Geotechnical Engineering - 1

Classification of Soil

Soils generally are called *gravel, sand, silt,* or *clay*, depending on the predominant size of particles within the soil. To describe soils by their particle size, several organizations have developed particle-size classifications.

Table Particle-Size Classifications

Name of organization	Gravel	Sand	Silt	Clay
	Grain size (mm)			
Massachusetts Institute of Technology (MIT)	>2	2 to 0.06	0.06 to 0.002	<0.002
U.S. Department of Agriculture (USDA)	>2	2 to 0.05	0.05 to 0.002	<0.002
American Association of State Highway and Transportation Officials (AASHTO)	76.2 to 2	2 to 0.075	0.075 to 0.002	<0.002
Unified Soil Classification System (U.S. Army Corps of Engineers, U.S. Bureau of Reclamation, and American Society for Testing and Materials)	76.2 to 4.75	4.75 to 0.075	Fines (i.e., silts and clays) <0.075	

Note: Sieve openings of 4.75 mm are found on a U.S. No. 4 sieve; 2-mm openings on a U.S. No. 10 sieve; 0.075-mm openings on a U.S. No. 200 sieve.

Grain-Size Distribution

In any soil mass, the sizes of the grains vary greatly. To classify a soil properly, you must know its grain-size distribution. The grain-size distribution of coarse-grained soil is generally determined by means of sieve analysis. For a fine-grained soil, the grain-size distribution can be obtained by means of hydrometer analysis.

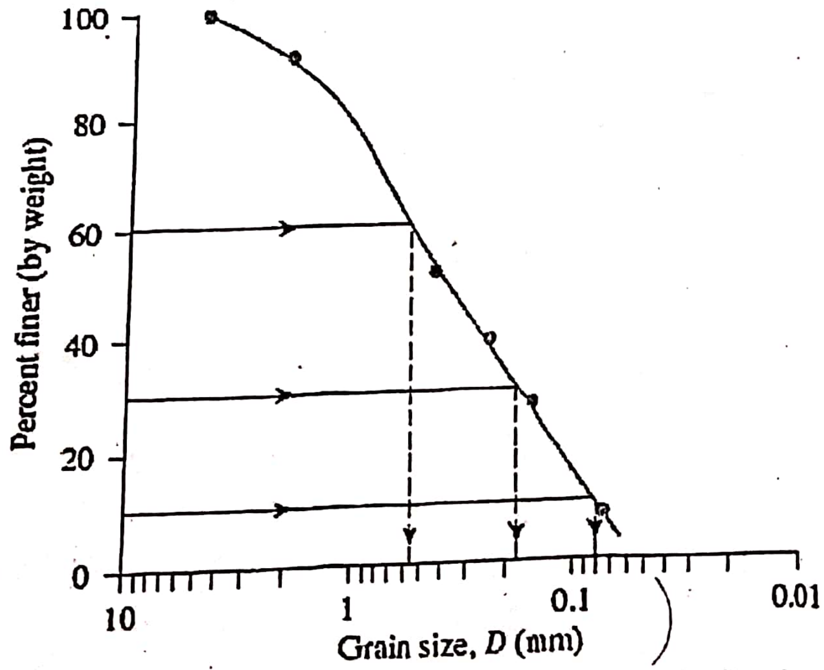

Figure: Grain-size distribution curve of a coarse grained soil

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Classification system	Grain size (mm)
Unified	Gravel: 75 mm to 4.75 mm Sand: 4.75 mm to 0.075 mm Silt and clay (fines): <0.075 mm
AASHTO	Gravel: 75 mm to 2 mm Sand: 2 mm to 0.05 mm Silt: 0.05 mm to 0.002 mm Clay: <0.002 mm

Figure: Soil-Separate Size Limits

Effective size (D_{10}): This parameter is the diameter in the particle-size distribution curve corresponding to 10% finer. The effective size of a granular soil is a good measure to estimate the hydraulic conductivity and drainage through soil.

Uniformity coefficient (C_u): The uniformity coefficient C_u is defined as the ratio of D_{60} by D_{10}.

$$C_u = \frac{D_{60}}{D_{10}}$$

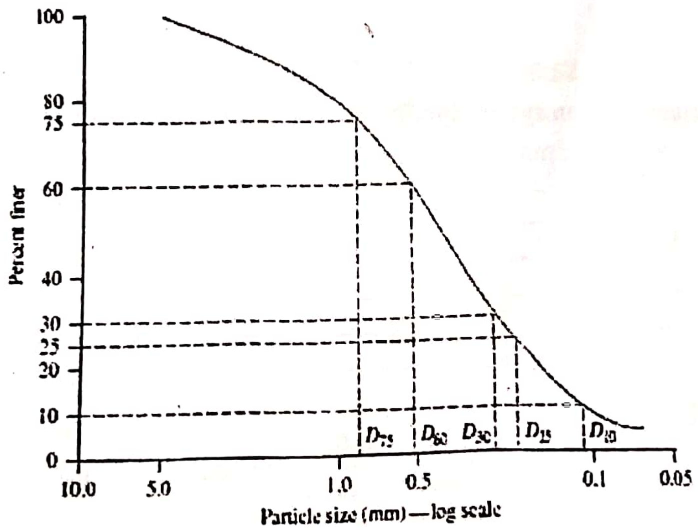
Where, D_{60} is the diameter corresponding to 60% finer.
$C_u > 4$ for well graded gravel
$C_u > 6$ for well graded sand
$C_u < 4$ for uniformly graded soil containing particles of the same size.

Coefficient of gradation (C_c): This parameter is defined as $C_c = \dfrac{D_{30}^2}{D_{30} \times D_{10}}$

Wherein D_{30} is the size of particle at 30 percent finer on the gradation curve. The soil is said to be well graded if C_c lies between 1 and 3 for gravels and sands.

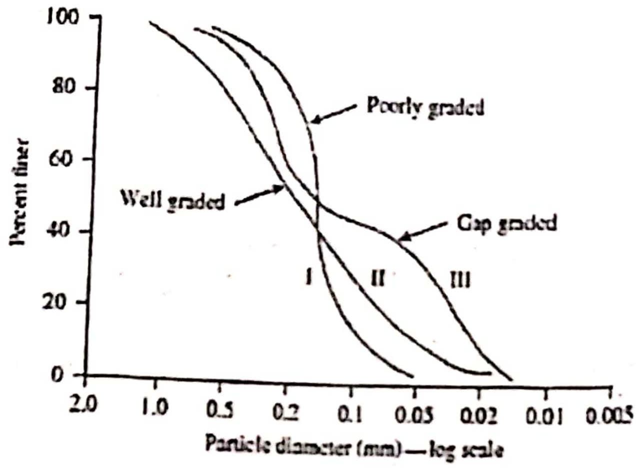
Question: Draw grain size distribution curve for D_{10}, D_{30} and D_{60} (BUET M. Sc. –2011)

Solution:



Question: Draw qualitative diagram of particle size distribution curve. (TGTDCL – 2014, DNCC – 2020)

Solution:



Question: What five letters used for unified soil classification system in their meaning. (RAJUK – 2014)

Solution:

The Unified Soil Classification System (USCS) is a soil classification system used in engineering and geology to describe the texture and grain size of a soil. The classification system can be applied to most unconsolidated materials, and is represented by a two-letter symbol. Five first-letter symbols are used:

→ G for gravel
→ S for sand
→ M for silt
→ C for clay
→ O for organic soil

Question: According to the unified soil classification system specify the meaning of the following types of soil. (TGTDCL – 2014)

Solution:

a. CL = Lean Clay
b. MH = Elastic Silt
c. SP = Poorly Graded Sand with silt.
d. OL = Organic Clay/Silt.
e. PT = Peat and others highly organic soil.

Question: Write five letters used for unified soil classification system with their meanings. (RAJUK – 2014)

Solution:

The Unified Soil Classification System (USCS) is a soil classification system used in engineering and geology to describe the texture and grain size of a soil. The classification system can be applied to most unconsolidated materials, and is represented by a two-letter symbol.

a. CL = Lean Clay
b. MH = Elastic Silt
c. SP = Poorly Graded Sand with silt.
d. OL = Organic Clay/Silt.
e. PT = Peat and others highly organic soil.
f. GW = Well graded gravels with sand.
g. GP = poorly graded gravels with sand.
h. SW = Well graded sand with gravel.
i. SP = poorly graded sand with gravel.
j. OH = Organic silt and clays.

Question: For the particle-size distribution curve D_{10}, D_{30} and D_{60} value is 0.15 mm, 0.17 mm and 0.27mm. Find uniformity coefficient and co-efficient of gradation.

Solution:

Coefficient of uniformity, $C_u = \frac{D_{60}}{D_{10}} = \frac{0.27}{0.15} = 1.8$

Coefficient of gradation, $C_c = \frac{D_{30}^2}{D_{30} \times D_{10}} = \frac{0.17^2}{0.27 \times 0.15} = 0.71$

Question: $D_{10} = 0.0162$ mm, $D_{30} = 0.1018$ mm, $D_{60} = 0.577$. Find the coefficient of uniformity. (DNCC – 2016)

Solution:

Coefficient of uniformity, $C_u = \frac{D_{60}}{D_{10}} = \frac{0.577}{0.0162} = 35.62$

Question: Grain size distribution of a soil are given bellow, find the uniformity co-efficient and the co-efficient of curvature & find out if the soil is well graded? (PGCB – 2019)

Particle size (mm)	Percentage finer
0.4	60
0.2	30
0.05	10

Solution:

Coefficient of uniformity, $C_u = \frac{D_{60}}{D_{10}} = \frac{0.4}{0.05} = 8$

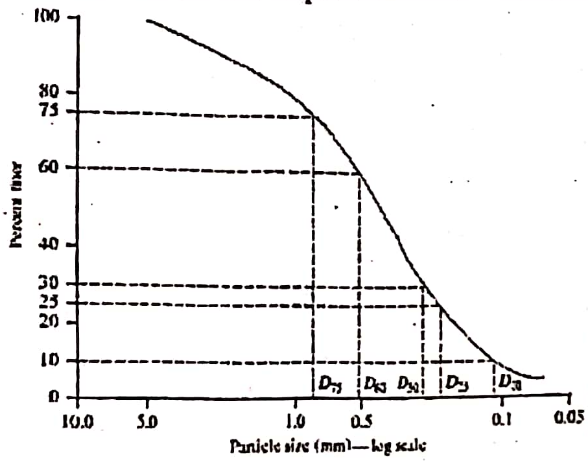
Coefficient of curvature, $C_c = \frac{D_{30}^2}{D_{30} \times D_{10}} = \frac{0.2^2}{0.4 \times 0.05} = 0.2$

$C_u > 6$ and $1 \le C_c \le 3$, so the soil is well graded.

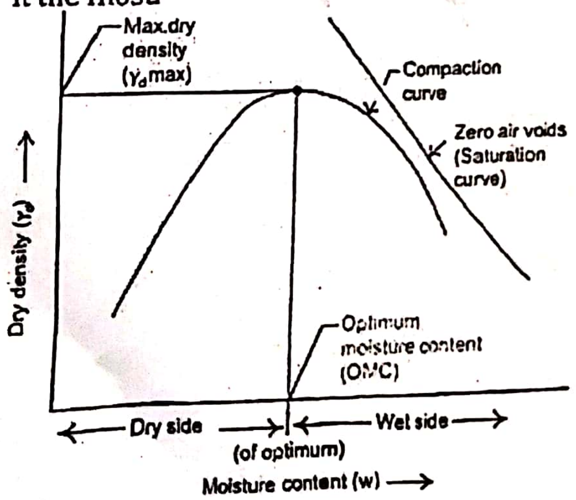
Question: Define i) Effective Grain Size and ii) Optimum Moisture Content. (SGFL – 2021)

Solution:

Effective Grain Size: Effective Grain Size means the diameter of filter sand or other aggregate that corresponds to the 10 percentile finer by dry weight on the grain size distribution curve or the sieve size that retains 90 percent of the materials.

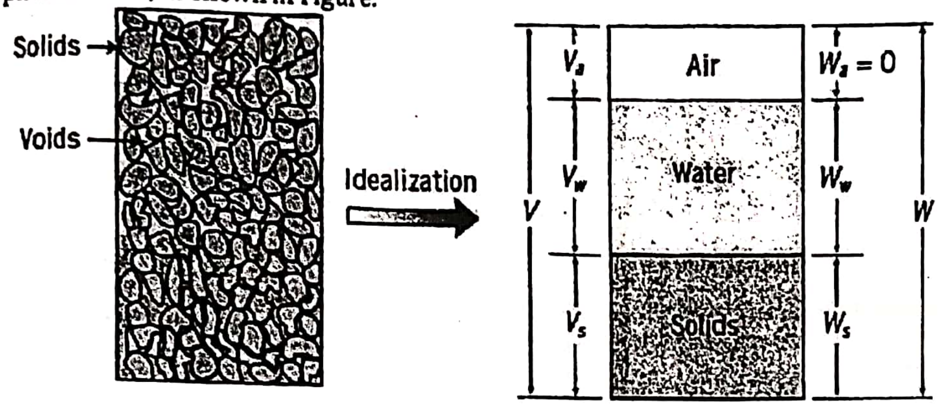


Optimum Moisture Content: The Optimum Water Content of soil is the water content at which a maximum dry unit weight can be achieved after a given compaction effort. A max dry unit weight would have no voids in the soil. If you were trying to compact a hard dry soil to make it more dense, you might want to get it wet. The OPT is the water content of the soil in which you could compact it the most.



Weight volume relationship

Soil is composed of solids, liquids, and gases. The solid phase may be minerals, organic matter, or both. The spaces between the solids (soil particles) are called voids. Water is often the predominant liquid and air is the predominant gas. We will use the terms water and air instead of liquid and gas. The soil water is called porewater and plays a very important role in the behavior of soils under load. If all the voids are filled by water, the soil is saturated. Otherwise, the soil is unsaturated. If all the voids are filled with air, the soil is said to be dry. We can idealize the three phases of soil, as shown in Figure.



(a) Soil (b) Idealized soil

$V = V_s + V_v = V_s + V_w + V_a$

Where, V_s = volume of soil solids

V_v = volume of voids

V_w = volume of water in the voids

V_a = volume of air in the voids

$W = W_s + W_w$

Where, W_s = weight of soil solids

W_w = weight of water

Void ratio (e) is defined as the ratio of the volume of voids to the volume of solids.

$$e = \frac{V_v}{V_s}$$

Porosity (n) is defined as the ratio of the volume of voids to the total volume.

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

Degree of saturation (S) is defined as the ratio of the volume of water to the volume of voids.

$$S = \frac{V_v}{V_w} = \frac{w\,G_s}{e}$$

If $S = 1$ or 100%, the soil is saturated. If $S = 0$, the soil is bone dry. It is practically impossible to obtain a soil with $S = 0$.

Moisture content, usually expressed in terms of percentage, is the ratio of the weight of water to the weight of solids.

$$w = \frac{W_w}{W_s} \times 100$$

Air content is defined as the ratio of volume of air voids to the volume of voids.

$$a_c = \frac{V_a}{V_v}$$

$V_a = V - V_w$, We get, $a_c = 1 - \frac{V_w}{V_v} = 1 - S$

Specific gravity of solid particles of soil is the ratio of the unit weight of solids (γ_w) to the unit weight of water (γ_s).

$$G_s = \frac{\gamma_s}{\gamma_w} = \frac{W_s}{V_s \gamma_w}$$

Unit weight is the weight of soil per unit volume.

$$\gamma = \frac{W}{V} = \frac{(G_s + S\,e)\,\gamma_w}{1+e} = \frac{G_s \gamma_w\,(1+w)}{1+e}$$

Dry unit weight, $\gamma_d = \frac{W_s}{V} = \frac{G_s \gamma_w}{1+e} = \frac{\gamma}{1+w}$

Submerged unit weight, $\gamma' = \gamma_{sat} - \gamma_w = \frac{(G_s - 1)\,\gamma_w}{1+e}$

Relative density is an index that quantifies the state of compactness between the loosest and densest possible state of coarse-grained soils. Relative density is commonly used to indicate the in situ denseness or looseness of granular soil.

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

e = in situ void ratio of the soil γ

e_{max} = void ratio of the soil in the loosest state

e_{min} = void ratio of the soil in the densest state

In terms of density,

$$D_r = \frac{\rho_d - \rho_{d(min)}}{\rho_{d(max)} - \rho_{d(min)}} \times \frac{\rho_{d(max)}}{\rho_d}$$

$$or, D_r = \frac{\gamma_d - \gamma_{d(min)}}{\gamma_{d(max)} - \gamma_{d(min)}} \times \frac{\gamma_{d(max)}}{\gamma_d}$$

$\gamma_{d(min)}$ = dry unit weight in the loosest condition when the void ratio of e_{max}

γ_d = in situ dry unit weight, at a void ratio of e

$\gamma_{d(max)}$ = dry unit weight in the densest condition when the void ratio of e_{min}

Relative density (%)	Description of soil deposit
0-15	Very loose
15-50	Loose
50-70	Medium
70-85	Dense
85-100	Very dense

Question: Draw phase diagram of saturated soil, partially saturated soil and dry soil. (RRI – 2015, RAJUK – 2014, BUET M.Sc. –2014, ISTT – 2015)

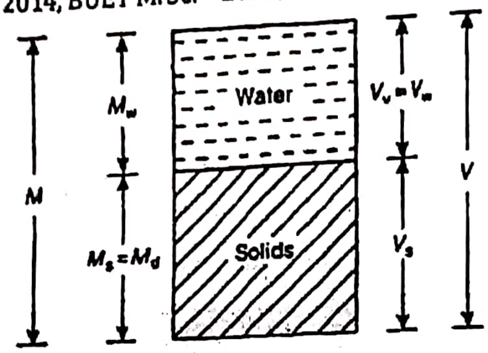


Figure Two-phase diagram of a fully saturated soil in terms of mass.

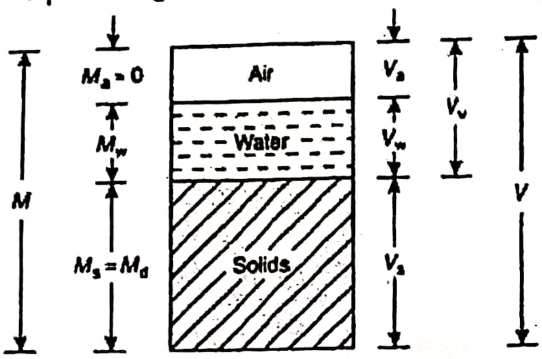


Figure Three-phase diagram of a partially saturated soil in terms of mass.

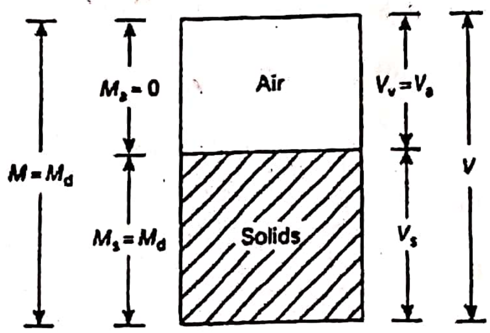


Figure Two-phase diagram of a dry soil in terms of mass.

Question: Show that $\gamma_{sat} = \frac{e}{w}\left(\frac{1+w}{1+e}\right)\gamma_w$ (BWDB – 2019)

Solution:

$$\gamma_{sat} = \frac{W}{V} = \frac{W_s + W_w}{V} = \frac{G_s\,\gamma_w + e\,\gamma_w}{1+e} = \frac{(G_s + e)\,\gamma_w}{1+e}$$

For saturated soil, $e = w\,G_s$

$$\gamma_{sat} = \frac{(G_s + w\,G_s)\,\gamma_w}{1+e} = \frac{G_s\,(1+w)\,\gamma_w}{1+e} = \frac{e}{w}\left(\frac{1+w}{1+e}\right)\gamma_w$$

Question: A soil has a porosity of 80%, calculate its void ratio. (PGCL – 2014)

Solution:

Void ratio, $e = \frac{n}{1-n} = \frac{0.8}{1-0.8} = 4$

Question: A saturated soil of, $w = 30\%$, $G = 2.65$, determine the dry unit weight. (PGCB – 2017)

Solution:

We know, $S\,e = w\,G_s$

For fully saturated soil, $S = 1$

$e = w\,G_s = 0.30 \times 2.65 = 0.795$

Dry unit weight, $\gamma_d = \frac{G_s\,\gamma_w}{1+e} = \frac{2.65 \times 9.81}{1+0.795} = 14.48\ \text{kN/m}^3$

Question: In its natural states a moist soil has a volume of 0.4 ft³ and weight 50lb, the oven dry weight of soil is 40lb. Calculate the bulk unit weight, dry unit weight and moisture content of the sample. (PGCB – 2015)

Solution:

Given, $V = 0.40\ \text{ft}^3$, $W = 50$ lb, $W_s = 40$ lb

Weight of water, $W_w = W - W_s = 50 - 40 = 10$ lb

Bulk unit weight, $\gamma = \frac{W}{V} = \frac{50}{0.4} = 125\ \text{lb/ft}^3$

Dry unit weight, $\gamma_d = \frac{W_s}{V} = \frac{40}{0.4} = 100\ \text{lb/ft}^3$

Moisture content, $w = \frac{W_w}{W_s} = \frac{10}{40} = 0.25 = 25\%$

Question: A soil sample was prepared by mixing a quantity of dry soil with 10% by mass of water. Find the mass of this wet mixture required to produce a cylindrical specimen of 15 cm diameter and 12.5 cm deep and having 5% air content. Also find the void ratio and the dry density of the specimen if G = 2.68. (38th BCS)

Solution:

$M_w = 10\%$ of dry mass of soil $= 0.1\, M_s$

Volume, $V = \frac{\pi d^2 h}{4} = \frac{\pi \times 15^2 \times 12.5}{4} = 2208.93 \text{ cm}^3$

$a_c = \frac{V_a}{V_v} = 0.05$

$V_a = 0.05\, V_v$

Volume of void, $V_v = V_a + V_w = 0.05\, V_v + V_w$

$V_v = \frac{V_w}{0.95}$

Total volume, $V = V_a + V_w + V_s = 0.05\, V_v + V_w + V_s$

$V = 0.05 \times \frac{V_w}{0.95} + V_w + V_s$

$V = 1.0526\, V_w + V_s$

$V = 1.0526 \times \frac{M_w}{\rho_w} + \frac{M_s}{G_s\, \rho_w}$

$V = 1.0526 \times \frac{M_w}{1} + \frac{M_s}{2.68 \times 1}$

$2208.93 = 1.0526 \times 0.1 \times M_s + 0.373\, M_s$

$M_s = 4618.68$ gm

Mass of wet mixture $= M_s + M_w = 4618.68 + 0.1 \times 4618.68 = 5080.54$ gm

Density, $\rho = \frac{M}{V} = \frac{5080.54}{2208.93} = 2.30$ gm/cc

Dry density, $\rho_d = \frac{\rho}{1+w} = \frac{2.30}{1+0.1} = 2.09$ gm/cc

We know, $\rho_d = \frac{G_s\, \rho_w}{1+e}$

Void ratio, $e = \frac{G_s\, \rho_w}{\rho_d} - 1 = \frac{2.68 \times 1}{2.09} - 1 = 0.28$

Question: Find the unit weight of saturated soil, γ_{sat}. Unit weight of dry soil, $\gamma_d = 110$ pcf and specific gravity, $G_s = 2.7$. (DPDC – 2014, PGCL – 2017)

Solution:

Dry unit weight, $\gamma_d = \frac{G_s \gamma_w}{1+e}$

$110 = \frac{2.7 \times 62.4}{1+e}$

$e = 0.531$

$\gamma_{sat} = \frac{(G_s + S\,e)\,\gamma_w}{1+e} = \frac{(2.7 + 1 \times 0.531)\,62.4}{1 + 0.531} = 131.69$ pcf

Question: A soil having 6' depth. Its relative density is 40% with minimum void ratio 0.46 and maximum void ratio is 0.9. Its specific gravity is 2.65. Find the dry density of the soil. (BHP – 2017)

Solution:

$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$

$0.4 = \frac{0.9 - e}{0.9 - 0.46}$

Void ratio, $e = 0.724$

Dry unit weight, $\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.65 \times 9.81}{1 + 0.724} = 15.08$ kN/m^3

Question: In a natural state, a moist soil has a volume of 0.33 ft^3 and weight 39.93 lb. The oven dry weight of soil is 34.54 lb. if the specific gravity is 2.75, calculate dry unit weight and void ratio. (BCIC – 2017)

Solution:

Dry unit weight, $\gamma_d = \frac{W_s}{V} = \frac{34.54}{0.33} = 104.66$ lb/ft^3

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

Void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{62.4 \times 2.75}{104.66} - 1 = 0.63$

Question: A saturated clay soil has 45% moisture content and $G_s = 2.7$; determine the unit weight of soil? (DPDC – 2019)

Solution:

We know, $S\,e = w\,G_s$

For fully saturated soil, $S = 1$

$e = w\,G_s = 0.45 \times 2.7 = 0.81$

$\gamma_{sat} = \frac{(G_s + S\,e)\,\gamma_w}{1+e} = \frac{(2.7 + 1 \times 0.81)\ 9.81}{1+0.81} = 19.02\ \text{kN/m}^3$

Question: Water content of a fully saturated soil sample is 50%, specific gravity of soil solid (G_s) = 2.65. Find the values of bulk unit weight, dry unit weight and void ratio. (SGCL – 2017)

Solution:

We know, $S\,e = w\,G_s$

For fully saturated soil, $S = 1$

$e = w\,G_s = 0.5 \times 2.65 = 0.1.325$

Dry unit weight, $\gamma_d = \frac{G_s\,\gamma_w}{1+e} = \frac{2.65 \times 9.81}{1+1.325} = 11.18\ \text{kN/m}^3$

Bulk unit weight, $\gamma = \gamma_d\,(1+w) = 11.18\,(1+0.5) = 16.77\ \text{kN/m}^3$

Question: The dry density of a soil is 1.58 gm/cc. If the saturation water content is 50% then what would be it saturated density and submerged density? (BWDB – 2013)

Solution:

$\gamma = \gamma_d\,(1+w) = 1.58\,(1+0.5) = 2.37\ \text{gm/cm}^3$

Submerged unit weight, $\gamma' = \gamma_{sat} - \gamma_w = 2.37 - 1 = 1.37\ \text{gm/cm}^3$

Question: A soil sample is dry weight 40 lb and moist weight 50 lb, volume of soil 0.21 ft³. Find the moisture content, bulk unit weight and dry unit weight. (GTCL – 2016)

Solution:

Bulk unit weight, $\gamma = \frac{W}{V} = \frac{50}{0.4} = 125\ \text{lb/ft}^3$

Dry unit weight, $\gamma_d = \frac{W_s}{V} = \frac{40}{0.4} = 100\ \text{lb/ft}^3$

Moisture content, $w = \frac{W_w}{W_w} = \frac{W - W_s}{W_w} = \frac{50-40}{40} = 0.25 = 25\%$

Question: Void ratio and volume of soil is given 0.876 and 40 cm³. Determine the amount of volume of voids and volume of solids. (WASA – 2014)

Solution:

Porosity, $n = \frac{V_v}{V} = \frac{e}{1+e} = \frac{0.876}{1+0.876} = 0.467$

Volume of voids, $V_v = n\,V = 0.467 \times 40 = 18.68\ \text{cm}^3$

We know, $e = \frac{V_v}{V_s}$

Volume of solids, $V_s = \frac{V_v}{e} = \frac{18.68}{0.876} = 21.32\ \text{cm}^3$

Question: A sample of a sand measured volume 12 cm³ and weight 32.3 gm. After oven dried the weight of sample is to be measured 31.2 gm. If the solids sand volume is 8.48 cm³, calculate the void ratio of the sample.

Solution:

Total volume, $V = 12\ cm^3$

Volume of solids, $V_s = 8.48\ cm^3$

Volume of void, $V_v = V - V_s = 12 - 8.48 = 3.52\ cm^3$

Void ratio, $e = \frac{V_v}{V_s} = \frac{3.52}{8.48} = 0.42$

Question: Percent of void is 50%. If specific gravity of soil solid 2.7 then calculate submerged unit weight. (BPDB – 2015)

Solution:

Porosity, $n = \frac{V_v}{V} = \frac{0.5}{1} = 0.5$

Void ratio, $e = \frac{V_v}{V_s} = \frac{n}{1-n} = \frac{0.5}{1-0.5} = 1$

Submerged unit weight, $\gamma' = \frac{(G_s - 1)\gamma_w}{1+e} = \frac{(2.7-1)\,9.81}{1+1} = 8.339\ kN/m^3$

Question: 100 cm³ sample of moist soil has a mass of 212.25 gm. Water content 16%. Specific gravity of soil is 2.68. Calculate bulk unit weight of soil, dry unit weight of soil, void ratio and porosity. (DNCC – 2016)

Solution:

Bulk unit weight, $\gamma = \frac{W}{V} = \frac{212.25}{100} = 2.12\ g/cm^3$

Dry unit weight, $\gamma_d = \frac{\gamma}{1+w} = \frac{2.12}{1+0.16} = 1.83\ g/cm^3$

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

Void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.68 \times 1}{1.83} - 1 = 0.465 = 46.50\%$

Porosity, $n = \frac{e}{1+e} = \frac{0.465}{1+0.465} = 0.317$

Question: In a field hole is cut off volume 1.1 ft³ and the wet mass of the hole is 130 lb and dry mass is 119 lb. determine the degree of saturation if specific gravity is 2.7. (BWDB – 2016)

Solution:

Bulk unit weight, $\gamma = \frac{W}{V} = \frac{130}{1.1} = 118.18\ lb/ft^3$

Dry unit weight, $\gamma_d = \frac{W_s}{V} = \frac{119}{01.1} = 108.18\ lb/ft^3$

We know, $\gamma_d = \frac{\gamma}{1+w}$

Water content, $w = \frac{\gamma}{\gamma_d} - 1 = \frac{118.18}{108.18} - 1 = 0.0924$

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

Void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 62.4}{108.18} - 1 = 0.557$

We know, $S\,e = w\,G_s$

Degree of saturation, $S = \frac{w\,G_s}{e} = \frac{0.0924 \times 2.7}{0.55} = 0.447 = 44.7\%$

Question: 100 cm^3 sample of moist soil has a mass of 212.25 gm. Water content 16%. Specific gravity of soil is 2.68. Calculate (a) void ratio (b) Dry unit weight of soil (c) Degree of saturation. (PGCB – 2018)

Solution:

Bulk unit weight, $\gamma = \frac{W}{V} = \frac{212.25}{100} = 2.12\ g/cm^3$

Dry unit weight, $\gamma_d = \frac{\gamma}{1+w} = \frac{2.12}{1+0.16} = 1.83\ g/cm^3$

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

Void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.68 \times 1}{1.83} - 1 = 0.465$

We know, $S\,e = w\,G_s$

Degree of saturation, $S = \frac{w\,G_s}{e} = \frac{2.68 \times 0.16}{0.465} = 0.92 = 92\%$

Question: A 27.50 lb soil sample has a volume of 0.220 cft, moisture content of 15.20% and specific gravity of soil solids of 2.67. Compute the bulk density, dry density, degree of saturation and void ratio. (32th BCS)

Solution:

Bulk density, $\rho = \frac{M}{V} = \frac{27.50}{0.220} = 125\ lb/ft^3$

Dry density, $\rho_d = \frac{\rho}{1+w} = \frac{125}{1+0.152} = 108.50\ lb/ft^3$

We know, $\rho_d = \frac{G_s \rho_w}{1+e}$

Void ratio, $e = \frac{G_s \rho_w}{\rho_d} - 1 = \frac{2.67 \times 62.4}{108.50} - 1 = 0.535$

We know, $S\,e = w\,G_s$

Degree of saturation, $S = \frac{w\,G_s}{e} = \frac{2.67 \times 0.152}{0.535} = 0.758 = 75.80\%$

Question: The dry density of a sand with a porosity of 0.381 is 1600 kg/m³. Find the void ratio of the soil and the specific gravity of the solids. (33th BCS)

Solution:

$$\text{Void ratio, } e = \frac{n}{1-n} = \frac{0.381}{1-0.381} = 0.615$$

$$\text{Dry density, } \rho_d = \frac{G_s \rho_w}{1+e}$$

$$G_s = \frac{\rho_d\,(1+e)}{\rho_w} = \frac{1600\,(1+0.615)}{1000} = 2.58$$

Question: Moist soil sample total volume 1.2 m³ and total mass 2350 kg, Specific gravity 2.71 and water content 8.6%. Find i) Porosity ii) Degree of saturation. (BB AD – 2018)

Solution:

$$\text{Bulk unit weight, } \gamma = \frac{W}{V} = \frac{2350}{1.2} = 1958.33\ \text{kg/m}^3 = 19.21\ \text{kN/m}^3$$

$$\text{Dry unit weight, } \gamma_d = \frac{\gamma}{1+w} = \frac{19.21}{1+0.086} = 17.68\ \text{kN/m}^3$$

$$\text{We know, } \gamma_d = \frac{G_s \gamma_w}{1+e}$$

$$\text{Void ratio, } e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.71 \times 9.81}{17.68} - 1 = 0.502$$

$$\text{We know, } S\,e = w\,G_s$$

$$\text{Degree of saturation, } S = \frac{w\,G_s}{e} = \frac{0.086 \times 2.71}{0.502} = 0.464 = 46.4\%$$

Question: An undisturbed sample of clayey soil have wet weight 285 N, dry weight 250 N and volume 14 x 10³ cm³. If G = 2.70, then calculate w, e and S. (HBFC – 2018)

Solution:

$$\text{Water content, } w = \frac{W_w}{W_w} = \frac{W - W_s}{W_w} = \frac{285-250}{250} = 0.14$$

$$\text{Dry unit weight, } \gamma_d = \frac{W_s}{V} = \frac{250}{14000} = 0.01786\ \text{N/cm}^3 = 17.86\ \text{kN/m}^3$$

$$\text{We know, } \gamma_d = \frac{G_s \gamma_w}{1+e}$$

$$\text{Void ratio, } e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.7 \times 9.81}{17.86} - 1 = 0.483$$

$$\text{We know, } S\,e = w\,G_s$$

$$\text{Degree of saturation, } S = \frac{w\,G_s}{e} = \frac{0.14 \times 2.7}{0.483} = 0.7812 = 78.12\%$$

Question: Determine the soil is saturated or not. Where porosity, n = 0.6, water content, w = 54.54% and specific gravity G_s = 2.75. If the soil is saturated find out the value of γ_d (BCIC – 2019)

Solution:

Void ratio, $e = \frac{n}{1-n} = \frac{0.6}{1-0.6} = 1.5$

We know, $S\ e = w\ G_s$

Degree of saturation, $S = \frac{w\ G_s}{e} = \frac{0.5454 \times 2.75}{1.5} = 1 = 100\%$

The soil is fully saturated

Dry unit weight, $\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.75 \times 9.81}{1+1.5} = 10.79\ kN/m^3$

Question: Water content, specific gravity and void ratio of a soil sample are 20%, 2.68 and 0.78. Now calculate dry unit weight, degree of saturation and porosity. (BIWTA – 2019)

Solution:

Dry unit weight, $\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.68 \times 9.81}{1+0.78} = 14.77\ kN/m^3$

We know, $S\ e = w\ G_s$

Degree of saturation, $S = \frac{w\ G_s}{e} = \frac{0.2 \times 2.68}{0.78} = 0.68 = 68\%$

Porosity, $n = \frac{e}{1+e} = \frac{0.78}{1+0.78} = 0.43$

Question: In a natural state, a moist soil has a volume of 0.33 ft³ and weight 39.93 lb. The oven dry weight of soil is 34.54 lb. if the specific gravity is 2.75, calculate dry unit weight and void ratio. (BUET M.Sc – 2019)

Solution:

Moisture content, $w = \frac{W_w}{W_w} = \frac{W - W_s}{W_w} = \frac{39.93 - 34.54}{34.54} = 0.156$

Dry unit weight, $\gamma_d = \frac{W_s}{V} = \frac{34.54}{0.33} = 104.66\ lb/ft^3$

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

Void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.75 \times 62.4}{104.66} - 1 = 0.63$

Question: Total volume of a soil sample is 150 cm^3, saturated weight is 250 gm and dry weight of the sample is 210 gm. If the specific gravity of the soil is 2.6, find out the saturation of the soil sample. (DESGO – 2019)

Solution:

Moisture content, $w = \frac{W_w}{W_w} = \frac{W - W_s}{W_w} = \frac{250 - 210}{210} = 0.19$

Dry unit weight, $\gamma_d = \frac{W_s}{V} = \frac{210}{150} = 1.4\ g/cm^3$

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

Void ratio, $e = \frac{G_s \gamma_w}{\gamma_d} - 1 = \frac{2.6 \times 1}{1.4} - 1 = 0.85$

We know, $S\,e = w\,G_s$

Degree of saturation, $S = \frac{w\,G_s}{e} = \frac{0.19 \times 2.6}{0.85} = 0.576 = 57.6\%$

Question: The difference between the maximum and minimum void ratio is 0.3 and field void ratio is 0.4. If relative density is 66.6% then find out the saturated density at its loosest condition. Specific gravity of the soil is 2.65 (BPDB – 2016, RPGCL – 2017, CPGCBL – 2018)

Solution:

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}} \times 100$$

$$66.6 = \frac{e_{max} - 0.4}{0.3} \times 100$$

$$e_{max} = 0.6$$

$$\gamma = \frac{(G_s + e_{max})\,\gamma_w}{1 + e_{max}} = \frac{(2.65 + 0.6)\ 9.81}{1 + 0.6} = 19.92\ kN/m^3$$

Question: A soil sample in its natural state has a weight of 5.05 lb and a volume of 0.041 ft^3. In an oven dried state, the dry weight of the sample is 4.49 lb. The specific gravity of the solids is 2.68. Determine the total unit weight, water content, void ratio, porosity and degree of saturation.

Solution:

$$\gamma_t = \frac{W}{V} = \frac{5.05}{0.041} = 123.2\ lb/ft^3$$

$$w = \frac{W_w}{W_w} = \frac{W - W_s}{W_w} = \frac{5.5 - 4.49}{4.49} = 0.125 \text{ or } 12.5\%$$

$$V_s = \frac{W_s}{G_s \gamma_w} = \frac{4.49}{2.68 \times 62.4} = 0.0268\ ft^3$$

$$V_v = V - V_s = 0.041 - 0.0268 = 0.0142\ ft^3$$

$e = \frac{V_v}{V_s} = \frac{0.0142}{0.0268} = 0.53$

$n = \frac{e}{1+e} = \frac{0.53}{1+0.53} = 0.3464$ or 36.64%

$S = \frac{w\,G_s}{e} = \frac{0.125 \times 2.68}{0.53} = 0.632 = 63.2\%$

Question: The moist weight of 0.0060 m³ of soil is 10 kg. If moisture content of the soil is 13% and specific gravity of the soil is 2.71, find the porosity of the soil sample. (TGTDCL – 2021)

Solution:

Given, $V = 0.006\ \text{m}^3, W = 10\ \text{kg} = 98.1\ \text{N} = 0.0981\ \text{KN}$

Bulk unit weight, $\gamma = \frac{W}{V} = \frac{0.0981}{0.006} = 16.35\ \text{kN/m}^3$

Dry unit weight, $\gamma_d = \frac{\gamma}{1+w} = \frac{16.35}{1+0.13} = 14.46\ \text{kN/m}^3$

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

Void ratio, $e = \frac{G_s\,\gamma_w}{\gamma_d} - 1 = \frac{2.71 \times 9.81}{14.46} - 1 = 0.83$

Porosity, $n = \frac{e}{1+e} = \frac{0.83}{1+0.83} = 0.45$

Question: Saturated unit weight and moisture content are 19.3 KN/m³ and 28% respectively. Calculate void ration e. (SGFL – 2021)

Solution:

$\gamma_{sat} = \frac{(G_s + S\,e)\,\gamma_w}{1+e} = \frac{(G_s + w\,G_s)\,\gamma_w}{1+e} = \frac{G_s\,(1+w)\,\gamma_w}{1+e}$

For fully saturated soil, $e = w\,G_s$

$\gamma_{sat} = \frac{e\,(1+w)\,\gamma_w}{w\,(1+e)}$

$19.3 \times 0.28\,(1+e) = e\,(1+0.28)\,9.81$

$e = 0.75$

Question: A soil has a total weight of 16.97 kN/m³ and a void ratio of 0.84. The specific gravity of solids is 2.70. Determine the moisture content, dry unit weight and degree of saturation of the sample.

Solution:

We know, $\gamma_t = \dfrac{(G_s + S\,e)\,\gamma_w}{1+e}$

$16.97 = \dfrac{(2.7 + 0.84\,S)\,9.81}{1+0.84}$ or $S = 58\%$

Dry unit weight, $\gamma_d = \dfrac{G_s\,\gamma_w}{1+e} = \dfrac{2.7 \times 9.81}{1+0.84} = 14.4 \text{ kN/m}^3$

Water content, $w = \dfrac{S\,e}{G_s} = \dfrac{0.58 \times 0.84}{2.7} = 0.18$ or 18%

Question: Dry unit weight of soil is 1.65 gm/cc and specific gravity 2.67. Determine saturated unit weight and void ratio. Calculate the hydraulic head that would produce a quick condition in sand stratum of thickness 1.476m. (RPGCL – 2017)

Solution:

We know, $\gamma_d = \dfrac{G_s\,\gamma_w}{1+e}$

Void ratio, $e = \dfrac{G_s\,\gamma_w}{\gamma_d} - 1 = \dfrac{2.67 \times 1}{1.65} - 1 = 0.61$

$\gamma_{sat} = \dfrac{(G_s + S\,e)\,\gamma_w}{1+e} = \dfrac{(2.67 + 1 \times 0.61)\,9.81}{1+0.61} = 19.98 \text{ kN/m}^3$

Hydraulic head = hydraulic gradient x thickness

Hydraulic head $= \dfrac{G-1}{1+e} \times H = \dfrac{2.67-1}{1+0.61} \times 1.476 = 1.53$ m

22

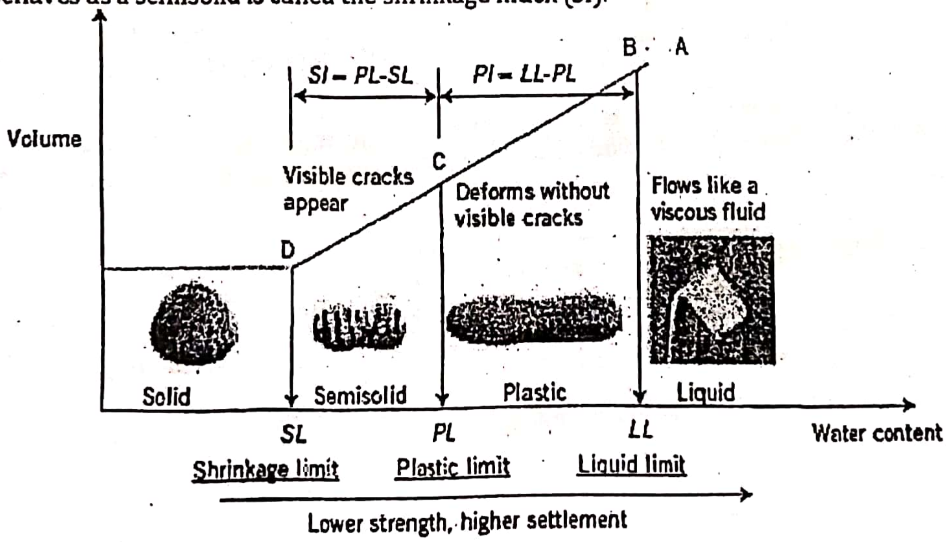
Plasticity and Structure of Soil

Atterberg Limits: The water contents corresponding to the transition from one state to another are termed as Atterberg Limits.

