears that the foundation would be affected by the construction of the new metro line in the area. Modern technology for underground metro construction tried to mitigate the impact of pressurized air pockets created by the excavation of large amounts of soil. But even with these safeguards, it was feared that the soil below the concert hall would not be stable.

From this, one can infer that

[GATE - 2017]

(a)The foundation of old buildings create pressurized air pockets underground, which are difficult to handle during metro construction.
(b)Metro construction has to be done carefully considering its impact on the foundations of existing buildings.
(c)Old buildings in an area form an impossible hurdle to metro construction in that area.
(d)Pressurized air can be used to excavate large amounts of soil from underground areas.

5. A 4m wide strip footing is founded at a depth of 1.5 m below the ground surface in a c-ϕ soil as shown in the figure. The water table is at a depth of 5.5 m below ground surface. The soil properties are: $c' = 35$ kN/m^2, $\phi' = 28.63o$, $\gamma_{sat} = 19$ kN/m^3, $\gamma_{bulk} = 17$ kN/m^3 and $\gamma_w = 9.81$ kN/m^3. The values of bearing capacity factors for different ϕ' are given below:

SECTION-B

FOUNDATION ENGINEERING

CHAPTER - 1
SHALLOW FOUNDATION

1.1 INTRODUCTION

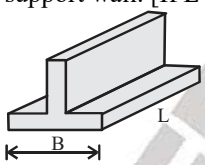
1. Footings are generally the lowermost supporting part of the structure known as sub-structure and are the last structural elements through which load is transferred to foundation comprising soil/rock.

2. Structural elements transfer the applied loads from one part of the building to the other. These are in turn transmitted to the foundation which transfers it to the underlying soil/rock.

1.2 TYPES OF FOOTING

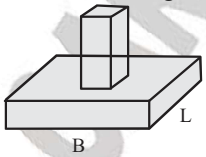
1. Strip Footing

These are also known as wall footing to support wall. [If L >>B] → Strip footing.



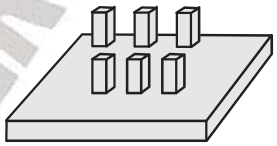
2. Isolated Footing

These are also known as spread footing. Isolated footing is used below the column.



3. Raft/Mat Foundation

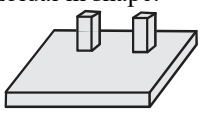
These type of foundations are large continuous footing which support all columns and walls of a structure and are constructed when soil is weak.



Raft Foundation

4. Combined Footings

These footings are usually constructed due to space limitations and support two or more columns. They may be either rectangular or trapezoidal in shape.



Combined Footing

WORKBOOK

Example 1. A 2m wide strip footing is formed at a depth of 1.5m below the ground level in a homogeneous bed of Dense Sand, having the following prosperities

ϕ = 36º, γ = 1.85 tm^3, Determine the ultimate, net ultimate

Net safe bearing capacity of the footing. Given for

ϕ = 36º, N_c = 60º N_q = 42, N_γ = 47. Assume a F.O.S of 3

Solution.

As ϕ = 36º, general shear failure likely to occur.

For Dense Sand c = 0, N_c = 60, N_q = 42, N_γ = 47

(i) Ultimate Bearing Capacity

$q_u = (cN_c + \gamma D\, N_q + 0.5\, B\gamma\, N_\gamma)$

$$= \left(0\times 60 + 1.85\times 1.5\times 42 + \frac{1}{2}\times 2\times 1.85\times 47\right)$$

$$= (116.55 + 86.95) = 203.5\,\text{t/m2}$$

(ii) Net ultimate Bearing Capacity

$q_{nu} = (q_u - \gamma D)$

= (203.5–1.85×1.5)

= 200.725 t/m^2

(iii) Net safe Bearing Capacity

$$q_{ns} = \left(\frac{q_{nu}}{\text{F.O.S}}\right) = \left(\frac{200.725}{3}\right) = 66.908\,\text{t/m}^2$$

(iv) Safe Bearing Capacity

$Q_s = (q_{ns} + \gamma D)$

= (66.908+1.85×1.5) = 69.68 t/m^2

Example 2. Determine the safe load that can be carried by a square footing of 2.2m × 2.2m size placed at a depth of 1.6 m below GL. The foundation soil has the following properties

γ = 1.65 t/m^3 c = 1.1 t/m^2, ϕ = 20º

Assume of F.O.S of 2.5. Given For ϕ = 20º

N_c = 17.7, N_q = 7.4 N_γ = 5.0

N_c' = 11.8 N_q' =3.8 N_γ' = 1.3

Solution.

