Soil Mechanics

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MODULE 1. Fundamentals of Soil Mechanics

LESSON 1. Introduction of Soil Mechanics

1.1 Soil and Soil mechanics

According to the Civil Engineering, the soil means all the naturally occurring, relatively unconsolidated organic or inorganic earth materials lie above the earth surface (Ranjan and Rao, 2000). The rocks are an agglomeration of mineral particles bonded together by strong molecular force. However, many hard soils can be termed as soft rock and vice versa. Rocks can be bed rocks or fragments of gravels, pebbles within the soil. The soil mechanics is the branch of Civil Engineering that uses the principles of mechanics, hydraulics and to some extent chemistry to solve the engineering problems related to soil. On the other hand, rock mechanics is a branch that applies the principles of mechanics to understand the behavior of rock masses.

1.2 Soil Formation

Based on the formation, soil can be divided into two groups: i) soils which are formed due to physical and chemical weathering of the parent rocks ii) soil which are of organic origin. The causes of the physical weathering of parent rocks are the impact and grinding action of flowing water, wind, ice and splitting action of ice, plants and animals. Gravel and sand are the soil those are formed due to physical weathering of parent rocks. The causes of the chemical weathering of the parent rocks are oxidation, hydration, carbonation and leaching by organic acids and water. Clay and some extent silt are the soils those are formed due to chemical weathering of parent rocks. Figure 1.1 shows the various types of soils based on their formation.

Figure 1.2 shows the various types of soils formed due to weathering of parent rocks. The soils those are formed due to the weathering of parent rocks can be divided into two groups: i) Transported soil and ii) Residual soil. The soil is called Transported soil if the products of rock weathering are transported from the place where they originated and re-deposited to any other place. The soil is called Residual soil if the products of rock weathering are still located at the place where they originated.

Depending upon the way of formation, the transported soil can be divided into five types: i) Alluvial deposit ii) Aeolian deposit iii) Glacial deposit iv) Lacustrine deposit v) Marine deposit. The Alluvial soils are deposited from the suspension of flowing water. The soils are called Aeolian if they have been transported by wind. If the soils have been transported by ice are called Glacial deposit. Lacustrine soils have been deposited from the suspension in still and fresh water of lakes. Marine soils have been deposited from the suspension in sea water.

1.3 Common Soils in India

The common Indian soils are presented below:

(i) Marine deposits: These soils are found along the coast in narrow tidal plains. These are very soft with low shear strength and high compressibility. Construction of structures on these soils is very challenging due to low bearing capacity and excessive settlement.

(ii) Laterites soils: These soils are found in Kerala, Karnataka, Maharashtra, Orissa, and West Bengal. These are formed due to the decomposition of rocks and reddish in color.

(iii) Black cotton soils: These are expansive soils found in Maharashtra, Gujarat, Madhya Pradesh, Karnataka, Tamil Nadu, Andhra Pradesh and Uttar Pradesh.

(iv) Alluvial soils: These are found in Assam to Punjab covering a large part of northern India. These soils have alternating layers of sand, silt and clay.

(v) Desert soils: These are found in large parts of Rajasthan.

(vi) Boulder deposits: These are found in the sub-Himalayan regions of Himachal Pradesh and Uttar Pradesh.