The physical and mechanical behavior of fine-grained soils is linked to four distinct states: solid, semisolid, plastic, and liquid, in order of increasing water content. Let us consider a soil initially in a liquid state that is allowed to dry uniformly. If we plot a diagram of volume versus water content as shown in Figure, we can locate the original liquid state as point A. As the soil dries, its water content reduces and, consequently, so does its volume.

At point B, the soil becomes so stiff that it can no longer flow as a liquid. The boundary water content at point B is called the liquid limit; it is denoted by *LL*. As the soil continues to dry, there is a range of water content at which the soil can be molded into any desired shape without rupture. The soil at this state is said to exhibit plastic behavior: the ability to deform continuously without rupture. But if drying is continued beyond the range of water content for plastic behavior, the soil becomes a semisolid. The soil cannot be molded now without visible cracks appearing. The water content at which the soil changes from a plastic to a semisolid is known as the plastic limit, denoted by *PL*, point C. The range of water contents over which the soil deforms plastically is known as the plasticity index, *PI*.

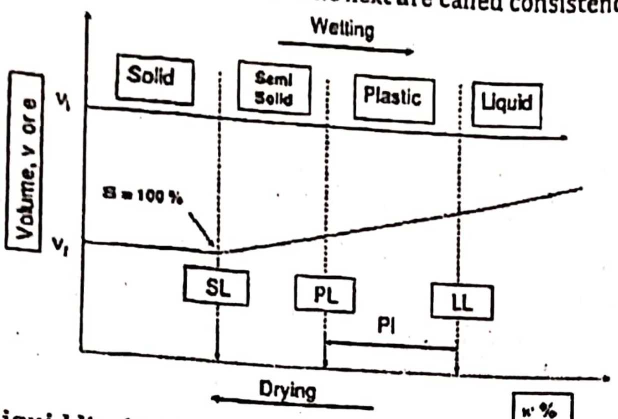
As the soil continues to dry, it comes to a final state called the solid state. At this state, no further volume change occurs because nearly all the water in the soil has been removed. The water content at which the soil changes from a semisolid to a solid is called the shrinkage limit, denoted by *SL*, point D. The shrinkage limit is useful for the determination of the swelling and shrinking capacity of soils. The range of water content from the plastic limit to the shrinkage limit for which the soil behaves as a semisolid is called the shrinkage index (*SI*).



Question: Write down the definition of Atterberg limits and plasticity chart with sketch. (WASA – 2014)

Solution:

Consistency varies with the water content of the soil. The consistency of a soil can range from (dry) solid to semi-solid to plastic to liquid (wet). The water contents at which the consistency changes from one state to the next are called consistency limits (or Atterberg limits).



Liquid limit: The water content at which soil changes from the liquid state to a plastic state or the water content at which the soil has such a small shear strength that it flows to close a groove of standard width when jarred in a specified manner.

Plastic limit: The water content at which a soil changes from the plastic state to a semisolid state or the water content at which the soil begins to crumble when rolled into threads of specified size.

Shrinkage limit: It is the maximum water content at which a reduction in water content will not cause a decrease in volume of the soil mass. It is the lowest water content at which soil can still be saturated.

Plasticity Index is the range of water content over which a soil behaves plastically. It indicates the degree of plasticity of the soil. PI is the difference between the liquid limit and the plastic limit.

$PI = LL - PL$

Plasticity index	Plasticity
0	Non-plastic
<7	Low plastic
7-17	Medium plastic
>17	Highly plastic

Table: Soil classification according to plasticity index

Liquidity Index: The index that is used to indicate the consistency of undisturbed soils is called the liquidity index.

$$I_L = \frac{w - PL}{LL - PL}$$

Consistency	I_L	I_C
Semisolid or solid state	Negative	>1
Very stiff state $(w_n - w_p)$	0	1
Very soft state $(w_n - w_l)$	1	0
Liquid state (when disturbed)	>1	Negative

Table: Values of I_L and I_C according to consistency of soil

Consistency Index: The index I_C reflects the state of the clay soil condition in the field in an undisturbed state

$$I_C = \frac{LL - w}{LL - PL}$$

It may be seen that values of I_L and I_C are opposite to each other for the same consistency of soil.

$I_L + I_C = 1$

Flow index is the slope of flow curve obtained by plotting water content as ordinate or natural scale against number of blows as abscissa on logarithmic scale.

$$I_F = \frac{w_1 - w_2}{\log\left(\frac{N_2}{N_1}\right)}$$

Toughness index: The shearing strength of clay at plastic limit is a measure of its toughness. It is the ratio of plasticity index to the flow index.

$$I_T = \frac{I_P}{I_F}$$

Activity of clays: Activity is the ratio of plasticity index to the percent by weight of soil particles of diameter smaller than two microns present in the soil.

$$\text{Activity} = \frac{\text{Plasticity index, } I_p}{\text{percent finer than 2 micron}}$$

Activity	Soil type
< 0.75	Inactive
0.75 – 1.40	Normal
< 1.4	Active

Table: Soil classification according to activity

Sensitivity of clays: The degree of disturbance of undisturbed clay sample due to remoulding is expressed by sensitivity which is defined as the ratio of its unconfined compression strength in the natural or undisturbed state to that in the remoulded state, without change in water content.

$$\text{Sensitivity, } S_t = \frac{q_u \text{ undisturbed}}{q_u' \text{ remolded}}$$

Sensitivity	Nature of clay	Sensitivity	Nature of clay
1	Insensitive clays	4-8	Sensitive clays
1-2	Low-sensitive clays	8-16	Extra-sensitive clays
2-4	Medium sensitive clays	>16	Quick clays

Table: Soil classification on the basis of sensitivity

Question: Write down the expression of liquidity index with meaning of each symbol. What is plasticity index? (PGCL – 2014)

Solution:

Liquidity index: The index that is used to indicate the consistency of undisturbed soils is called the liquidity index.

$$I_L = \frac{w - PL}{LL - PL} = \frac{w - PL}{I_P}$$

w = Natural moisture content, LL = Liquid limit, PL = Plastic limit

Plasticity Index: Plasticity Index is the range of water content over which a soil behaves plastically. It indicates the degree of plasticity of the soil. PI is the difference between the liquid limit and the plastic limit.

$$I_P = LL - PL$$

Question: Water content 48%, LL 50%, PL 30%, SL 10%. Explain whether it is soft compressible or less compressible. (MES – 2015)

Solution:

$$I_L = \frac{w - PL}{LL - PL} = \frac{48 - 30}{50 - 30} = 0.9, \text{ so the clay is less compressible}$$

Question: The values of liquid limit, plastic limit and water contents are 20%, 20% and 40%. Determine plasticity index and liquidity index. (BUET M.Sc – 2019)

Solution:

$$\text{Plasticity index, } I_P = LL - PL = 50 - 20 = 30\%$$

$$\text{Liquidity index, } I_L = \frac{w - PL}{LL - PL} = \frac{40 - 20}{50 - 20} = 0.66$$

Question: A soil have liquid limit LL = 72 and plastic limit PL = 18. Classify the soil by USCS system plasticity chart A line $I_P = 0.73\ (LL - 10)$ (BADC – 2020)

Solution:

Plasticity index, $I_P = LL - PL = 72 - 18 = 54\%$
From A line, $I_P = 0.73\ (LL - 10) = 0.73\ (72 - 10) = 37.96$
The soil is above A line and LL > 50%, so the soil is CH (High plasticity inorganic clay)

Question: What is relative density/index? Is it possible to have relative index = 0? Explain. (MES – 2015)

Solution:

Relative Density: The term relative density is commonly used to indicate the in situ denseness or looseness of granular soil. It may be expressed as,

$$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$$

The values of D_r vary from minimum of 0% for very loose soil to a maximum of 100% for very dense soil.

Question: What is expansive clay? LL = 70, PI = 40. Will it be an expansive clay? (MES – 2015)

Solution:

Expansive Clay: Expansive clay is a type of clay known as lightweight aggregate that is prone to large volume changes (swelling and shrinking) that are directly related to changes in water content.

Degree of expansion	LL %	PL %	Natural soil suction, tsf
High	>60	>35	>4.0
Marginal	50-60	25-35	1.5-4.0
Low	<50	<25	<1.5

Question: The values of liquid limit, plastic limit and water contents are 35%, 13% and 40%. D_{10} = 0.0162 mm, D_{30} = 0.1018 mm, D_{60} = 0.577. Find (a) The coefficient of uniformity (b) coefficient of curvature and (c) Plasticity index. (PGCB – 2018)

Solution:

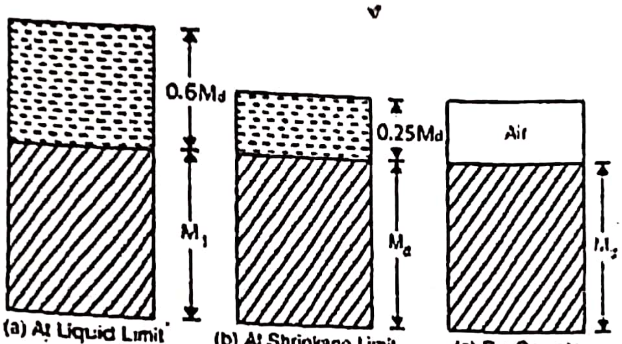
$$(a) C_u = \frac{D_{60}}{D_{10}} = \frac{0.577}{0.0162} = 35.62$$

$$(b)\ C_c = \frac{D_{30}^2}{D_{30} \times D_{10}} = \frac{0.1018^2}{0.577 \times 0.0162} = 1.11$$

$$(c)\ I_P = LL - PL = 35 - 13 = 22\%$$

Question: Clay sample has atterberg limits: Liquid limit = 60, Plastic limit = 40, Shrinkage limit = 25. The sample Shrink from 15 cm³ to 9.57 cm³, when the moisture content is decreased from LL to the SL. What is the dry specific gravity of the clay sample? (GTCL – 2018)

Solution:



(a) At Liquid Limit (b) At Shrinkage Limit (c) Dry Sample

Difference of volume of water content = 15 – 9.5 = 5.43 cm³

Difference of mass of water = 5.43 g

Now, $(0.6 - 0.25)\ M_d = 5.43$

$M_d = 15.51$ g

Mass of water at shrinkage limit = 0.25 x 15.51 = 3.87 g

Volume of water at shrinkage limit = 3.87 cc

Volume of solids = 9.57 – 3.87 = 5.7 cm³

Hence, $\rho_s = \frac{M_d}{V_s} = \frac{15.57}{5.7} = 2.72$

$G_s = \frac{\gamma_s}{\gamma_w} = \frac{\rho_s}{\rho_w} = \frac{2.72}{1} = 2.72$

Question: The laboratory tests on a sample of soil gave the following results: w = 24%, LL, = 62%, PL = 28%, percentage of particles less than 2 μ = 23%. Determine: (a) The liquidity index, (b) activity (c) consistency and nature of soil.

Solution:

$I_p = LL - PL = 62 - 28 = 34\%$

Since I_p is greater than 17% the soil is highly plastic.

$I_L = \frac{w - PL}{LL - PL} = \frac{24 - 28}{62 - 28} = -0.12$

Since I_L is negative, the consistency of the soil is very stiff to extremely stiff (semi-solid state).

$$\text{Activity, } A = \frac{\text{Plasticity index, } I_P}{\text{\% of particles less than } 2\,\mu} = \frac{34}{23} = 1.48$$

Since A is greater than 1.40, the soil is active and is subject to significant volume change.

Question: The natural moisture content of an excavated soil is 32%. Its liquid limit is 60% and plastic limit is 27%. Determine the plasticity index of the soil and comment about the nature of the soil.

Solution:

Plasticity index, $I_P = LL - PL = 60 - 27 = 33\%$

The nature of the soil can be judged by determining its liquidity index,

$$I_L = \frac{w - PL}{LL - PL} = \frac{32 - 27}{60 - 27} = 0.15$$

Since the value of it is very close to 0, the nature of the soil is very stiff.

Question: A soil with a liquidity index of -0.20 has a liquid limit of 56% and a plasticity index of 20%. What is its natural water content? What is the nature of this soil?

Solution:

$I_P = LL - PL$

$PL = I_P - LL = 56 - 20 = 36$

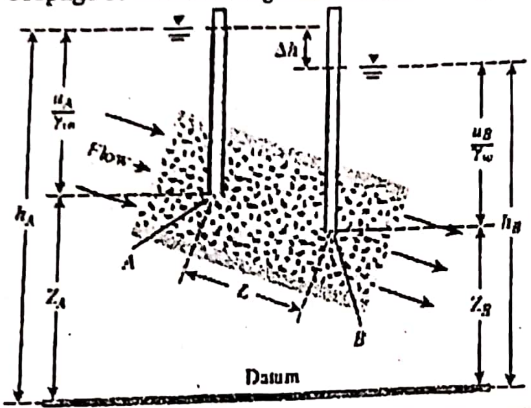
$$I_L = \frac{w - PL}{I_P}$$

$w = I_L \times I_P + PL = -0.20 \times 20 + 36 = 32$

Since I_L is negative, the soil is in a semisolid or solid state.

Soil Permeability and Seepage

Permeability: Permeability is a property of porous material which permits the passage or seepage of water through its interconnecting voids.



Darcy's law: For laminar flow condition in a saturated soil, the rate of flow or the discharge per unit time is proportional to the hydraulic gradient.

$q = k\,i\,A$

Where k is termed the hydraulic conductivity (or coefficient of permeability)

$v_s = \frac{v}{n} = \left(\frac{1+e}{e}\right) v$

v_s = seepage velocity
v = discharge velocity.

Empirical formula for determination of coefficient of permeability
Allen hazen's formula, $K = C D_{10}^2$
Where C is 100 when D_{10} is in cm

Methods of determination of hydraulic Conductivity of soils

Methods that are in common use for determining the coefficient of permeability k can be classified under laboratory and field methods.

Laboratory methods:
1. Constant head permeability method. (For coarse grained soil)
2. Falling head permeability method. (For fine grained soil)

Field methods:
1. Pumping tests
2. Bore hole tests

For Constant – head permeability test, $K = \frac{Q\,L}{A\,h\,t}$

Q = volume of water collected
A = area of cross section of the soil specimen

t = duration of water collection
L = length of the specimen
h = Constant − head difference

For falling head permeability test, $K = \frac{a\,l}{A\,t} \ln \frac{h_o}{h_1}$

a = cross − sectional area of the standpipe
h_o = initial head difference
h_1 = final head difference

Question: What is meant by coefficient of permeability of soil? List the factors affecting the permeability of soil. (38th BCS)

Solution:

The coefficient of permeability of a soil describes how easily a liquid will move through a soil. It is also commonly referred to as the hydraulic conductivity of a soil. The rate of flow under laminar flow conditions through a unit cross sectional are of porous medium under unit hydraulic gradient is defined as coefficient of permeability.

1. Size of soil particle
2. Shape of soil particle
3. Void ratio
4. Soil structure
5. Degree of saturation
6. Water properties
7. Temperature
8. Adsorbed water
9. Impurities in the water

Question: Refer to the constant-head permeability arrangement, a test gives these values:

- L = 30 cm
- A = area of the specimen = 177 cm²
- Constant-head difference, h = 50 cm
- Water collected in a period of 5 min = 350 cm³

Calculate the hydraulic conductivity in cm/sec.

Solution:

Given, $Q = 350\ cm^3, L = 30\ cm, A = 177\ cm^2, h = 50$ cm and $t = 5$ min

We know, $K = \frac{Q\,L}{A\,h\,t}$

$$K = \frac{350 \times 30}{177 \times 50 \times 5 \times 60} = 3.95 \times 10^3 \text{ cm/sec}$$

Question: Piezometric head of a soil at two point A and B are found $h_A = 20$ cm and $h_B = 10$ cm. Length between two point is 10 cm. Permeability of the soil is 0.15 mm/sec. Calculate the rate of flow between these two points through cross section are of 10 cm². (BADC – 2020)

Solution:

Given, k = 0.15 mm/sec = 0.015 cm/sec

Head difference, $h = h_A - h_B = 20 - 10 = 10$ cm

We know, $k = \frac{QL}{Aht}$

$0.015 = \frac{Q}{t} \times \frac{10}{10 \times 10}$

$\frac{Q}{t} = 0.15$ cm³/sec

Question: A field sample of an unconfined aquifer is packed in a test cylinder. The length and diameter of the cylinder are 50 cm and 6 cm respectively. The field sample tested for a period of 3 min under a constant dead difference of 16.3 cm. As a result of 45.2 cm³ of water is collected at the outlet. Determine the hydraulic conductivity of the aquifer sample. (SGFL – 2021)

Solution:

Area of the sample, $A = \frac{\pi d^2}{4} = \frac{\pi \times 6^2}{4} = 28.27$ cm²

$k = \frac{Q}{t}\frac{L}{h}\frac{1}{A} = \frac{45.2}{3 \times 60} \times \frac{50}{16.3} \times \frac{1}{28.27} = 0.02724$ cm/sec

Question: A sand sample of 35 cm² cross sectional area and 20 cm long was tested in a constant head permeameter. Under a head of 60 cm, the discharge was 120 ml in 6 min. The dry weight of sand used for the test was 1 120 g, and $G_s = 2.68$. Determine (a) the hydraulic conductivity in cm/sec (b) the discharge velocity, and (c) the seepage velocity.

Solution:

We know, $K = \frac{QL}{Aht} = \frac{120 \times 20}{35 \times 60 \times 6 \times 60} = 3.174 \times 10^{-3}$ cm/sec

Discharge velocity, $v = ki = 3.174 \times 10^{-3} \times \frac{60}{20} = 8.52 \times 10^{-3}$ cm/sec

Dry density, $\gamma_d = \frac{W_s}{V} = \frac{1120}{35 \times 20} = 1.6$ g/cm³

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

Void ratio, $e = \frac{\gamma_w G_s}{\gamma_d} - 1 = \frac{1 \times 2.68}{1.6} - 1 = 0.675$

Porosity, $n = \frac{1}{1+e} = \frac{1}{1+0.675} = 0.403$

Seepage velocity, $v_s = \frac{v}{n} = \frac{9.52 \times 10^{-3}}{0.403} = 2.36 \times 10^{-2}$ cm/sec

Question: For a falling-head permeability test, the following values are given:

- Length of specimen = 8 in.
- Area of soil specimen = 1.6 in^2
- Area of standpipe = 0.06 in.2
- Head difference at time $t = 0$ = 20 in.
- Head difference at time t = 180 sec = 12 in.

Determine the hydraulic conductivity of the soil in in/sec.

Solution:

We are given, $a = 0.06 \text{ in}^2, L = 8 \text{ in}, A = 1.6 \text{ in}^2, t = 180 \text{ sec}, h_1 = 20$ in and $h_1 = 300$ mm

We know, $K = \frac{a\,l}{A\,t} \ln \frac{h_0}{h_1} = \frac{0.06 \times 8}{1.6 \times 180} \ln \frac{20}{12} = 8.52 \times 10^{-4}$ in/sec

Question: A falling head permeability was performed in a permeameter with an inside diameter of 5 cm. The inside diameter of standard pipe was 2 mm. The sample had a length of 8 cm. During period of 6 min, the head on the sample decreased from 100 to 50 cm. compute the value of K (TGTDCL – 2018)

Solution:

We know, $K = \frac{a\,l}{A\,t} \ln \frac{h_0}{h_1} = \frac{\frac{\pi}{4} \times \left(\frac{2}{10}\right)^2 \times 8}{\frac{\pi}{4} \times 5^2 \times 6 \times 60} \ln \frac{100}{50} = 2.5 \times 10^{-5}$ cm/s

Permeability Test in the Field by Pumping from Wells

In the field, the average hydraulic conductivity of a soil deposit in the direction of flow can be determined by performing pumping tests from wells.

For an unconfined aquifer, $K = \frac{2.303\, q}{\pi\,(h_2^2 - h_1^2)} \log \frac{r_2}{r_1}$

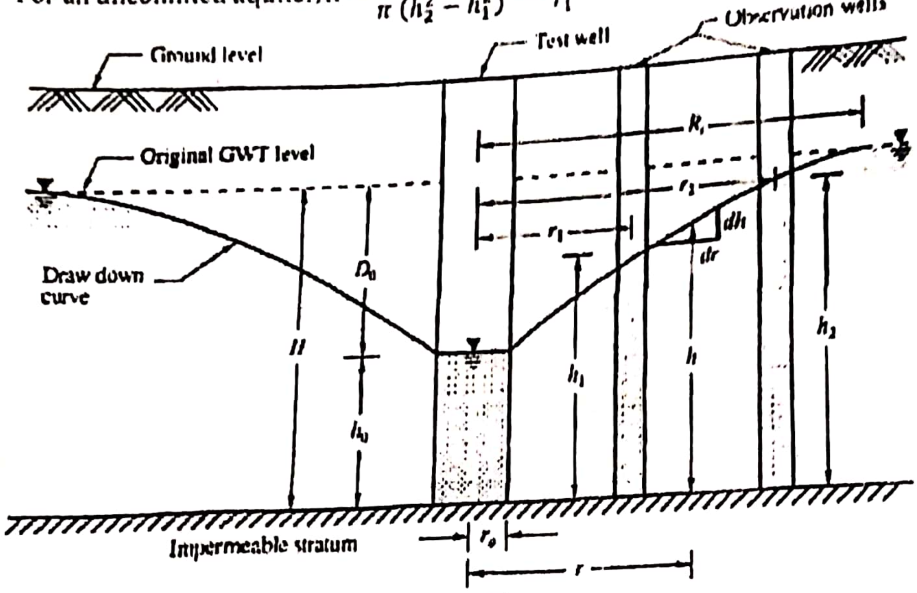


Figure: Pumping test in an unconfined aquifer

For an confined aquifer, $K = \frac{2.303\, q}{2\,\pi\, H_0(h_2 - h_1)} \log \frac{r_2}{r_1}$

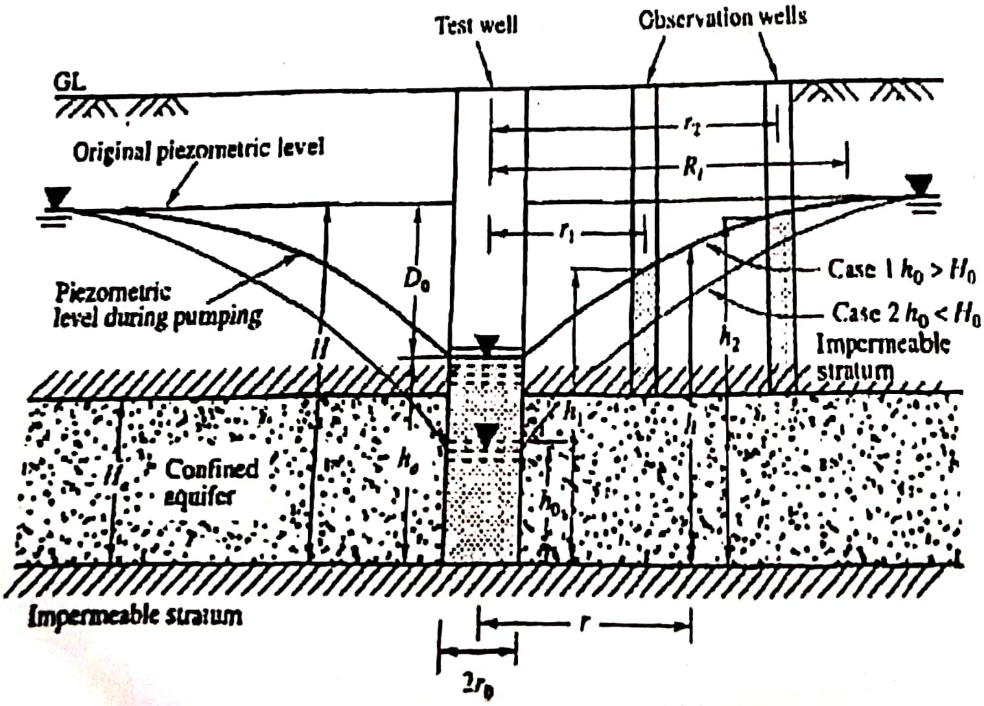


Figure: Pumping test in confined aquifer

Question: A pumping test was made in pervious gravels and sands extending to a depth of 50 ft, where a bed of clay was encountered. The normal ground water level was at the ground surface. Observation wells were located at distances of 10 and 25 ft from the pumping well. At a discharge of 761 ft3 per minute from the pumping well, a steady state was attained in about 24 hr. The draw-down at a distance of 10 ft was 5.5 ft and at 25 ft was 1.21 ft. Compute the hydraulic conductivity in ft/sec.

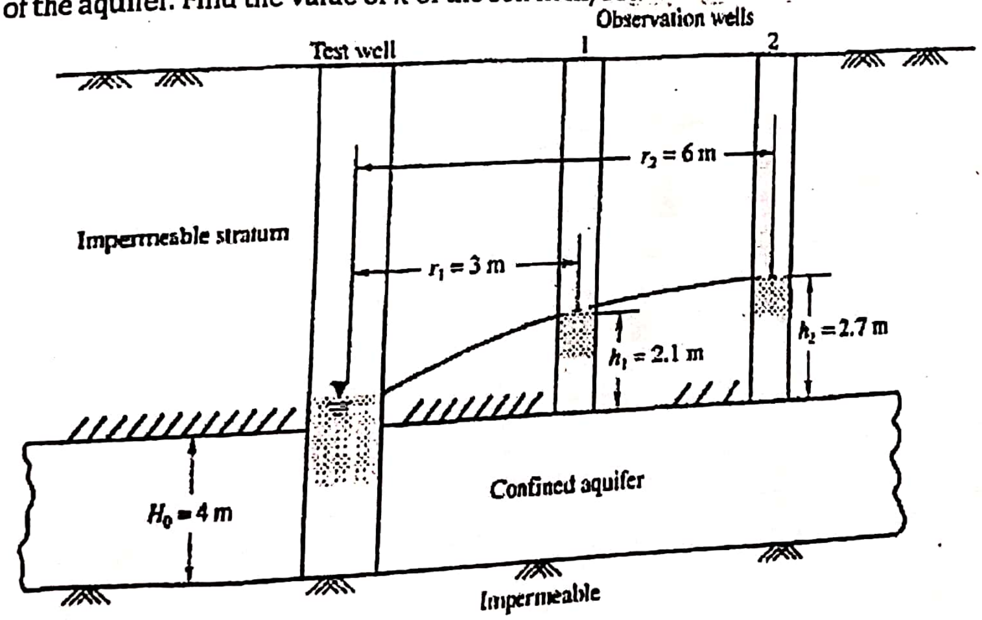
Solution:

$$q = \frac{761}{60} = 12.683 \text{ ft}^3/\text{sec}$$

$r_1 = 10$ ft, $r_2 = 25$ ft, $h_2 = 50 - 1.21 = 48.79$ ft, $h_1 = 50 - 5.5 = 44.5$ ft

$$k = \frac{2.303\, q}{\pi\,(h_2^2 - h_1^2)} \log\frac{r_2}{r_1} = \frac{2.303 \times 12.683}{\pi\,(48.79^2 - 44.5^2)} \log\frac{25}{10} = 9.2 \times 10^{-3} \text{ ft/sec}$$

Question: A field pumping test was conducted from an aquifer of sandy soil of 4 m thickness confined between two impervious strata. When equilibrium was established, 90 liters of water was pumped out per hour. The water elevation in an observation well 3.0 m away from the test well was 2.1 m and another 6.0 m away was 2.7 m from the roof level of the impervious stratum of the aquifer. Find the value of *k* of the soil in m/sec.



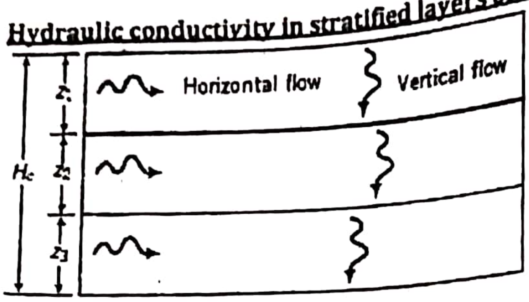
Solution:

$q = 90 \times 10^3$ cm³/hr. $= 90 \times 10^{-6}$ m³/sec

$$k = \frac{2.303\, q}{2\pi H_0\,(h_2 - h_1)} \log\frac{r_2}{r_1}$$

$$= \frac{2.303 \times 25 \times 10^{-6}}{2 \times 3.14 \times 4\,(2.7 - 2.1)} \log\frac{6}{3} = 11.48 \times 10^{-6} \text{ m/sec}$$

Hydraulic conductivity in stratified layers of soils



Flow in the Horizontal Direction

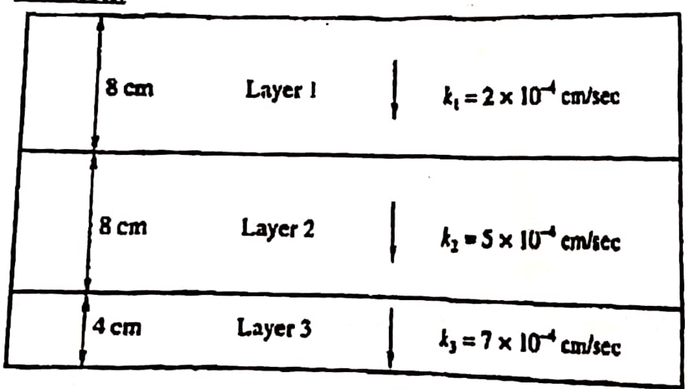
$$k_h = \frac{k_1 z_1 + k_2 z_2 + \ldots + k_n z_n}{z}$$

Flow in the Vertical Direction

$$k_v = \frac{z}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \ldots + \frac{z_n}{k_n}}$$

Question: In a falling head permeameter, the sample used is 20 cm long having a cross-sectional area of 24 cm². Calculate the time required for a drop of head from 25 to 12 cm if the cross-sectional area of the stand pipe is 2 cm². The sample of soil is made of three layers. The thickness of the first layer from the top is 8 cm and has a value of $k_1 = 2 \times 10^{-4}$ cm/sec, the second layer of thickness 8 cm has $k_2 = 5 \times 10^{-4}$ cm/sec and the bottom layer of thickness 4 cm has & $k_3 = 7 \times 10^{-4}$ cm/sec. Assume that the flow is taking place perpendicular to the Layers.

Solution:



$$k_v = \frac{z}{\frac{z_1}{k_1} + \frac{z_2}{k_2} + \frac{z_3}{k_3}} = \frac{20}{\frac{8}{2 \times 10^{-4}} + \frac{8}{2 \times 10^{-4}} + \frac{8}{2 \times 10^{-4}}} = 3.24 \times 10^{-4} \text{ cm/sec}$$

We know, $K = \frac{a\,l}{A\,t} \ln \frac{h_o}{h_1}$

$$t = \frac{a\,l}{A\,k} \ln \frac{h_o}{h_1} = \frac{2.3 \times 2 \times 20}{24 \times 3.24 \times 10^{-4}} \ln \frac{25}{12} = 3771 \text{ sec} = 62.9 \text{ minutes}$$

Seepage: Seepage is the slow movement of water through the continuous void sample.

Seepage force: The pressure that is exerted on the soil due to the seepage of water is called the seepage force or pressure.

Flow net: A flow net for an isometric medium is a network of flow lines and equipotential lines intersecting at right angles to each other.

Flow lines: A flow line is a line along which a water particle will travel from upstream to the downstream side in the permeable soil medium.

Equipotential line: An equipotential line is a line along which the potential head at all points is equal. Equipotential lines are lines that intersect the flow lines at right angles.

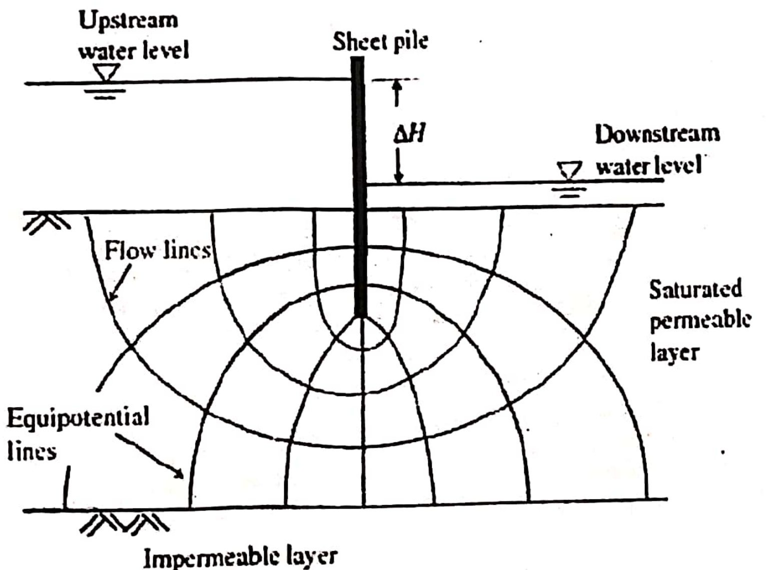


Figure: Flow net for seepage around a sheet pile wall.

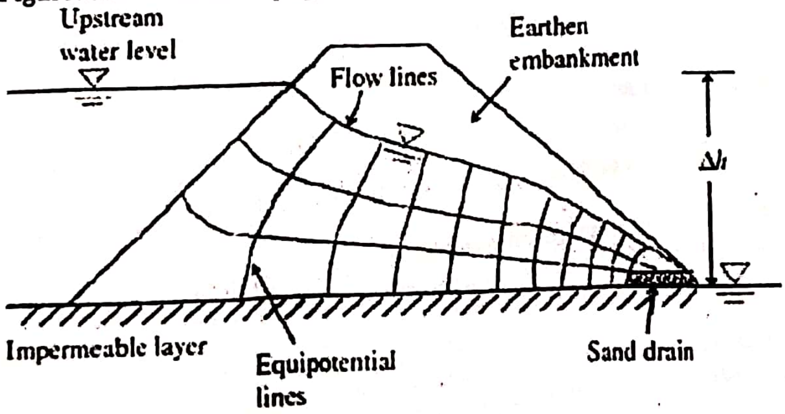


Figure: Flow net for seepage in an earthen dam.

Properties of a Flow Net

The properties of a flow net can be expressed as given below:
1. Flow and equipotential lines are smooth curves.
2. Flow lines and equipotential lines meet at right angles to each other.
3. No two flow lines cross each other.
4. No two flow or equipotential lines start from the same point.

Determination of quantity of seepage

$$q = K\,H\,\frac{N_f}{N_d}$$

N_f = Number of flow channel
N_d = Number of potential drops
H = Head difference between the upstream and downstream sides.
q = The quantity of seepage is calculated per unit length of the section.

Critical hydraulic gradient: The effective pressure reduces to zero when the hydraulic gradient attains a maximum value which is equal to the ratio of the submerged unit weight of soil and the unit weight of water. This gradient is known as the critical hydraulic gradient. In such cases, cohesionless soils lose all of their shear strength and bearing capacity and a visible agitation of soil grains is observed. This phenomenon is known as boiling or a quick sand condition.

$$i_c = \frac{G_s - 1}{1 + e}$$

Question: In order to compute the seepage loss through the foundation of a cofferdam, flownets were constructed. The result of the flownet study gave $N_f = 6$, $N_d = 16$. The head of water lost during seepage was 19.68 ft. If the hydraulic conductivity of the soil is $k = 13.12 \times 10^{-5}$ ft/min, compute the seepage loss per foot length of dam per day.

Solution:

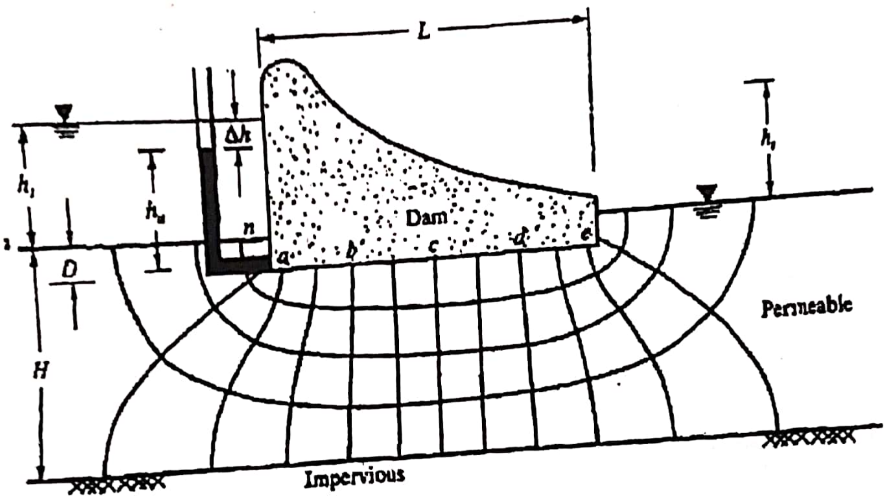
The equation for seepage loss is

$$q = k\,H\,\frac{N_f}{N_d}$$

$$q = 13.12 \times 10 - 5 \times 19.68 \times \frac{6}{16}$$

$$= 9.683 \times 10^{-4}\ \text{ft}^3/\text{min} = 1.39\ \text{ft}^3/\text{day per ft length of dam.}$$

Question: A concrete dam is constructed across a river over a permeable stratum of soil of limited thickness. The water heads are upstream side 16m and 2 m on the downstream side. The flow net constructed under the dam gives $N_f = 4$ and $N_d = 12$. Calculate the seepage loss through the subsoil if the average value of the hydraulic conductivity is 6×10^{-3} cm/sec horizontally and 3×10^{-4} cm/sec vertically. Calculate the exit gradient if the average length of the last field is 0.9 m. Assuming e = 0.56, and $G_s = 2.65$, determine the critical gradient. Comment on the stability of the river bed on the downstream side.



Solution:

Upstream side, $h_1 = 16$ m and downstream side, $h_2 = 2$ m, therefore $h = 16 - 2 = 14$ m

$N_f = 4, N_d = 12, k_h = 6 \times 10^{-3}$ cm/sec, $k_v = 3 \times 10^{-4}$ cm/sec

$k_e = \sqrt{k_h k_v} = \sqrt{6 \times 10^{-3} \times 3 \times 10^{-4}} = 1.34 \times 10^{-3}$ cm/sec

$q = k\,H\,\frac{N_f}{N_d} = 1.34 \times 10^{-3} \times (14 \times 100) \times \frac{4}{12} = 0.626$ cm3/sec

The head loss per potential drop $= \frac{h}{N_d} = \frac{4}{12} = 1.17$ m

Exit gradient, $i = \frac{\Delta h}{l} = \frac{1.17}{0.9} = 1.30$

Critical gradient, $i_c = \frac{G_s - 1}{1 + e} = \frac{2.65 - 1}{1 + 0.56} = 1.06$

Since the exit gradient is greater than the critical gradient, the river bed on the downstream side will be subjected to a quick condition. One solution would be to provide a sheet pile wall on the upstream side below the dam to prevent this condition.

Effective Stress and Pore Water Pressure

Effective pressure: The pressure transmitted through grain to grain at the contact points through a soil mass is termed as inter-granular or effective pressure. This pressure is responsible for the decrease in the void ratio or increase in the frictional resistance of a soil mass.

Pore water pressure: If the pores of a soil mass are filled with water and if a pressure induced into the pore water, tries to separate the grains, this pressure is termed as pore water pressure or neutral stress. The effect of this pressure is to increase the volume or decrease the frictional resistance of the soil mass.

The expulsion of water from the pores decreases the pore water pressure and correspondingly increases the intergranular pressure.

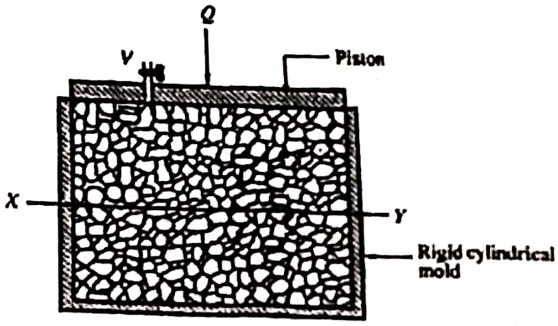
Total pressure, $\sigma = \frac{Q}{A}$ = Intergranular pressure + pore water pressure

or, $\sigma = \sigma' + u$

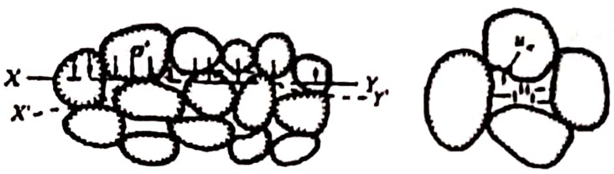
Final equilibrium will be reached when there is no expulsion of water. At this stage the pore water pressure $u = 0$. All the pressure will be carried by the soil grains.