The low value of unit weight suggests that the soil is in the loose state. Moreover Q = 20º < 29º. Hence a local shear failure is likely to occur.

$$q_{nu} = 1.3c'\,N_c' + \gamma D\times(N_q' - 1) + 0.4\,B\gamma\,N_\gamma'$$

$$c' = \frac{2}{3}c = \frac{2}{3}\times 1.1$$

$$= \frac{2}{3}\times 1.1 = 0.73\,\text{t/m}^2$$

$$\therefore q_{nu} = (1.3\times 0.73\times 11.8) + 1.65\times 1.6(3.8-1)$$

$$+(0.4\times 1.65\times 2.2\times 1.3) = 20.52\,\text{t/m}^2$$

The safe bearing capacity of the footing

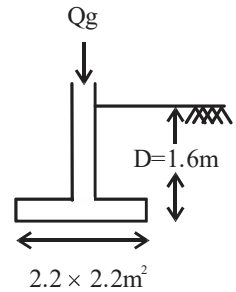
$$q_s = \left(\frac{q_{nu}}{\text{F.O.S}}\right) + \gamma D$$

$$= \left(\frac{20.52}{2.5}\right) + 1.65\times 1.6 = 10.85\,\text{t/m}^2$$

$\therefore$ Gross safe load to be carried by the footing

$= q_s \times (\text{Area of footing})$

$= 10.85\times(2.2)^2 = 52.52$ tonn.



Example 3. A concrete strip footing rectangular in cross section is located at ground level and extends 1.2 m below the ground level. It carries UDL of 15000 kg/m. the soil profile consists of homogeneous clay 6m thick over laying rock. The clay properties are as under.

Saturated unit bulk weight = 1750 kg/m^3

Shear strength (undrained) = 8500 kg/m^2

Compressibility = 1×10^{-4} m^2/100 kg

Determine,

(i) Width of footing for F.O.S. of F = 2

(ii) Ultimate consolidation settlement for F = 2

Assume bulk unit weight of concrete = 2500 kg/m^3

Neglect the spread of load beneath the footing any side cohesion on the foundation

CHAPTER - 2
DEEP FOUNDATION

2.1 INTRODUCTION

In situations where soil at shallow depth is poor, in order to transmit load safely, the depth of foundation has to be increased till the suitable soil strata is met. In view of increased depth, such foundations are called Deep foundation. Well foundation. Pile-Foundation and Pier Foundation are Deep foundations.

Pile is a small dia shaft which can be driven or installed into ground, Where as Piers and well Foundation are large dia shafts, constructed by excavation and sunk to the required depth.

2.2 CLASSIFICATION BASED ON MODE OF TRANSFER OF LOAD

1. End-Bearing Piles

(i) Used to transfer load through the pile tip to a suitable bearing stratum, passing soft soil or water.

2. Friction Piles

(i) Used to transfer loads to a depth in; frictional material by means of skin friction along the surface area of the pile.

(ii) Friction piles are also called as Floating piles, as they do not reach the hard stratum.

3. Combined End Bearing and Friction Pile

(i) Used to transfer, load through the combine action of end bearing and friction along the surface area of pile.

2.3 SINGLE PILE LOAD CAPACITY

1. When a compressive load (P) is applied at-the top of pile, the pile will tend to move vertically downward relative to soil. Due to this shear or friction develops between soil and surface of shaft. As a result applied load is distributed as frictional load along certain length of pile.

2. As load is increased full frictional resistance is mobilised over complete length of pile but by that time point bearing resistance will be very less.