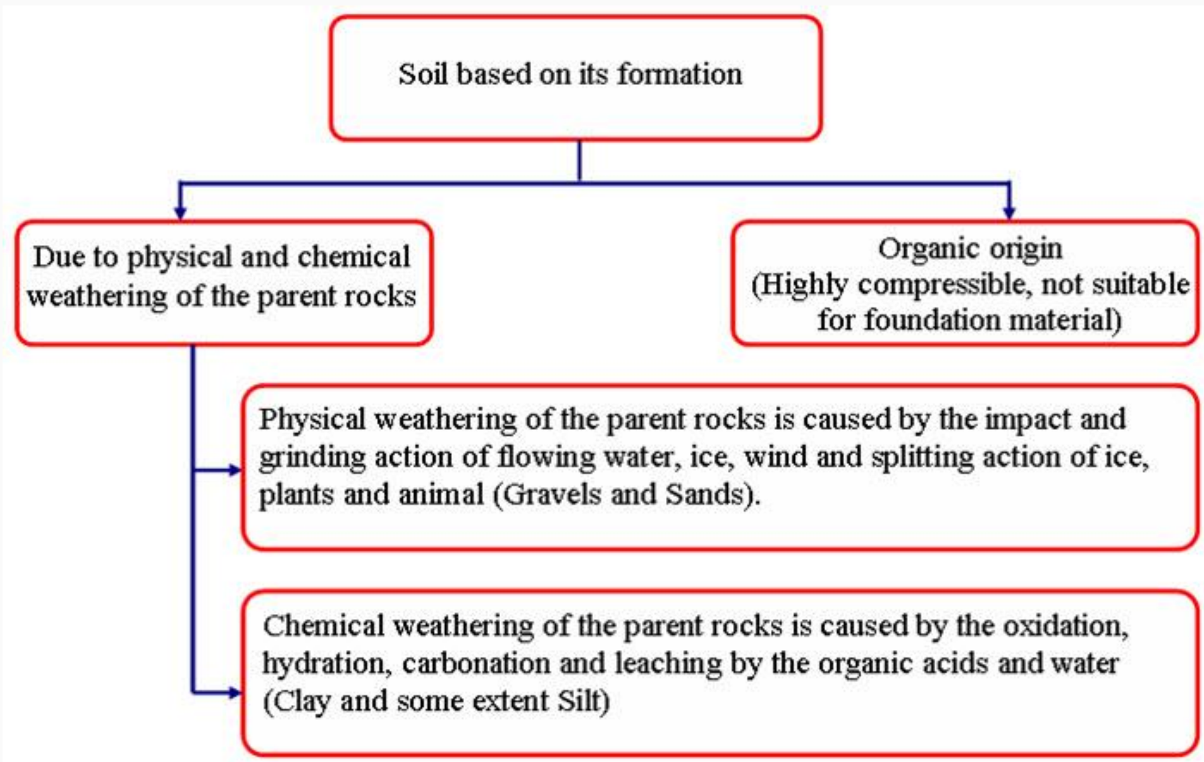


Figure 1.1: Various types of soils based on their formation.