$\sigma = \sigma' + u$

The pore pressure us equal to piezometric head h_w times the unit weight of water.



(a) Soil under load in a rigid container



(b) Intergranular pressure (c) Porewater pressure, u_w

Figure: Effective and pore water pressures

Effective pressure reduces to zero when the hydraulic gradient attains a maximum value which is equal to the ratio of the submerged unit weight of soil and the unit weight of water. This gradient is known as the critical hydraulic gradient i_c. In such cases, cohesionless soils lose all of their shear strength and bearing capacity and a visible agitation of soil grains is observed. This phenomenon is known as boiling or a quick sand condition.

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$$\gamma_b = \frac{(G_s - 1)\gamma_w}{1+e}$$

$$i_c = \frac{G_s - 1}{1+e}$$

Question: Define Total stress, Pore water stress and Effective stress. (BUET M.Sc.–2011)

Solution:

Effective stress: The stress carried by the solid particles or the solid portion of the soil is known as effective stress.

Pore water pressure: The stress carried by the pore water is known as pore water pressure.

Total stress: The compressive stress at a point which may or may not consists of geostatic stresses and induced stresses, is carried partially by the solid portion of the soil and partially by the pore water.

Question: Calculate the total pressure and effective pressure of a swimming pool having 5 m of water level from ground. (BPDB- 2016)

Solution:

Total pressure, $\sigma = \gamma h$ = 9.81 x 5 = 49.05 KN/m²

Pore water pressure, $U = \gamma_w h$ = 9.81 x 5 = 49.05 KN/m²

Effective pressure, $\sigma' = \sigma - U = 0$

Question: Find the effective stress and total stress in a depth of 8m, where unit weight of saturated soil is 18 KN/m³ and unit weight of water 9.81 KN/m³. (BWDB- 2013)

Solution:

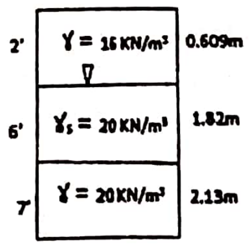
Total stress, $\sigma = \gamma h$ = 18 x 8 = 144 KN/m²

Water pressure, $U = \gamma_w h$ = 9.81 x 8 = 78.48 KN/m²

Effective pressure, $\sigma' = \sigma - U$ = 144 – 78.48 = 65.52 KN/m²

Question: A sand layer has 8 feet depth; W.T is at 2 feet depth below GL. The sand layer is overlying a clay layer of large depth. The moist (above W.T) is 16 KN/m³ and saturated unit weight of sand is 20 KN/m³ and saturated unit weight of clay is 20 KN/m³. Find the total effective pressure at 15 feet depth. (GTCL- 2016)

Solution:

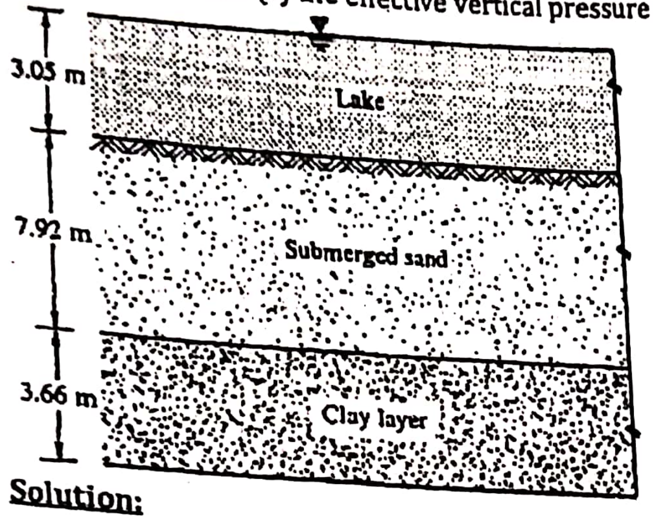


Total pressure = 0.609 x 16 + 1.82 x 20 +2.13 x 20 = 88.74 KN/m²

Water pressure = (1.82 + 2.13) x 9.81 = 38.84 KN/m²

Effective pressure = 88.74 – 38.84 = 49.89 KN/m²

Question: A clay layer 3.66 m thick rests beneath a deposit of submerged sand 7.92 m thick. The top of the sand is located 3.05 m below the surface of a lake. The saturated unit weight of the sand is 19.62 kN/m³ and of the clay is 18.36 kN/m³. Compute (a) the total vertical pressure, (b) the pore water pressure, and (c) the effective vertical pressure at mid height of the clay layer.



Solution:

Total pressure at the mid height of clay layer, $\sigma = 3.05\,\gamma_w + 7.92\,\gamma_{sand} + \frac{3.66}{2}\,\gamma_{clay}$

$\sigma = 3.05 \times 9.81 + 7.92 \times 19.62 + 1.83 \times 18.36 = 218.9\ \text{kN/m}^2$

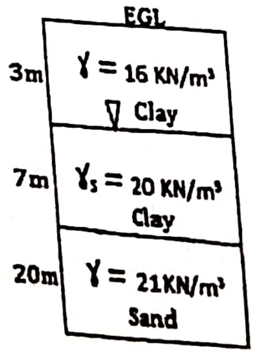
Pore water pressure, $u = 3.05\,\gamma_w + 7.92\,\gamma_w + \frac{3.66}{2}\,\gamma_w$

$u = 3.05 \times 9.81 + 7.92 \times 9.81 + 1.83 \times 9.81 = 125.6\ \text{kN/m}^2$

Effective vertical pressure, $\sigma' = \sigma - u = 218.9 - 125.6 = 93.3\ \text{kN/m}^2$

Question: A clay layer of 10 m thickness underlying E.G.L, water table is lying 3 m from E.G.L. A sand layer of 20 m thickness underlying of the clay layer. Unit weight of clay over water table, under water table and saturated unit weight of sand is 16 KN/m³, 20 KN/m³ and 21 KN/m³. Find total stress, effective stress and effective pore water pressure at 30m from E.G.L (DNCC – 2016)

Solution:



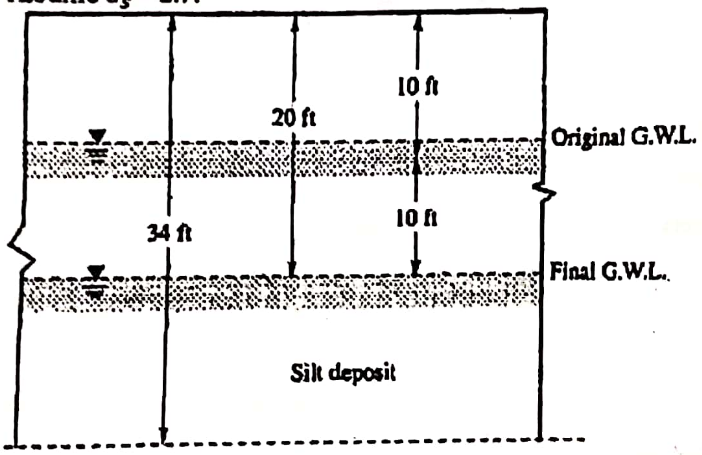
Total pressure $= 3 \times 16 + 7 \times 20 + 20 \times 21 = 608\ \text{KN/m}^2$

Effective pressure $= 3 \times 16 + 7 \times (20 - 9.81) + 20 \times (21 - 9.81)$

$= 343.13\ \text{KN/m}^2$

Pore water pressure $= (7 + 20) \times 9.81 = 264.87\ \text{KN/m}^2$

Question: The water table is lowered from a depth of 10 ft to a depth of 20 ft in a deposit of silt. All the silt is saturated even after the water table is lowered. Its water content is 26%. Estimate the increase in the effective pressure at a depth of 34 ft on account of lowering the water table. Assume $G_s = 2.7$.



Solution:

Effective pressure before lowering the water table

The water table is at a depth of 10 ft and the soil above this depth remains saturated but not submerged. The soil from 10 ft to 20 ft remains submerged. Therefore, the effective pressure at 34 ft depth is

$\sigma_1' = 10\,\gamma_{sat} + (34 - 10)\,\gamma_b$

$Now. S\,e = w\,G_s$

for saturated soil $S = 1, e = 0.26 \times 2.7 = 0.70$

$$\gamma_{sat} = \frac{\gamma_w(G_s + e)}{1 + e} = \frac{62.4\,(2.7 + 0.7)}{1 + 0.7} = 124.8 \text{ lb/ft}^3$$

$$\gamma_b = \frac{\gamma_w(G_s - 1)}{1 + e} = \frac{62.4\,(2.7 - 1)}{1 + 0.7} = 62.4 \text{ lb/ft}^3$$

$\sigma_1' = 10 \times 124.8 + 24 \times 62.4 = 2745.6 \text{ lb/ft}^2$

Effective pressure after lowering of water table

After lowering the water table to a depth of 20 ft, the soil above this level remains saturated but effective and below this submerged. Therefore, the altered effective pressure is

$\sigma_2' = 20\,\gamma_{sat} + (34 - 20)\,\gamma_b$

$= 20 \times 124.8 + 14 \times 62.4 = 3369.6 \text{ lb/ft}^2$

The increase in the effective pressure is

$\Delta\sigma' = \sigma_2' - \sigma_1' = 3369.6 - 2745.6 = 624.0 \text{ lb/ft}^2$

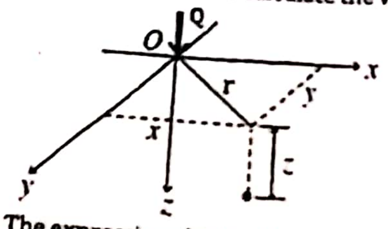
Stress distribution in soils due to Surface loads

Boussinesct's formula for point loads

Assumptions:

1. The soil mass is elastic, isotropic, homogeneous and semi-infinite.
2. The soil is weightless.
3. The load is a point load acting on the surface.

Consider we want to calculate the vertical stress increase at point A in figure below:



The expression obtained by Boussinesq for computing vertical stress σ_z at point P due to a point load Q is

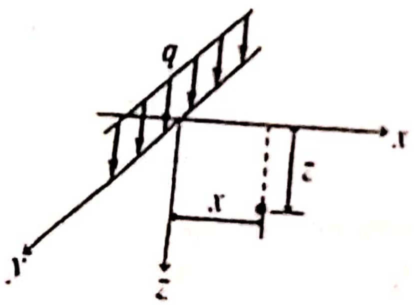
$$\sigma_z = \frac{3\,Q}{2\,\pi\,z^2}\frac{1}{[1+(r/z)^2]^{5/2}} = \frac{Q}{z^2}\,I_B$$

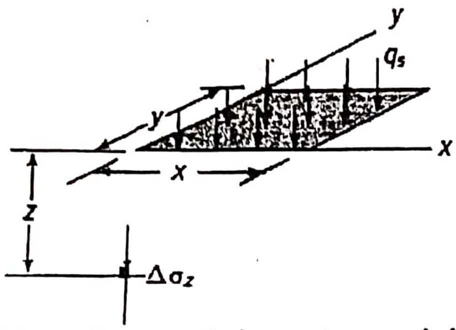
r = the horizontal distance between an arbitrary point P below the surface and the vertical axis through the point load Q.
z = the vertical depth of the point P from the surface.
I_B = Boussinesq stress coefficient

Boussinesct's formula for line loads

The vertical stress σ_z at point P may be written in rectangular coordinates as,

$$\sigma_z = \frac{2\,q}{\pi\,z}\frac{1}{[1+(x/z)^2]^2}$$





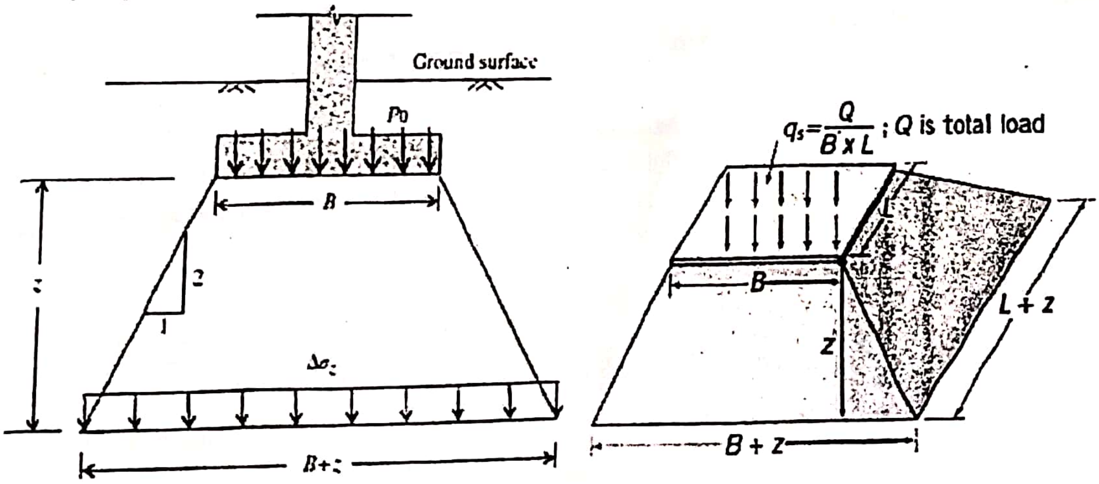
Only applies to vertical stress increase below the *corner*.
Under corner: $\Delta\sigma_z = q_s I_z$

$$I_z = \frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right) + \tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2-m^2n^2+1}\right)\right]$$

$m = B/z$ and $n = L/z$

Approximate 2:1 method

In this method, the stress is assumed to be distributed uniformly over areas lying below the foundation. The size of the area at any depth is obtained by assuming that the stresses spread out at an angle of 2 (vertical) to 1 (horizontal) from the edges of the loaded areas. The average stress at any depth *z* is



Under centre of the loaded area,

$$\Delta\sigma = \frac{q_s\,B\,L}{(B+z)\,(L+z)} = \frac{Q}{(B+z)\,(L+z)}$$

The approximate method is only used to estimate the vertical stress increase under the center of rectangular (or square) surface loads.

Question: A concentrated load of 1000 kN is applied at the ground surface. Compute the vertical pressure (*i*) at a depth of 4 m below the load, (*ii*) at a distance of 3 m at the same depth. Use Boussinesq's equation.

Solution:

(*i*) Vertical pressure at a depth of 4 m below the load

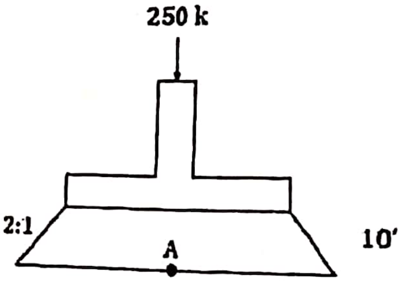
$$\sigma_z = \frac{3\,Q}{2\,\pi\,z^2}\frac{1}{[1+(r/z)^2]^{5/2}} = \frac{3 \times 1000}{2 \times 3.14 \times 4^2}\frac{1}{[1+(0/4)^2]^{5/2}} = 30\ \text{kN/m}^2$$

(*ii*) Vertical pressure at a distance of 3 m at the same depth

$$\sigma_z = \frac{3\,Q}{2\,\pi\,z^2}\frac{1}{[1+(r/z)^2]^{5/2}} = \frac{3 \times 1000}{2 \times 3.14 \times 4^2}\frac{1}{[1+(3/4)^2]^{5/2}} = 9.8\ \text{kN/m}^2$$

Question: Compute the stress at point A which is 10' below the base of the single column foundation carrying $Q = 250$ kip load and size = 10' x 10'; $\gamma = 110\ \text{lb/ft}^3$ (WASA - 2014)

Solution:



Pressure increases at point A due to 250 k load,

$$\Delta\sigma = \frac{Q}{(B+z)(L+z)} = \frac{250}{(10+10) \times (10+10)}$$

$\Delta\sigma = 0.625\ \text{k/ft}^2 = 625\ \text{lb/ft}^2$

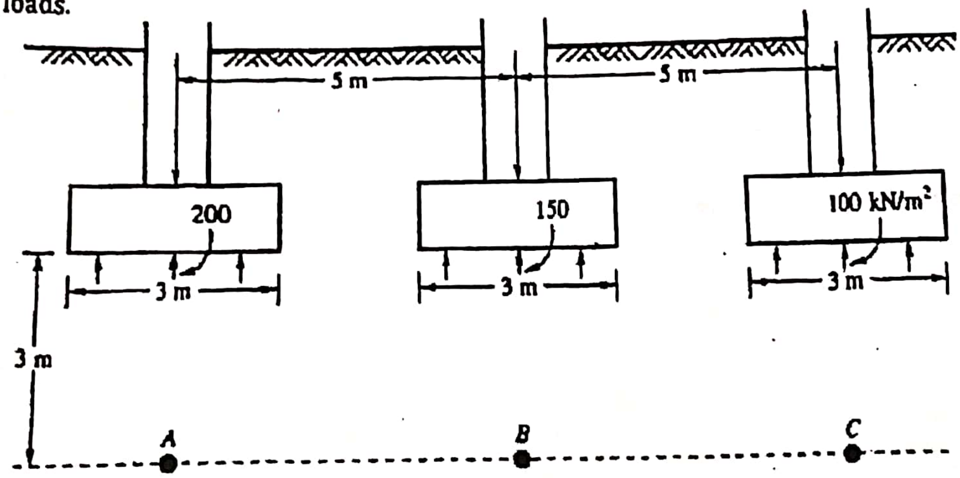
Stress at point A $= 10 \times 110 + 625 = 1725\ \text{lb/ft}^2$

Question: A square footing (2.5 m x 2.5 m) is to be placed on a homogeneous layer at 3 m depth. The load on footing base from corresponding column is 50 KN. Determine the net pressure on the base. (BWDB - 2014)

Solution:

$$\Delta q = \frac{Q}{B \times L} = \frac{50}{2.5 \times 2.5} = 8\ \text{kN/m}^2$$

Question: Three parallel strip footings 3 m wide each and 5 m apart center to center transmit contact pressures of 200, 150 and 100 kN/m²respectively. Calculate the vertical stress due to the combined loads beneath the centers of each footing at a depth of 3 m below the base. Assume the footings are placed at a depth of 2 m below the ground surface. Use Boussinesq's method for line loads.



Solution:

We know, $\sigma_z = \frac{2\,q}{\pi\,z}\frac{1}{[1+(x/z)^2]^2}$

The stress at A

$$\sigma_{z\,(A)} = \frac{2 \times 200}{3.14 \times 3}\frac{1}{[1+(0/3)^2]^2} + \frac{2 \times 150}{3.14 \times 3}\frac{1}{[1+(5/3)^2]^2} + \frac{2 \times 100}{3.14 \times 3}\frac{1}{[1+(10/3)^2]^2}$$

$\sigma_{z\,(A)} = 45\ \text{kN/m}^2$

The stress at B

$$\sigma_{z\,(B)} = \frac{2 \times 200}{3.14 \times 3}\frac{1}{[1+(5/3)^2]^2} + \frac{2 \times 150}{3.14 \times 3}\frac{1}{[1+(0/3)^2]^2} + \frac{2 \times 100}{3.14 \times 3}\frac{1}{[1+(5/3)^2]^2}$$

$\sigma_{z\,(B)} = 36.3\ \text{kN/m}^2$

The stress at C

$$\sigma_{z\,(C)} = \frac{2 \times 200}{3.14 \times 3}\frac{1}{[1+(10/3)^2]^2} + \frac{2 \times 150}{3.14 \times 3}\frac{1}{[1+(5/3)^2]^2} + \frac{2 \times 100}{3.14 \times 3}\frac{1}{[1+(10/3)^2]^2}$$

$\sigma_{z\,(C)} = 23.74\ \text{kN/m}^2$

COMPRESSIBILITY AND CONSOLIDATION

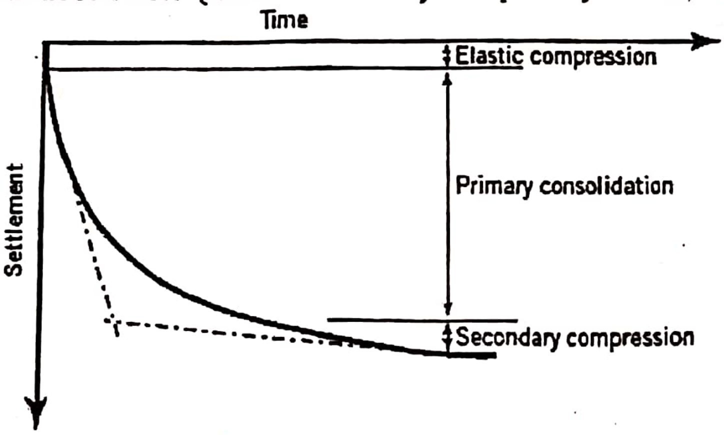
Settlement: Foundations should be designed for both shear failure and allowable settlement. So the allowable settlement of shallow foundations may control the allowable bearing capacity. The allowable settlement itself may be controlled by local building codes. For example; the maximum allowable settlement for mat foundation is 50 mm, and 25 mm for isolated footing.

Immediate or elastic settlement: Elastic or immediate settlement occurs during or immediately after the application of the load (construction of structure) without change in the moisture content of the soil. Elastic settlement is the settlement of a geosystem that can be recoverable upon unloading

Consolidation is the time-dependent settlement of soils resulting from the expulsion of water from the soil pores.

Primary consolidation is the change in volume of a fine-grained soil caused by the expulsion of water from the voids and the transfer of stress from the excess pore-water pressure to the soil particles.

Secondary compression is the change in volume of a fine-grained soil caused by the adjustment of the soil fabric (internal structure) after primary consolidation has been completed.



$S = S_i + S_c + S_s$

S = total settlement

S_i = Immediate elastic settlement

S_c = Primary consolidation settlement

S_s = Secondary consolidation settlement

Soil compaction: Soil compaction is defined as the method of mechanically increasing the density of soil. Air during compaction of soil is expelled from the void space in the soil mass and therefore the mass density is increased. Compaction increases the strength characteristics of soils, which increase the bearing capacity of foundations constructed over them. Compaction also decreases the amount of undesirable settlement of structures and increases the stability of slopes of embankments.

Consolidation is the reduction in volume of a soil due to the dissipation of excess pore water pressures. This phenomenon only happens in cohesive soils, since granular soils drain rapidly and do not generate the excess pore water pressures in the first place.

Normally consolidated clay: whose present effective overburden pressure is the maximum pressure that the soil was subjected to in the past is called normally consolidated clay.

Over consolidated clay: whose present effective overburden pressure is less than that which the soil experienced in the past is called over consolidated clay. The maximum effective past pressure is called the pre-consolidation pressure.

The overconsolidation ratio (OCR) for a soil is the ratio of preconsolidation pressure of a specimen to present effective vertical pressure.

Question: Write down the difference between compaction and consolidation. (DPDC – 2014, RAJUK – 2014, PGCB – 2017)

Solution:

Compaction	Consolidation
Compaction is a process where mechanical pressure is used to compress the soil mass for the purpose of soil improvement.	Consolidation is a process where steady and static pressure causes compression of saturated soil.
Dynamic loads by rapid mechanical methods like tamping, rolling and vibration are applied for a small interval in soil compaction.	Static and sustained loading is applied for a long interval in soil consolidation.
Compaction of soil is mainly used for sandy soil.	Consolidation of soil is mainly used for clayey soil.
Compaction process, soil volume is reduced by removing air void from the saturated and dry soil	Consolidation is due to expulsion of pore water from voids.
Compaction is done before the construction of structure	The process of consolidation starts as soon as the construction work begins.

Question: Define (a) normally consolidated soil (b) over consolidated soil (c) SPT (BHP – 2017)

Solution:

Normally consolidated clay: whose present effective overburden pressure is the maximum pressure that the soil was subjected to in the past is called normally consolidated clay.

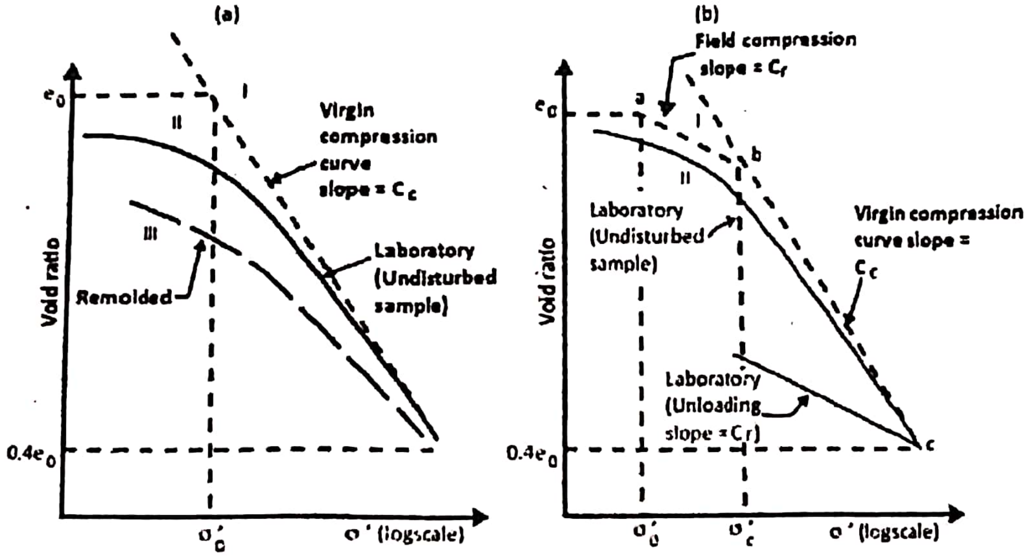
Over consolidated clay: whose present effective overburden pressure is less than that which the soil experienced in the past is called over consolidated clay. The maximum effective past pressure is called the pre-consolidation pressure.

The Standard Penetration test (SPT) is a common in situ testing method used to determine the geotechnical engineering properties of subsurface soils. It is a simple and inexpensive test to estimate the relative density of soils and approximate shear strength parameters. It produces an N-value, which represents the number of blows of a standardized sampler driven into the soil a standardized distance.

Question: Define virgin consolidation curve. Which one do you prefer as s soil engineer in between laboratory consolidation curve and virgin consolidation curve? Explain with necessary diagram and examples. (JB – 2017)

Solution:

Soil Samples collected from field are somehow disturbed. When consolidation tests are conducted on these samples, we obtain e vs. log σ' plots that are slightly different from those in the field. This is demonstrated in Figure.



Curve I in shows the nature of the e vs. log σ' variation that an undisturbed normally consolidated clay (present effective overburden pressure; void ratio) in the field would exhibit. This is called the virgin compression curve. A laboratory consolidation test on a carefully recovered sample would result in e vs. log plot such as curve II. Due to obtain approximately corrected result laboratory compression curve is preferred to obtain actual form of virgin compression curve. As example the compression index as determined from the laboratory e-log σ' curve different from that encountered in the field. Primary reason is that soil remolds to some degree during field exploration. The virgin compression curve intersects the laboratory compression curve at $0.42e_0$. Knowing the value of e_0 *and* p_c (Pre-Consolidation Pressure) can be easily construct the virgin compression curve to calculate compression index.

Question: What is meant by OCR? Difference between OC clay and NC clay. How will you find the preconsolidation pressure for clay? (32th BCS, BKB – 2018)

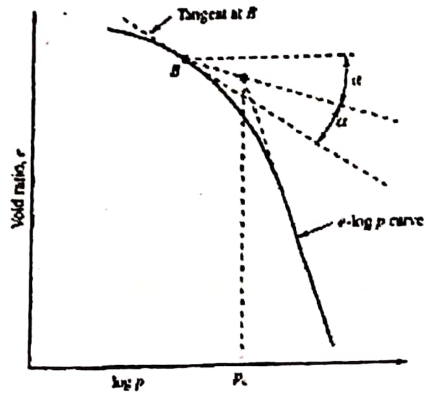
Solution:

The "over-consolidation ratio" (OCR) is defined as the highest stress experienced divided by the current stress. A soil that is currently experiencing its highest stress is said to be "normally consolidated" and has an OCR of one.

Normally consolidated clay: whose present effective overburden pressure is the maximum pressure that the soil was subjected to in the past is called normally consolidated clay.

Over consolidated clay: whose present effective overburden pressure is less than that which the soil experienced in the past is called over consolidated clay. The maximum effective past pressure is called the pre-consolidation pressure.

There are a few graphical methods for determining the preconsolidation pressure based on laboratory test data. No suitable criteria exists for appraising the relative merits of the various methods. The earliest and the most widely used method was the one proposed by Casagrande (1936). The method involves locating the point of maximum curvature, B, on the laboratory e-log *p* curve of an undisturbed sample as shown in Fig. From *B*, a tangent is drawn to the curve and a horizontal line is also constructed. The angle between these two lines is then bisected. The abscissa of the point of intersection of this bisector with the upward extension of the inclined straight part corresponds to the preconsolidation pressure p_c.



Question: Define over consolidation ratio (OCR) & Permeability. (BUET M. Sc. 2013)

Solution:

The pre-consolidation stress is defined to be the maximum effective stress experienced by the soil. If the current effective stress is less than the pre-consolidation stress, then the soil is said to be over-consolidated (OC). If OCR=1 the soil is normally consolidated (NC), if OCR>1 the soil is over consolidated (OC). Soil deformation is higher in normally consolidated soil than it is over consolidated soil.

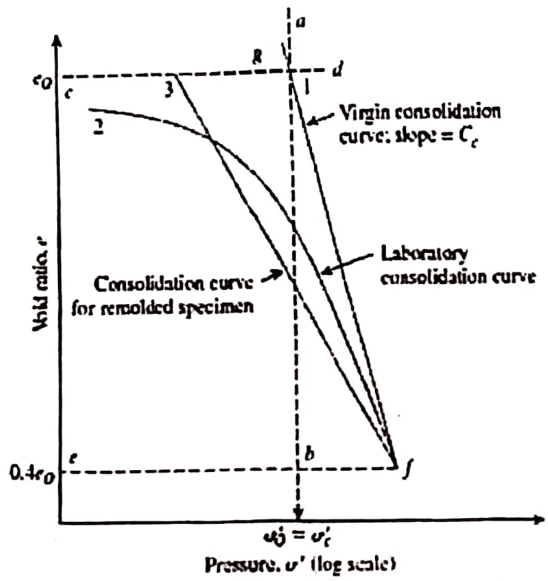
Permeability in fluid mechanics and the earth sciences is a measure of the ability of a porous material (often, a rock or an unconsolidated material) to allow fluids to pass through it.

Question: What do you know by normally loaded and preloaded clay? Draw e-log P curve for both Condition.

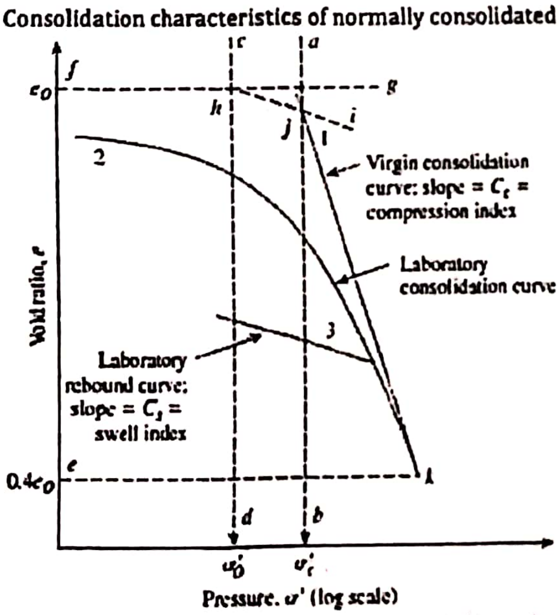
Solution:

- If the initial vertical effective stress is approximately equals to the pre-consolidation stress then this condition is known as normally consolidated." Or "The soil whose present effective overburden pressure is the maximum pressure that the soil was subjected to in the past.
- Over consolidated soil may be defined as "If the initial vertical effective stress of the soil sample is less than the pre-consolidation stress, then the vertical effective stress in the field was

once higher than its current magnitude, and the soil is called over consolidated." Or "The soil whose present effective overburden pressure is less than that which the soil experienced in the past is called over consolidated soil.



Consolidation characteristics of normally consolidated clay of low to medium sensitivity



Consolidation characteristics of overconsolidated clay of low to medium sensitivity

Settlement Calculation for normally consolidated Clay Soil

Compression Index: The compression index, C_c is the slope of the straight-line portion of the loading curve. The compression index as determined from the laboratory e–log σ curve, will be somewhat different from that encountered in the field. The primary reason is that the soil remolds itself to some degree during the field exploration. The nature of variation of the e–log σ curve in the field for a normally consolidated clay is shown in Figure. The curve generally referred to as the virgin compression curve.

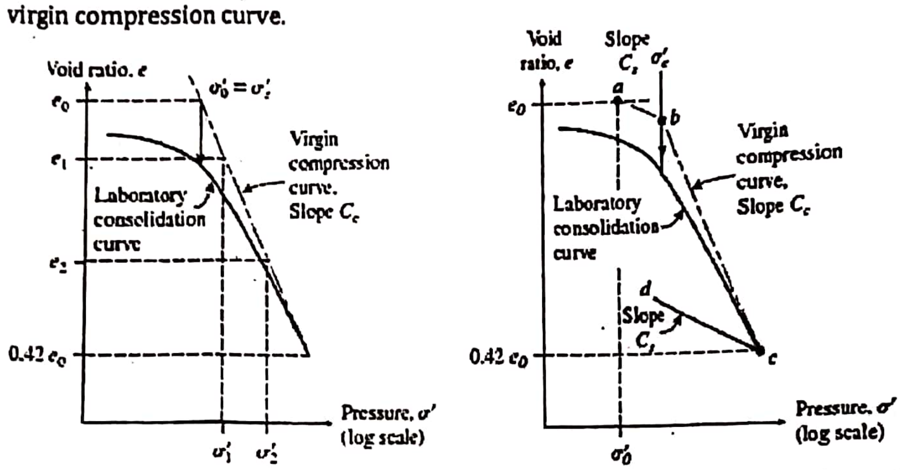


Figure: Construction of virgin compression curve for normally consolidated clay and Construction of field consolidation curve for overconsolidated clay.

$$C_c = \frac{e_1 - e_2}{\log \sigma - \log \sigma_0} = \frac{\Delta e}{\log \sigma / \sigma_0}$$

For normally consolidated clays, $C_c = 0.009\,(LL - 10)$
For remolded clays, $C_c = 0.007\,(LL - 10)$

Swelling Index: The swelling index, C_s is the slope of the unloading portion of the e–log σ curve. The swelling index is also referred to as the recompression index.

$$C_s = \frac{C_c}{5}$$

Calculation of Primary Consolidation Settlement

The one-dimensional primary consolidation settlement (caused by an additional load) of a clay layer having a thickness H may be calculated as

$$S_c = \frac{\Delta e}{1 + e_0} H$$

For normally consolidated clay

$$\Delta e = C_c \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

$$S_c = \frac{C_c H}{1 + e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

S_c = primary consolidation settlement
Δe = total change of void ratio caused by the additional load application
e_0 = void ratio of the clay before the application of load

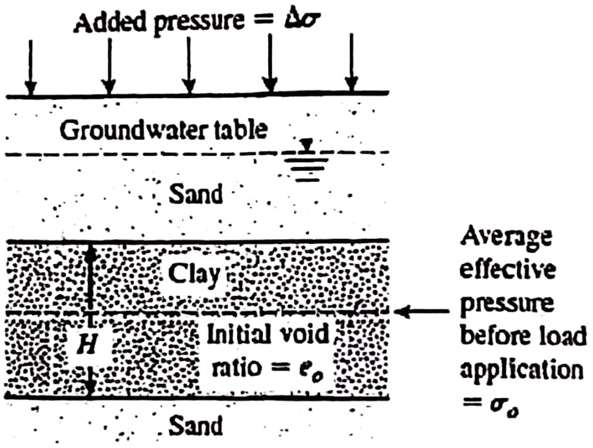


Figure: One-dimensional settlement calculation

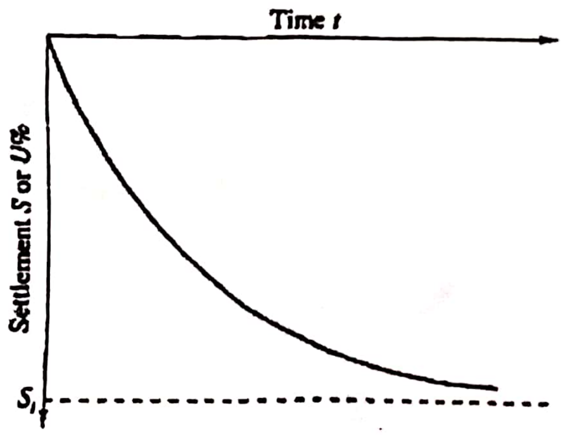
<u>For over consolidated Clay Soil</u>

Settlement Computation, if $\sigma_o + \Delta\sigma \leq \sigma_c$

$$S_c = \frac{C_s H}{1+e_0} log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

Settlement Computation, if $\sigma_o < \sigma_c < \sigma_o + \Delta\sigma$

$$S_c = \frac{C_s H}{1+e_0} log \frac{\sigma_c}{\sigma_0} + \frac{C_c H}{1+e_0} log \frac{\sigma_0 + \Delta\sigma}{\sigma_c}$$



Time-settlement curve

Time Rate of Consolidation

Consolidation is the result of the gradual dissipation of the excess pore water pressure from a clay layer. The dissipation of pore water pressure increases the effective stress which induces settlement. Hence, to estimate the degree of consolidation of a clay layer at some time *t* after the load is applied, you need to know the rate of dissipation of the excess pore water pressure.

Coefficient of consolidation, $C_v = \dfrac{k}{m_v\, \gamma_w}$

Coefficient of volume compressibility, $m_v = \dfrac{a_v}{1 + e_{av}}$

Coefficient of compressibility, $a_v = \dfrac{\Delta e}{\Delta \sigma} = \dfrac{e_0 - e}{\sigma - \sigma_0}$

$$S_c = \frac{\Delta e}{1 + e_{av}} H$$

$$S_c = \frac{a_v\, \Delta\sigma\, H}{1 + e_{av}} = \frac{a_v}{1 + e_{av}} H\, \Delta\sigma = m_v\, H\, \Delta\sigma$$

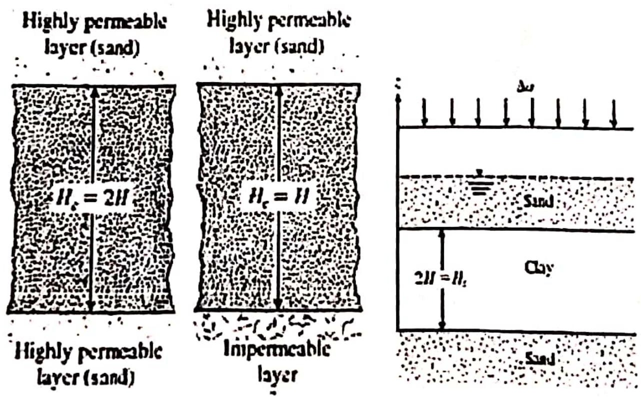
k = co-efficient of permeability in cm/sec
γ_w = unit weight of water in gm/cc
m_v = coefficient of volume change in cm²/gm
C_v = Coefficient of compressibility in cm²/sec

Time factor, $T_v = \dfrac{C_v\, t}{H^2}$

For $U = 0$ *to* $60\%, T_v = \dfrac{\pi}{4}\left(\dfrac{U\,\%}{100}\right)^2$

$U > 60\%, T_v = 1.781 - 0.933 \log\,(100 - U\%)$

U = degree of consolidation
t = time required for U% consolidation in sec
H = maximum length of drainage path in cm
C_v = Co-efficient of permeability in cm²/sec



Question: A recently completed fill was 32.8 ft thick and its initial average void ratio was 1.0. The fill was loaded on the surface by constructing an embankment covering a large area of the fill. Some months after the embankment was constructed, measurements of the fill indicated an average void ratio of 0.8. Estimate the compression of the fill.

Solution:

$$S_c = \frac{\Delta e}{1+e_0}H = \frac{1-0.8}{1+1} \times 32.8 = 3.28 \text{ ft}$$

Question: The intial void ratio of a 5 m thick clay layer is 1.1, If the difference of void ratio is 0.3 than find out the the settlement. (BUET M.Sc – 2018)

Solution:

$$S_c = \frac{\Delta e}{1+e_0}H = \frac{0.3}{1+1.1} \times 5 = 0.71 \text{ m}$$

Question: The initial void ratio of a 5m thick underlying saturated clay layer is 1.4. Due to the weight of a superimposed embankment the void ratio of clay layer decrease from 1.4 to 1.1. Determine the consequent settlement of that clay layer. (DPDC – 2019)

Solution:

$$S_c = \frac{\Delta e}{1+e_0}H = \frac{1.4-1.1}{1+1.4} \times 5 = 0.625 \text{ m}$$

Question: A 3 m thick layer (double drainage) of saturated clay under a surcharge loading underwent 90% primary consolidation in 75 days. Find the coefficient of consolidation of clay for the pressure range. (PGCL – 2017)

Solution:

$U > 60\%, T_v = 1.781 - 0.933 \log (100 - U\%)$
$T_v = 1.781 - 0.933 \log (100 - 90) = 0.848$
Because the clay layer has two way drainage, H = 3/2 = 1.5 m

$$\text{Time factor}, T_v = \frac{C_v t}{H^2}$$

$$C_v = \frac{T_v H^2}{t} = \frac{0.848 \times 1.5^2}{75 \times 24 \times 60 \times 60} = 0.00294 \text{ cm}^2/\text{sec}$$

Question: In a laboratory consolidation test, the void ratio of the sample reduced from 0.85 to 0.73 as the pressure was increased from 1 to 2 kg/cm². If the co-efficient of permeability of the soil be 3.3×10^{-4} cm/sec, determine the coefficient of volume change and Co-efficient of consolidation.