3. When full point bearing resistance is mobilised, the frictional stress will drop from its maximum value.

4. The maximum frictional resistance mobilised is called (Q_f) when the load exceeds (Q_f), the point bearing starts mobilising. This load is known as point load (at base). This point load goes on increasing till failure occurs by punching shear.

5. Load in bearing at this stage is called ultimate point load (Q_{pu}).

6. Hence, for calculation purpose, *we* take Ultimate load Q_u where Q_u = (Q_u = (Q_{pu} + Q_f). although this is not correct because when maximum point bearing is developed, friction reduced from its maximum value.

7. If [Q_{pu} >>> Q_F] Pile is called **Point bearing point**.

WORKBOOK

Example 1. A 12 m long, 300 mm diameter concrete pile is driven in a uniform deposit of dense sand. Water table is at great depth and is not likely to rise. The average dry unit wt. of sand is 18 kN/m^3. Use N_q = 137, Calculate the safe load capacity of a angle pile with a F.O.S. of 2.5, ϕ = 40º.

Solution.

1. Driven/Bored Driven pile
2. Sand/clay→ Dense sand

So, 1. Driven pile

2. Pile is in dense sand.

Here, in case of pile we only work for $P_{ultimate}$ and not for $P_{net\ ultimate}$

$q_{pu} = 0 + \sigma.N_q + 0.5\ B.\gamma.N_\gamma$

Here, 3rd term is negligible. Hence, it is neglected.

$\therefore\ q_{pu} = 0 + \sigma.N_q$

Calculation for σ

We know that, in case of dense sand $\bar{\sigma}$ is calculated at a depth of 20 × D.

20 × D = 20 × 0.3 = 6 m

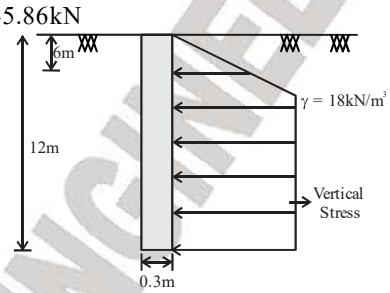
$\bar{\sigma} = \gamma \times 20 \times D$

= 18 × 20 × 0.3 = 108 kN/m^2

$Q_{pu} = q_{pu}.\ A_b$

$$Q_{pu} = [0 + 108 \times 137] \times \frac{\pi}{4}(0.3)^2$$

$$= 1045.86\text{kN}$$



Also $Q_f = f_s \cdot A_s$

where, f_s is $k\bar{\sigma}_{avg}\tan(\delta)$

Here $Q_f = f_{s1}.A_1 + f_{s2}.A_2$

$$\delta = \left(\frac{3}{4}\phi\right) = \frac{3}{4} \times 40 = 30^\circ$$

$$Q_f = 2.\tan 30^\circ \left[\left(\frac{108+0}{2} \times \pi \times 0.3 \times 6\right) + (108 \times \pi \times 0.3 \times 6)\right]$$

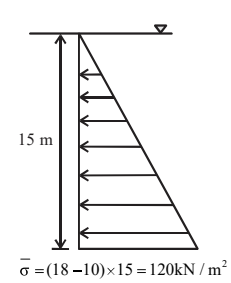
$$= 1057.807\text{kN}$$

$$Q_{up} = 1057.807 + 1045.86\text{kN} = 2103.667\text{kN}$$

$$Q_{safe} = \left[\frac{Q_{up}}{\text{F.O.S.}}\right] = 841.452\text{kN}$$

Example 2. For 450 mm side square section concrete pile 15 m long is driven in a deep deposite of uniform clay. The laboratory unconfined compression test on undisturbed sample indicates an avg. value of (Q_u) unconfined compressive Strength = 75 kN/m^2. Calculate the ultimate load capacity of pile. Take α = 0.8. γ_{sat} = 18 kN/m^3, γ_w = 10 kN/m^3.

Solution.