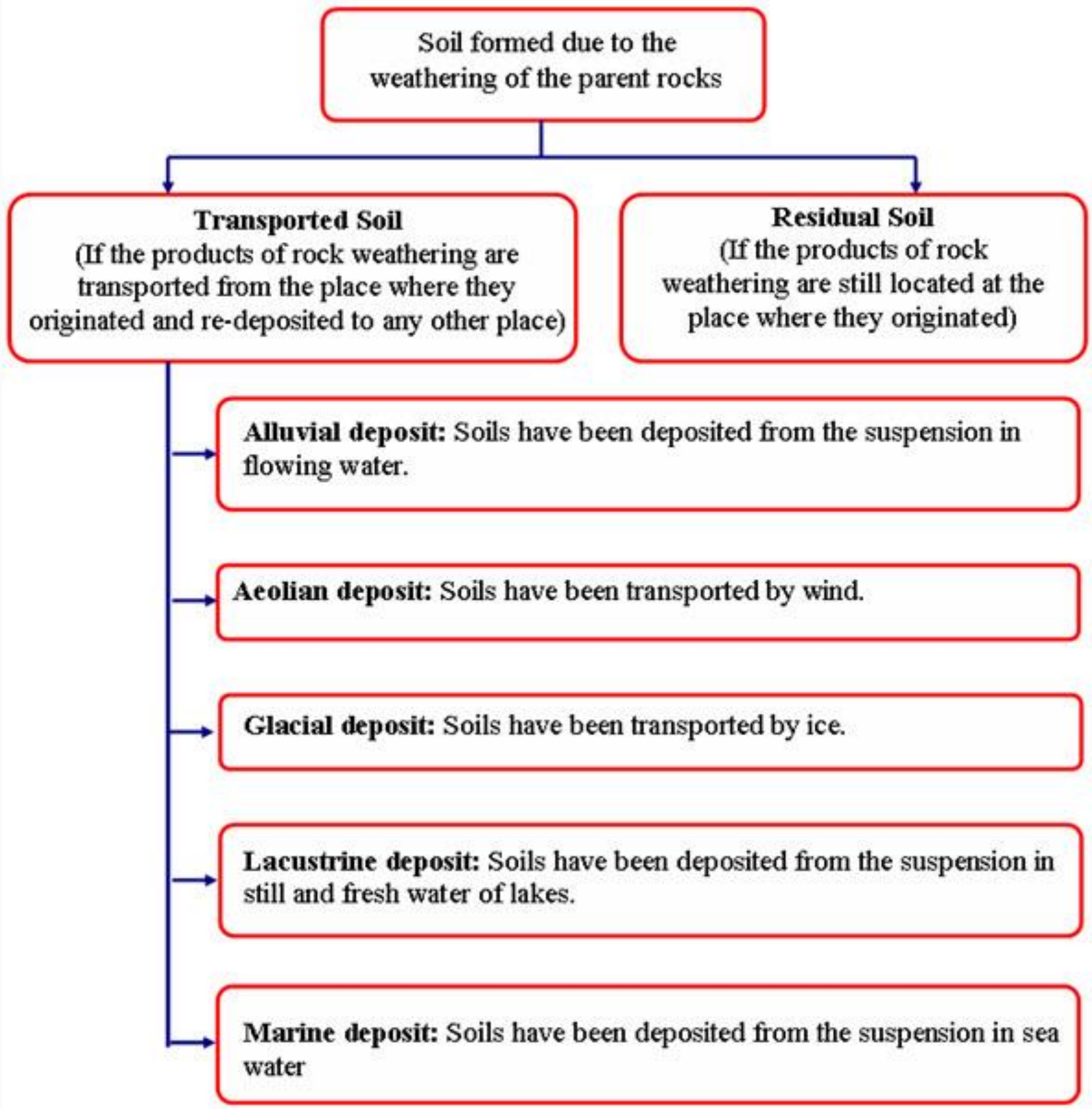


Figure 1.2: Various types of soils formed due to weathering of parent rocks.

LESSON 2. Phase Diagram

2.1 What is Phase Diagram?

In general, soil is a three-phase system composed of solid, liquid and gaseous matter. The voids in between solid soil particles are filled with either water or air or both. In completely saturated soil, the voids are filled with water only whereas, incase of completely dry soil voids are filled with air only. In case of partially saturated soil, the voids are filled with both water and air. The representation of different phases of soil with diagrams is called as phase diagram. Figure 2.1(a) shows the phase diagram of partially saturated soil (three-phase system: solid, water and air). Figure 2.1(b) and Figure 2.1(c) show the phase diagram of completely saturated (two-phase system: solid and water) and dry soil (two-phase system: solid and air), respectively. In the figures, weight of air (W_a) is assumed to be zero. The weight of the voids is equal to the weight of water (W_w). The weight of solid is represented by W_s. The total weight of the partially and completely saturated soil is $W = W_s + W_w$ whereas, the total weight of completely dry soil is $W = W_s$. The volume of solid, water and air are represented by V_s, V_w and V_a, respectively. The total volume of the soil is $V = V_v + V_s$, where V_v is the volume of voids ($V_v = V_w + V_a$). In case of completely saturated and dry soil the volume of voids $V_v = V_w$ and $V_v = V_a$, respectively.

2.2 Definitions

Void Ratio

Void ratio (e) of a soil is the ratio of the volume of voids to the volume of solid. Thus,

$$e={{{V_v}} \over {{V_s}}} \qquad (2.1)$$

The void ratio is greater than zero (as soil has to contain some voids), but there is no upper limit to the value of void ratio. As this the ratio of two volumes, it is unit less.

Porosity

The porosity (n) of a soil is the ratio of the volume of voids to the total volume of the soil. Thus,

$$n={{{V_v}} \over V} \times 100 \qquad (2.2)$$

The porosity cannot be greater than 100%. As this the ratio of two volumes, it is unit less.

Water content

The water content or moisture content (w) of a soil is the ratio of weight of water to the weight of solids present in the soil. Thus,

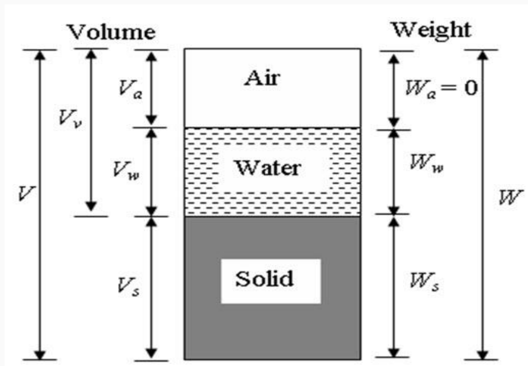
$$w={{{w_w}} \over {{w_s}}} \times 100 \qquad (2.3)$$

The water content is greater than or equal to zero, but there is no upper limit to the value of water content. As this the ratio of two weights, it is unit less.