Solution:

$\Delta e = 0.85 - 0.73 = 0.12$
$\Delta\sigma = 2 - 1 = 1 \text{ cm}^2/\text{kg}$

$$\text{Coefficient of volume compressibility}, m_v = \frac{a_v}{1+e_0} = \frac{\Delta e}{\Delta\sigma(1+e_0)}$$

$$m_v = \frac{\Delta e}{\Delta\sigma\,(1 + e_0)} = \frac{0.12}{1\,(1 + 0.85)} = 0.065\ cm^2/kg = 6.5 \times 10^{-5}\ cm^2/g$$

$$C_v = \frac{k}{m_v\,\gamma_w} = \frac{3.3 \times 10^{-4}}{6.5 \times 10^{-5} \times 1} = 5.07\ cm^2/sec$$

Question: The time required to reach 50% consolidation for a soil specimen of 3 cm thick tested in a consolidometer under single drainage condition was 30 minutes. Determine the time required for the same soil of 4 m thick to reach the same degree of consolidation, it has double drainage paths. (30th BCS)

Solution:

For first case:

For 50% consolidatio, $T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 = \frac{\pi}{4}\left(\frac{50}{100}\right)^2 = 0.196$

$H = 3$ cm and time for 50% consolidation, $t_{50} = 30$ minutes $= 0.5$ hour

Coefficient of consolidatio, $C_v = \frac{T_v\,H^2}{t_{50}} = \frac{0.196 \times 3^2}{0.5} = 3.528\ cm^2/hr$

For second case:

$H = 4/2 = 2\ m = 200$ cm (double drainage)

Time required for 50% consolidation, $t_{50} = \frac{T_v\,H^2}{C_v} = \frac{0.196 \times 200^2}{3.528} = 2222.22$ hour

Question: A raft footing is to be constructed on a 7.5 m thick clay layer which lies between two sand layers. In order to predict the time rate of settlement of the building, a 2.5 cm thick undisturbed sample of the soil was tested in the laboratory under double drainage condition. The sample was found to have undergone 50% consolidation in 12.5 minutes. Determine the time required for 50% settlement of the building.

Solution:

As $U = 50\%$, $T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$

$T_v = \frac{\pi}{4}\left(\frac{50}{100}\right)^2 = 0.197$

Because the clay layer has two way drainage, H = 2.5/2 = 1.25 cm

Time factor, $T_v = \frac{C_v\,t}{H^2}$

$$C_v = \frac{T_v\,H^2}{t} = \frac{0.197 \times 1.25^2}{12.5} = 0.0246\ cm^2/min$$

In case of the actual building,

$$H = \frac{7.5 \times 100}{2} = 375\ cm$$

$$t = \frac{T_v\,H^2}{C_v} = \frac{0.197 \times 375^2}{0.0246} = 1126146.29\ min = 782\ days$$

Question: Differentiate between normally consolidated and over consolidated soil. A clay stratum 5 m thick has initial void ratio of 1.50 and the effective overburden pressure of 120 kN/m², when the sample is subjected to an increased pressure of 120 kN/m² the void ratio reduces to 1.44. Determine the coefficient of volume compressibility and the final settlement of the stratum. (35th BCS)

Solution:

Normally consolidated clay: whose present effective overburden pressure is the maximum pressure that the soil was subjected to in the past is called normally consolidated clay.

Over consolidated clay: whose present effective overburden pressure is less than that which the soil experienced in the past is called over consolidated clay. The maximum effective past pressure is called the pre-consolidation pressure.

$\Delta e = 1.50 - 1.44 = 0.06$
$\Delta\sigma = 120\ kN/m^2$

Coefficient of volume compressibility, $m_v = \dfrac{\Delta e}{\Delta\sigma\,(1 + e_0)}$

$$m_v = \frac{0.06}{120\,(1 + 1.5)} = 2 \times 10^{-4}\ m^2/kN$$

$$\text{Settlement, } S_c = \frac{\Delta e}{1+e_0}H = \frac{0.06}{1+1.5} \times 5 = 0.12\ m$$

Question: 2 m deep clay layer, coefficient of volume compressibility = 0.02 cm²/kg. Effective pressure increases from 2 to 4 kg/cm². What is the settlement of the clay layer? (NPCBL – 2017)

Solution:

$H = 2\,m = 200\ cm, \Delta\sigma = 4 - 2 = 2\ cm^2/kg$

$S_c = m_v\, H\, \Delta\sigma = 0.02 \times 2 \times 200 = 8\ cm$

Question: Determine the co-efficient of volume change if the settlement is 5 cm for 7.5 m clay layer having increasing pressure of 80 kg/cm². (CPGCBL – 2018)

Solution:

$S_c = m_v\, H\, \Delta\sigma$

$5 = m_v \times 80 \times 7.5 \times 100$

$m_v = 8.33 \times 10^{-5}\ cm^2/kg$

Question: A clay layer of 2 m thick has a coefficient of volume compressibility 0.02 cm²/kg. If effective stress increases to 2 kg/cm² what is the settlement of the clay layer? (NPCBL – 2019)

Solution:

$H = 2\,m = 200\ cm, m_v = 0.02\ cm^2/kg, \Delta\sigma = 2\ kg/cm^2$

$S_c = m_v\, H\, \Delta\sigma = 0.02 \times 2 \times 200 = 8\ cm$

Question: A clay layer 5 m depth has a settlement of 7.5 cm. find the coefficient of volume change if effective overburden pressure increases 20 KN/m² due to construction of a building. (CBGCBL – 2018)

Solution:

$H = 7.5 \text{ cm} = 0.075 \text{ m}$

$S_c = m_v\, H\, \Delta\sigma$

$0.075 = m_v \times 5 \times 20$

Coefficient of volume change, $m_v = 8.33 \times 10^{-5} \text{ m}^2/\text{kN}$

Question: A clay stratum 5 m thick has the initial void ratio of 1.50 and the effective overburden pressure of 120 KN/m². When the sample is subjected to an increased pressure of 120 KN/m², the void ratio reduces to 1.44. Determine the co-efficient of volume compressibility and final settlement of the stratum. (34th BCS)

Solution:

$\Delta e = 1.50 - 1.44 = 0.06$

$\Delta\sigma = 120 \text{ kN/m}^2$

Coefficient of volume compressibility, $m_v = \dfrac{\Delta e}{\Delta\sigma\,(1 + e_0)}$

$$m_v = \frac{0.06}{120\,(1 + 1.5)} = 2 \times 10^{-4} \text{ m}^2/\text{kN}$$

$$\text{Settlement, } S_c = \frac{\Delta e}{1 + e_0} H = \frac{0.06}{1 + 1.5} \times 5 = 0.12 \text{ m}$$

Question: A strata of normally consolidated clay of thickness 10 ft is drained on one side only. It has a hydraulic conductivity of $k = 1.863 \times 10^{-8}$ in/sec and a coefficient of volume compressibility $m_v = 8.6 \times 10^{-4}$ in²/lb. determine the ultimate value of the compression of the stratum by assuming a uniformly distributed load of 5250 lb/ft² and also determine the time required for 20 percent and 80 percent consolidation.

Solution:

$$\text{Total compression, } S_c = m_v\, H\, \Delta\sigma = 8.6 \times 10^{-4} \times 10 \times 12 \times 5250 \times \frac{1}{144} = 3.763 \text{ in}$$

$$\text{For 20\% consolidatio, } T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 = \frac{\pi}{4}\left(\frac{20}{100}\right)^2 = 0.0314$$

$$\text{For 80\% consolidatio, } T_v = 1.781 - 0.933 \log\,(100 - U\%)$$

$$= 1.781 - 0.933 \log\,(100 - 80) = 0.567$$

$$\text{Coefficient of consolidatio, } C_v = \frac{k}{m_v\,\gamma_w} = \frac{1.863 \times 10^{-8}}{8.6 \times 10^{-4} \times 3.61 \times 10^{-2}} = 6 \times 10^{-4} \text{ in}^2/\text{sec}$$

Time required for 20% consolidation, $t_{20} = \frac{T_v H^2}{C_v} = \frac{0.0314 \times (10 \times 12)^2}{6 \times 10^{-4} \times 60 \times 60 \times 24} = 8.72$ days

Time required for 80% consolidation, $t_{80} = \frac{T_v H^2}{C_v} = \frac{0.567 \times (10 \times 12)^2}{6 \times 10^{-4} \times 60 \times 60 \times 24} = 157.5$ days

Question: A clay layer 4 m thick us subjected to a pressure of 55 m²/kN. If the layer has a double drainage and undergoes 50% consolidation in one year, determine the coefficient of consolidation. Take $T_v = 0.196$. If the coefficient of permeability is 0.02 m/yr, determine the settlement in one year and rate of flow of water per unit area in one year. (38th BCS)

Solution:

$$C_v = \frac{T_v H^2}{t} = \frac{0.196 \times 2^2}{1} = 0.784 \text{ m}^2/\text{yr}$$

$$m_v = \frac{k}{C_v \gamma_w} = \frac{0.02 \times 1000}{0.784 \times 1000 \times 9.81} = 2.60 \times 10^{-3} \text{ m}^2/\text{kN}$$

$$S_c = m_v H_0 \Delta\sigma = 2.60 \times 10^{-3} \times 4 \times 55 = 0.572 \text{ m}$$

Settlement after one year $= 0.5 \times 0.572 = 0.286$ m

Since U is proportional to $\sqrt{t}$ for $U < 0.60$, the settlement is also proportional to $\sqrt{t}$

$s^2 \propto t$ or $t = C s^2$

When $t = 1$ year, $s = 0.286$ m

$$C = \frac{1}{0.286^2} = 12.226$$

Thus, $t = 12.226\, s^2$

$$\frac{ds}{dt} = \frac{1}{2 \times 12.226\, s} = \frac{1}{24.452 \times 0.286} = 0.143 \text{ m/yr}$$

Discharge per unit area per surface $= \frac{0.143}{2} = 0.072$ m³/yr /m²

Question: A layer of clay 2.0 m thick is subjected to a loading of 0.5 kg/cm². After one year loading the average consolidation is 50%. The layer has double drainage; (*i*) what is the co-efficient of consolidation? (*ii*) If the co-efficient of permeability is 3 mm/yr, what is the settlement after one year? (*iii*) How much time will the layer takes to reach 90% consolidation assume, $T_{v2} = 0.848$. (37th BCS)

Solution:

Co-efficient of consolidation

As $U = 50\%$, $T_v = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2$

$$T_v = \frac{\pi}{4}\left(\frac{50}{100}\right)^2 = 0.197$$

Time required for 50% consolidation, $t = 1$ year

Because the clay layer has two way drainage, $H = 2/2 = 1.0$ m

61

Time factor, $T_v = \frac{C_v t}{H^2}$

Coefficient of consolidation, $C_v = \frac{T_v H^2}{t} = \frac{0.197 \times 1^2}{1} = 0.197\ m^2/year$

Settlement after one year

co – efficient of permeability, $k = 3\ mm/yr = 3 \times 10^{-3}\ m/yr$

$\Delta\sigma = 0.5\ kg/cm^2 = \frac{0.5 \times 9.81 \times 100 \times 100}{1000} = 49.05\ kN/m^2$

co – efficient of permeability, $k = C_v m_v \gamma_w$

$m_v = \frac{k}{C_v \gamma_w} = \frac{3 \times 10^{-3}}{0.197 \times 9.81} = 1.55 \times 10^{-3}\ m^2/kN$

$S_c = m_v H_0 \Delta\sigma$

$S_c = 1.55 \times 10^{-3} \times 1 \times 49.05 = 0.076\ m$

Settlement after one year $= 0.5 \times 0.076 = 0.038\ m$

Time to reach 90% consolidation

Time required for 90% consolidation, $t_{90} = \frac{T_v H^2}{C_v} = \frac{0.848 \times 1^2}{0.197} = 4.304$ years

Question: Compute consolidation settlement of a 2.5 m thick clay layer due to an increase of 30 KN/m² pressure at the mid height of the layer. If vertical stress at the mid height of the layer is 130 KN/m². Given, initial void ratio (e_0) = 0.8 and compression index (C_c) = 0.28. (WRGCL – 2014)

Solution:

$$S_c = \frac{C_c H}{1 + e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

$$S_c = \frac{0.28 \times 2.5}{1 + 0.8} \log \frac{130 + 30}{130} = 0.035\ m$$

Question: Compute consolidation settlement of a 2.5 m thick clay layer due to an increase of 30 KN/m² pressure at the mid height of the layer. If vertical stress at the mid height of the layer is 40 KN/m². The value of initial void ratio is 0.7 and compression index is 0.28. (PGCB – 2015, GTCL – 2016, SGCL – 2017, ERL – 2017, BWDB – 2018, NHA – 2020)

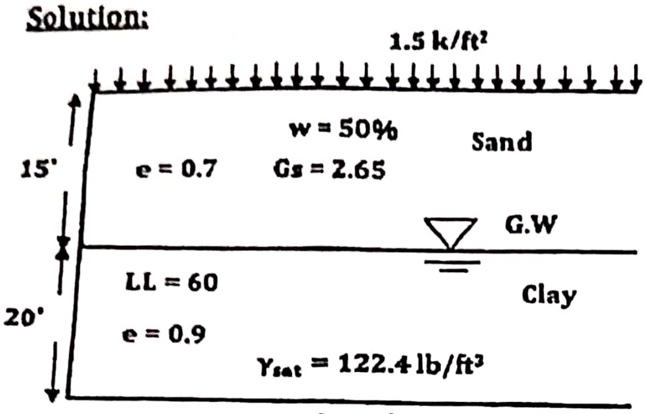
Solution:

$$S_c = \frac{C_c H}{1 + e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

$$S_c = \frac{0.28 \times 2.51}{1 + 0.7} \log \frac{40 + 30}{40} = 0.100\ m$$

Question: Determine the primary settlement of the clay layer. The soil is normally consolidated. (PGCB – 2018)

Solution:



For sand layer, $\gamma_{bulk} = \dfrac{G_s\,\gamma_w(1+w)}{1+e} = \dfrac{2.65 \times 62.4(1+0.5)}{1+0.7} = 145.9\ \text{lb/ft}^3$

Existing presure at centre of clay layer, $\sigma_0 = \gamma\, h + (\gamma_{sat} - \gamma_w)\, h_1$

$$\sigma_0 = 145.9 \times 15 + (122 - 62.4)\frac{20}{2} = 2784.5\ \text{lb/ft}^2 = 2.7845\ \text{k/ft}^2$$

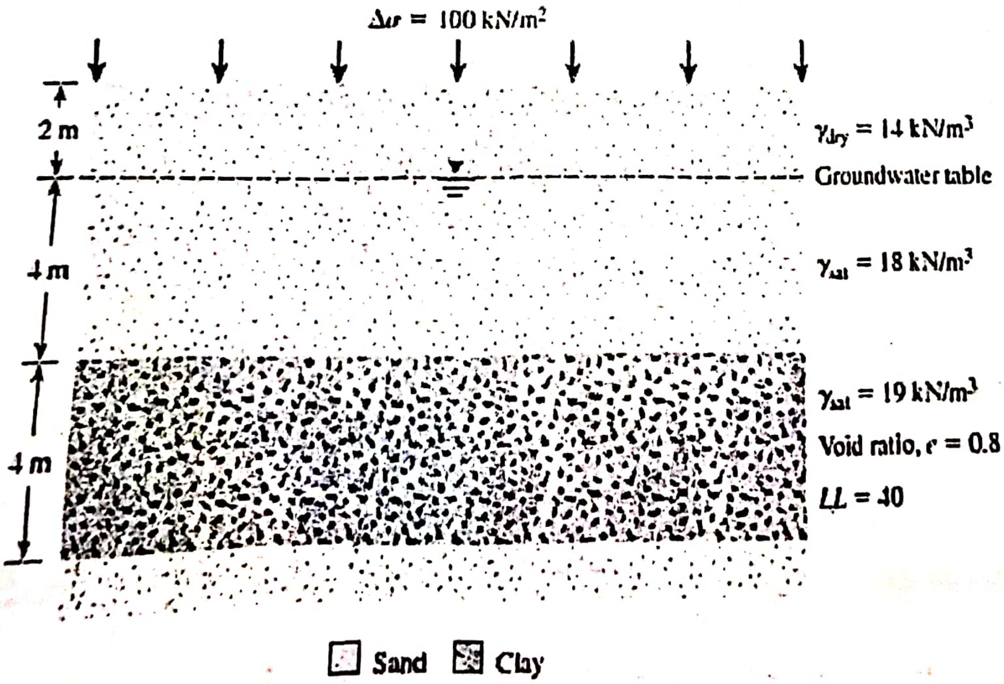
Overburden pressure, $\Delta\sigma = 1.5\ \text{k/ft}^2$

Compression index, $C_c = 0.009\,(LL - 10) = 0.009\,(60 - 10) = 0.45$

For normally consolidated clay,

$$\Delta H = \frac{C_c\, H}{1+e_0} log \frac{\sigma_0 + \Delta\sigma}{\sigma_0} = \frac{0.45 \times 20}{1+0.9} \log \frac{2.7845 + 1.5}{2.7845} = 0.9\ \text{ft}$$

Question: A soil profile is shown in Figure. If a uniformly distributed load $\Delta\sigma$, is applied at the ground surface, what is the settlement of the clay layer caused by primary consolidation? If the clay is normally consolidated the pre-consolidation pressure 160 KN/m². Cc = 0.27, Cs = 0.045 (BWDB – 2019)



63

idated.

Solution:

The average effective stress at the middle of the clay layer is,

$\sigma_0 = 2 \times 14 + 4 \times (18 - 9.81) + 2\,(19 - 9.81) = 79.14\ \text{KN/m}^2$

Overburden pressure, $\Delta\sigma = 100\ \text{KN/m}^2$

Pre – consolidation pressure, $\sigma_c = 160\ \text{KN/m}^2$

Here, $\sigma_0 < \sigma_c < \sigma_0 + \Delta\sigma$

$$S_c = \frac{C_c H}{1 + e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_c} + \frac{C_s H}{1 + e_0} \log \frac{\sigma_c}{\sigma_0}$$

$$S_c = \frac{0.27 \times 4}{1 + 0.8} \log \frac{79.14 + 100}{160} + \frac{0.045 \times 4}{1 + 0.8} \log \frac{160}{79.14}$$

$S_c = 0.029 + 0.030 = 0.059\ \text{m}$

Question: Soil investigation at a site gave the following information. Fine sand exists to a depth of 10.6 m and below this lies a soft clay layer 7.60 m thick. The water table is at 4.60 m below the ground surface. The submerged unit weight of sand γ_b is 10.4 kN/m³, and the wet unit weight above the water table is 17.6 kN/m³. The water content of the normally consolidated clay w_n = 40%, its liquid limit w_l = 45%, and the specific gravity of the solid particles is 2.78. The proposed construction will transmit a net stress of 120 kN/m² at the center of the clay layer. Find the average settlement of the clay layer.

Solution:

$C_c = 0.009\,(LL - 10) = 0.009\,(45 - 10) = 0.32$

Void ratio, $e_0 = \dfrac{w\,G}{S} = \dfrac{0.40 \times 2.78}{1} = 1.11$

Saturated unit weight, $\gamma_{sat} = \dfrac{(G_s + S\,e_0)\gamma_w}{1 + e_0} = \dfrac{(2.78 + 1 \times 1.11)\,9.81}{1 + 1.11} = 18.1\ \text{KN/m}^3$

Submerged unit weight, $\gamma_b = \gamma_{sat} - \gamma_w = 18.1 - 9.81 = 8.28\ \text{KN/m}^3$

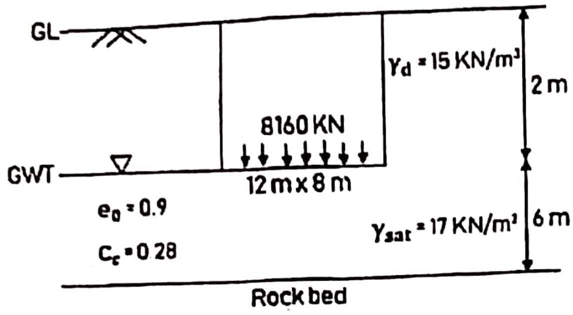
Effective vertical stress σ_0 at the mid height of the clay layer is

$$\sigma_0 = 4.60 \times 17.6 + 6 \times 10.4 + \frac{7.60}{2} \times 8.28 = 174.8\ \text{KN/m}^2$$

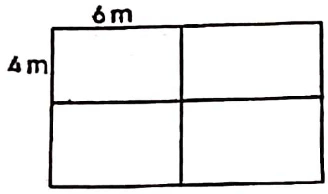
$$S_c = \frac{C_c H}{1 + e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0} = \frac{0.32 \times 7.60}{1 + 1.11} \log \frac{174.8 + 120}{174.8} = 0.26\ \text{m} = 26\ \text{cm}$$

Average settlement = 26 cm.

Question: Determine primary settlement at the center of raft foundation. Assume normally consolidated soil. (RPGCL – 2017)



Solution:



$P_0 = 15 \times 2 + 3 \times (17 - 10) = 51 \text{ KN/m}^2$

$m = \frac{x}{z} = \frac{6}{3} = 2$

$m = \frac{x}{z} = \frac{4}{3} = 1.33$

z = distance between bottom of footing and mid clay layer

$q = \frac{8160}{8 \times 12} = 85 \text{ KN/m}^2$

Contact pressure, $q_0 = 85 - 2 \times 15 = 55 \text{ KN/m}^2$

$$I = \frac{1}{4\pi}\left[\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+m^2n^2+1}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right)+\tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2-m^2n^2+1}\right)\right]$$

$I = 0.107$

$\Delta P = 4I \times q_0 = 4 \times 0.107 \times 55 = 38.52 \text{ KN/m}^2$

$C_c = 0.28, e_o = 0.9, H = 3\text{m} = 3000 \text{ mm}$

$$\Delta H = \frac{C_c H}{1 + e_o} \log \frac{P_0 + \Delta P}{P_o}$$

$$\Delta H = \frac{0.28 \times 3000}{1 + 0.9} \log \frac{55 + 38.52}{55} = 101.92 \text{ mm}$$

Question: In a consolidation test, the void ratio of the specimen which was 1.068 under the effective pressure of 214 KN/m², changed to 0.994 when the pressure was increased to 429 KN/m². Calculate the coefficient of compressibility, compression index and the coefficient of volume compressibility. Find the settlement of foundation resting on above type of clay, if thickness of layer is 8 m and the increase in pressure is 10 KN/m².

Solution:

Here, $e_0 = 1.068, e = 0.994, \sigma_0 = 214 \text{ kN/m}^2, \sigma = 429 \text{ kN/m}^2$

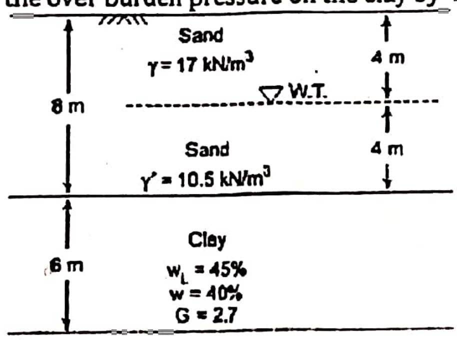
Coefficient of compressibility, $a_v = \frac{\Delta e}{\Delta \sigma} = \frac{e_0 - e}{\sigma - \sigma_0} = \frac{1.068 - 0.994}{429 - 214} = 3.442 \times 10^{-4} \text{ m}^2/\text{kN}$

Compression index, $C_c = \frac{\Delta e}{\log \sigma/\sigma_0} = \frac{1.068 - 0.994}{\log 429/214} = 0.245$

Coefficient of volume change, $m_v = \frac{a_v}{1 + e_0} = \frac{3.442 \times 10^{-4}}{1 + 1.068} = 1.664 \times 10^{-4} \text{ m}^2/\text{kN}$

Settlement, $S_c = \frac{C_c H}{1 + e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0} = \frac{0.245 \times 8}{1 + 1.068} \log \frac{214 + 10}{214} = 0.0188 \text{ m}$

Question: A stratum of clay with an average liquid limit of 45% is 6 m thick. Its surface is located at a depth of 8 m below the ground surface. The natural water content of clay is 40% and the specific gravity is 2.7. Between ground surface and clay the subsoil consists of fine sand. The water is located at a depth of 4 m below the ground surface. The average submerged unit weight of sand is 10.5 kN/m³ and the weight of building that will be constructed on the clay above clay increases the over burden pressure on the clay by 40 kN/m². Estimate the settlement of building.



Solution:

$e_0 = w\,G = 0.4 \times 2.7 = 1.08$

For clay layer, $\gamma_{bulk} = \frac{G_s \gamma_w (1 + w)}{1 + e_0} = \frac{2.7 \times 9.81\,(1 + 0.4)}{1 + 1.08} = 17.82 \text{ kN/m}^3$

Pressure on the top of clay due to overburden, $\sigma_0 = 17 \times 4 + 10.5 \times 4 + 3 \times 17.82$

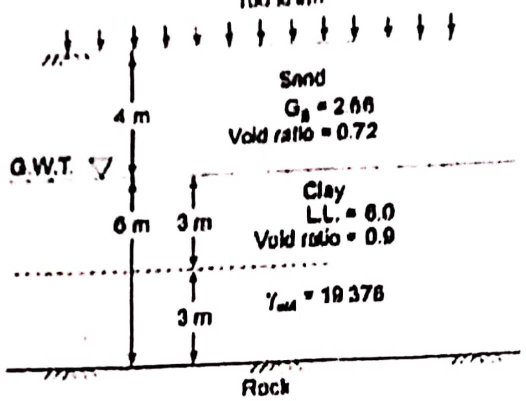
$\sigma_0 = 163.46 \text{ kN/m}^2$

Increase in pressure due to construction of building, $\Delta\sigma = 40 \text{ kN/m}^2$

$C_c = 0.009\,(LL - 10) = 0.009\,(45 - 10) = 0.315$

Settlement, $S_c = \frac{C_c H}{1 + e_0} log \frac{\sigma_0 + \Delta\sigma}{\sigma_0} = \frac{0.315 \times 6}{1 + 1.08} \log \frac{163.46 + 40}{163.46} = 0.086$ m

Question: A soil profile shown in figure. Calculate the settlement due to primary consolidation for the 6 m clay layer due to a surcharge of 100 kN/m². Ground water table is flush with the top of the clay layer.



Solution:

For sand, $\gamma = \gamma_w = \frac{G\,\gamma_w}{1 + e} = \frac{2.66 \times 9.81}{1 + 0.72} = 15.17$ kN/m³

For clay, $\gamma' = \gamma_{sat} - \gamma_w = 19.376 - 9.81 = 9.566$ kN/m³

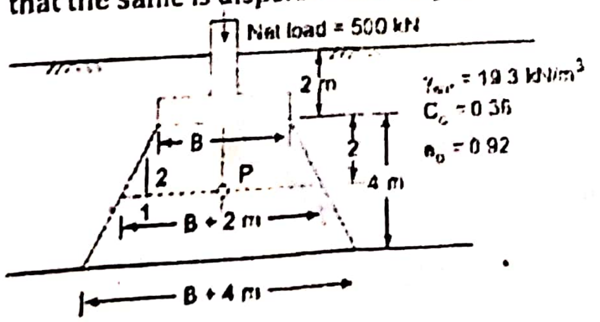
Overburden Pressure, $\sigma = 15.17 \times 4 + 9.566 \times 3 = 89.378$ kN/m²

Increase in pressure, $\Delta\sigma = 100$ kN/m²

$C_c = 0.009(LL - 10) = 0.009(60 - 10) = 0.45$

Settlement, $S_c = \frac{C_c H}{1 + e_0} log \frac{\sigma_0 + \Delta\sigma}{\sigma_0} = \frac{0.45 \times 6}{1 + 1.09} \log \frac{89.378 + 100}{89.378} = 0.463$ m

Question: A square footing is to be established in a clayey soil at a depth of 2 m. Where water table has risen upto the ground level as shown in figure. Determine the width of the footing if it is permitted to settle by 120 mm for the given data. Assume that the net load given is a constant and that the same is dispersed into clay as shown. Take γ_w = 10 kN/m³



Solution:

Let the size of the footing be $B \times B$

Width of spread at the centre of the depth below footing $= B + 2$

$$\Delta\sigma = \frac{500}{(B+2)^2} \text{ kN/m}^2$$

Here, $S_c = 120$ mm $= 0.12$ m, $C_c = 0.36$, $e_0 = 0.92$, $H = 4$ m

At P, $\sigma_0 = 4\gamma' = 4(19.3 - 10) = 37.2$ kN/m^2

$$S_c = \frac{C_c H}{1+e_0} \log \frac{\sigma_0 + \Delta\sigma}{\sigma_0}$$

$$0.12 = \frac{0.36 \times 4}{1+0.92} \log \frac{37.2 + \Delta\sigma}{37.2}$$

$\Delta\sigma = 16.57$ kN/m^2

$$\text{Now, } 16.57 = \frac{500}{(B+2)^2}$$

$B = 3.49$ m

Settlement Based on the Theory of Elasticity

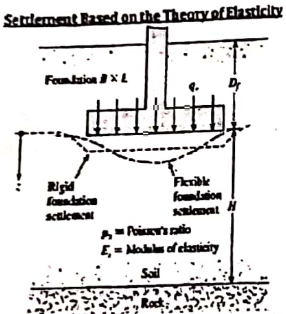


Figure: Elastic settlement of flexible and rigid foundations

The magnitude of the elastic settlement will depend on the flexibility of the foundation (flexible or rigid), and on the type of material (soil) that the foundation will rest on it (i.e. this method is valid for both sand and clay).

$$S_i = q\,B\left(\frac{1-\mu^2}{E}\right) I_p$$

q = Intensity of loading
B = Least lateral dimension of loaded area
I_p = Influence factor
μ = Poisson's ratio of soil

E = Modulus of elasticity
I_p = Influence factor
= 0.88 for rigid circular footing
= 0.82 for rigid square footing
= 1.06 for rigid rectangular footing with $L/B = 1.5$
= 1.70 for rigid rectangular footing with $L/B = 5$

Question: Determine the elastic settlement of a footing 3 m x 3m resting on sandy soil given Es = 45000 kpa and μ = 0.3. Footing carries a load of 2000 KN. Take I_p = 0.82 (51 BMA)

Solution:

Intensity of loading, $q = \frac{2000}{3 \times 3} = 222.22$ KPa

Least lateral dimension of loaded area, $B = 3$ m

Influence factor, $I_p = 0.82$

Poisson's ratio of soil, $\mu = 0.3$

Modulus of elasticity, $E = 45000$ kpa

$$S_i = q\,B\left(\frac{1-\mu^2}{E}\right) I_p = 222.22 \times 3 \times \left(\frac{1-0.3^2}{4500}\right) \times 0.82 = 0.011 \text{ m} = 11 \text{ mm}$$

SHEAR STRENGTH OF SOIL

Shear strength: The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. The fundamental shear strength equation proposed by the French engineer Coulomb is

$\tau = c + \sigma \tan \varphi$

c = cohesion
φ = angle of internal friction
σ = normal stress on the failure plane
τ = shear strength
for cohesionless soil $c = 0$

In saturated soil, the total normal stress at a point is the sum of the effective stress (σ') and pore water pressure (u).
$\sigma = \sigma' + u$

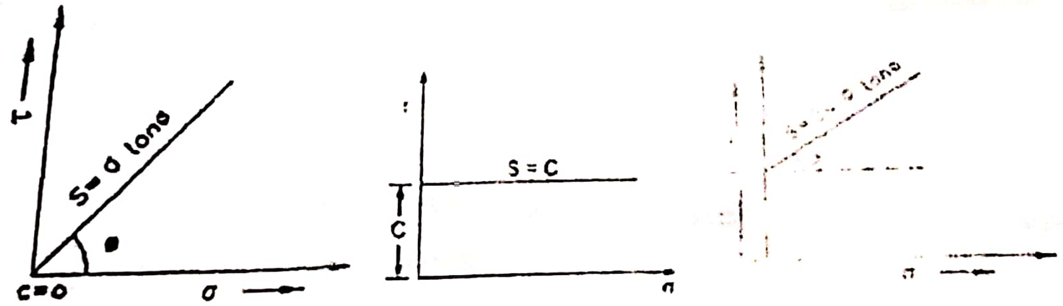
in terms of effective stress, $\tau = c' + \sigma' \tan \varphi'$

Question: Define shear strength of soil. Explain the shear characteristics of cohesive and cohesionless soil. (30th BCS)

Solution:

The shear strength of a soil mass is the internal resistance per unit area that the soil mass can offer to resist failure and sliding along any plane inside it. The fundamental shear strength equation proposed by the French engineer Coulomb is
$\tau = c + \sigma \tan \varphi$
c = cohesion
φ = angle of internal friction
σ = normal stress on the failure plane
τ = shear strength

Cohesionless soil: Purely granular soils possess no cohesion ($c = 0$). The shear strength of such soils are given by, $\tau = \sigma \tan \varphi$



Cohesive soil: Fine soils possess cohesion and no friction ($\varphi = 0$). The shear strength of such soils are given by, $\tau = c$

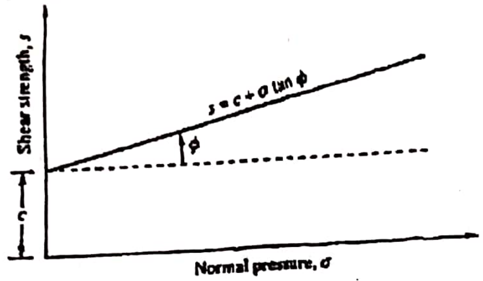
Cohesive frictional soil: clayey sand, silty sand has cohesion and friction. The shear strength of such soils are given by, $\tau = c + \sigma \tan \varphi$

Question: What are the strength parameters of soil? Write down the expression of relating those. (PGCL – 2014)

Solution:

Strength parameters of soil: Cohesion and angle of internal friction.

$\tau = c + \sigma \tan \varphi$



Question: Write the names of soil strength test. What tests are used for cohesive soil? (BUET M.Sc. –2011)

Solution:

The following tests are used to measure the shear strength of the soil,

- Direct shear test
- Triaxial compression test
- Unconfined compression test
- Vane shear test
- Consolidation test
- Compaction test
- Permeability test

For cohesive soil

- Unconfined compression test
- Triaxial test

Question: Write down the parameters of shear strength of soil. Also write the name of the tests to calculate these. (BCIC – 2017)

Solution:

Parameters of shear strength of soil is given below,

1. Angle of internal friction
2. Cohesion
3. Normal stress on failure plane

Following test are used to determine shear strength of soil

1. Direct shear test
2. Tri-axial shear test
3. Vane shear test
4. Unconfined compression test

There are several laboratory methods now available to determine the shear strength parameters of various soil specimens in the laboratory. They are as follows:

- Direct shear test
- Triaxial test
- Direct simple shear test
- Plane strain triaxial test
- Torsional ring shear test

Direct Shear Test: Dry sand can be conveniently tested by direct shear tests. The sand is placed in a shear box that is split into two halves. First a normal load is applied to the specimen. Then a shear force is applied to the top half of the shear box to cause failure in the sand. The normal and shear stresses at failure are,

$$\sigma' = \frac{N}{A} \text{ and } s = \frac{R}{A}$$

Where, A = area of the failure plane in soil that is the cross-sectional area of the shear box. Several tests of this type can be conducted by varying the normal load. The angle of friction of the sand can be determined by plotting a graph of s against σ'

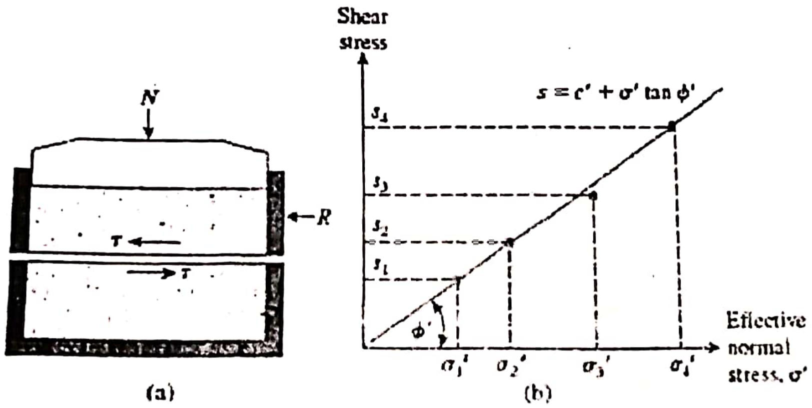
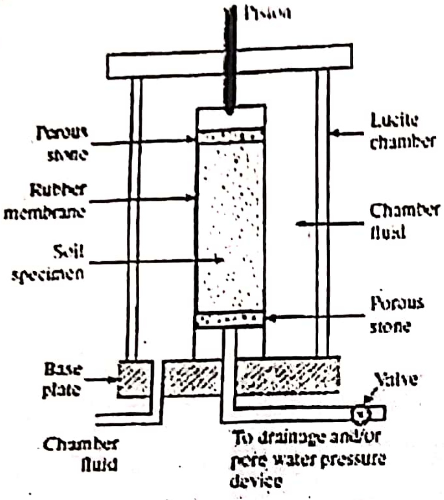
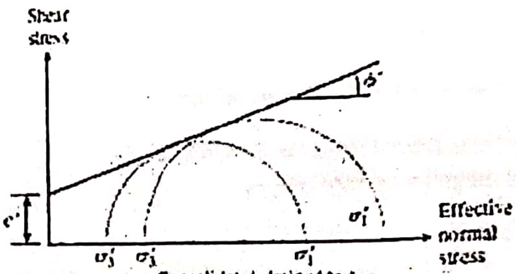


Figure: Direct shear test in sand:

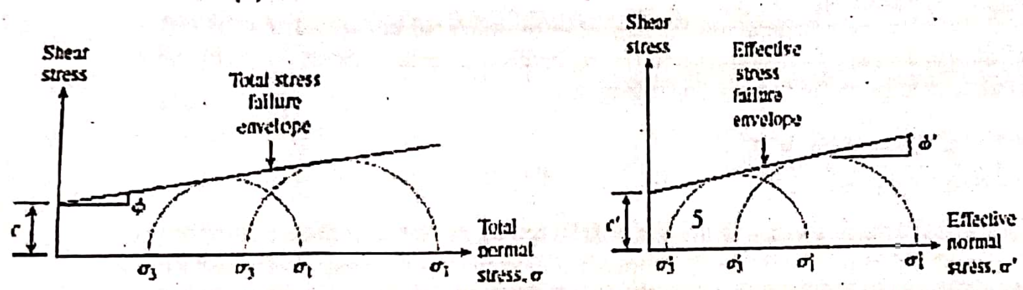
Triaxial Tests: Triaxial compression tests can be conducted on sands and clays. Figure shows aschematic diagram of the triaxial test arrangement. Essentially, the test consists of placing a soil specimen confined by a rubber membrane into a lucite chamber and then applying an all-around confining pressure σ_3 to the specimen by means of the chamber fluid (generally, water or glycerin). An added stress $\Delta\sigma$ can also be applied to the specimen in the axial direction to cause failure (σ_3 = $\Delta\sigma_f$ at failure). Drainage from the specimen can be allowed or stopped, depending on the condition being tested. For clays, three main types of tests can be conducted with triaxial equipment.



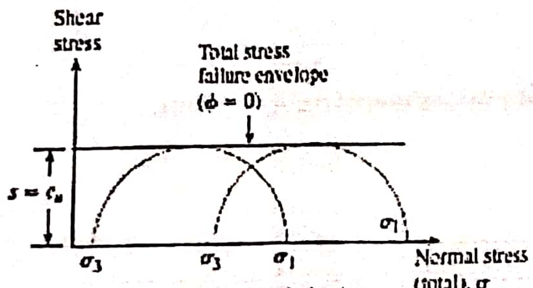
Schematic diagram of triaxial test equipment



Consolidated-drained test



Consolidated-undrained test



Unconsolidated-undrained test

1. Consolidated – drained test (CD test)
2. Consolidated – undrained test (CU test)
3. Unconsolidated – undrained test (UU test)

Consolidated-Drained Tests: At failure,

Major principal effective stress $= \sigma_3 + \Delta\sigma_f = \sigma_1 = \sigma_1'$
Minor principal effective stress $= \sigma_3 = \sigma_3'$

$$\sigma_1' = \sigma_3' \tan^2\left(45 + \frac{\varphi'}{2}\right) + 2\,c' \tan\left(45 + \frac{\varphi'}{2}\right)$$

Note: For normally consolidated clay, $c' = 0$

Consolidated-Undrained Tests: At failure,

Major principal total stress $= \sigma_3 + \Delta\sigma_f = \sigma_1$
Minor principal total stress $= \sigma_3$
Major principal effective stress $= (\sigma_3 + \Delta\sigma_f) - u_f = \sigma_1'$
Minor principal effective stress $= \sigma_3 - u_f = \sigma_3'$
$s = c + \sigma \tan\varphi$
Note: For normally consolidated clay, $c = 0$

Unconsolidated-Undrained Tests: At failure,

Major principal total stress $= \sigma_3 + \Delta\sigma_f = \sigma_1$
Minor principal total stress $= \sigma_3$

$$s = c_u = \frac{\Delta\sigma_f}{2}$$

c_u = undrained cohesion (or undrained shear strength).