$Q_u = C_{ub}.\ N_c.\ A_b + \alpha.\ C_{uavg}.\ A_s$

37.5kN/m^2 9 0.45x0.450.8 q. 0.45x4x15

$$C_{uavg} = \frac{75}{2} = 37.5\text{kN/m}^2$$

Here, (C_{ub}) at the base of pile is not given. Hence, average values will be taken as (C_{ub})

Q_u = 878.34 kN

Example 3. Determine the allowable pile load capacity of 40 cm dia driven concrete pile as shown below. take N_2 = 160

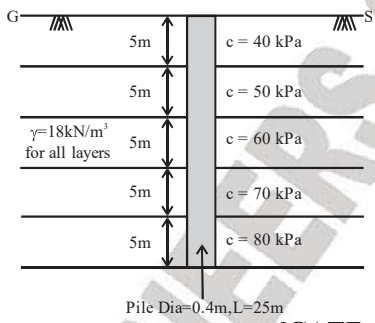
GATE QUESTIONS

1. A 0.5 m × 0.5 m square concrete pile is to be driven in a homogenous clayey soil having undrained shear strength, cu = 50 kPa and unit weight, γ = 18.0 kN/m^3. The design capacity of the pile is 500 kN. The adhesion factor α is given 0.75. The length of the pile required for the above design load with a factor of safety of 2.0 is

[GATE - 2018]

(a) 5.2 m (b) 5.8 m
(c) 11.8 m (d) 12.5 m

2. A pile of diameter 0.4 m is fully embedded in a clay stratum having 5 layers, each 5 m thick as shown in the figure below. Assume a constant unit weight of soil as 18 kN/m^3 for all the layers. Using λ-method (λ = 0.15 for 25 m embedment length) and neglecting the end bearing component, the ultimate pile capacity (in kN/) is _____.



[GATE - 2015]

3. A single vertical friction pile of diameter 500 mm and length 20 m is subjected to a vertical compressive load. the pile is embedded in a homogeneous sandy stratum where: angle of internal friction (φ) = 30º, dry nit weight (γ_d) = 20 kN/m^3. Considering the coefficient of laeral earth pressure (K) = 2.7 and the bearing capacity factor (N_q) = 25, the ultimate bearing capacity of the pile (in kN) is _____.

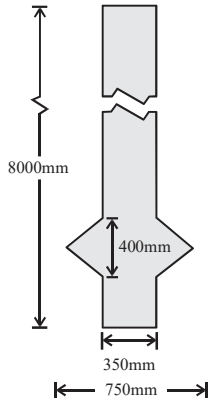
[GATE - 2014]

4. The action of negative skin friction on the pile is to

[GATE - 2014]

(a) Increase the ultimate load on the pile
(b) Reduce the allowable load on the pile
(c) Maintain the working load on the pile
(d) Reduce the settlement of the pile

5. A singly under-reamed, 8-m long, RCC pile (shown in the adjoining figure) weighing 20 kN with 350 mm shaft diameter and 750 mm under-ream diameter is installed within stiff, saturated silty clay (undrained shear strength is 50 kPa, adhesion factor is 0.3, and the applicable bearing capacity factor is 9) to counteract the impact of soil swelling on a structure constructed above. Neglecting suction and the contribution of the under-team to the adhesive shaft capacity, what would be the estimated ultimate tensile capacity (rounded off to the nearest integer value of kN) of the pile



[GATE - 2011]

(a) 132 kN (b) 156 kN
(c) 287 kN (d) 301 kN

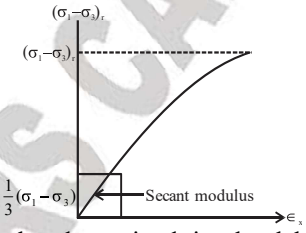
6. The ultimate load capacity of a 10 m long concrete pile of square cross section 500 mm × 500 mm driven into a homogeneous clay layer having undrained cohesion value of 40 kPa is 700kN. If the cross section of the pile is reduced to 250 mm × 250 mm and the length of the pile