__Unconfined Compression Test__

The unconfined compression test is a special type of unconsolidated undrained triaxial test in which the confining pressure $\sigma_3 = 0$ as shown in Figure. In this test, an axial stress $\Delta\sigma$ is applied to the specimen to cause failure ($\Delta\sigma = \Delta\sigma_f$). For this case,

Major Principal total stress $= \Delta\sigma_f = q_u$
Minor principal total stress $= 0$

The axial stress at failure, $\Delta\sigma_f = q_u$, is generally referred to as the unconfined compression strength. The unconfined compression test is generally applicable to saturated clays for which the apparent angle of shearing resistance $\varphi = 0$. The shear strength of saturated clays under this condition,

$$s = c_u = \frac{q_u}{2}$$

q_u = unconfined compressive strength at failure
The compressive stress is calculated on the basis of changed cross-sectional area A_2 at failure,

$$A_2 = \frac{V}{L_1 - \Delta L} = \frac{A_1}{1 - \frac{\Delta L}{L_1}}$$

L_1 = initial length of the specimen
ΔL = change in length at failure

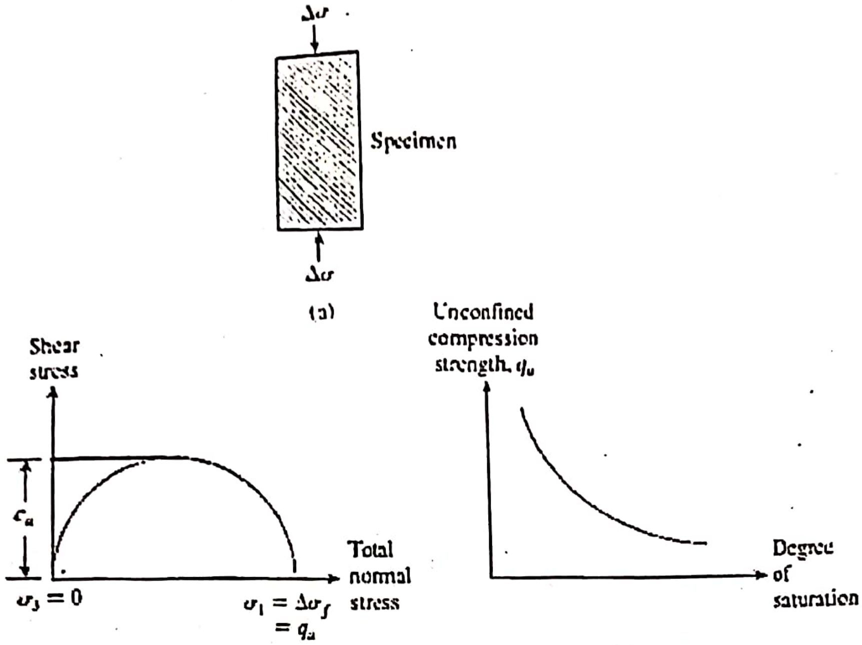


Figure: Unconfined compression test

Vane shear test: Vane test can be used as a reliable in-situ test for determining the shear strength of soft-sensitive clays. The vane consists of a steel rod having at one end four small projecting blades or vanes parallel to its axis, and situated at 90° intervals around the rod. A post hole borer is first employed to bore a hole up to a point just above the required depth. The rod is pushed or driven carefully until the vanes are embedded at the required depth. At the other end of the rod above the surface of the ground a torsion head is used to apply a horizontal torque and this is applied at a uniform speed of about 0.1° per sec until the soil fails, thus generating a cylinder of soil. The area consists of the peripheral surface of the cylinder and the two round ends. The first moment of these areas divided by the applied moment gives the unit shear value of the soil.

$$c_u = \frac{T}{2\pi r^2 (L + 0.67\, r)}$$

r = radius of the cylinder
c_u = undrained shear strength
T = torque

Question: A vane 11.25 cm long, and 7.5 cm in diameter was pressed into soft clay at the bottom of a borehole. Torque was applied to cause failure of soil. The shear strength of clay was found to be 37 kN/m^2. Determine the torque that was applied.

Solution:

Here, $c_u = 37\ kN/m^2 = 3.7\ kN/cm^2$

Torque, $T = c_u\, 2\pi r^2 (L + 0.67\, r)$

$T = 3.7 \times 2 \times 3.14 \times 3.75^2\ (11.25 + 0.67 \times 3.75) = 4500\ N-cm$

Question: A cylinder of soil fails under an axial vertical stress of 160 kN/m^2, when it is laterally unconfined. The failure plane makes an angle of 50^0 with the horizontal. Calculate the value of cohesion and the angle of internal friction of the soil.

Solution:

$\alpha = 50^0 = 45^0 + \frac{\varphi}{2}$

$\varphi = 2\,(50^0 - 45^0) = 10^0$

As the sample is unconfined, $\sigma_3 = 0$

Now, $\sigma_1 = \sigma_3\, tan^2\alpha + 2\,c\,tan\,\alpha$

$160 = 2\,c\,\tan 50^0$

$c = \frac{160}{1.192} = 67.1\ kN/m^2$

Question: A lateral pressure in a triaxial compression test in a cohesive soil gave the following results: angle of shearing resistance $\varphi = 17.5$ degree, cohesion= 3.0 kg/cm^2 and total axial stress at failure = 18 kg/cm^2. Determine the lateral pressure.

Solution:

$\varphi = 17.5^0, c = 3.0\ kg/cm^2, \sigma_1 = 18\ kg/cm^2, \sigma_3 = ?$

$\alpha = 45^0 + \frac{\varphi}{2} = 45^0 + \frac{17.5^0}{2} = 53.75^0$

Now, $\sigma_1 = \sigma_3\, tan^2\alpha + 2\,c\,\tan\alpha$

$18 = \sigma_3 \times (\tan 53.75^0)^2 + 2 \times 3 \times \tan\ 53.75^0$

$\sigma_3 = 5.28\ kg/cm^2$

Question: In an unconfined compression test sample of clay 8 cm long and 4 cm diameter fails under a load 12 kg at 7% strain. Calculate the un-drained shear strength of clay taking in account that the effect of change in cross-section of sample. (BPDB -- 2015)

Solution:

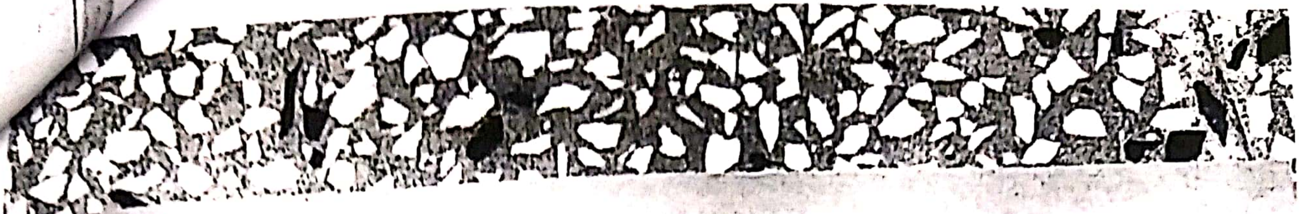
Strain, $\varepsilon = \frac{\Delta L}{L} = 0.07$

Initial cross − sectional area, $A = \frac{\pi \times 4^2}{4} = 12.56\ cm^2$

Changed cross − sectional area, $A_f = \frac{A}{1-\varepsilon} = \frac{12.56}{1-0.07} = 13.50\ cm^2$

unconfined compression strength, $q_u = \frac{P_f}{A_f} = \frac{12}{13.50} = 0.88\ kg/cm^2$

undrained shear strength, $c_u = \frac{q_u}{2} = \frac{0.88}{2} = 0.44\ kg/cm^2$



Question: What is the shear strength in terms of effective stress on a plane within a saturated soil mass at a point where the total normal stress is 295 kPa and the pore water pressure 120 kPa? The effective shear strength parameter are $c' = 12\ kPa$ and $\varphi' = 30^0$.

Solution:

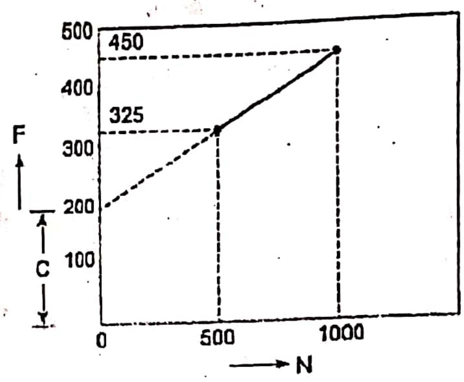
Effective stress, $\sigma' = \sigma - u = 295 - 120 = 175$ kPa

Shear strength, $s = c' + \sigma' \tan\varphi' = 12 + 175 \tan 30^0 = 113.04$ kPa

Question: A cohesive soil specimen was tested under triaxial compression with a cell pressure of P_s. Failure of the specimen occurred under a total pressure of 35 N/cm^2. With the same soil direct shear test was also done. Shearing forces at failure were 450 N and 325 N under normal loads of 1000 N and 500 N respectively. The sectional area of the shear box was 36 cm^2. Find the cell pressure P_s at failure.

Solution:

If C is the cohesion and m is the slope of the line,



$$m = \frac{450 - 325}{1000 - 500} = \frac{450 - C}{1000 - 0}$$

$$450 - C = 1000 \times \frac{125}{500}$$

C = 200 N

Unit cohesion, $C = \frac{200}{36} = 5.6\ \text{N/cm}^2$

$\alpha = 45^0 + \frac{\varphi}{2} = 45^0$, since the soil is cohesive $\varphi = 0$

Now, for the triaxial test, $\sigma_1 = \sigma_3\ tan^2\alpha + 2\,C \tan\alpha$

$\sigma_1 = \sigma_3\ tan^2 45^0 + 2\,C \tan 45^0 = \sigma_3 - 2\,C$

$\sigma_3 = \sigma_1 - 2\,C = 35 - 2 \times 5.56 = 23.89\ \text{N/cm}^2$

Question: The unconfined compressive strength 120 KN/m². Find undrained shear strength. (WASA – 2017, EGCB – 2020, BCMCL – 2020, APSCL – 2020)

Solution:

undrained shear strength, $c_u = \frac{q_u}{2} = \frac{120}{2} = 60\ \text{kN/m}^2$

Question: A sample of dry sand was tested in direct shear apparatus under a normal load of 36 kg. The sample failed under a shearing load of 58 lb. The sample size was 2" x 2". What is the angle of internal friction? (GTCL – 2016, SGCL – 2017, BWDB – 2018, NHA – 2020)

Solution

For cohesionless soil, $c = 0$

Shear stress, $\tau = \frac{R}{A} = \frac{58}{2 \times 2} = 14.5$ psi

Normal stress, $\sigma = \frac{N}{A} = \frac{36 \times 2.205}{2 \times 2} = 19.84$ psi

Shear strength, $\tau = c + \sigma \tan\varphi$

$14.5 = 0 + 19.84 \tan\varphi$

$\varphi = tan^{-1}\left(\frac{14.5}{19.84}\right) = 36.16^0$

Question: A 6m sand layer having properties, γ_{sat} = 20 KN/m³, γ_d = 15 KN/m³. Water table is 3 m below the ground level. If the cohesion and angle of internal friction is 52 kPa and 36⁰ determine the shear strength of the sand. (B – R Powegen – 2019)

Solution:

Normal stress, $\delta = 15 \times 3 + 3 \times (20 - 9.81) + 75.57$ KN/m²

Shear strength, $\tau = c + \delta \tan\varphi = 52 + 75.57 \tan 36^0 = 106.9$ KN/m²

Question: A soil layer having properties, γ_d = 16 KN/m³, γ_{sat} = 20 KN/m³, φ = 30⁰. Water table is 3m below the ground level and C = 50 KN/m². Determine shear strength at a depth of 5 m from its base. (DSCC – 2019)

Solution:

Normal Stress, $\sigma = 3 \times 16 + 2 \times (20 - 9.81) = 68.38$ KN/m²

Shear Strength, $\tau = c + \sigma \tan\varphi = 50 + 68.38 \tan 30^0 = 89.47$ KN/m²

Question: A sand layer having properties, γ_d = 16 KN/m³, γ_{sat} = 18 KN/m³, Gs = 2.6, φ = 36⁰. Water table is 2.4 m below the ground level. Determine shear strength at a depth of 5 m from its base. (BPDB – 2018)

Solution:

Normal Stress, $\sigma = 2.4 \times 16 + 2.6 \times (18 - 9.81) = 59.69$ KN/m²

Shear Strength, $\tau = c + \sigma \tan\varphi = 0 + 59.69 \tan 36^0 = 43.37$ KN/m²

78

Question: From a direct shear test, the value of normal stress of two sample is found 100 & 200 kip/ft^2 respectively. The value of shear stress of two sample is 80 & 120 kip/ft^2 respectively. What is the value of c & φ value? (Combined Bank – 2020)

Solution:

Here, $\sigma_1 = 100\ kip/ft^2, \sigma_2 = 200\ kip/ft^2, \tau_1 = 80\ kip/ft^2, \tau_2 = 120\ kip/ft^2$

Now, $\tau_1 = c + \sigma_1 \tan\varphi$

$80 = c + 100 \tan\varphi$

Again, $\tau_2 = c + \sigma_2 \tan\varphi$

$120 = c + 200 \tan\varphi$

Solving the two equations, $\varphi = 21.80^0$ & $c = 40\ kip/ft^2$

Question: Properties of a soil layer are $c = 15\ kN/m^2, \varphi = 30^0, e = 0.6$ and $G = 2.7$. Determine the shear strength of the soil along a horizontal plane at a depth of 5 m in soil layer, if; (i) Water table is at the ground surface and (ii) Water table is at a depth of 3 m from the ground surface. (37th BCS)

Solution:

Dry unit weight, $\gamma_d = \frac{G_s \gamma_w}{1+e} = \frac{2.7 \times 9.81}{1+0.6} = 16.55\ kN/m^3$

$\gamma_{sat} = \frac{(G_s + S\,e)\,\gamma_w}{1+e} = \frac{(2.7 + 1 \times 0.6)\,9.81}{1+0.6} = 20.23\ kN/m^3$

Water table is at the ground surface

Normal Stress, $\sigma = 5 \times (20.23 - 9.81) = 52.1\ KN/m^2$

Shear Strength, $\tau = c + \sigma \tan\varphi = 15 + 52.1 \tan 30^0 = 45.077\ KN/m^2$

Water table is at a depth of 3 m from the ground surface

Normal Stress, $\sigma = 3 \times 16.55 + 2 \times (20.23 - 9.81) = 70.49\ KN/m^2$

Shear Strength, $\tau = c + \sigma \tan\varphi = 15 + 70.49 \tan 30^0 = 55.69\ KN/m^2$

Question: Angle of internal friction 30°. Maximum principle stress is 70 kpa at failure concerned, Soil mass submerged by 2 m of water. What is minor principal stress? (NPCBL – 2017)

Solution:

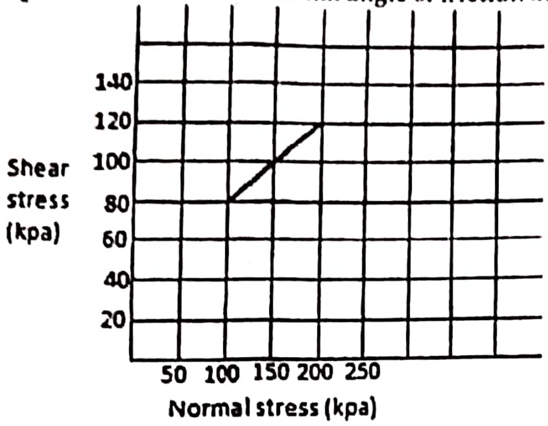
$\sigma_1 = \sigma_3 \tan^2\left(45^0 + \frac{\varphi}{2}\right) + 2\,c \tan\left(45^0 + \frac{\varphi}{2}\right)$

$70 = \sigma_3 \tan^2\left(45^0 + \frac{30}{2}\right)$

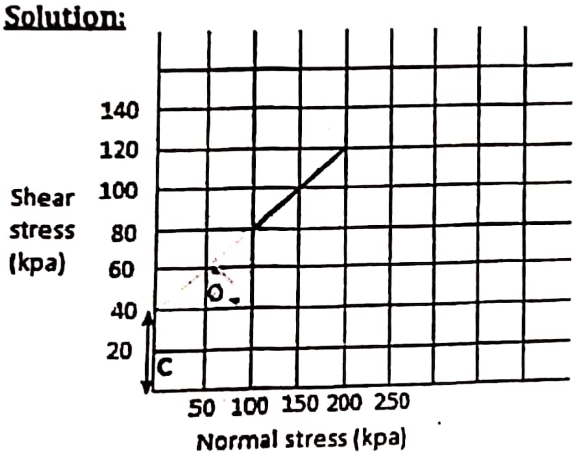
$\sigma_3 = 23.34$ kPa

Principle stress is the normal stress acting on principle plane. Principle Plane is the plane on which shear stress is zero and normal stress may or may not be. The maximum value of Principal stress is known as major principal stress and minimum value of principal stress is known as minor principal stress.

Question: Find the internal angle of friction and cohesion. (GTCL – 2018)



Solution:



$$\tan\varphi = \frac{F_2 - F_1}{N_2 - N_1} = \frac{120 - 80}{200 - 100} = 0.40$$

$\varphi = \tan^{-1}(0.40) = 21.80$

Now, $F_1 = C + N_1 \tan\varphi$

$80 = C + 100 \times \tan 21.80$

$C = 40$ kPa

LATERAL EARTH PRESSURE

Retaining wall: Structures that are built to retain vertical or nearly vertical earth banks or any other material are called retaining walls. Retaining walls may be constructed of masonry or sheet piles.

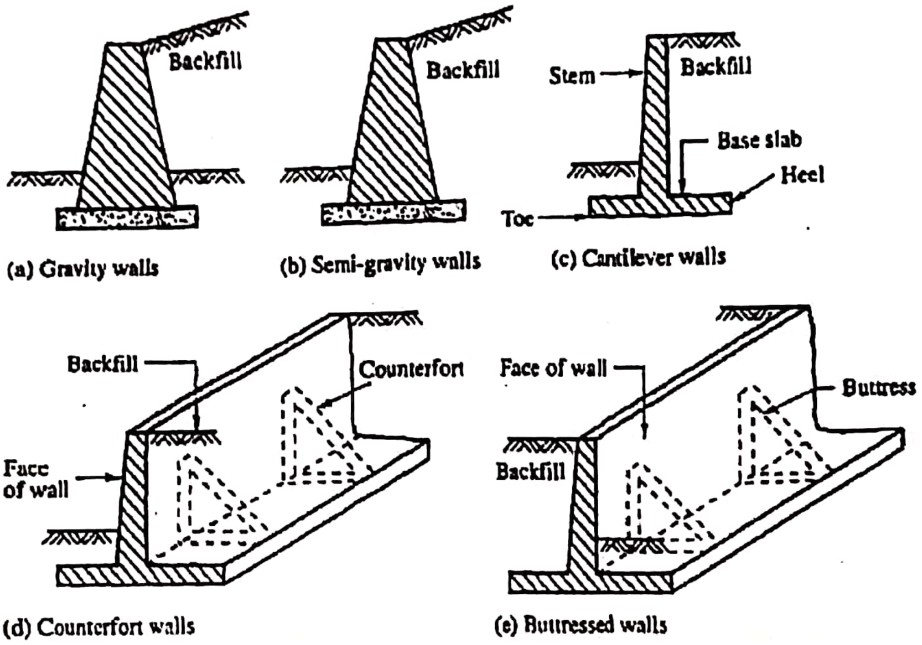


Figure: Principal types of rigid retaining walls

Question: What is earth pressure intensity? (BUET M. Sc – 2011)

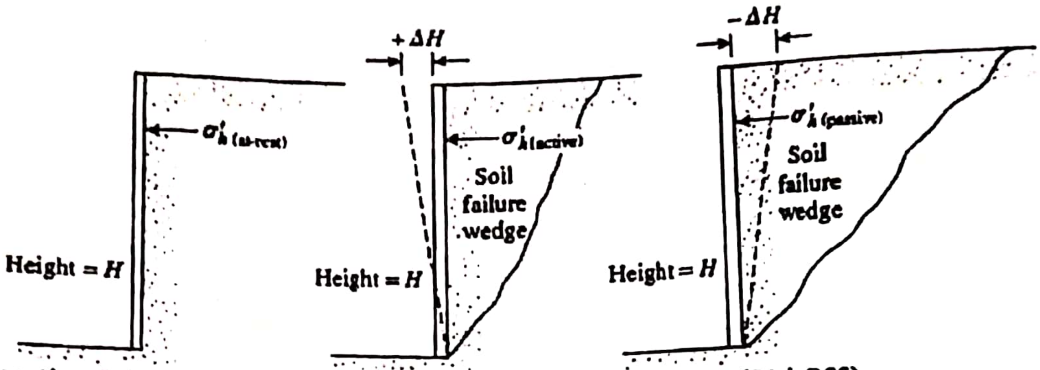
Lateral earth pressure is the pressure that soil exerts in the horizontal direction. The lateral earth pressure is important because it affects the consolidation behavior and strength of the soil and because it is considered in the design of geotechnical engineering structures such as retaining walls, basements, tunnels, deep foundations and braced excavations. There are three types of earth pressures on the basis of the movement of the wall.

- Earth pressure at rest
- Active Earth Pressure
- Passive Earth Pressure

Earth pressure at rest: When the retaining wall is at rest then the ratio between the lateral earth pressure and the vertical pressure is called the co-efficient of the earth pressure at rest,

Active earth pressure: When the retaining wall is moving away from the backfill the ratio between lateral earth pressure and vertical earth pressure is called coefficient of active earth pressure.

Passive earth pressure: When the retaining wall is moving towards the backfill, then the ratio between the lateral earth pressure and the vertical earth pressure is called the coefficient of passive earth pressure.



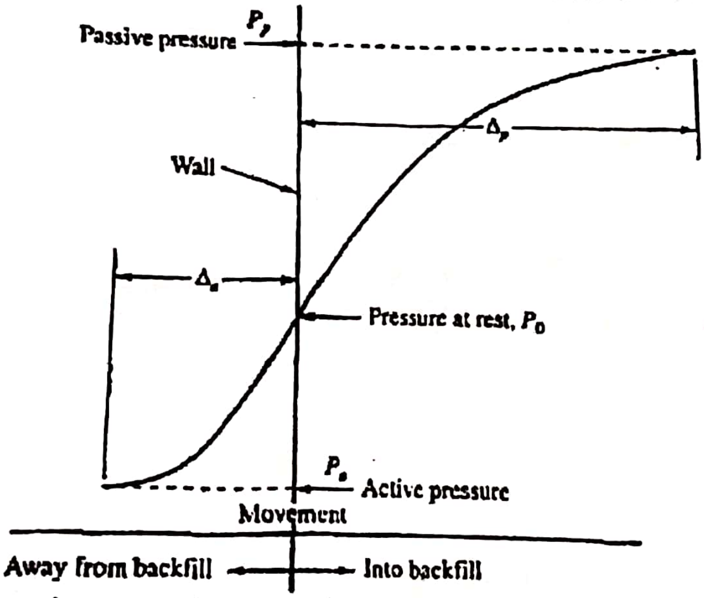
Question: What do you mean by active and passive earth pressure? (34th BCS)

Solution:

Active earth pressure: When the retaining wall is moving away from the backfill the ratio between lateral earth pressure and vertical earth pressure is called coefficient of active earth pressure.

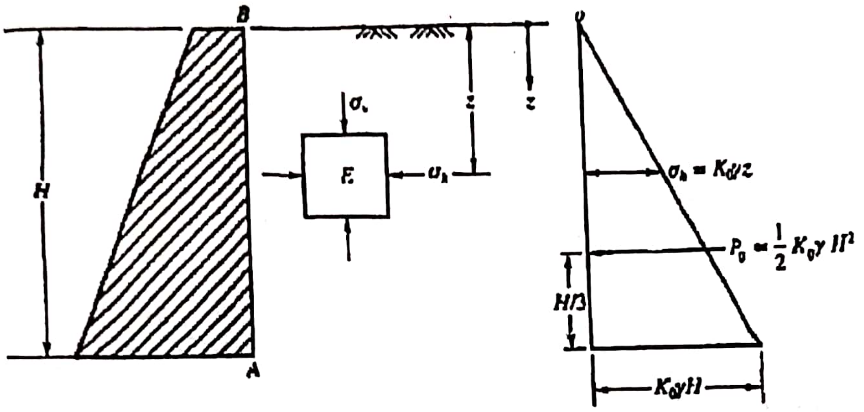
Passive earth pressure: When the retaining wall is moving towards the backfill, then the ratio between the lateral earth pressure and the vertical earth pressure is called the coefficient of passive earth pressure.

LATERAL EARTH PRESSURE FOR AT REST CONDITION



Development of active and passive earth pressures

If the wall is rigid and does not move with the pressure exerted on the wall, the soil behind the wall will be in a state of *elastic equilibrium.* Consider a prismatic element *E* in the backfill at depth *z* shown in Fig.



Lateral earth pressure for at rest condition

Element E is subjected to the following pressures.
Vertical pressure $= \sigma_v = \gamma z$; lateral pressure $= \sigma_h$

$\gamma =$ Effective unit weight of the soil. If we consider the backfill is homogeneous then both σ_v and σ_h increase linearly with depth z. In such a case, the ratio of σ_h to $<\sigma_v$ remains constant with respect to depth, that is

$$\frac{\sigma_h}{\sigma_v} = \frac{\sigma_h}{\gamma z} = \text{constant} = K_0$$

Where, K_0 is called the coefficient of earth pressure for the at rest condition or at rest earth pressure coefficient. The lateral earth pressure σ_h acting on the wall at any depth z may be expressed as

$$\sigma_h = K_0 \gamma z$$

The expression for σ_h at depth H, the height of the wall is

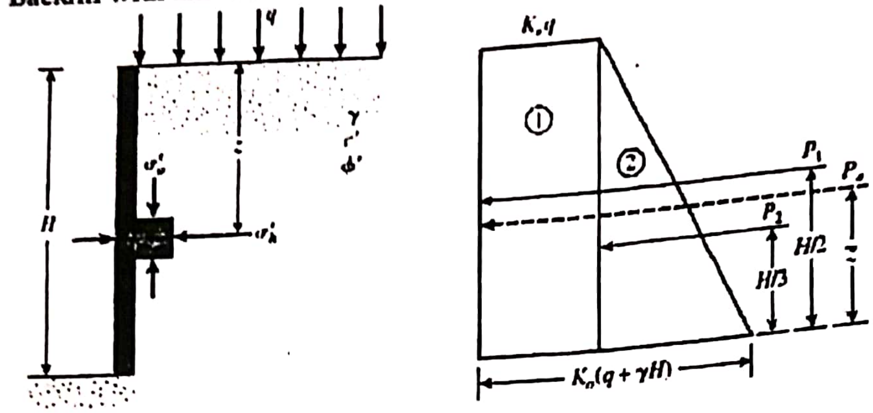
$$\sigma_h = K_0 \gamma H$$

The total pressure P_0 for the soil for the rest condition is,

$P_0 = \frac{1}{2} K_0 \gamma H^2$ acting at H / 3 above the base of the wall.

$$K_0 = 1 - \sin\varphi$$

Backfill with uniform surcharge

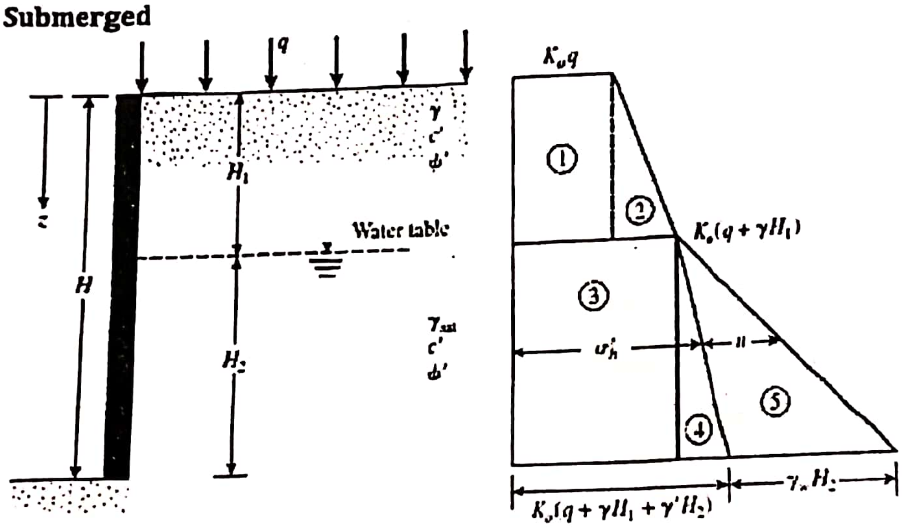


Lateral earth pressure at $z = 0, \sigma_h = K_0\, q$

Lateral earth pressure at $z = H, \sigma_h = K_0\, q + K_0\, \gamma\, H$

Total force per unit length, $P_0 = K_0\, q\, H + \frac{1}{2} K_0\, \gamma\, H^2$

Submerged



Effective unit weight of soil below the water table, $\gamma' = \gamma_{sat} - \gamma_w$

Lateral earth pressure at $z = 0, \sigma_h = K_0\, q$

Lateral earth pressure at $z = H_1, \sigma_h = K_0\, q + K_0\, \gamma\, H_1$

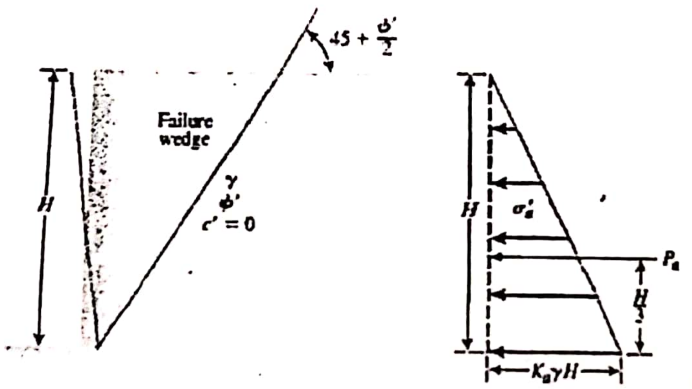
Lateral earth pressure at $z = H_2, \sigma_h = K_0\, q + K_0\, \gamma\, H_1 + K_0\, \gamma'\, H_2$

Total force per unit length, $P_0 = K_0\, q\, H_1 + \frac{1}{2} K_0\, \gamma\, H_1^2 + K_0(q + \gamma\, H_1)\, H_2 + \frac{1}{2} K_0\, \gamma'\, H_2^2 + \frac{1}{2} \gamma_w\, H_2^2$

Rankin's theory: Rankin theory of lateral earth pressure is applied to uniform cohesionless soils only. Later it was extended to include cohesive soils by Resal and Bell. The theory has also been extended to stratified, partially immersed and submerged soils. This theory is based mainly on the assumption of neglecting friction between the soil and the wall, so no shear forces are developed on soil particles.

Backfill—Cohesionless Soil with Horizontal Ground Surface

Active case:



Pressure distribution against a retaining wall for cohensionless soil backfill with horizontal ground surface: Rankine's active state

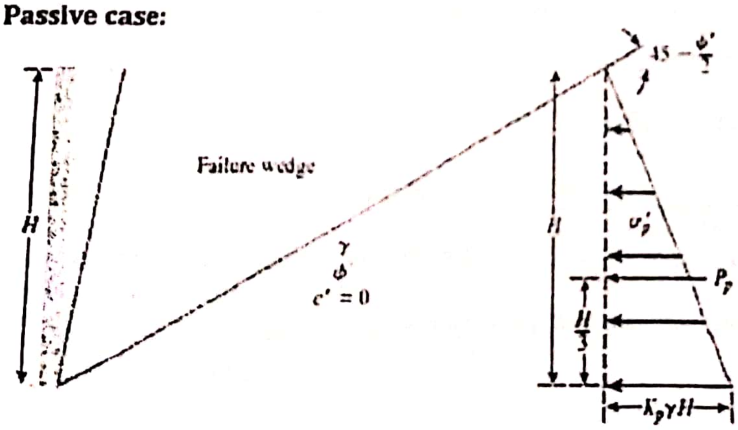
Active earth pressure, $\sigma_a = K_a \gamma H$

K_a = coefficient of active earth pressure = $tan^2\left(45 - \frac{\varphi}{2}\right) = \frac{1 - \sin\varphi}{1 + \sin\varphi}$

Total active earth pressure P_a,

$P_a = \frac{1}{2} K_a \gamma H^2$ acting at H/3 above the base of the wall.

Passive case:



Pressure distribution against a retaining wall for cohensionless soil backfill with horizontal ground surface: Rankine's passive state

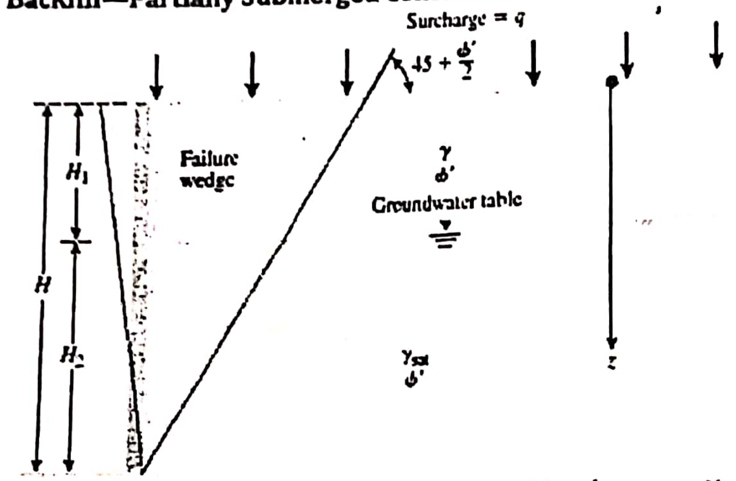
Passive earth pressure, $\sigma_p = K_p \gamma H$

K_p = coefficient of active earth pressure $= tan^2\left(45 + \frac{\varphi}{2}\right) = \frac{1 + \sin\varphi}{1 - \sin\varphi}$

Total passive earth pressure P_p,

$P_p = \frac{1}{2} K_p \gamma H^2$ acting at H/3 above the base of the wall.

Backfill—Partially Submerged Cohesionless Soil Supporting a Surcharge

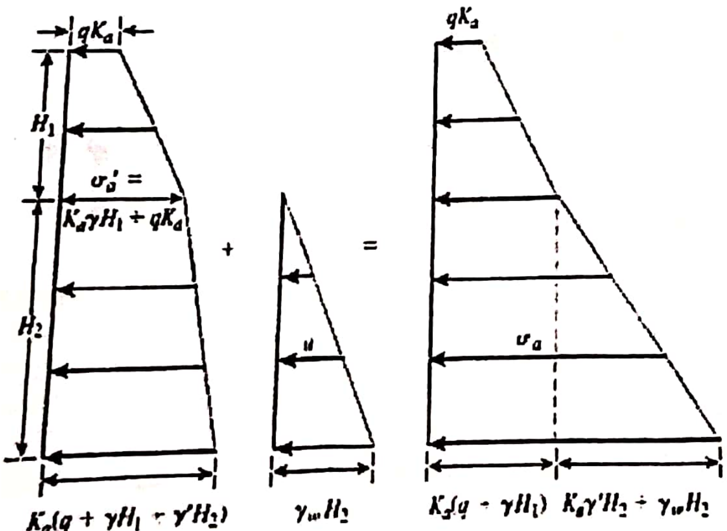


Effective unit weight of soil below the water table, $\gamma' = \gamma_{sat} - \gamma_w$

Lateral earth pressure at $z = 0, \sigma_h = K_0\, q$

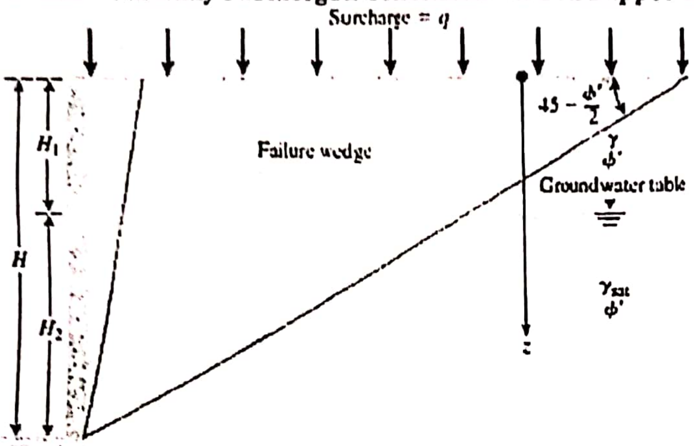
Lateral earth pressure at $z = H_1, \sigma_h = K_0\, q + K_0\, \gamma\, H_1$

Lateral earth pressure at $z = H_2, \sigma_h = K_0\, q + K_0\, \gamma\, H_1 + K_0\, \gamma'\, H_2$



Total force per unit length, $P_a = K_a\, q\, H + \frac{1}{2} K_a\, \gamma\, H_1^2 + K_a\, \gamma\, H_1 H_2 + \frac{1}{2} K_a\, \gamma'\, H_2^2 + \frac{1}{2}\gamma_w\, H_2^2$

Backfill—Partially Submerged Cohensionless Soil Supporting a Surcharge (Passive case)

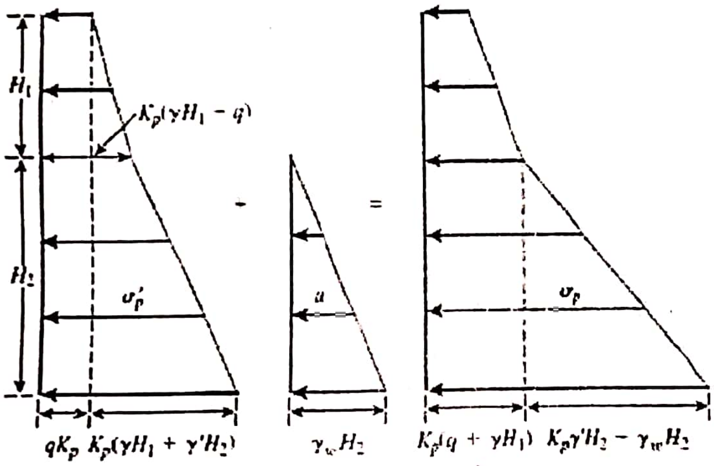


Effective unit weight of soil below the water table, $\gamma' = \gamma_{sat} - \gamma_w$

Lateral earth pressure at $z = 0$, $\sigma_h = K_0\, q$

Lateral earth pressure at $z = H_1$, $\sigma_h = K_0\, q + K_0\, \gamma\, H_1$

Lateral earth pressure at $z = H_2$, $\sigma_h = K_0\, q + K_0\, \gamma\, H_1 + K_0\, \gamma'\, H_2$



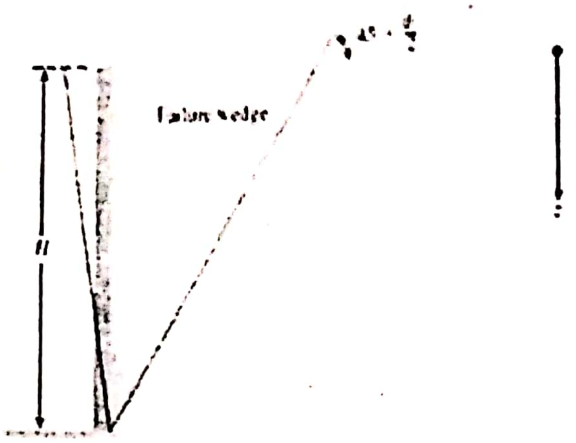
Total force per unit length, $P_p = K_p\, q\, H + \frac{1}{2} K_p\, \gamma\, H_1^2 + K_p\, \gamma\, H_1 H_2 + \frac{1}{2} K_p\, \gamma'\, H_2^2 + \frac{1}{2} \gamma_w\, H_2^2$

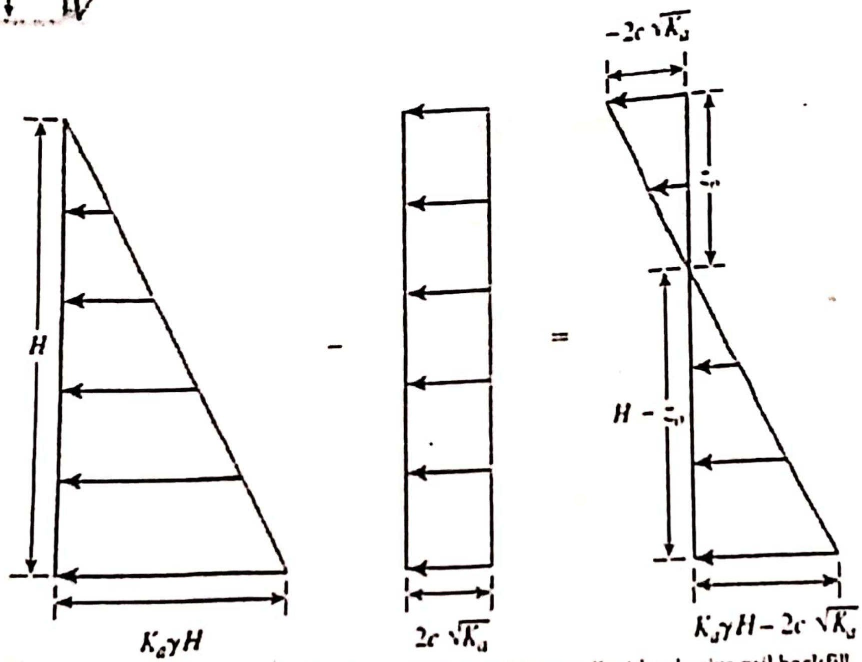
Backfill—Cohesive Soil with Horizontal Backfill

Active earth pressure for cohesive soil

Figure shows a frictionless retaining wall with a cohesive soil backfill. The active pressure against the wall at any depth below the ground surface can be expressed as

$\sigma_a = K_a\, \gamma\, z - 2\, c\, \sqrt{K_a}$





Rankine's active earth-pressure distribution against a retaining wall with cohesive soil backfill

The depth z_0 at which the active pressure becomes equal to 0

$$K_a\,\gamma\,z - 2\,c\,\sqrt{K_a} = 0$$

$$z_0 = \frac{2\,c}{\gamma\,\sqrt{K_a}}$$

For the undrained condition, $\varphi = 0, K_a = tan^2 45 = 1$ *and* $c = c_u$ *(undrained condition)*

$$z_0 = \frac{2\,c}{\gamma}$$

The depth z_0 is usually referred to as the *depth of tensile crack*, because the tensile stress in the soil will eventually cause a crack along the soil-wall interface. Thus, the total Rankine active force per unit length of the wall before the tensile crack occurs is

$$P_a = \frac{1}{2} K_a \gamma H^2 - 2c\sqrt{K_a} H$$

For the $\varphi = 0$, $\quad P_a = \frac{1}{2} \gamma H^2 - 2cH$

After the tensile crack appears, the force per unit length on the wall will be caused only by the pressure distribution between depths $z = z_0$ and $z = H$. only the active pressure distribution against the wall between z_0 and H is considered.

$$P_a = \frac{1}{2}(K_a \gamma H - 2c\sqrt{K_a})(H - z_0) = \frac{1}{2}(K_a \gamma H - 2c\sqrt{K_a})\left(H - \frac{2c}{\gamma\sqrt{K_a}}\right)$$

$$P_a = \frac{1}{2} K_a \gamma H^2 - 2c\sqrt{K_a} H + 2\frac{C^2}{\gamma}$$

Maximum depth of unsupported excavation = 2 z_0

Passive earth pressure for cohesive soil

Rankine's passive pressure against the wall at depth z can be given by

$\sigma_p = K_p \gamma z + 2c\sqrt{K_p}$

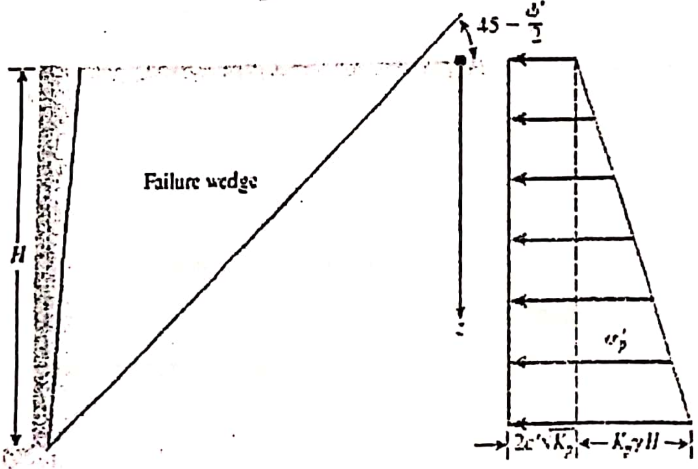
at $z = 0$, $\quad \sigma_p = 2c\sqrt{K_p}$

at $z = H$, $\quad \sigma_p = K_p \gamma H + 2c\sqrt{K_p}$

The passive force per unit length of the wall

$$P_p = \frac{1}{2} K_p \gamma H^2 + 2c\sqrt{K_p} H$$

For the $\varphi = 0$, $\quad P_a = \frac{1}{2} \gamma H^2 + 2cH$



Rankine's passive earth-pressure distribution against a retaining wall with cohesive soil backfill

Question: If a retaining wall 5 m high is restrained from yielding, what will be the at-rest earth pressure per meter length of the wall? Given: the backfill is cohesionless soil having $\varphi = 30°$ and $\gamma = 18\ kN/m^3$. Also determine the resultant force for the at-rest condition.

Solution:

$K_0 = 1 - \sin\varphi = 1 - \sin 30^0 = 0.5$

$\sigma_h = K_0\,\gamma\,H = 0.5 \times 18 \times 5 = 45\ kN/m^2$

$P_0 = \frac{1}{2}K_0\,\gamma\,H^2 = \frac{1}{2}\times 0.5 \times 18 \times 5^2 = 112.5\ kN/m$ length of wall

Question: Determine active and passive pressure diagram of a 3 m deep retaining wall where $\gamma = 20\ KN/m^3$ and $\varphi = 35^0$ (JB – 2017)

Solution:

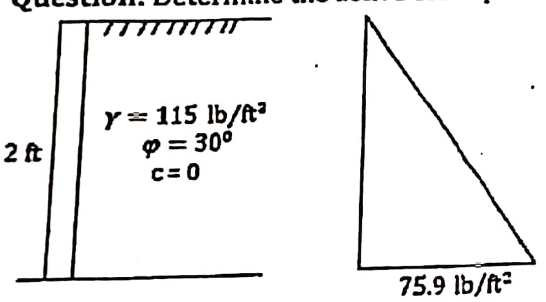
$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 35^0}{1+\sin 35^0} = 0.27$$

$$K_p = \frac{1+\sin\varphi}{1-\sin\varphi} = \frac{1+\sin 35^0}{1-\sin 35^0} = 3.65$$

Active pressure $= K_a\,\gamma\,Z = 0.27 \times 20 \times 3 = 16.2\ kN/m^2$

Passive pressure $= K_p\,\gamma\,Z = 3.65 \times 20 \times 3 = 219\ kN/m^2$

Question: Determine the active earth pressure of following soil profile. (EGCB – 2015)



Solution:

$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 30^0}{1+\sin 30^0} = 0.33$$

Active earth pressure, $\sigma_a = K_a\,\gamma\,H = 0.33 \times 115 \times 2 = 75.9\ lb/ft^2$

Question: Determine the active earth pressure of a sandy soil at a depth 4.5m, where $\Phi = 37^0$ and $\gamma = 16.5$ KN/m³ (PGCB – 2019)

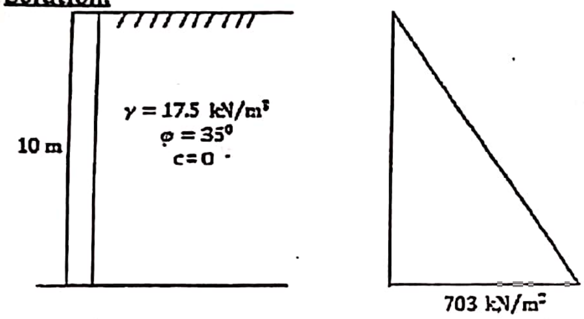
Solution:

$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 37^0}{1+\sin 37^0} = 0.398$$

Active earth pressure, $\sigma_a = K_a\, \gamma\, H = 0.398 \times 16.5 \times 4.5 = 29.55\ \text{kN/m}^2$

Question: Determine the passive pressure on a sheet pile 10m height if the soil retaining has following properties, $\varphi = 35^0$ and $\gamma = 19$ KN/m³. (DESCO – 2015)

Solution:



$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 35^0}{1+\sin 35^0} = 0.37$$

at $z = 0,\ \sigma_p = k_p \gamma\, z = 3.7 \times 19 \times 0 = 0$

at $z = 10, \sigma_p = k_p \gamma\, h = 3.7 \times 19 \times 10 = 703\ \ \text{kN/m}^2$

Question: A rigid retaining wall 15 feet high supports a backfill of cohesionless soil with $\varphi = 30°$, the water table below the base of the wall. The backfill is dry and has a unit weight of 115 pcf. Determine Rankin's passive earth pressure per meter length of the wall.
(BCIC – 2017, BUET M.Sc – 2019)

Solution:

The coefficient of passive earth pressure K_P is

$$K_P = \frac{1+\sin\varphi}{1-\sin\varphi} = \frac{1+\sin 30}{1-\sin 30} = 3$$

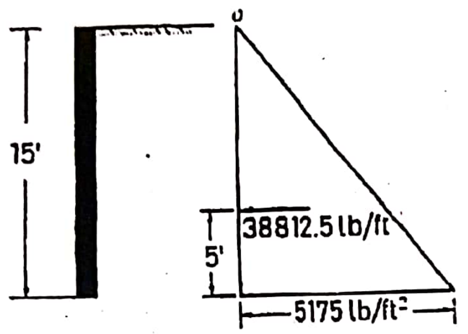
At base level, the passive earth pressure is

$\sigma_P = K_P\, \gamma\, H = 3 \times 115 \times 15 = 5175$ lb/ft2

Total passive thrust,

$$P_P = \frac{1}{2} K_P\, \gamma\, H^2 = 0.5 \times 3 \times 115 \times 15^2 = 38812.5 \text{ lb/ft.}$$

Location of force $= \frac{H}{3} = \frac{15}{3} = 5$ feet from the base of the wall



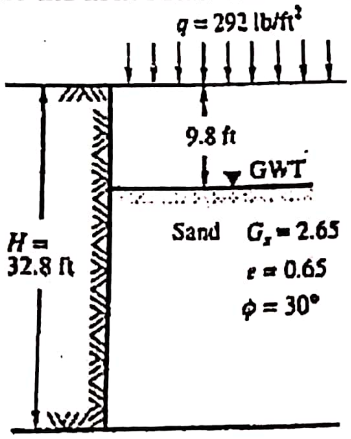
Question: A retaining wall has a granular soil backfill with a level top. For the un-drained condition ($\varphi = 0$) of the backfill, determine the maximum depth of the tensile crack if $\gamma = 16$ KN/m³ and $C_u = 17$ KN/m² (BUET M.Sc – 2019)

Solution:

Active pressure coefficient, $K_a = \frac{1 - Sin\varphi}{1 + Sin\varphi} = \frac{1 - Sin0}{1 + Sin0} = 1$

Depth of the tensile crack, $Z_0 = \frac{2C}{\gamma\sqrt{K_a}} = \frac{2 \times 17}{16 \times \sqrt{1}} = 2.125$ m

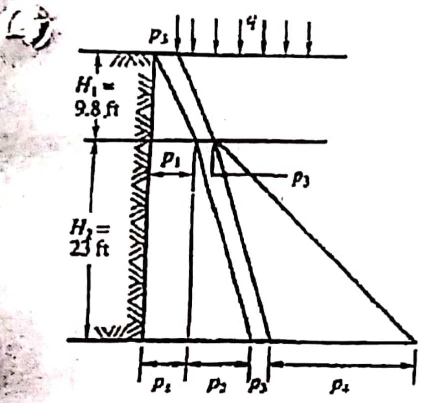
Question: For the earth retaining structure shown in Fig, construct the earth pressure diagram for the active state and determine the total thrust per unit length of the wall.



Solution:

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi} = \frac{1 - \sin 30^0}{1 + \sin 30^0} = 0.33$$

Dry unit weight, $\gamma_d = \frac{G_s\,\gamma_w}{1 + e} = \frac{2.65 \times 62.4}{1 + 0.65} = 100.22$ lb/ft³



Assuming the soil above the water table is dry,

$\sigma_1 = K_a \gamma_d H_1 = 0.33 x 100.22 x 9.8 = 327.39\ lb/ft^2$

$\sigma_2 = K_a \gamma_b H_2 = 0.33 x 62.4 x 23 = 478.4\ lb/ft^2$

$\sigma_3 = K_a q = 0.33 x 292 = 97.33\ lb/ft^2$

$\sigma_4 = \gamma_w H_2 = 62.4 x 23 = 1435.2\ lb/ft^2$

Total thrust

= *summation of the areas of the different parts of the pressure diagram*

$= \frac{1}{2} \sigma_1 H_1 + \sigma_1 H_2 + \frac{1}{2} \sigma_2 H_2 + \sigma_3 (H_1 + H_2) + \frac{1}{2} \sigma_4 H_2$

$= 0.5 x 327.39 x 9.8 + 327.39 x 23 + 0.5 x 478.4 x 23 + 97.33 x 32.8$
$+ 0.5 x 1435.2 x 23$

$= 34333\ lb/ft = 34.3\ kips/ft\ of\ wall$

Question: A rigid retaining wall 19.69 ft high has a saturated backfill of soft clay soil. The properties of the clay soil are γ_{sat} = 111.76 lb/ft^3, and unit cohesion c_u = 376 lb/ft^2. Determine (a) the expected depth of the tensile crack in the soil (b) the active earth pressure before the occurrence of the tensile crack, and (c) the active pressure after the occurrence of the tensile crack. Neglect the effect of water that may collect in the crack.

Solution:

Since, $\varphi = 0, K_a = tan^2 45 = 1$

at $z = 0$, $\sigma_a = K_a \gamma z - 2c\sqrt{K_a} = 1 x 111.76 x 0 - 2 x 376 x 1 = -752$ lb/ft^2

at $z = H$, $\sigma_a = K_a \gamma H - 2c\sqrt{K_a} = 1 x 111.76 x 19.69 - 2 x 376 x 1 = 1449$ lb/ft^2

The depth of tensile crack, $z_0 = \frac{2c}{\gamma\sqrt{K_a}} = \frac{2 x 376}{111.76 x 1} = 6.73$ ft

The active earth pressure before the crack occurs.

$P_a = \frac{1}{2} K_a \gamma H^2 - 2c\sqrt{K_a} H$

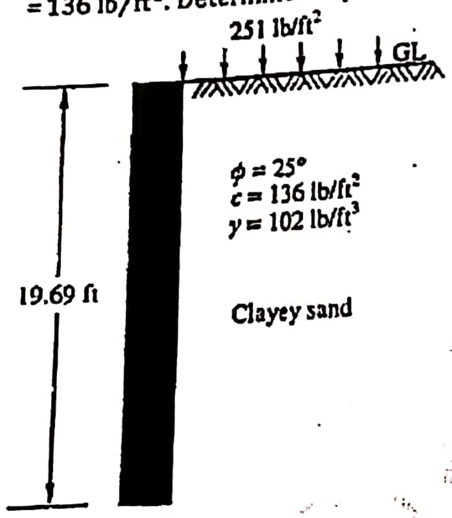
$P_a = \frac{1}{2} x\ 1\ x\ 111.76\ x\ 19.69^2 - 2\ x\ 376\ x\ 1\ x\ 19.69 = 6857$ lb/ft

The active earth pressure after the tensile crack occurs.

$P_a = \frac{1}{2}(K_a \gamma H - 2c\sqrt{K_a})(H - z_0)$

$P_a = \frac{1}{2}(1\ z\ 111.76\ x\ 19.69 - 2\ x\ 376\ x\ 1)(19.69 - 6.73)9387 = 9387$ lb/ft

Question: A smooth rigid retaining wall 19.69 ft high carries a uniform surcharge load of 251 lb/ft². The backfill is clayey sand with the following properties: $\gamma = 102$ lb/ft³, $\varphi = 25°$, and $c = 136$ lb/ft². Determine the passive earth pressure and draw the pressure diagram.



Solution:

$$K_p = \frac{1 + \sin\varphi}{1 - \sin\varphi} = \frac{1 + \sin 25^0}{1 - \sin 25^0} = 2.47$$

Passive earth pressure, $\sigma_p = K_p q + K_p \gamma z + 2c\sqrt{K_p}$

at $z = 0$, $\sigma_p = K_p q + 2c\sqrt{K_p} = 2.47\ x\ 251 + 2\ x\ 136\ x\ \sqrt{2.47} = 1047.5$ lb/ft²

at $z = H$, $\sigma_p = K_p q + K_p \gamma H + 2c\sqrt{K_p}$

$\sigma_p = 2.47\ x\ 251 + 2.47\ x\ 102\ x\ 19.62 + 2\ x\ 136\ x\ \sqrt{2.47} = 6007\ lb/ft^2$

Total passive pressure acting on the wall,

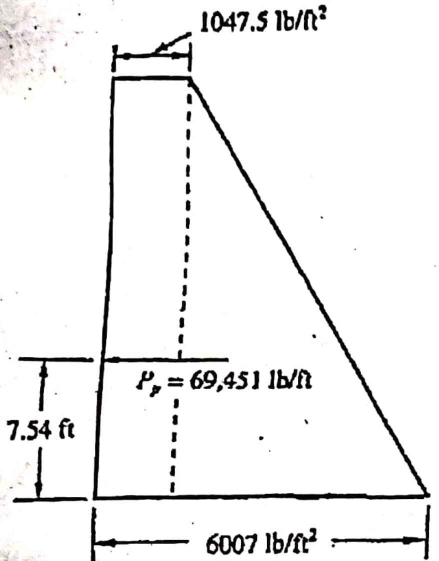
$$P_p = 1047.5\ x\ 19.69 + \frac{1}{2}\ x\ 19.69\ x\ (6007 - 1047.5) = 69451\ lb/ft\ of\ wall$$

Location of resultant

Taking moments about the base

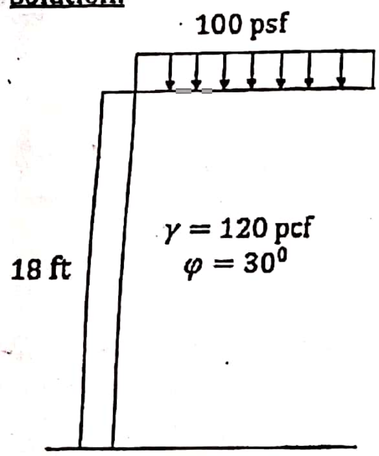
$$P_p\ x\ h = \frac{1}{2}\ x\ 19.69^2\ x + 1047.5 + \frac{1}{6}\ x\ 19.69^2\ x\ 4959.5 = 523518\ lb/ft$$

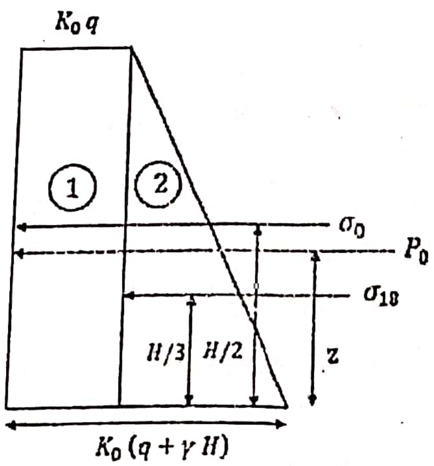
$\bar{y}_h = \frac{523518}{P_p} = \frac{523518}{69451} = 7.54\ ft$



Question: Determine the earth pressure at rest over the length of an 18′ retaining wall under a surcharge load of 100 psf supporting a backfill with unit weight 120 pcf and angle 30°. Also determine the point of application of pressure. (SGFCL – 2017)

Solution:





$K_0 = 1 - \sin\varphi = 1 - \sin 30^\circ = 0.5$

Lateral earth pressure at $z = 0,\ \sigma_0 = K_0\, q = 0.5 \times 100 = 50\, psf$

Lateral earth pressure at $z = H,\ \sigma_{18} = K_0\, q + K_0\, \gamma H = 0.5 \times 100 + 0.5 \times 120 \times 18$

$= 753.33\, psf$

Total force per unit length, $P_0 = K_0 q H + \frac{1}{2} K_0 \gamma H^2$

$P_0 = 0.5 \times 100 \times 18 + 0.5 \times 0.5 \times 120 \times 18^2 = 7079.94$ lb/ft

Location of resultant force

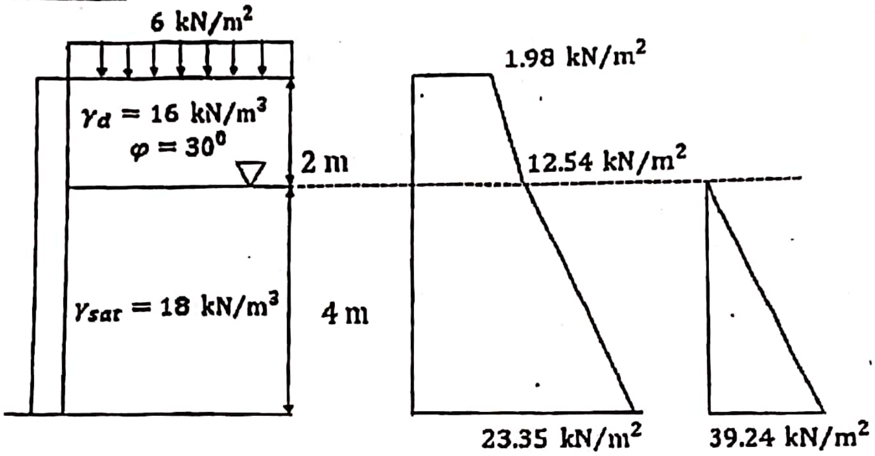
Taking moments about the base

$$P_0 x h = 33.33 x 18 x 9 + \frac{1}{2} x (753.33 - 33.33) x 18 x 6 = 44279.46\ lb/ft$$

$$h = \frac{44279.46}{P_p} = \frac{44279.46}{7079.94} = 6.25\ ft$$

Question: A retaining wall is 6 m height. Uniform load acting on the wall is 6 KN/m². Non cohesive soil backfill, γ = 18 KN/m³, φ = 30⁰, water table 2 m from top. Find out the resultant active force per unit length. (BUET M.Sc – 2014, BPDB – 2018)

Solution:



$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi} = \frac{1 - \sin 30^0}{1 + \sin 30^0} = 0.33$$

Effective unit weight of soil below the water table, $\gamma' = \gamma_{sat} - \gamma_w = 18 - 9.81$
$= 8.19\ kN/m^2$

Lateral earth pressure at $z = 0,\ \sigma_a = K_a q = 0.33 x 6 = 1.98\ kN/m^2$

Lateral earth pressure at $z = 2\ m,\ \sigma_a = K_a q + K_a \gamma_d H_1$

$\sigma_2 = 0.33 x 6 + 0.33 x 16 x 2 = 12.54\ kN/m^2$

Lateral earth pressure at $z = 6\ m,\ \sigma_a = K_a q + K_a \gamma_d H_1 + K_a \gamma' H_2$

$\sigma_6 = 0.33 x 6 + 0.33 x 16 x 2 + 0.33 x 8.19 x 4 = 23.35\ kN/m^2$

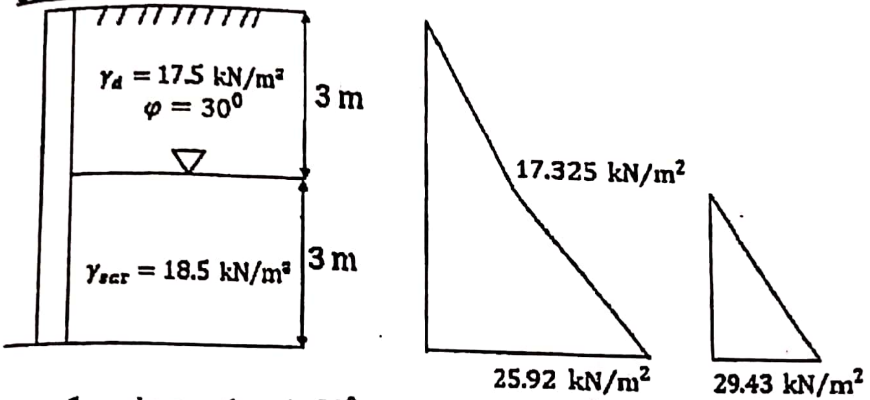
Pore water pressure $= \gamma_w z = 9.81 x 4 = 39.24\ kN/m^2$

Total active force, $P_a = 0.5\ (1.98 + 13.86) \times 2 + 0.5\ (13.86 + 27.31) \times 4 + 0.5 \times 4 \times 39.24$

= 176.66 kN/m

Question: A retaining wall is 6 m height, water table 3 m from top. Unit weight of soil above and below the water table is respectively, γ = 17.5 KN/m³ and γ = 18.5 KN/m³, φ = 30⁰ .Draw active pressure diagram and find out the resultant active force per unit length. (DESCO – 2019)

Solution:



$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 30^0}{1+\sin 30^0} = 0.33$$

Effective unit weight of soil below the water table, $\gamma' = \gamma_{sat} - \gamma_w = 18.5 - 9.81$

$= 8.69\ kN/m^2$

Lateral earth pressure at $z = 0$, $\sigma_a = 0\ kN/m^2$

Lateral earth pressure at $z = 3\ m$, $\sigma_a = K_a \gamma_d H_1$

$\sigma_2 = 0.33 \times 17.5 \times 3 = 17.325\ kN/m^2$

Lateral earth pressure at $z = 6\ m$, $\sigma_a = K_a \gamma_d H_1 + K_a \gamma' H_2$

$\sigma_6 = 0.33 \times 17.5 \times 3 + 0.33 \times 8.69 \times 3 = 25.92\ kN/m^2$

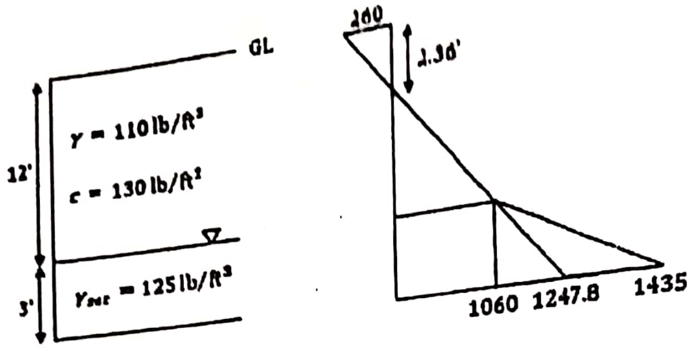
Pore water pressure $= \gamma_w z = 9.81 \times 3 = 29.43\ kN/m^2$

Total active force, $P_a = 0.5 \times 17.325 \times 3 + 0.5\ (17.325 + 25.92) \times 3 + 0.5 \times 3 \times 29.4 = 135\ kN/m$

Question: A retaining wall has to support 15 ft soil above its base level and the water table is 12 ft below the ground level. The backfill material is pure clay having λ = 110 lb/ft³ and λ_{sat} = 125 lb/ft³. The value of cohesion, C = 130 lb/ft², determine the stresses at different location and draw the pressure diagram. (JB – 2017)

Solution:

97



For pure clay, $\varphi = 0$, $K_a = \tan^2 45 = 1$

at top $= 2\,c\,\sqrt{K_a} = 2 \times 130 \times \sqrt{1} = 260$ lb/ft²

Location of zero stress, $z_0 = \dfrac{2\,c}{\gamma\sqrt{K_a}} = \dfrac{2 \times 130}{110 \times \sqrt{1}} = 2.36$ ft

at $z = 12$ ft (top of water table), $\sigma_a = K_a\,\gamma\,H_1 - 2\,c\,\sqrt{K_a}$

$\sigma_a = 1 \times 110 \times 12 - 2 \times 130 \times \sqrt{1} = 1060\ \text{lb/ft}^2$

at $z = 15$ ft, $\sigma_a = K_a\,\gamma\,H_1 + K_a\,\gamma' H_2 - 2\,c\,\sqrt{K_a}$

$\sigma_a = 1 \times 110 \times 12 + 1 \times (125 - 62.4) \times 3 - 2 \times 130 \times \sqrt{1} = 1247.8\ \text{lb/ft}^2$

Water pressure $= \gamma_w\,H_2 = 62.4 \times 3 = 187.2\ \text{lb/ft}^2$

Question: Determine the stresses at the top and bottom of a vertical cut 4.5 m deep in soil with $\varphi = 16^0$ and $c = 19.1\ kN/m^2$ and $\gamma = 18.5\ kN/m^3$. What could be the depth of the potential crack? What is the maximum depth of excavation that can be left unsupported?

Solution:

$$K_a = \frac{1 - \sin\varphi}{1 + \sin\varphi} = \frac{1 - \sin 16^0}{1 + \sin 16^0} = 0.567$$

Active earth pressure for $c - \varphi$ soil, $\sigma_a = K_a\,\gamma\,z - 2\,c\,\sqrt{K_a}$

at $z = 0$, $\sigma_a = 0.567 \times 18.5 \times 0 - 2 \times 19.1 \times \sqrt{0.567} = -28.787\ \text{kN/m}^2$ (tension)

at $z = 4.5$ m, $\sigma_a = 0.567 \times 18.5 \times 4.5 - 2 \times 19.1 \times \sqrt{0.567} = 18.489\ \text{kN/m}^2$

Depth of potential crack is at z_0 where $\sigma_a = 0$

Location of zero stress, $z_0 = \dfrac{2\,c}{\gamma\sqrt{K_a}} = \dfrac{2 \times 19.1}{18.5 \times \sqrt{0.567}} = 2.74$ m

Maximum depth of unsupported excavation $= 2\,z_0 = 2 \times 2.74 = 5.48$ m

Question: A vertical bank was formed during the excavation of a soil having $\varphi = 15^0$ and unit weight of 1800 kg/m^3. When the depth of excavation reached 5.5 m the bank failed. What was the approximate value of cohesion of clay?

Solution:

$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 15^0}{1+\sin 15^0} = 0.589$$

$$\text{Critical depth, } z_0 = \frac{2\,c}{\gamma\sqrt{K_a}} = \frac{2\,c}{1800 \times \sqrt{0.589}} = 1.448 \times 10^{-3}\,c$$

Maximum unsupported depth = $2\,z_0 = 2.896 \times 10^{-3}\,c$

$c = 1899\ \text{kg/m}^2$

Question: An unsupported excavation is to be made in a clay layer if $\gamma = 18.5\ \text{kN/m}^3$, $c = 19.1\ \text{kN/m}^2$ and $\varphi = 10^0$. Calculate the depth of tension cracks, the maximum possible unsupported depth and draw the active pressure distribution diagram.

Solution:

$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 10^0}{1+\sin 10^0} = 0.704$$

Active earth pressure for c − φ soil, $\sigma_a = K_a\,\gamma\,z - 2\,c\,\sqrt{K_a}$

at $z = 0$, $\sigma_a = 0.704 \times 18 \times 0 - 2 \times 30 \times \sqrt{0.704} = -50.343\ \text{kN/m}^2$ (tension)

$$\text{Depth of tension crack, } z_0 = \frac{2\,c}{\gamma\sqrt{K_a}} = \frac{2 \times 30}{18 \times \sqrt{0.704}} = 3.97\ \text{m}$$

Maximum depth of unsupported excavation = $2\,z_0 = 2 \times 3.97 = 7.94$ m

Question: The unconfined compression of the soil is 50 Kpa. Determine the depth of excavation without any lateral support. (BWDB – 2014)

Solution:

$$\text{Undrained shear strength, } c_u = \frac{q_u}{2} = \frac{50}{2} = 25\ \text{kpa}$$

As for unconfined compression test $\varphi = 0$, $K_a = 1$

$$\text{Critical depth, } z_0 = \frac{2\,c}{\gamma\sqrt{K_a}} = \frac{2 \times 25}{20 \times \sqrt{1}} = 2.5\ \text{m}$$

Maximum unsupported depth = $2\,z_0 = 5$ m

Question: A rigid retaining wall 6 m high has a saturated backfill of soft clay soil. The properties of the clay soil are $\gamma_{sat} = 16.5\ \text{kN/m}^3$, and unit cohesion $c_u = 10\ \text{kN/m}^2$. For undrained condition ($\varphi = 0$) etermine (a) the expected depth of the tensile crack in the soil (b) the active earth pressure before the occurrence of the tensile crack, and (c) the active pressure after the occurrence of the tensile crack. Neglect the effect of water that may collect in the crack. (BWDB – 2020)

Solution:

Since, $\varphi = 0, K_a = \tan^2 45 = 1$

at $z = 0$, $\sigma_a = K_a \gamma z - 2c\sqrt{K_a} = 1 \times 16.5 \times 0 - 2 \times 10 \times 1 = -20 \text{ kN/m}^2$

at $z = H$, $\sigma_a = K_a \gamma H - 2c\sqrt{K_a} = 1 \times 16.5 \times 6 - 2 \times 10 \times 1 = 99 \text{ kN/m}^2$

The depth of tensile crack, $z_0 = \dfrac{2c}{\gamma\sqrt{K_a}} = \dfrac{2 \times 10}{16.5 \times 1} = 1.21 \text{ m}$

The active earth pressure before the crack occurs.

$$P_a = \frac{1}{2} K_a \gamma H^2 - 2c\sqrt{K_a} H$$

$$P_a = \frac{1}{2} \times 1 \times 16.5 \times 6^2 - 2 \times 10 \times 1 \times 6 = 177 \text{ kN/m}$$

The active earth pressure after the tensile crack occurs.

$$P_a = \frac{1}{2}(K_a \gamma H - 2c\sqrt{K_a})(H - z_0)$$

$$P_a = \frac{1}{2}(1 \times 16.5 \times 6 - 2 \times 10 \times 1)(6 - 1.21) = 189.205 \text{ kN/m}$$

Question: How many types of foundation are there? (EED – 2015)

Solution:

Following are different types of foundations used in construction:

1. Shallow foundation
    - Individual footing or isolated footing
    - Combined footing
    - Strip foundation
    - Raft or mat foundation
2. Deep Foundation
    - Pile foundation
    - Drilled Shafts or caissons

Question: Draw and discuss different types of footing. (32th BCS, 33th BCS)

Solution:

- **Shallow Foundation**

Wall Footing: This type of foundation runs continuous along the direction of the wall and helps to transmit the load of the wall into the ground. Wall footing is suitable where loads to be transmitted are small and are economical in dense sands and gravels.

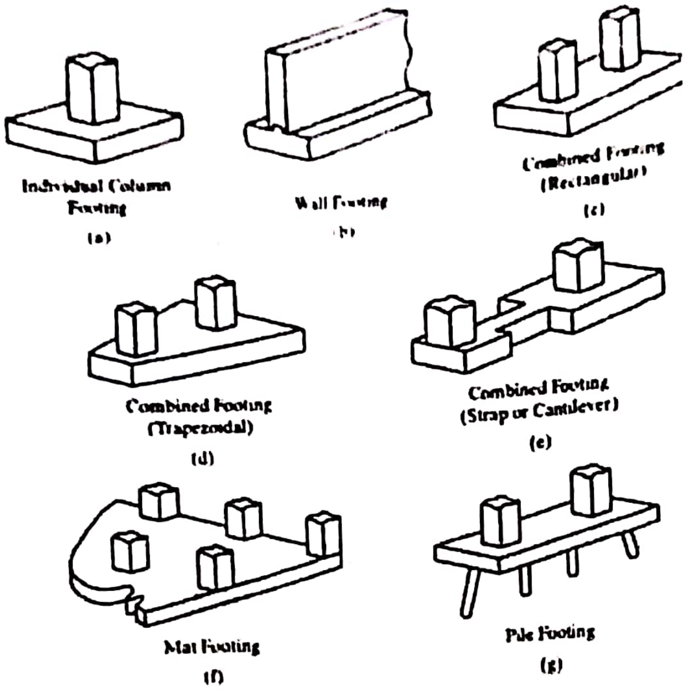
Column Footing: A column footing (or isolated or pad) footing is provided to support an individual column. Column footing is suitable and economical for the depth greater than 1.5m. In this type of foundation the base of the column is enlarged.

Combined Footing: Combined footings are those foundations that are made common for two or more columns in a row. It is used when the footing for a column may extend beyond the property line. It is also suitable when the two columns are closely spaced and the soil on which the structure resist is of low bearing capacity.

Strap Footing: When an edge footing cannot be extended beyond the property line the edge footing is linked up with the other interior footing by means of a strap beam. Such footings are called as strap footing. It is also known as cantilever footing.

Mat Foundation: A mat foundation is a combined footing which covers the entire area beneath of a structure and supports all the walls and columns. It is also known as raft foundation. Mat foundation is applicable when:

- Allowable bearing pressure is low.
- The structure is heavy.
- The site is with highly compressible layer.



- **Deep Foundation**

Pile Foundation: Pile is a slender member with small area of cross-section relative to its length. They can transfer load either by friction or by bearing. Pile foundation is used when:

- The load is to be transferred to stronger or less compressible stratum, preferably rock.
- The granular soils need to be compacted.
- The horizontal and the inclined forces need to be carried from the bridge abutments and the retaining walls.

Pier Foundation: Pier foundation is underground cylindrical structural member that support heavier load of the structure which shallow foundations cannot resist. Unlike pile foundation, pier foundation can only transfer load by bearing. Pier foundation is shallower in depth than the pile foundation. Pier foundation is used when:

- The top strata are a decomposed rock underlying as sound rock strata.
- The soil is stiff clay that occurs large resistance for driving the bearing pile.

Well (Caissons) Foundation: The term caisson refers to box or a case. These are hollow inside and are usually constructed at the site and sunk in place into hard bearing strata. As they are expensive in construction, they are usually restricted to major foundation works. Well foundation is suitable when the soil contains large boulders obstructing the penetration during installation of pier or pile foundations. Caissons are used for bridge piers, abutments in rivers and lakes and other shore protection works. They are used to resist heavy vertical and horizontal loads and are used in the construction of large water front structures as pump houses.

Question: Deep foundation and shallow foundation. (HED – 2017)

Solution:

Foundation which is placed near the surface of the earth or transfers the loads at shallow depth is called the shallow foundation. The depth of shallow foundation is generally about 3 meters or the depth of foundation is less than the footing width. Shallow foundations transfer loads mostly by end bearing. Isolated foundation, strip foundation, mat foundation, combined foundation etc. are the types of shallow foundation.

Foundation which is placed at a greater depth or transfers the loads to deep strata is called the deep foundation. Deep foundations rely both on end bearing and skin friction, with few exceptions like end-bearing pile. Deep foundations are generally more expensive than shallow foundations. Pier foundation, pile foundation, caissons etc. are the types of deep foundation.

Question: What do you mean by bearing capacity of soil? What is the utility of soil test for construction of a Building? (EED – 2015)

Solution:

Bearing capacity: Bearing capacity of soil as the capacity of soil to resist external load applied to it. In simple terms, it is the capacity of soil to support the load without failure. In other words, bearing capacity of the soil is defined as the maximum coverage contact pressure between the foundation and the soil which should not produce shear failure in the soil.

Utility of soil test: Soil testing is a vital and necessary step in the construction process. The soil properties once established, such as settlement of the soil and other relevant data, can be used by engineers and builders to:

1. Determine the suitability of the soil, allowing you to assess whether the construction project can be accommodated at the location.

2. By drilling in multiple different locations on the site, you can help to identify the different types of soil located on the site and where they are.

3. Test the soil for strength, density, compaction, contamination, sand content, etc. and assess what the impact of the soil may have on the construction project.

4. Get data that is necessary to compile technical and safety data reports that can help support you in getting planning permissions from council.

5. Receive precise results and ensure maximum quality and safety for the project.

Question: Write major factors affecting the bearing capacity of shallow foundation? (DPDC – 2014, BWDB – 2016, PGCL – 2017)

Solution:

The following are some of the more important ones which affect bearing capacity:

- Nature of soil and its physical and engineering properties;
- Nature of the foundation and other details such as size, shape, depth below the ground surface.
- Total and differential settlements that the structure can withstand without functional failure;

- Location of ground water table relative to the level of the foundation;
- Initial stresses, if any.

Question: What are the main considerations for foundation construction in the site? (MES – 2015)

Solution:

- General landforms
- Evidence of a risk of landslide
- Soil types for load-bearing capacity
- Drainage and runoff
- The water table, and presence of natural springs or waterlogged soils
- The proximity of the site or proposed building to excavations or exposed banks
- The presence of expansive clays
- Previous use of the site such as buried structures, contamination, earthworks and uncompacted fill

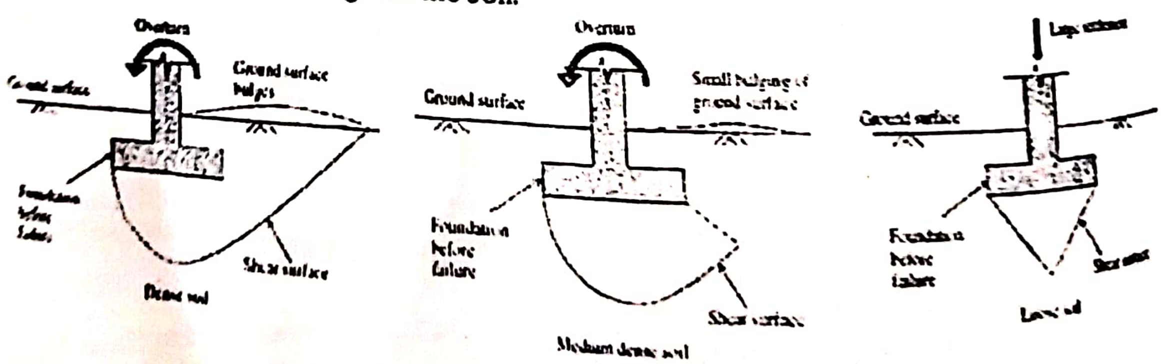
Question: What is bearing capacity of soil? Write four methods to improve bearing capacity of soil? (Army – 2014)

Solution:

The bearing capacity of soil is defined as the capacity of the soil to bear the loads coming from the foundation. The pressure which the soil can easily withstand against load is called allowable bearing pressure. Some of the methods to improve bearing capacity of soils:

- By increasing the depth of the footing
- Draining the soil
- Compacting the soil
- By blending granular material, like sand, gravel or crushed stone into the natural soil.
- By confining the soil in an enclosed area with the help of sheet piles
- By driving sand piles
- Stabilizing the soil with chemicals

Shear Failure: Also called "Bearing capacity failure" and it's occur when the shear stresses in the soil exceed the shear strength of the soil.



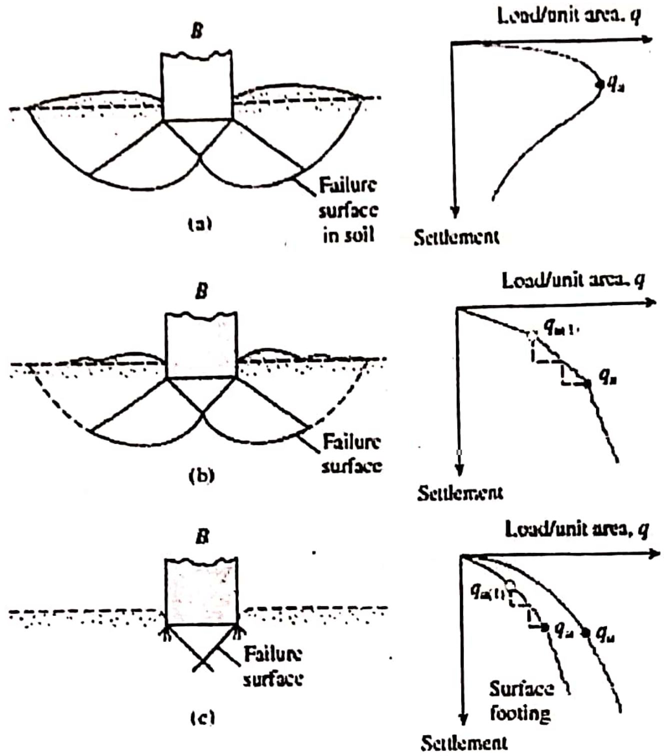


Figure: Modes of bearing capacity failure

For $\varphi > 36^0$, General shear failure
For $\varphi < 28^0$, Local shear failure

Terzaghi's bearing capacity equation

The ultimate bearing capacity of soil for a strip footing may be given by

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

q_u = Ultimate bearing capacity of the underlying soil (KN/m^2)
c = Cohesion of underlying soil (KN/m^2)
q = Effective stress at the bottom of the foundation (KN/m^2)
The terms N_c, N_q and N_γ are called the bearing capacity factors

$$N_q = e^{\pi \tan\varphi} \tan^2\left(45 + \frac{\varphi}{2}\right)$$

$$N_c = (N_q - 1)\cot\varphi$$

$$N_\gamma = (N_q - 1)\tan\,(1.4\,\varphi) \text{ or } 2\,(N_q + 1)\tan\varphi$$

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

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

Rectangular foundations: $q_u = c\,N_c\left(1 + \frac{0.3\,B}{L}\right) + q\,N_q + \frac{1}{2}\gamma\,B\,N_\gamma\left(1 - \frac{0.2\,B}{L}\right)$

B = width or diameter, L = length of footing

For an undrained condition with $\varphi = 0$, the bearing capacity factors are $N_c = 5.14, N_q = 1, N_\gamma = 0$

Now for local shear failure the above three equations:

For strip footing, $q_u = \frac{2}{3} c\, N_c' + q\, N_q' + \frac{1}{2}\gamma\, B\, N_\gamma'$

For square footing, $q_u = 0.867\, c\, N_c' + q\, N_q' + 0.4\gamma\, B\, N_\gamma'$

For square footing, $q_u = 0.867\, c\, N_c' + q\, N_q' + 0.3\gamma\, B\, N_\gamma'$

For rectangular footing: $q_u = 0.67\, c\, N_c'\left(1 + \frac{0.3\,B}{L}\right) + q\, N_q' + \frac{1}{2}\gamma\, B\, N_\gamma'\left(1 - \frac{0.2\,B}{L}\right)$

N_c', N_q' and N_γ' = Modified bearing capacity factors

$\tan\varphi$ (General shear) $= \frac{2}{3}\tan\varphi$ (Local shear)

$\tan\bar{\varphi} = 0.67\tan\varphi$

$\bar{\varphi} = tan^{-1}(0.67\ \tan\varphi)$

Ultimate bearing capacity (q_u) is the maximum value the soil can bear it. If the bearing stress from foundation exceeds the ultimate bearing capacity of the soil, shear failure in soil will be occur. So we must design a foundation for a bearing capacity less than the ultimate bearing capacity to prevent shear failure in the soil. This bearing capacity is "Allowable Bearing Capacity".

$$q_{all} = \frac{q_u}{FS}$$

q_{all} = allowable bearing capacity

q_u = ultimate bearing capacity

FS = Factor of safety for bearing capacity ≥ 3

The net ultimate bearing capacity is defined as the ultimate pressure per unit area of the foundation that can be supported by the soil in excess of the pressure caused by the surrounding soil at the foundation level. If the difference between the unit weights of concrete used in the foundation and the unit weight of soil surrounding is assumed to be negligible,

$$q_{net(u)} = q_u - q$$

$$q_{all(net)} = \frac{q_u - q}{FS} = \frac{q_u - \gamma\, D}{FS}$$

q = Effective stress at the level of foundation level.