CHAPTER - 3
VERTICAL STRESSES

3.1 INTRODUCTION

1. Stresses are induced in a soil mass due to weight of overlying soil and due. to the applied loads.
2. These stresses are required to design a foundation such that the shear stress on any stratum of soil below it does not exceed, after providing Factor of safety for bearing capacity of soil.
3. Further the vertical stresses transmitted to the soil layers below the foundation will lead to vertical deformation in the soil, causing foundation settlement.
4. This settlement again should not be allowed to exceed the permissible settlement.
5. Hence the know ledge of distribution of stresses with in a soil mass, induced by loads applied on the surface of soil, is a prerequisite for foundation design.
6. The stress induced in soil due to applied loads depend upon its stress strain characteristics. The stress strain behavior of soil is extremely complex and it depends upon a large number of factors, such as drainage conditions, water content, void ratio, rate of loading, the load level, and the stress path.
7. Generally the stress strain relationship is assumed to be linear, and fortunately these results are good enough for the problems usually encountered in practice.
8. Theory of Elasticity is used 10 determine the stresses in soil mass.
9. The main stress-strain parameters required for the application of elastic theories are modulus of Elasticity (E) and Poisson's ratio (μ).

3.1.1 Modulus of Elasticity



1. Modulus of Elasticity (E) can be determined in the laboratory by conducting a triaxial compression test.
2. The stress strain curve is plotted between the deviator stress (σ_1 - σ_3) and the axial strain ($\in_x$).

1. For Saturated, Cohesive Soil

Unconsolidated untrained (U U) test or unconfined compression test is performed.

2. For Cohesionless Soil

Consolidated drained (CD) test is performed.

(i) The value of modulus of elasticity is generally taken as the secant modulus (1/2 to 1/3) of the peak stress. Sometimes, instead of secant modulus, the initial tangent modulus or the tangent modulus at (1/2 to 1/3) of the peak stress is also used.

3.1.2 Poission's Ratio

1. For elastic material generally poission's ratio varies from 0 - 0.5
2. For Undrained conditions, the value of poission's ratio is 0.50.
3. For drained condition value is less than 0.50.

CHAPTER - 4
SOIL EXPLORATION

4.1 WASH BORING

1. In this method a casing pipe is driven through a heavy drop hammer supported by a tripod and pully.

2. Water is forced under pressure through the hollow drill rod and it is rotated up and down inside the casing pipe.

3. The lower end of the drill rod is fitted with a sharp cutting edge or chopping bit which cuts the soil.

4. Soil water mixture floats up through the annular space between the casing pipe and the drill rod.

5. Slurry or soil water mixture flowing out provides an indication of the soil type. Whereas change in soil strata can be determined from the rate of progress and slurry flowing out.

6. As the sample of soil is obtained in the form of soil water mixture, has no value because it is highly disturbed.

7. It cannot be used efficiently in hard soils, rocks and soil containing boulders.

8. This method is not suitable for taking good quality undisturbed samples above ground water table, as the wash water enters the strata below the bottom of the hole and causes an increase in its water content.

4.2 PERCUSSION BORING

1. In this method a heavy drilling bit is alternately dropped and raised in such a manner that it grinds the underlying hard material to the consistency of a sand or silt.

2. The bore hole is kept dry and only a small amount of water is added to form. A slurry with the material cut from the bit.

3. The soil samples are obtained with the means of bailer, and the changes in character of the soil stratum can be determined by the rate of progress.

4. In bouldery and gravelly stratum percussion boring is only suitable method.

4.3 ROTATORY BORING

Rotator Boring is of two types

1. Mud Rotatory Boring

A hole is drilled with the help of a rotating bit. Bentonite soil with some admixture is used as a drilling mud. Drill mud comes upwards through the anular space between the drill rod and the side of the hole.

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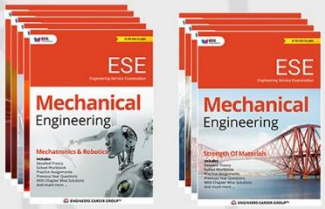

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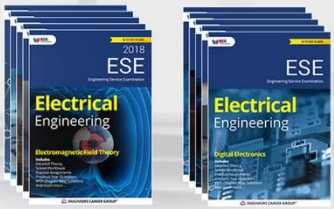

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