Ultimate Bearing Capacity(q_u): It's the minimum load per unit area of the foundation that causes shear failure in the underlying soil. Or, it's the maximum load per unit area of the foundation can be resisted by the underlying soil without occurs of shear failure (if this load is exceeded, the shear failure will occur in the underlying soil). The load per unit area of the foundation at which shear failure in soil occurs is called the ultimate bearing capacity

Allowable Bearing Capacity (q_{all}): It's the load per unit area of the foundation can be resisted by the underlying soil without any unsafe movement occurs (shear failure) and if this load is exceeded, the shear failure will not occur in the underlying soil till reaching the ultimate load.

Question: Discuss the effect of water table on the bearing capacity of soil. (34,35th BCS, 38 BCS)

Solution:

The ultimate bearing capacity of soil for a strip footing may be given by

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

Previous equation give the ultimate bearing capacity, based on the assumption that the water table is located well below the foundation. However, if the water table is close to the foundation, some modifications of the bearing capacity equations will be necessary.

Case I: If the water table is located so that $0 \le D_1 \le D_f$, the factor q in the bearing capacity equations takes the form

q = effective surcharge = $D_1\gamma + D_2(\gamma_{sat} - \gamma_w)$

γ_{sat} = saturated unit weight of soil

γ_w = unit weight of water

Also, the value of γ in the last term of the equations has to be replaced by $\gamma' = \gamma_{sat} - \gamma_w$

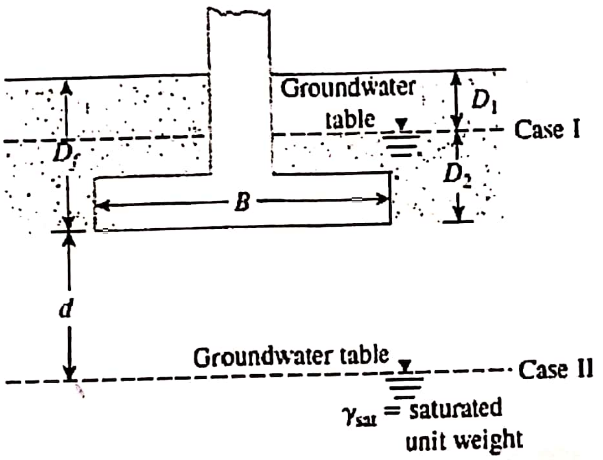


Figure: Modification of bearing capacity equations for water table

Case II: For a water table located so that $0 \le d \le B$

$q = \gamma D_f$

In this case, the factor γ in the last term of the bearing capacity equations must be replaced by the factor

$$\bar{\gamma} = \gamma' + \frac{d}{B}(\gamma - \gamma')$$

Case III: When the water table is located so that $d \ge B$, the water will have no effect on the ultimate bearing capacity.

Question: Express the bearing capacity equation and describe the terms. (WASA – 2014, BEUT M. Sc. –2014)

Solution:

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

$c\,N_c$ = Effect of cohesion

$q\,N_q$ = Effect of surcharge

$\frac{1}{2}\gamma\,B\,N_\gamma$ = Effect of soil weight in shear zone

Question: Determine the ultimate bearing capacity of a strip footing of 1 m is laid at a depth 0.5m. Water table is at 8 m bellow the ground level. Given, C = 30 KN/m², γ = 16 KN/m³, N_c = 17, N_q = 12, N_γ = 5 (PGCB – 2019)

Solution:

$q_u = 1.3\,c\,N_c + q\,N_q + 0.5\,\gamma\,B\,N_\gamma$

$q_u = 30 \times 17 + 16 \times 0.5 \times 12 + 0.5 \times 16 \times 1 \times 5 = 646$ KN/m²

Question: Using Terzaghi equation, find the bearing capacity of a footing. Necessary data is given. Footing size 2 m x 2 m and having the depth of foundation is 1 m, if take C - φ soil, C = 18 KN/m², φ = 35°, N_c = 57.7, N_q = 14.4, N_γ = 42.4. (DESCO – 2015)

Solution:

$c = 18$ KN/m², $\varphi = 35^0$, $N_c = 57.7$, $N_q = 14.4$, $N_\gamma = 42.4$, $B = 2$ m, $D = 1$ m,

$q = \gamma\,D_f = 18 \times 1 = 18$ KN/m²

For square footing we know,

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

$q_u = 1.3 \times 18 \times 57.7 + 18 \times 14.4 + 0.4 \times 18 \times 2 \times 42.4 = 2219.94$ KN/m²

Question: Determine the ultimate bearing capacity of a strip footing 1.2 m wide and having the depth of foundation is 1.5 m. Take for C-soil C = 30 KN/m², γ = 20 KN/m³, φ = 0, N_c = 5.14, N_q = 1.0, N_γ = 2.4. (DESCO – 2015)

Solution:

$c = 30$ KN/m², $\gamma = 20$ KN/m³, $\varphi = 0$, $N_c = 5.14$, $N_q = 1.0$, $N_\gamma = 0$, $B = 1.2$ m

$q = \gamma\,D_f = 20 \times 1.5 = 30$ KN/m²

For strip footing we know, $q_u = c\,N_c + q\,N_q + 0.5\,\gamma\,B\,N_\gamma$

$q_u = 30 \times 5.14 + 30 \times 1 + 0 = 184.2$ KN/m²

Question: A square footing 2 m x 2 m rests on a C-Ø soil, with its base at 5 m below the ground surface. Calculate the allowable bearing capacity using a factor of safety 3. The soil has following parameters, γ_{sat} = 20.4 KN/m², φ = 20⁰, C = 20 kpa and the value of N_c = 14.83, N_q = 6.40, N_γ = 5. (BCPCL – 2016)

Solution:

$c = 20$ KN/m², $\varphi = 20^0$, $N_c = 14.83$, $N_q = 6.4$, $N_\gamma = 5$, $B = 2$ m, $D = 5$ m,

$q = \gamma D_f = 20 \times 5 = 100$ KN/m²

For square footing we know,

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

$q_u = 1.3 \times 20 \times 14.83 + 100 \times 6.4 + 0.4 \times 20 \times 2 \times 5 = 1105.58$ KN/m²

Question: Determine the ultimate bearing capacity of a continuous footing having 1.2 m width and the depth of foundation is 0.9 m. Take for C-soil C = 20 KN/m², γ = 17.2 KN/m³, N_c = 17.69, N_q = 7.44, N_γ = 3.64. (APSCL – 2020, BCMCL – 2020)

Solution:

$c = 20$ KN/m², $\gamma = 17.2$ KN/m², $N_c = 17.69$, $N_q = 7.44$, $N_\gamma = 3.64$, $B = 1.2$ m

$q = \gamma D_f = 1.72 \times 0.9 = 15.48$ KN/m²

For strip footing we know, $q_u = c\, N_c + q\, N_q + 0.5\, \gamma\, B\, N_\gamma$

$q_u = 20 \times 17.69 + 15.48 \times 7.44 + 0.5 \times 17.2 \times 1.2 \times 3.64 = 506.536$ KN/m²

Question: A stripe footing of 1 m width q_u = 50 kpa, N_c = 9, γ = 18 KN/m³, N_q = 2, N_γ = 1, Calculate the ultimate bearing capacity. (WASA – 2017)

Solution:

$q_u = c\, N_c + q\, N_q + 0.5\, \gamma\, B\, N_\gamma$

$q_u = 25 \times 9 + 18 \times 0.5 \times 2 + 0.5 \times 18 \times 1 \times 1 = 252$ KN/m²

Question: A 3 m x 3 m spread footing with a depth of 1.5 m is to be placed on a homogenous clay layer having C = 30 KN/m², NMC = 20% and e = 50%. Determine the gross allowable load capacity if factor of safety is two. (SGFCL – 2017)

Solution:

$$\gamma = \frac{G_s\, \gamma_w\, (1 + w)}{1 + e} = \frac{2.65 \times 9.81\,(1 + 0.2)}{1 + 0.5} = 20.79 \text{ KN/m}^3$$

For pure clay soil, $\varphi = 0$; $N_c = 5.14$, $N_q = 1$, $N_\gamma = 0$

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

$= 1.3 \times 50 \times 5.14 + 20.79 \times 1.5 \times 1 + 0.4 \times 20.79 \times 3 \times 0$

$= 365.285$ KN/m²

$$\text{Allowable bearing capacity, } q_a = \frac{\text{Ultimate bearing capacity}}{\text{Factor of safety}}$$

$q_a = \frac{365.285}{2} = 182.64\ KN/m^2$

Allowable load, $Q_a = 182.64 \times 3 \times 3 = 1643.78\ KN$

Question: A 4 m x 4 m square footing has its base at 3 m below the ground level. Unit weight of the soil is 18 KN/m^3. Find the allowable bearing capacity of the soil if C = 35 kpa, $N_c = 9$, $N_q = 2$, N_γ = 1 and use factor factor of safety 3. (BCIC – 2017)

Solution:

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

$q_u = 1.3 \times 35 \times 9 + 3 \times 18 \times 2 + 0.4 \times 18 \times 4 \times 1 = 546.3\ KN/m^2$

Allowable bearing capacity, $q_a = \frac{\text{Ultimate bearing capacity}}{\text{Factor of safety}}$

$q_a = \frac{546.3}{3} = 182.1\ KN/m^2$

Question: Determine the ultimate bearing capacity of the continuous footing 1.2 m wide and having the depth of footing is 0.8 m. Unit weight of the soil is, $\gamma = 17.79\ KN/m^3$, Cohesion C = 9.6 KN/m^2 and $\varphi = 20^0$. Ground water table is far more than footing. (EGCB – 2020)

Solution:

$N_q = e^{\pi \tan\varphi} \tan^2\left(45 + \frac{\varphi}{2}\right) = e^{\pi \tan 20} \tan^2\left(45 + \frac{20}{2}\right) = 6.40$

$N_c = (N_q - 1) \cot\varphi = (6.40 - 1) \cot 20 = 14.83$

$N_\gamma = (N_q - 1) \tan\ (1.4\,\varphi) = (6.40 - 1) \tan\ (1.4 \times 20) = 2.87$

Ultimate bearing capacity, $q_u = c\, N_c + q\, N_q + 0.5\, \gamma\, B\, N_\gamma$

$q_u = 9.6 \times 14.83 + (0.8 \times 17.79) \times 6.4 + 0.5 \times 17.79 \times 1.2 \times 2.87 = 264.07\ KN/m^2$

Question: A strip footing carrying a load of 340 KN/m which is located at EGL. Water table is located at EGL. $\gamma_{sat} = 21\ KN/m^3$, width of footing = 2 m and depth of water = 1 m, FS = 3, Ultimate pressure is 530 kN/m^2. Use $q_n = q_u - \gamma D_f$. Prove that the size of footing is safe. (TGTDCL – 2018)

Solution:

$\gamma = \gamma_{sat} - \gamma_w = 21 - 9.81 = 11.19\ KN/m^3, D_f = 1\ m,$

Net Ultimate Bearing capacity, $q_n = q_u - \gamma\, D_f = 530 - 11.19 \times 1 = 518.81$ KN/m2

Allowable bearing capacity, $q_a = \frac{\text{Net Ultimate Bearing capacity}}{\text{Factor of safety}} = \frac{518.81}{3} = 172.93\ KN/m^2$

Applied pressure $= \frac{340}{2} = 170\ KN/m$

Which is less than q_a. Hence the size of footing is safe.

Question: A square footing carry a gross mass of 35,000 kg column load. If the factor of safety is 3, determine the width of the footing. (BWDB – 2019)

Solution:

Total column load, Q_{all} = 35,000 kg = 343.35 KN

For square footing we know,

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

$N_q = e^{\pi \tan\phi'} \tan^2(45 + \frac{\varphi'}{2}) = e^{\pi \tan 30} \tan^2(45 + \frac{30}{2}) = 18.40$

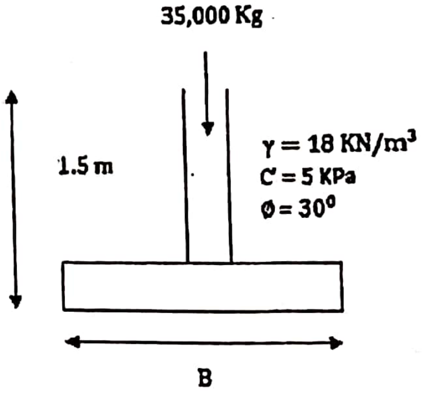
$N_c = (N_q - 1) \cot \varphi' = 30.14$

$N_\gamma = (N_q - 1) \tan(1.4\varphi') = 15.66$

With a factor of safety 3,

$q_{all} = \frac{q_u}{3} = \frac{1}{3}(1.3\, C'\, N_c + q\, N_q + 0.4\, \gamma\, B\, N\gamma)$

and $q_{all} = \frac{Q_{all}}{B^2} = \frac{343.35}{B^2}$



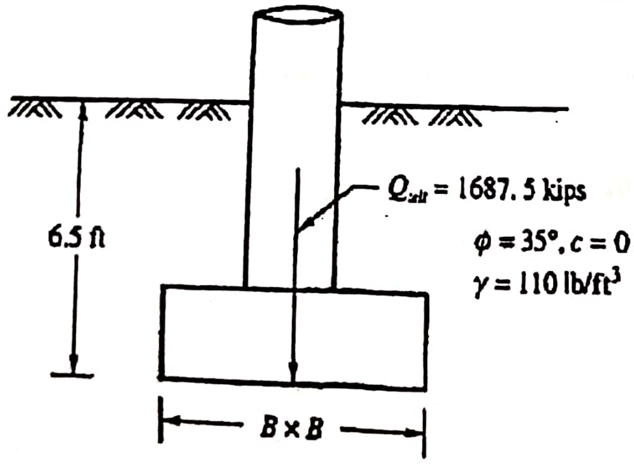
Now, $\frac{343.35}{B^2} = \frac{1}{3}(1.3\, C'\, N_c + q\, N_q + 0.4\, \gamma\, B\, N\gamma)$

$\frac{343.35}{B^2} = \frac{1}{3}$ [1.3 x 5 x 30.14 + (18 x 1.5) x 18.40 + 0.4 x 18 x B x 15.66]

$\frac{343.35}{B^2} = 230.90 + 37.58\, B$

By trial and error, $B = 1.12$ m

Question: A square footing fails by general shear in a cohesionless soil under an ultimate load of Q_{ult} = 1687.5 kips. The footing is placed at a depth of 6.5 ft below ground level. Given φ = 35°, and γ = 110 lb/ft³, determine the size of the footing if the water table is at a great depth.



Solution:

For square footing we know.

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

$N_q = e^{\pi \tan\varphi} \tan^2\left(45 + \frac{\varphi}{2}\right) = e^{\pi \tan 35} \tan^2\left(45 + \frac{35}{2}\right) = 33.3$

$N_c = (N_q - 1)\cot\varphi = (33.3 - 1)\cot 35 = 46.12$

$N_\gamma = (N_q - 1)\tan(1.4\,\varphi) = (33.3 - 1)\tan(1.4 \times 35) = 37.15$

$q_u = \frac{Q_u}{B^2} = \frac{1687.5 \times 10^3}{B^2}$

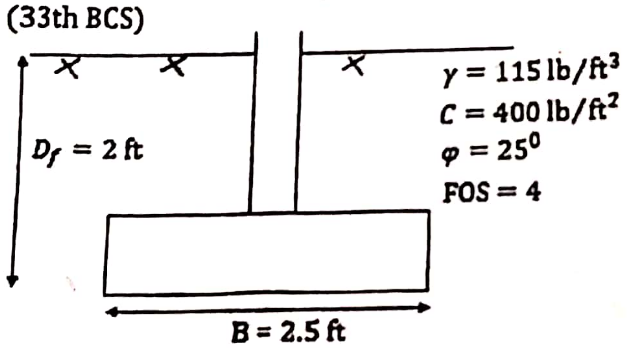
Now, $q_u = 1.3\,c\,N_c + q\,N_q + 0.4\,\gamma\,B\,N_\gamma$

$\frac{1687.5 \times 10^3}{B^2} = 1.3 \times 0 \times 46.12 + 110 \times 6.5 \times 33.3 + 0.4 \times 110 \times B \times 37.15$

$1687.5 \times 10^3 = (23809.5 + 1634.6B)\,B^2$

$B = 6.93$ ft

Question: A continuous footing is shown in the figure below, Using Terzaghi's bearing capacity factors, determine the gross allowable load per unit area (q_{au}) that the footing can carry. (33th BCS)



Given for, $\varphi = 25^0, N_c = 25.13, N_q = 12.72, N_\gamma = 8.34$

Solution:

$c = 400$ lb/ft^2, $\gamma = 115$ lb/ft^3, $\varphi = 25^0, N_c = 25.13, N_q = 12.72, N_\gamma = 8.34, B = 2.5$ ft

$q = \gamma\,D_f = 115 \times 2 = 230$ lb/ft^2

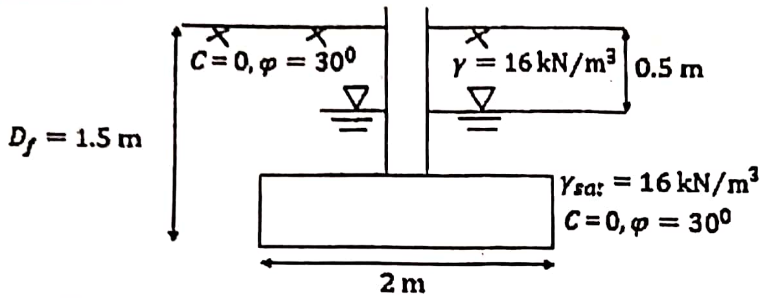
For strip footing we know, $q_u = c\,N_c + q\,N_q + 0.5\,\gamma\,B\,N_\gamma$

$q_u = 400 \times 25.13 + 230 \times 12.72 + 0.5 \times 115 \times 2.5 \times 8.34 = 14176.45$ lb/ft^2

$q_{all} = \frac{q_u}{FS} = \frac{14176.45}{4} = 3544.11$ lb/ft^2

Question: A square footing is shown is figure. Determine the gross allowable load that the footing can carry. Use Terzaghi's bearing capacity equation for general shear failure. Given factor of safety $= 3, N_c = 30, N_q = 18, N_\gamma = 15$. (40th BCS)

Solution:



q = effective surcharge $= D_1 \gamma + D_2 (\gamma_{sat} - \gamma_w)$

$q = 0.5 \times 16 + 1 \times (19 - 9.81) = 17.19$ KN/m^2

$\gamma' = \gamma_{sat} - \gamma_w = 19 - 9.81 = 9.19$ KN/m^3

Here, $\varphi = 30^0$; $N_c = 30, N_q = 18, N_\gamma = 15, C = 0, B = 2$ m

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

$q_u = 1.3 \times 0 \times 30 + 17.19 \times 18 + 0.4 \times 9.19 \times 2 \times 15$

$q_u = 419.7$ KN/m^2

Allowable bearing capacity, $q_a = \dfrac{\text{Ultimate bearing capacity}}{\text{Factor of safety}}$

$q_a = \dfrac{419.7}{3} = 139.9$ KN/m^2

Allowable load, $Q_a = 139.9 \times 2 \times 2 = 559.6$ KN

Bearing capacity of mat foundation

The mat foundation, which is sometimes referred to as a raft foundation, is a combined footing that may cover the entire area under a structure supporting several columns and walls. Mat foundations are sometimes preferred for soils that have low load-bearing capacities, but that will have to support high column or wall loads. Under some conditions, spread footings would have to cover more than half the building area, and mat foundations might be more economical. Mats may be supported by piles, which help reduce the settlement of a structure built over highly compressible soil. Where the water table is high, mats are often placed over piles to control buoyancy. For mats constructed over sand, a factor of safety of 3 should normally be used. For saturated clays with $\varphi = 0$ and a vertical loading condition.

$q_u = C_u N_c F_{cs} F_{cd} + q$

C_u = undrained cohesion

For, $\varphi = 0$ the value of $N_c = 5.14, N_q = 1, N_\gamma = 0$

$F_{cs} = \left(1 + \frac{0.195\,B}{L}\right)$ and $F_{cs} = \left(1 + 0.4\frac{D_f}{B}\right)$

So ultimate bearing capacity, $q_u = 5.14\,c_u\left(1 + \frac{0.195\,B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right) + q$

Net ultimate bearing capacity, $q_{net(u)} = q_u - q = 5.14\,c_u\left(1 + \frac{0.195\,B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right)$

$q_{net(all)} = \frac{q_{net(u)}}{FS}$

The net pressure applied on a foundation, $q = \frac{Q}{A} - \gamma D_f$

Q = dead weight of the structure and the live load

A = area of the raft

In all cases, q should be less than or equal to allowable $q_{net(all)}$

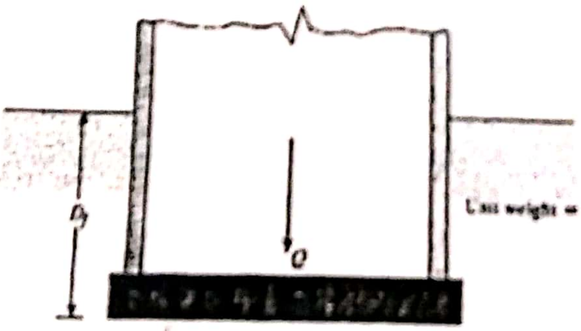


Figure: Definition of net pressure on soil caused by a mat foundation

Compensated Foundation

The net pressure increase in the soil under a mat foundation can be reduced by increasing the depth D_f of the mat. This approach is generally referred to as the *compensated foundation design* and is extremely useful when structures are to be built on very soft clays.

The net average applied pressure on soil is

$$q = \frac{Q}{A} - \gamma D_f$$

For no increase in the net pressure on soil below a mat foundation, q should be zero

$$D_f = \frac{Q}{A\gamma}$$

This relation for D_f is usually referred to as the depth of a *fully compensated foundation.* The factor of safety against bearing capacity failure for partially compensated foundations ($D_f < Q/A\gamma$) may be given as

$$FS = \frac{q_{net(u)}}{q} = \frac{q_{net(u)}}{\frac{Q}{A} - \gamma D_f}$$

Question: When and why raft foundation is used? (32th BCS)

Solution:

Rafts are most often used these days when the strata is unstable or (because of this) a normal strip foundation would cover more than 50% of the ground area beneath the building. There are also situations (usually in areas where mining has occurred) where there may be areas of movement in the strata. A raft foundation is usually preferred under a number of circumstances:

a. The soil-bearing capacity is low.
b. Column loads are heavy.
c. Single footings cannot be used.
d. Piles are not used.
e. Differential settlement must be reduced through the entire footing system.

Question: For a fully compensated mat foundation, find the depth of foundation. Given, Area = 20 m x 30 m. Unit weight of soil 20 KN/m³, Load 6000 KN. (BUET M.Sc – 2017)

Solution:

For of a fully compensated mat foundation, $D_f = \frac{Q}{A\gamma}$

$$D_f = \frac{6000}{20 \times 30 \times 20} = 0.5 \text{ m}$$

Question: Determine the net ultimate bearing capacity of a mat foundation measuring 20 m x 8 m on a saturated clay with $C_u = 85\ \text{kN/m}^2$, $\varphi = 0$ and $D_f = 1.5$ m

Solution:

$$q_{net(u)} = 5.14\, c_u \left(1 + \frac{0.195\, B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right)$$

$$= 5.14 \times 85 \left(1 + \frac{0.195 \times 8}{20}\right)\left(1 + \frac{4 \times 1.5}{8}\right) = 506.3\ \text{kN/m}^2$$

Question: Find the allowable bearing capacity of a raft foundation of 10 m x 10 m and depth of the foundation is 5 m below the ground level. Undrained cohesion of the soil is 40 kN/m² and bearing capacity factor N_C is 5.14. Consider factor of safety two.
(BIFPCL – 2015, BCIC – 2016. MPL – 2017, JOCL – 2018)

Solution:

$$q_{net(u)} = N_c\, c_u \left(1 + \frac{0.195\, B}{L}\right)\left(1 + 0.4\frac{D_f}{B}\right)$$

$$q_{net(u)} = 5.14 \times 40\left(1+\frac{0.195 \times 10}{10}\right)\left(1+\frac{0.4 \times 5}{10}\right) = 294.83 \text{ kN/m}^2$$

$$q_{net(all)} = \frac{q_{net(u)}}{FS} = \frac{294.83}{2} = 147.41 \text{ kN/m}^2$$

Question: If a mat foundation 10 m x 10 m and depth of foundation 1.5 m. Undrained cohesion 75 kpa. Factor of safety 2.5. Bearing capacity factor 5.14, unit weight of the soil, $\gamma = 18$ KN/m³. What is the net bearing capacity? (LGD – 2018)

Solution:

$$q_{net(u)} = N_c\, c_u\left(1+\frac{0.195\,B}{L}\right)\left(1+0.4\frac{D_f}{B}\right)$$

$$= 5.14 \times 75\left(1+\frac{0.195 \times 10}{10}\right)\left(1+\frac{0.4 \times 1.5}{10}\right) = 488.31 \text{ kN/m}^2$$

DEEP FOUNDATION: PILE FOUNDATION

The structural loads may be transferred to deeper firm strata by means of piles. Piles are long slender columns either driven, bored or cast-in-situ.

The major uses of piles are:

1. To carry vertical compression load.
2. To resist uplift load.
3. To resist horizontal or inclined loads.

The factors that govern the selection of piles are:

1. Length of pile in relation to the load and type of soil
2. Character of structure
3. Availability of materials
4. Type of loading
5. Factors causing deterioration

The bearing capacity of a single pile depends upon

1. Type, size and length of pile,
2. Type of soil,
3. The method of installation.

Methods of determining ultimate load bearing Capacity of a single vertical pile

1. By the use of static bearing capacity equations.
2. By the use of SPT and CPT values.
3. By field load tests.
4. By dynamic method.

Question: When and why pile foundation is more suitable than shallow foundation? (32th BCS)

Solution:

Pile foundation is required when the soil bearing capacity is not sufficient for the structure to withstand.

1. Compressible or weak upper soil layer: When one or more upper soil layers are highly compressible and too weak to support the load transmitted by the superstructure, piles are used to transmit the load to underlying bedrock or a stronger soil laye

2. Presence of horizontal forces: When subjected to horizontal forces (see Figure-c), pile foundations resist by bending, while still supporting the vertical load transmitted by the superstructure.

3. Presence of expansive soils: Pile foundations may be considered as an alternative when piles are extended beyond the active zone, which is where swelling and shrinking occur.

4. Subjected to uplifting forces: The foundations of some structures, such as transmission towers, offshore platforms, and basement mats below the water table, are subjected to uplifting forces.

5. Soil erosion: Bridge abutments and piers are usually constructed over pile foundations to avoid the loss of bearing capacity that a shallow foundation might suffer because of soil erosion at the ground surface.

Question: when and why sheet piling is used? (32th BCS)

Solution:

The term sheet piling in general is used for a wall that resists horizontal loads, as opposed to Bearing piles, which are isolated and take load. Sheet piles are sections of sheet materials with interlocking edges that are driven into the ground to provide earth retention and excavation support. Sheet piles are most commonly made of steel, but can also be formed of timber or reinforced concrete.

Sheet pile walls are generally used for the following:
1. Water front structures, for example, in building wharfs, quays, and piers
2. Building diversion dams, such as cofferdams
3. River bank protection
4. Retaining the sides of cuts made in earth.

Question: Differentiate between friction piles and end bearing pile. (31th BCS)

Solution:

End Bearing Piles: These piles are used to transfer the load through water or soft soil ground to a suitable hard bearing stratum. End bearing piles are used to transfer load through water or soft soil to a suitable bearing stratum. Such piles are used to carry heavy loads safely to hard strata. Multi-storied buildings are invariably founded on end bearing piles, so that the settlements are minimized. End bearing piles are typically driven through soft soil, such as a loose silt-bearing stratum underlain by compressible strata. Remember this factor when determining the load the pile can support safely.

Friction Piles: These piles are used to transfer to a depth of a friction load carrying material by mean of a skin friction along the length of piles. Friction piles are used to transfer loads to a depth of a friction-load-carrying materials by means of skin friction along the length of the pile. Such piles are generally used in granular soil where the depth of hard stratum is very great. Some piles transfer the super-imposed load both through side friction as well as end bearing. Such piles are more common, especially when the end bearing piles pass through granular soils.

Friction piles are used in the soil of fairly uniform consistency and the tip is not seated in a hard layer, the load carrying capacity of the pile is developed by skin friction. The load is transferred downward and laterally to the soil.

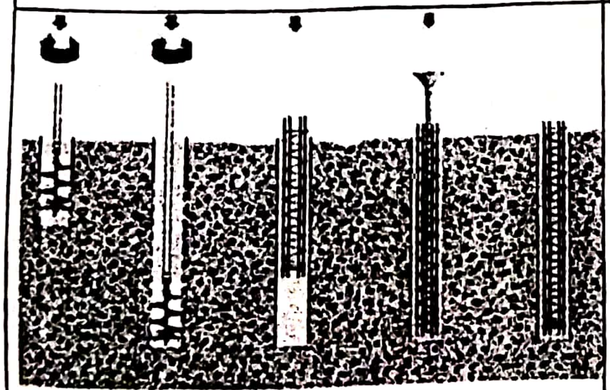
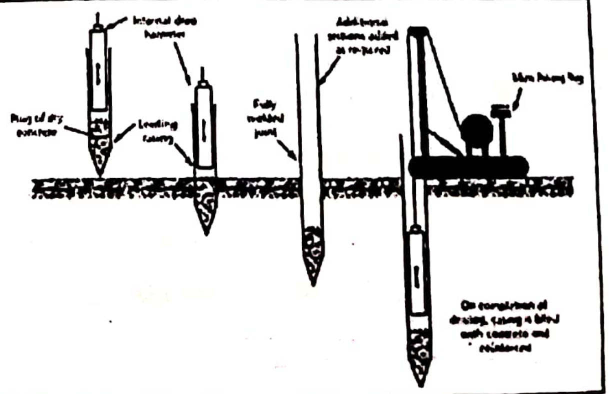
১ *Question:* Driven Pile এবং Bored Pile এর প্রার্থক্য প্রয়োজনীয় চিত্রের মাধ্যমে ব্যাখ্যা করুন। (LGED – 2019)

Solution:

Driven Pile: Driven piles, also known as displacement piles, are a commonly-used form of building foundation that provide support for structures, transferring their load to layers of soil or rock that have sufficient bearing capacity and suitable settlement characteristics. Driven piles are commonly used to support buildings, tanks, towers, walls and bridges and can be the most cost-effective deep foundation solution. They can also be used in applications such as embankments, retaining walls, bulkheads, anchorage structures and cofferdams.

Bored Pile: Bored pile also called drilled shaft, is a type of reinforced-concrete foundation that supports structures with heavy vertical loads. A bored pile is a cast-in-place concrete pile, meaning the pile is cast on the construction site. When used for foundations, bored piles are specified for buildings and other structures that impose loads in the thousands of tons. They are also particularly well suited to unstable or difficult soil conditions.

Difference between Driven Pile & Bored Pile:

Bored Pile	Driven Pile
A bore is construct by removing the soil and the pile is casted at site in the bore.	The pile is casted in a yard brought to the site and driven by some mechanism into the soil.
Bored pile foundations are preferred in a location where the top strata consist of decomposed rock overlying a strata of sound rock.	In such a condition, it becomes difficult to drive the bearing piles through decomposed rock.
Bored pile foundation is shallower in depth.	Driven pile foundation is deeper than Bored Pile.
Bored pile transfer the load only through bearing.	Driven piles are driven straight in and transfer the load through friction and/or bearing.
Less disruption to adjacent soil occurs.	Vibrations & disruption occurs to adjacent soil
Can be installed in narrow areas.	Can't be installed in narrow areas, wide area is required.
Easy to construct.	Advance planning is required for handling and driving.
	

Question: Explain the various types of pile foundation. (34th BCS)

Solution:

Sheet Piles: This type of pile is mostly used to provide lateral support. Usually, they resist lateral pressure from loose soil, the flow of water, etc. They are usually used for cofferdams, trench sheeting, shore protection, etc.

Load Bearing Piles: This type of pile foundation is mainly used to transfer the vertical loads from the structure to the soil. These foundations transmit loads through the soil with poor supporting property onto a layer which is capable of bearing the load.

End Bearing Piles: In this type of pile, the loads pass through the lower tip of the pile. The bottom end of the pile rests on a strong layer of soil or rock. Usually, the pile rests at a transition layer of a weak and strong slayer. As a result, the pile acts as a column and safely transfers the load to the strong layer.

Friction Pile: Friction pile transfers the load from the structure to the soil by the frictional force between the surface of the pile and the soil surrounding the pile such as stiff clay, sandy soil, etc. Friction can be developed for the entire length of the pile or a definite length of the pile, depending on the strata of the soil.

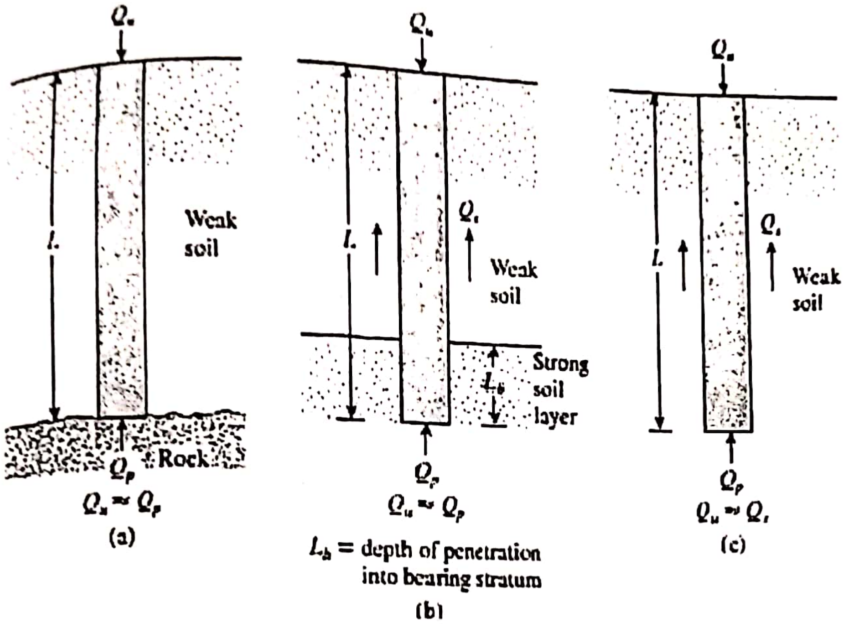
Soil Compactor Piles: Sometimes piles are driven at placed closed intervals to increase the bearing capacity of soil by compacting.

General equation for ultimate bearing capacity

Piles can be divided into three major categories, depending on their lengths and the mechanisms of load transfer to the soil: (a) point bearing piles, (b) friction piles, and (c) compaction piles.

The ultimate capacity of the piles depends entirely on the load-bearing capacity of the underlying material; thus, the piles are called point bearing piles.

When no layer of rock or rocklike material is present at a reasonable depth at a site, point bearing piles become very long and uneconomical. In this type of subsoil, piles are driven through the softer material to specified depths. These piles are called friction piles, because most of their resistance is derived from skin friction.



(a) and (b) Point bearing piles; (c) friction piles

Compaction Piles: Under certain circumstances, piles are driven in granular soils to achieve proper compaction of soil close to the ground surface. These piles are called compaction piles.

Load Transfer Mechanism

When the ultimate load applied on the top of the pile is Q_u a part of the load is transmitted to the soil along the length of the pile and the balance is transmitted to the pile base. The load transmitted to the soil along the length of the pile is called the ultimate friction load or skin load Q_f and that transmitted to the base is called the base or point load Q_b. The total ultimate load Q_u is expressed as the sum of these two,

$$Q_u = Q_b + Q_f = q_b A_b + f_s A_s$$

Q_u = ultimate load applied on the top of the pile
Q_b = ultimate unit bearing capacity of the pile at the base
A_b = bearing area of the base of the pile
A_s = total surface area of pile embedded below ground surface
f_s = unit skin friction (ultimate)

The ultimate capacity of a single pile driven into sand, $Q_u = Q_p + Q_s$

$Q_p = \sigma N_q A_b \leq A_b q_l$

$q_l = 50 N_q \tan\varphi$, σ = vertical pressure at the pile base, A_b = area of the pile base

Static methods for driven piles in saturated clay (Cohesive soil)

For cohesive soils such as saturated clays (normally consolidated), we have for $\varphi = 0$, $N_q = 1$ and $N_\gamma = 0$. The ultimate base load,

$$Q_u = c_b N_c A_b + \alpha c_u A_s$$

α = adhesion factor
c_u = average undrained shear strength of clay along the shaft
c_b = undrained shear strength of clay at the base level
N_c = bearing capacity factor

The value of the bearing capacity factor N_c that is generally accepted is 9 which is the value proposed by Skempton for circular foundations for an L/B ratio greater than 4.

Static methods for driven piles in sand (Cohesionless soil)

For cohesionless soils, $c = 0$. The ultimate base load,

$$Q_u = Q_b + Q_f = q_b A_b + f_s A_s$$
$$Q_u = \sigma N_q A_b + k\,\sigma_{avg} \tan\delta\ A_s$$

A_s = surface area of the embedded length of the pile
A_b = bearing area of the base of the pile
σ = effective overburden pressure at the base level of pile
σ_{avg} = average effective overburden pressure over the embedded depth of the pile
k = average lateral earth pressure coefficient
δ = soil – pile friction angle
N_q = bearing capacity factor

		Values of k	
Pile material	δ	Low D_r (Loose soil)	Low D_r (Dense soil)
Steel	20^0	0.5	1.0
Concrete	$0.75\,\varphi$	1.0	2.0
Wood	$0.67\,\varphi$	1.5	4.0

The unit skin friction increases with depth more or less linearly to a depth of L' and remains constant thereafter. This depth is called critical depth and the magnitude of the critical depth L' may be 15 to 20 pile diameters. A conservative estimate would be, $L' = 15D$

$L' = 15D$ Loose to medium sand
$L' = 20D$ Dense sand

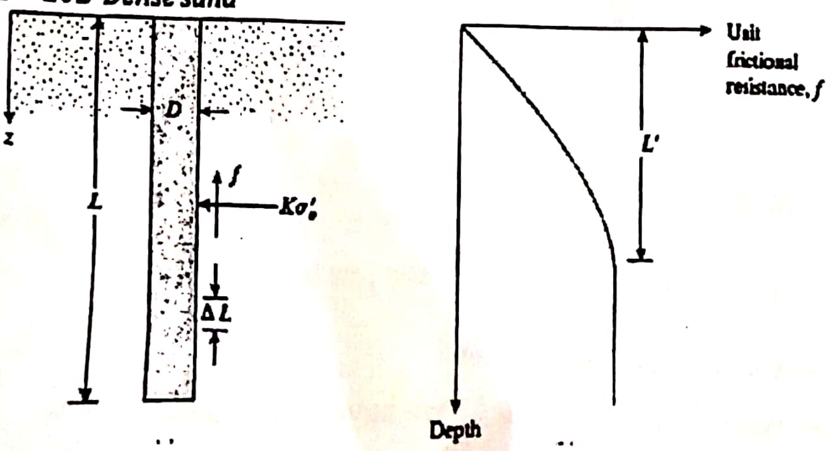


Figure: Unit frictional resistance for piles in sand

Static methods for bored piles in saturated clay (Cohesive soil)

For cohesive soils such as saturated clays (normally consolidated), we have for $\varphi = 0$, $N_q = 1$ and $N_\gamma = 0$. The ultimate base load,

$$Q_u = c_b N_c A_b + \alpha c_u A_s$$

α = adhesion factor
c_u = average undrained shear strength of clay along the shaft
c_b = undrained shear strength of clay at the base level
N_c = bearing capacity factor

The value of the bearing capacity factor N_c that is generally accepted is 9 which is the value proposed by Skempton for circular foundations for an L/B ratio greater than 4.

Static methods for bored piles in saturated clay (Cohesionless soil)

For cohesionless soils, $c = 0$. The ultimate base load,

$$Q_u = Q_b + Q_f = q_b A_b + f_s A_s$$
$$Q_u = \sigma N_q A_b + k \sigma_{avg} \tan\delta A_s$$

A_s = surface area of the embedded length of the pile
A_b = bearing area of the base of the pile
σ = effective overburden pressure at the base level of pile
σ_{avg} = average effective overburden pressure over the embedded depth of the pile
k = average lateral earth pressure coefficient
δ = soil − pile friction angle
N_q = bearing capacity factor

The sand in bored piles is loosened as a result of the boring operation even though it may initially be in a dense or medium dense state. An approximate value of K can be obtained from the following equation, $K = 1 - \sin\varphi$

The value of K generally varies between 0.3 and 0.75. An average value of 0.5 is usually adopted. The value of $\tan\delta$ can be taken equal to $\tan\varphi$ for bored piles excavated in dry soil. If a slurry has been used during excavation, the value of $\tan\delta$ should be reduced.

The ultimate bearing capacity of a bored and cast-in-situ-pile in cohesionless soil is reduced considerably due to disturbance of the soil. The base resistance for bored and cast-in-situ pile is about one-third of the ultimate base resistance of a driven pile.

$$q_b(\text{bored pile}) = \frac{q_b(\text{driven pile})}{3}$$

Question: A concrete pile 18 inch in diameter and 50 ft long is driven into a homogeneous mass of clay soil of medium consistency. The water table is at the ground surface. The unit cohesion of the soil under undrained condition is 1050 lb/ft² and the adhesion factor $\alpha = 0.75$. Compute Q_u and Q_a with $F_s = 2.5$.

Solution:

Here, $C_u = C_b = 1050 \text{ lb/ft}^2$

$$A_b = \frac{\pi D^2}{4} = \frac{\pi \times 1.5^2}{4} = 1.766 \text{ ft}^2$$

$$A_s = \pi D L = \pi \times 1.5 \times 50 = 235.5 \text{ ft}^2$$

$$Q_u = c_b N_c A_b + \alpha c_u A_s = \frac{1050 \times 9 \times 1.766}{1000} + \frac{235.5 \times 0.75 \times 1050}{1000} = 202.15 \text{ kips}$$

$$Q_a = \frac{Q_u}{FS} = \frac{202.15}{2.5} = 81 \text{ kips}$$

Question: Calculate the ultimate capacity of 24" diameter pile with an embedment length of 30' is saturated clay. Given the unconfined strength of clay and adhesion factor are 25 psi and 0.8 respectively. (PGCL – 2014, WRGCL – 2014, MES – 2015)

Solution:

Here, $c_b = c_u = \frac{q_u}{2} = \frac{25}{2} = 12.5 \text{ psi} = 1800 \text{ psf}$

$$Q_u = c_b N_c A_b + \alpha c_u A_s$$

$$Q_u = c_b N_c \frac{\pi D^2}{4} + \alpha c_u \pi D L$$

$$Q_u = 1800 \times 9 \times \frac{3.14 \times 2^2}{4} + 0.8 \times 1800 \times 3.14 \times 2 \times 30$$

$$Q_u = 322.33 \text{ kip}$$

Question: Calculate the ultimate load bearing capacity of a 25 ft long and 24" diameter pile which is embedded in clayey soil. Coefficient of adhesion = 0.5 and unconfined compression strength of soil = 2000 psf. (DPDC – 2014, BWDB – 2016, PGCL – 2017, MPA – 2019)

Solution:

Here, $c_b = c_u = \frac{q_u}{2} = \frac{2000}{2} = 1000 \text{ psf}$

$$Q_u = c_b N_c A_b + \alpha c_u A_s$$

$$Q_u = c_b N_c \frac{\pi D^2}{4} + \alpha c_u \pi D L$$

$$Q_u = 1000 \times 9 \times \frac{3.14 \times 2^2}{4} + 0.5 \times 1000 \times 3.14 \times 2 \times 25$$

$$Q_u = 106814.15 \text{ lb} = 106.81 \text{ kip}$$

Question: Concrete pile of 24" diameter embedded in clay unconfined compression strength of the soil is 4 ksf and the embed length of the pile 30 ft. Calculate ultimate load carrying capacity of pile if adhesive friction between clay and pile is 0.45, unit weight of clay 100 lb/ft³ and water table exists at ground. (BPDB – 2015)

Solution:

$$\text{Here, } c_b = c_u = \frac{q_u}{2} = \frac{4}{2} = 2 \text{ ksf}$$

$$Q_u = c_b\, N_c\, A_b + \alpha\, c_u\, A_s$$

$$Q_u = c_b\, N_c\, \frac{\pi D^2}{4} + \alpha\, c_u\, \pi\, D\, L$$

$$Q_u = 2 \times 9 \times \frac{3.14 \times 2^2}{4} + 0.45 \times 2 \times 3.14 \times 2 \times 30$$

$$Q_u = 226.19 \text{ kip}$$

Question: A nine pile group consisting of 600 mm diameter concrete pile was cast in situ in a soil whose unconfined compressive strength is 80 KN/m². Each pile is 15 m long. Using a reduction factor α = 0.5 and factor of safety is 3, calculate allowable skin friction of single pile. (NHA – 2020)

Solution:

$$\text{Here, } c_u = \frac{q_u}{2} = \frac{80}{2} = 40 \text{ kN/m}^2$$

$$Q_s = \alpha\, c_u\, A_s$$

$$Q_s = \alpha\, c_u\, \pi\, D\, L$$

$$Q_u = 0.5 \times 40 \times 3.14 \times 0.6 \times 15$$

$$Q_u = 565.46 \text{ kN}$$

$$\text{Allowable skin friction} = \frac{565.46}{3} = 188.49 \text{ kN}$$

Question: A nine pile group consisting of 18" diameter concrete pile was cast in situ in a clayey soil whose unconfined compressive strength is 1000 lb/ft². Each pile is 60ft long and the pile spacing is 2.5 times the pile diameter. Using an adhesion factor α=0.5 and factor of safety is 3, calculate allowable skin friction of single pile. (PGCB – 2015, DNCC – 2016, GTCL – 2016, SGCL – 2017, ERL – 2017, BWDB – 2018, BIWTA – 2019)

Solution:

$$\text{Here, } c_u = \frac{q_u}{2} = \frac{1000}{2} = 500 \text{ lb/ft}^2$$

$$Q_s = \alpha\, c_u\, A_s$$

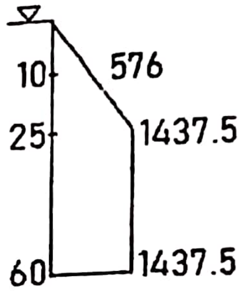
$$Q_s = \alpha\, c_u\, \pi\, D\, L$$

$$Q_u = 0.5 \times 500 \times 3.14 \times 1.5 \times 60$$

$$Q_u = 70676 \text{ lb} = 70.686 \text{ kip}$$

$$\text{Allowable skin friction} = \frac{70.686}{3} = 23.56 \text{ kip}$$

Question: Determine the skin friction capacity of a cast in situ bored RCC pile of diameter 20 inch and length 50 ft in homogeneous sand layer. The top of pile is at 10ft below the ground surface. Unit weight at saturated condition 120 pcf and angle of internal friction 30°. GWT is at the ground surface. (BWDB – 2014)



Solution:

We know, $Q = Q_f + Q_f = \bar{\sigma} N_q A_b + k\, \sigma_{avg} \tan\delta\ A_f$

So, $Q_f = k\, \sigma_{avg} \tan\delta\ A_f$

Critical depth $= 15D = 15 \times \frac{20}{12} = 25'$

Stresses at 10' depth $= (120 - 62.4) \times 10 = 576$ psf

Stresses at 25' depth $= (120 - 62.4) \times 25 = 1437.5$ psf

For loose sand, $k = 1$ and for concrete $\delta = \frac{3}{4}\varphi = 22.50$

Now, $Q_f = k\, \sigma_{avg1} \tan\delta\ A_f + k\, \sigma_{avg2} \tan\delta\ A_f$

$= k \tan\delta\ \pi D\ (\sigma_{avg1} \times L_1 + \sigma_{avg2} \times L_2)$

$= 1 \times \tan 22.5 \times 3.1416 \times \frac{20}{12}\left[\frac{1}{2}(576 + 1437.5) \times 15 + 1437.5 \times 35\right]$

$= 141.87$ kip

Group Efficiency of piles

In most cases, piles are used in groups as shown in Figure, to transmit the structural load to the soil. A pile cap is constructed over group piles. The cap can be in contact with the ground, as in most cases or well above the ground, as in the case of offshore platforms.

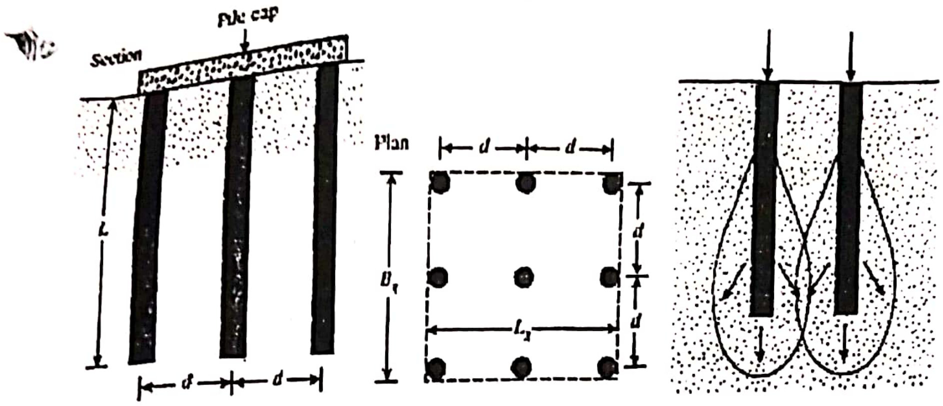
Determining the load-bearing capacity of group piles is extremely complicated and has not yet been fully resolved. When the piles are placed close to each other, a reasonable assumption is that the stresses transmitted by the piles to the soil will overlap, reducing the load-bearing capacity of the piles. Ideally, the piles in a group should be spaced so that the load-bearing capacity of the group is not less than the sum of the bearing capacity of the individual piles. In practice, the minimum center to- center pile spacing 'd' is 2.5D and in ordinary situations, is actually about 3 to 3.5D. The efficiency of the load-bearing capacity of a group pile may be defined as,

$$\eta = \frac{Q_{g(u)}}{\Sigma Q_u}$$

η = Group efficiency

$Q_{g(u)}$ = Ultimate load-bearing capacity of the group pile

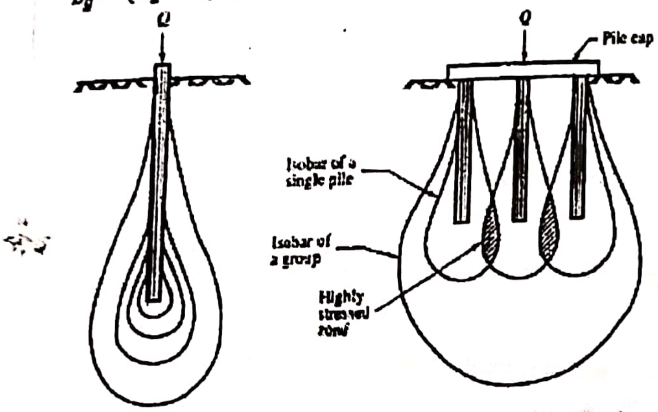
Q_u = Ultimate load-bearing capacity of each pile without the group effect



Number of piles in group = $n_1 \times n_2$ ($L_g \geq B_g$)

$L_g = (n_1 - 1)\, d + 2\,(D/2)$

$B_g = (n_2 - 1)\, d + 2\,(D/2)$



(a) Single pile

(b) Group of piles closely spaced

Ultimate Capacity of Group Piles for Sand

(a) End bearing piles: For driven piles bearing on dense, compact sand with a spacing equal to or greater than 3B, the group capacity is generally taken equal to the sum of individual capacity.

$Q_g = n\, Q_u$

For spacing less than 3B the group capacity is found for block of piles group.

(b) Friction piles: When several closely spaced piles are grouped together, it is reasonable to expect that the soil pressure developed in the soil as resistance will overlap. The bearing capacity of a pile group may or may not be equal to the sum of the bearing capacity of individual piles constituting a group. Theory and tests have shown that the total bearing value Q_{ug} of a group of friction piles particularly in clay may be less than the product of the friction bearing value Q_{up} of an individual pile multiplied by the number of piles n in a group. However there is no reduction due to grouping occurs in end bearing piles.

$Q_{ug} = n\, Q_{up}\, \eta_g$

Q_{ug} = load carried by group of friction piles

Q_{up} = load carried by each friction piles

n = number of piles

η_g = efficiency of group pile

$$\eta_g = 1 - \frac{\theta}{90}\left[\frac{(n-1)\,m + (m-1)\,n}{mn}\right]$$

m = number of rows

n = number of piles in a row

$\theta = tan^{-1}\frac{d}{s}$, d = diameter of pile, s = spacing of pile

Ultimate Capacity of Group Piles in Saturated Clay

The bearing capacity of single pile in clay is mainly due to friction and the point bearing resistance may be negligible. In a pile group, the piles are connected at its top by a pile cap which is rigid. Hence the failure of a pile group is likely to occur at a load which may be smaller than the ultimate load and carried by each pile multiplied by the number of piles in the group. The area of the pile group along failure surface is approximately equal to the perimeter P of the pile group multiplied by the length L of the pile.

The ultimate load, $Q_{ug} = \alpha\, c_u\, A_f + c_b\, N_c\, A_b$

$Q_{ug} = c_u\, P\, L + c_b\, N_c\, A_b$

P = Perimeter of pile group $= 2\,(B + H)$

A_b = cross sectional area of pile group $= B \times H$

α = adhesion factor = 1 (for soft clay)

If piles of the group are so spaced that they act individually rather than acting in the group, the total load capacity of n piles is, $Q_{un} = n\, Q_{up}$

Question: A group of 9 piles in a row was driven into a soft clay extending from ground level to a great depth. The diameter and the length of the piles were 30 cm and 10 m respectively. The unconfined compressive strength of the clay is 70 kPa. If the piles were placed 90 cm center to center, compute the allowable load on the pile group on the basis of a shear failure criterion for a factor of safety of 2.5.

Solution:

(a) Block failure

Here, $c_u = c_b = \frac{q_u}{2} = \frac{70}{2} = 35\ \text{kN/m}^2$

$A_b = 2.1 \times 2.1 = 4.4\ \text{m}^2, P = 4\,B = 4 \times 2.1 = 8.4\ \text{m}$

$Q_{ug} = c_u\, P\, L + c_b\, N_c\, A_b = 35 \times 8.4 \times 10 + 35 \times 9 \times 4.4 = 4326\ \text{kN}$

$$Q_a = \frac{Q_{ug}}{FS} = \frac{4326}{2.5} = 1730\ \text{kN}$$

(*b*) Individual pile failure

$Q_u = c_b\, N_c\, A_b + \alpha\, c_u\, A_s$

$Q_u = c_b\, N_c\, \frac{\pi\, D^2}{4} + \alpha\, c_u\, \pi\, D\, L$

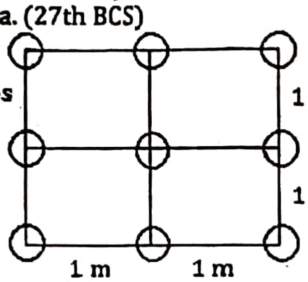
$Q_u = 35 \times 9 \times \frac{\pi \times 0.3^2}{4} + 1 \times 35 \times \pi \times 0.3 \times 10 = 352$ kN   Assume, $\alpha = 1$

$Q_{gu} = n\, Q_u = 9 \times 352 = 3168$ kN

$Q_a = \frac{Q_{ug}}{FS} = \frac{3168}{2.5} = 1267$ kN

The allowable load is 1267 kN

Question: A factor of safety of 3 is required for the group of piles shown below. Find the maximum load as determined by (i) the piles acting as individual (ii) the piles acting as a group. The piles are 20.00 m long and are driven in clay having a $q_u = 90$ kN/m². Assume reasonable values for any necessary additional data. (27th BCS)



Solution:

(*i*) the piles acting as individual

$Q_u = c_b\, N_c\, A_b + \alpha\, c_u\, A_s$

$Q_u = c_b\, N_c\, \frac{\pi\, D^2}{4} + \alpha\, c_u\, \pi\, D\, L$

$Q_u = 45 \times 9 \times \frac{\pi \times 0.3^2}{4} + 1 \times 45 \times \pi \times 0.3 \times 20 = 876.85$ kN   Assume, $\alpha = 1$

$Q_{gu} = n\, Q_u = 9 \times 876.85 = 7891.73$ kN

$Q_a = \frac{Q_{ug}}{FS} = \frac{7891.73}{3} = 2630.57$ Kn

(*ii*) the piles acting as a group

Here, $c_u = c_b = \frac{q_u}{2} = \frac{90}{2} = 45$ kN/m²

$B = 1 + 1 + \frac{0.3}{2} + \frac{0.3}{2} = 2.3$ m

$A_b = B \times B = 2.3 \times 2.3 = 5.29$ m², $P = 4\, B = 4 \times 2.3 = 9.2$ m

$Q_{ug} = c_u\, P\, L + c_b\, N_c\, A_b = 45 \times 9.2 \times 20 + 45 \times 9 \times 5.29 = 10422.45$ kN

$Q_a = \frac{Q_{ug}}{FS} = \frac{10422.45}{3} = 3474.15$ kN

Question: Briefly explain the field procedure of pile load test. (31th BCS)

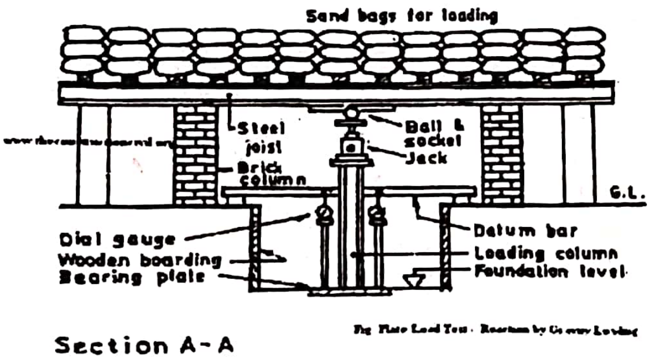
Solution:

Plate Load Test is a field test for determining the ultimate bearing capacity of soil and the likely settlement under a given load. The Plate Load Test basically consists of loading a steel plate placed at the foundation level and recording the settlements corresponding to each load increment. The test load is gradually increased till the plate starts to sink at a rapid rate. The total value of load on the plate in such a stage divided by the area of the steel plate gives the value of the ultimate bearing capacity of soil. The ultimate bearing capacity of soil is divided by suitable factor of safety (which varies from 2 to 3) to arrive at the value of safe bearing capacity of soil.

1. Test Setup: A test pit is dug at site up to the depth at which the foundation is proposed to be laid. The width of the pit should be at least 5 times the width of the test plate. At the centre of the pit a small square depression or hole is made whose size is equal to the size of the test plate and bottom level of which corresponds to the level of actual foundation. The depth of the hole should be such that the ratio of depth to width of the loaded area is approximately the same as the ratio of the actual depth to width of the foundation. The mild steel plate (also known as bearing plate) used in the test should not be less than 25 mm in thickness and its size may vary from 300 to 750 mm. The plate could be square or circular in shape. Circular plate is adopted in case of circular footing and square plate is used in all other types of footings. The plate is machined on side and edges.

2. Testing Procedure: The load is applied to the test plate through a centrally placed column. The test load is transmitted to the column by one of the following two methods.

(i) By gravity loading or reaction loading method
(ii) By loading truss method.



Section A-A

SOIL IMPROVEMENT

Question: What is soil improvement? Write down the various methods of soil improvement? (31th BCS, DSCC – 2019)

Solution:

Soil-improvement techniques involve changing soil characteristics by a physical action, such as vibration, or by the inclusion or mixing in the soil of a stronger material. The aim of this process is as follows:

- Increase the load-bearing capacity and/or the shear strength.
- Reduce both absolute and differential settlements or in certain cases, accelerate them,
- To mitigate or remove the risk of liquefaction in the event of an earthquake or major vibrations.

The various techniques of soil improvement are:-

1. Vertical drains
2. Stone columns
3. Micro-piles
4. Vibro Compaction (Vibroflotation)
5. Dynamic compaction and replacement
6. Pre-compression and consolidation
7. Grouting and Injection
8. Chemical Stabilization
9. Soil Reinforcement
10. Geo-textiles and Geo-membranes

Question: Soil improvement কি বাংলাদেশে ব্যবহার হয় এমন একটি ?soil improvement পদ্ধতির বর্ননা করুন (HED – 2020)

Solution:

Soil improvement: Soil improvement in its broadest sense is the alteration of any property of a soil to improve its engineering performance such as strength, reduced compressibility, reduced permeability, or improved ground water condition.

Soil improvement in Bangladesh: There are various techniques used in Bangladesh for the improvement of the soil based on the construction activity and type of soil. The soil improvement techniques are

Surface Compaction: Most common method of soil improvement is surface compaction. Construction of a new road, a runway, an embankment or any soft or loose site needs a compacted base for laying the structure. If the depth is less, the surface compaction alone can solve the problem. The usual surface compaction devices are rollers, tampers and rammers. All conventional rollers like smooth wheel, rubber-tyred, sheep foot, vibratory and grid rollers are be used.

Drainage Method: Ground water is one of the most difficult problems in excavation work. The presence of water increases the pore water pressure and decreases the shear strength. Heavy inflow of water during excavation collapses the side of open excavations. Drainage method used to control the ground water and ensure a safe and economical construction scheme.

Vibration Method: Vibration method can be effectively used for rapid densification of saturated non-cohesive soils. Some of the mostly adopted vibration methods are vibratory rollers, vibro-displacement Compaction Piles, Vibro-floatation, Heavy Tamping etc.

Pre-Compression and Consolidation: This method aims to consolidate the soil before construction. Various techniques adopted are Preloading and Surcharge Fills, Vertical Drains, Dynamic Consolidation etc.

Soil Reinforcement: Soil Reinforcement is in the form of a weak soil reinforced by high-strength thin horizontal membranes. A large variety of materials such as rubber, aluminum and thermoplastics have been used successfully.

Geotextiles and Geomembranes: Geotextiles are porous fabrics manufactured from synthetic materials, which are primarily petroleum products and others, such as polyester, polyethylene, polypropylene and polyvinyl chloride, nylon, fibre-glass and various mixtures of these. Geotextiles are used as separators, filters, Drains, reinforcement, geo-membranes etc.

Question: What is bearing capacity of soil? Write four methods to improve bearing capacity of soil. (ARMY – 2014)

Solution:

The bearing capacity of soil is defined as the capacity of the soil to bear the loads coming from the foundation. The pressure which the soil can easily withstand against load is called allowable bearing pressure. Some of the methods to improve bearing capacity of soil-

- By increasing the depth of the footing
- Draining the soil
- Compacting the soil
- By blending granular material, like sand, gravel or crushed stone into the natural soil.
- By confining the soil in an enclosed area with the help of sheet piles.
- By driving sand piles.
- Stabilizing the soil with chemicals.

Question: What is geo-textile? (Army – 2014)

Solution:

Geotextiles are textiles in the traditional sense; however, the fabrics usually are made from petroleum products such as polyester, polyethylene, and polypropylene. They also may be made from fiberglass. Geotextiles are not prepared from natural fabrics, which decay too quickly. They may be woven, knitted, or nonwoven.

Geotextiles have four major uses:

1. Drainage: The fabrics can channel water rapidly from soil to various outlets.

2. Filtration: When placed between two soil layers, one coarse grained and the other fine grained, the fabric allows free seepage of water from one layer to the other. At the same time, it protects the fine-grained soil from being washed into the coarse grained soil.

3. Separation: Geotextiles help keep various soil layers separate after construction. For example, in the construction of highways, a clayey subgrade can be kept separate from a granular base course.

4. Reinforcement: The tensile strength of geotextiles increases the load-bearing capacity of the soil.

Question: When and why geotextile is used? (32th BCS)

Solution:

Geotextiles are a kind of geosynthetic material that has become more and more popular over the past fifteen years. The material owes its success in more than 80 applications to a large extent to its resistance to biodegradation. Geotextiles are indeed textiles, however not in the traditional sense of the word. They are no natural materials like cotton, wool or silk. Geotextiles are synthetic fibers that can be made into a flexible, porous, nonwoven needlefelt fabric. They are porous to water flow, to a varying degree. Because of this wide variety, they can be applied in at least five different ways:

1. Separation: Geotextiles will prevent two soil layers of different particle sizes from mixing with each other, as is illustrated the image below.

2. Drainage: Geotextiles will efficiently collect superfluous water from structures, such as rainwater or surplus water, from the soil and discharge it.

3. Filtration: Geotextiles are an ideal interface for reverse filtration in the soil adjacent to the geotextile. In all soils water allows fine particles to be moved. Part of these particles will be halted at the filter interface; some will be halted within the filter itself while the rest will pass into the drain. The complex needle-punched structure of the geotextile enables the retention of fine particles without reducing the permeability of the drain.

4. Reinforcement: Heavy geotextiles can be used to reinforce earth structures by means of fill materials. Thanks to their high soil fabric friction coefficient and high tensile strength, they are an ideal reinforcement solution.

5. Protection: Geotextiles are an ideal protection from erosion of earth embankments by wave action, currents or repeated drawdown. A layer of geotextiles can be placed so as to prevent leaching of fine material. They can be used for rock beaching or as mattress structures. They can even easily be placed under water.

Question: Quick sand এবং Sand drain বলতে কি বোঝায় ?(HED – 2020)

Solution:

Quick sand: Quick sand condition is the floatation of particles of cohesion less soil, like fine gravel and sand, due to vertical upward seepage flow. As sand boiling occurs, the bearing capacity and shear strength of the cohesion less soil decrease and the agitations of soil particles become apparent. Quick sand condition is not a type of soil but a flow condition that occurs in cohesion less soils. Practically, boiling condition may occur when excavations are made below the water table and water is pumped out from the excavation pit to keep the area free from water.

Sand drain: Sand Drain is based on principles of rapidly and centrally dewatering system. Sand Drain is a process of radial consolidation which increases rate of drainage in the embankment by driving casing into the embankment and making vertical boreholes. The holes are backfilled with suitable grade of sand.

Question: Define liquefaction and liquefaction index. (BUET M.Sc – 2013)

Solution:

Liquefaction is a phenomenon in which the strength and stiffness of a soil is reduced by earthquake shaking. Liquefaction occurs in saturated soils, that is, soils in which the space between individual particles is completely filled with water. When liquefaction occurs, the strength of the soil decreases and, the ability of a soil deposit to support foundations for buildings and bridges is reduced. Liquefied soil also exerts higher pressure on retaining walls, which can cause them to tilt or slide. This movement can cause settlement of the retained soil and destruction of structures on the ground surface.

Liquefaction potential index is frequently used to assess the liquefaction susceptibility of soils. The liquefaction potential index (LPI) was used to prepare quantitative maps of liquefaction hazard area.

Question: What is negative skin friction? Write the equation of it and also write the value of adhesion factor α_c for stiff and sandy clay. (BCIC – 2019)

Solution:

Negative skin friction is usually a downward shear drag acting on a pile or pile group due to downward sinking of surrounding soil relative to the piles. This shear drag movements are expected to occur when a segment of the pile penetrates a compressible soil stratum that can consolidate. Downward drag may be caused by (1) Placement of fill on compressible soils, lowering of the groundwater table (2) Placement of fill on Under-consolidated natural or compacted soils.

Negative skin friction of piles in cohesive soil, $F_n = P L_C C_a$

Where: F_n = Negative skin friction

P = Perimeter of the pile

L_C = Pile length in compressible soil

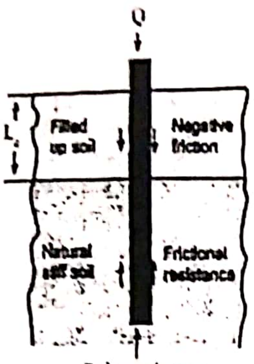
C_a = Unit adhesion, $C_a = \alpha C_u$

α = Adhesion factor

C_u = Undrained cohesion of the compressible layer

Adhesion factor value for stiff clay = 0.48 – 0.75

Adhesion factor value for sandy clay = 0.96 – 1



Question: Write down the short notes on

a. Bearing capacity
b. Compaction
c. Permeability
d. Shear strength (52 BMA)

Solution:

Bearing capacity: It is defined as the capacity of the soil to bear the loads coming from the foundation. The pressure against which the soil can easily withstand against load is called allowable bearing pressure.

Compaction: Soil compaction is defined as the method of mechanically increasing the density of soil. Air during compaction of soil is expelled from the void space in the soil mass and therefore the mass density is increased. Compaction increases the strength characteristics of soils, which increase the bearing capacity of foundations constructed over them.

Permeability: Permeability in fluid mechanics and the earth sciences is a measure of the ability of a porous material to allow fluids to pass through it.

Shear strength: Shear strength is the maximum shear stress which a material can withstand without rupture.

The process of identifying the layers of deposits that underlie a proposed structure and their physical characteristics is generally referred to as subsurface exploration. The purpose of subsurface exploration is to obtain information that will aid the geotechnical engineer in

1. Selecting the type and depth of foundation suitable for a given structure.
2. Evaluating the load-bearing capacity of the foundation.
3. Estimating the probable settlement of a structure.
4. Determining potential foundation problems (expansive soil, collapsible soil and landfill).
5. Determining the location of the water table.
6. Predicting the lateral earth pressure for structures such as retaining walls, sheet pile.
7. Establishing construction methods for changing subsoil conditions.

Question: What are principal objectives of soil exploration? What are common safety measures that should be taken during soil excavation? (36th BCS)

Solution:

Site investigation or Sub-Soil explorations are done for obtaining the information about subsurface conditions at the site proposed for construction. Soil exploration consists of determining the profile of the natural soil deposits at the site, taking the soil samples and determining the engineering properties of soils using laboratory tests as well as in-situ testing methods.

Following are the objectives of site investigation or subsurface exploration.

1. To know about the order of occurrence of soil and rock strata.
2. To know about the location of the groundwater table level and its variations.
3. To determine engineering properties of soil.
4. To select a suitable type of foundation.
5. To estimate the probable and maximum differential settlements.
6. To find the bearing capacity of the soil.
7. To predict the lateral earth pressure against retaining walls and abutments.
8. To select suitable soil improvement techniques.
9. To select suitable construction equipment.
10. To forecast problems occurring in foundations and their solutions.

Safety measures for excavation

1. Prior to excavation work a complete knowledge of underground structures such as sewers, water pipes, gas main etc. is essential so as to proper precaution to prevent accidents.
2. The works must be provided with all protective devices
3. When the depth of excavation exceeds 2 m the trenches should be securely shored and timbered
4. Sheathing should be placed against the side of the trench
5. Excavated materials should be kept away from the edge of trench
6. Heavy equipment such as excavation machine trucks etc, should be kept away at suitable distance from the excavation sides.
7. A fence or barricade should be erected and at night the area must be properly lighted
8. Adequate barricade and excavation sign board.

There are two types of samples:

Disturbed Samples: These types of samples are disturbed but representative, and may be used for the following types of laboratory soil tests:

- Grain size analysis.
- Determination of liquid and plastic limits.
- Specific gravity of soil solids.

- Determination of organic content.
- Classification of soil.
- But disturbed soil samples cannot be used for consolidation, hydraulic conductivity, or shear tests, because these tests must be performed on the same soil of the field without any disturbance (to be representative)

The major equipment used to obtain disturbed sample is (Split Spoon) which is a steel tube has inner diameter of 34.93 mm and outer diameter of 50.8mm.

Undisturbed Samples: These types of samples are used for the following types of laboratory soil tests:

- Consolidation test.
- Hydraulic Conductivity test.
- Shear Strength tests.

These samples are more complex and expensive, and it's suitable for clay, however in sand is very difficult to obtain undisturbed samples. The major equipment used to obtain undisturbed sample is (Thin-Walled Tube).

Degree of Disturbance

If we want to obtain a soil sample from any site, the degree of disturbance for a soil sample is usually expressed as:

$$A_R(\%) = \frac{D_o^2 - D_i^2}{D_i^2} \times 100$$

A_R = area ratio (ratio of disturbed area to total area of soil)
D_o = outside diameter of the sampling tube.
D_i = inside diameter of the sampling tube.
If $(A_R) \leq 10\% \rightarrow$ the sample is undisturbed.
If $(A_R) > 10\% \rightarrow$ the sample is disturbed.

For a standard split-spoon sampler (which sampler for disturbed samples):

$$A_R(\%) = \frac{50.8^2 - 34.93^2}{34.93^2} \times 100 = 111.5\% > 10\% \text{ (disturbed)}$$

Question: Define disturbed and undisturbed sampling? Where are they used?
(52 BMA, 31th BCS)

Solution:

Disturbed soil sample (DS): When the natural conditions of a sample such as its structure, texture, density, natural water contents or the stress conditions are disturbed then the sample is called as disturbed soil sample. By using shovel from auger cutting these samples can be recovered.

Undisturbed soil samples: Without disturbing the natural conditions of a soil sample such as its structure, texture, density, natural water contents or the stress condition the sample obtained is called undisturbed soil sample. This type of soil sample cannot be recovered and it retains the original properties of the soil mass as much as possible.

Usually, scientists test the disturbed samples of soil for texture, soil type, moisture content, as well as the nutrient and contaminant analysis. Undisturbed samples allow the engineer to identify the properties of strength, permeability, compressibility, as well as the fracture patterns among others.

Factors Affecting Compaction

The factors affecting compaction are
1. The moisture content
2. The compactive effort

The compactive effort is defined as the amount of energy imparted to the soil. For a particular compactive effort, there is only one moisture content which gives the maximum dry unit weight. The moisture content that gives the maximum dry unit weight is called the optimum moisture content. If the compactive effort is increased, the maximum dry unit weight also increases, but the optimum moisture content decreases. The following tests are normally carried out in a laboratory.

1. Standard Proctor test (ASTM Designation D-698)
2. Modified Proctor test (ASTM Designation D-1557)

A soil at a selected water content is placed in layers into a mold of given dimensions with each layer compacted by 25 or 56 blows of a 5.5 lb (2.5 kg) hammer dropped from a height of 12 in (305 mm), subjecting the soil to a total compactive effort of about 12,375 fl-lb/ft³ (600 kNm/m³). The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of water contents to establish a relationship between the dry unit weight and the water content of the soil. This data, when, plotted, represents a curvilinear relationship known as the compaction curve or moisture-density curve.

A soil at a selected water content is placed in five layers into a mold of given dimensions, with each layer compacted by 25 or 56 blows of a 10 lb (4.54 kg) hammer dropped from a height of 18 in. (457 mm) subjecting the soil to a total compactive effort of about 56,250 ft-lb/ft³ (2700 kN-m/m³). The resulting dry unit weight is determined. The procedure is repeated for a sufficient number of water contents to establish a relationship between the dry unit weight and the water content for the soil. This data, when plotted, represents a curvilinear relationship known as the compaction curve or moisture-dry unit weight curve.

Question: What is the difference between standard proctor test and modified proctor test?

Solution:

Standard proctor test	Modified proctor test
Material pass #4	Material pass #4
4" diameter mold	4" diameter mold
3 layers	5 layers
5.5 lb (2.5 kg) hammer	10 lb (4.5 kg) hammer
12" falling distance	18" falling distance
25 blows per layer	25 blows per layer

Question: What is standard Proctor Test (SPT)? How is it conducted? Explain with its uses. (31th BCS)

Solution:

SPT: The standard penetration test (SPT) is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil.
Collection Process: The test uses a thick-walled sample tube, with an outside diameter of 50.8 mm and an inside diameter of 35 mm, and a length of around 650 mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a mass of 63.5 kg (140 lb) falling through a distance of 760 mm (30 in). The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm (6 in) up to a depth of 450 mm (18 in) is recorded. The sum of the number of blows required for the second and third 6 in. of penetration is termed the "standard penetration resistance" or the "N-value". In cases where 50 blows are insufficient to advance it through a 150 mm (6 in) interval the penetration after 50 blows is recorded. The blow count provides an indication of the density of the ground, and it is used in many empirical geotechnical engineering formulae.

SPT *N* value is widely used as it is an index for quick strength characterization due to its simplicity. In estimation of other parameters also, SPT *N* value is used for immediately understand the soil profile, the probable foundation depth, the probable Safe bearing capacity, the other parameters like liquefiablilty of soil etc. The N value is used for working out c i.e. cohesion (in clayey soils) and Ø (angle of internal friction for Sandy soils) with direct empirical co-relations.

Question: What is meant by SPT of soil? How is it carried out in field? Explain the relationship of SPT value with bearing capacity of soil. (30th BCS)

Solution:

SPT-N value: The standard penetration test (SPT) is an in-situ dynamic penetration test designed to provide information on the geotechnical engineering properties of soil.
Collection Process: The test uses a thick-walled sample tube, with an outside diameter of 50.8 mm and an inside diameter of 35 mm, and a length of around 650 mm. This is driven into the ground at the bottom of a borehole by blows from a slide hammer with a mass of 63.5 kg (140 lb) falling through a distance of 760 mm (30 in). The sample tube is driven 150 mm into the ground and then the number of blows needed for the tube to penetrate each 150 mm (6 in) up to a depth of 450 mm (18 in) is recorded. The sum of the number of blows required for the second and third 6 in. of penetration is termed the "standard penetration resistance" or the "N-value". In cases where 50 blows are insufficient to advance it through a 150 mm (6 in) interval the penetration after 50 blows is recorded. The blow count provides an indication of the density of the ground, and it is used in many empirical geotechnical engineering formula.

SPT *N* value is used for immediately understand the soil profile, the probable foundation depth, the probable Safe bearing capacity, the other parameters like liquefiablilty of soil etc.

Meyerhof's equations:
For footing width, 4 feet or less: $q_a = (N/4)/K$
For footing width, greater than 4 feet: $q_a = (N/6)[(B+1)/B]^2/K$

Bowles' equations:
For footing width, 4 feet or less: $q_a = (N/2.5)/K$
For footing width, greater than 4 feet: $q_a = (N/4)[(B+1)/B]^2/K$

q_a = Allowable soil bearing capacity in kip/ft^2
N = SPT numbers below the footing
B = Footing width in feet
D = Depth from ground level to the bottom of footing in feet.
$K = 1 + 0.33(D/B) \leq 1.33$

Question: In SPT test for each 15 cm penetration needs 10, 15 & 20 SPT value. Calculate N value. (EGCB – 2020, BCMCL – 2020, APSCL – 2020)

Solution:

N value = 2nd penetration + 3rd penetration = 15 + 20 = 35

Question: প্রতি ৬ ইঞ্চি পর পর ধারাবাহিক penetration এর জন্য blow লাগে 4, 6, 8 টি তাহলে SPT এর মান কত? (HED – 2020)

Solution:

SPT value = 6 + 8 = 14

Question: The soil used in an embankment construction found to have MDD of 19 kN/m^3 from standard proctor test. The specifications are; minimum 95% compaction. After finishing by roller a block of soil is extracted from the compaction layer using a pipe section of 100 mm length and 75 mm internal dia. Wet and dry weight of extracted soil is 865 gm and 810 gm respectively. Does the layer meet specifications? (DMTCL – 2019)

Solution:

Maximum dry density MDD, $\gamma_{d,max} = 19\ kN/m^3$

Volume of the sample, $V = \frac{\pi d^2 h}{4} = \frac{\pi \times 75^2 \times 100}{4} = 441787.5\ mm^3 = 4.417 \times 10^{-4}\ m^3$

Dry weight of the sample, $W_s = 810\ gm = \frac{810 \times 9.81}{1000 \times 1000} = 0.00794\ kN$

Dry density in the field, $\gamma_d = \frac{W_s}{V} = \frac{0.00794}{4.417 \times 10^{-4}} = 17.97\ kN/m^3$

Percent compaction $= \frac{\text{Dry density in the field}}{\text{Max dry density in the lab}} = \frac{17.97}{19} \times 100 = 94.57\%$

Percent compaction = 94.57% which is less than the specified 95% compaction.
So the layer doesn't met the specifications.

Solution: Evaluate the ratio of energy imparted by Modified Proctor Test and Standard Proctor Test. (BPDB – 2016)

Solution:

Comparative energy per unit volume by Modified Proctor Test, $E = \frac{N\,n\,w\,h}{V}$

Here, Number of blows per layer, $N = 25$
No. of layer, $n = 5$
Height of mould, $h = 18" = 0.4572$ m
Wight of hammer, $w = 10$ lb $= 4.54$ kg
Volume of mould, $V = 944$ cc
Now, $E = \frac{N\,n\,w\,h}{V}$

$$E = \frac{25 \times 5 \times 4.54 \times 0.4572 \times 9.81}{944 \times 10^{-6}} = 2695126.033\ \text{J/m}^3 = 2695.126\ \text{kJ/m}^3$$

Comparative energy per unit volume by Standard Proctor Test, $E = \frac{N\,n\,w\,h}{V}$

Here, Number of blows per layer, $N = 25$
No. of layer, $n = 3$
Height of mould, $h = 12" = 0.305$ m
Wight of hammer, $w = 5.5$ lb $= 2.5$ kg
Volume of mould, $V = 944$ cc

Now, $E = \frac{N\,n\,w\,h}{V} = \frac{25 \times 3 \times 2.5 \times 0.305 \times 9.81}{944 \times 10^{-6}} = 594.29\ \text{kJ/m}^3$

So, Modified proctor test imparts 4.53 times energy per unit volume than Standard proctor test.

Cone Penetration Test is a versatile sounding method that can be used to determine the materials in a soil profile and estimate their engineering properties. The test is also called the static penetration test and no boreholes are necessary to perform it. In the original version, a 60° cone with a base area of 10 cm² (1.55 in²) was pushed into the ground at a steady rate of about 20 mm/sec (=0.8 in/sec) and the resistance to penetration (called the point resistance) was measured.

Question: Advantage of CPT over SPT. disadvantage of CPT. (BUET M.Sc – 2014)

Solution:

Advantages of CPT over SPT test:
→ Rapid and inexpensive
→ Greater accuracy
→ Continuous soil profile
→ Reduce cost over SPT

Disadvantages:

→ Inability to penetrate through gravel and cobbles
→ Lack of sampling
→ Elimination of operator error
→ Require special equipment and skilled operator

142 ✓