Degree of saturation

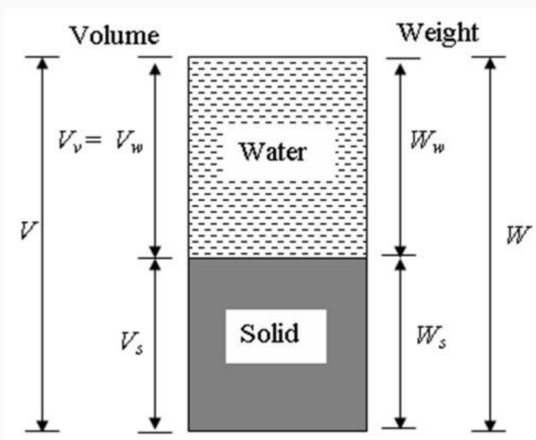
The degree of saturation (S) is the ratio of the volume of water to the volume of voids. Thus,

$$S={{V_w} \over {V_v}} \times 100 \qquad (2.4)$$

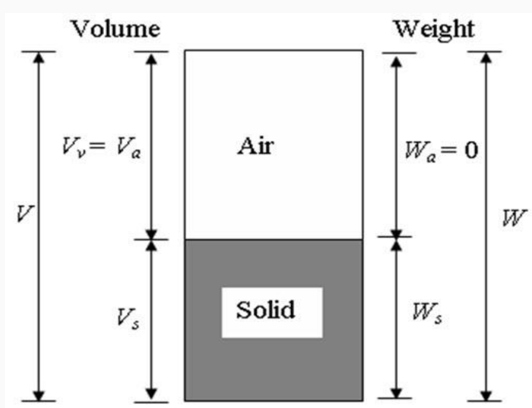


e

(a) Partially saturated soil (Three-phase system)



(b) Completely saturated soil (Two-phase system)



(b) Completely dry soil (Two-phase system)

Figure 2.1. Phase diagrams.

For fully saturated soil, $V_v = V_w$, Thus, S = 100% or 1. Similarly, for completely dry soil, V_w = 0, Thus, S = 0. For partially saturated soil, S value can be in between 0 to 100%. As this the ratio of two volumes, it is unit less.

Unit weight

Depending upon the state of the soil, the unit weight of the soil also changes. Bulk unit weigh*t* (${\gamma _{bulk}}$ or ${\gamma _t}$) of the soil is defined as the total weight of the soil mass per unit of total volume. The bulk unit weight is the unit weight of the soil in its natural condition. In SI unit, it is expressed as 'kN/m^3'. Thus,

$${\gamma _t}={W \over V}={{{W_s} + {W_w}} \over {{V_s} + {V_w} + {V_a}}} \qquad (2.5)$$

Saturated unit weight (${\gamma _{sat}}$) of the soil is defined as the total weight of the soil when it is fully saturated per unit of total volume. This is the unit weight of soil under completely saturated condition. Thus,

$${\gamma _{sat}}={{{W_{sat}}} \over V} \qquad (2.6)$$

Dry unit weight (${\gamma _d}$) of the soil is defined as the weight of soil solids per unit of total volume. This is the unit weight of soil under completely dry condition. Thus,

$${\gamma _d}={{{W_s}} \over V} \qquad (2.7)$$

Submerged unit weight ($\gamma '$) of the soil is equal to the saturated unit weight of soil minus unit weight of the water (${\gamma _w}$). The unit weight of water is taken to be 9.8 kN/m^3 or approximately 10 kN/m^3. Thus,

Unit weight of solids (${\gamma _s}$) is the ratio of weight of solid to the volume of solids. Thus,

$${\gamma _s}={{{W_s}} \over {{V_s}}} \qquad (2.9)$$

Specific gravity

The specific gravity (G_s) of the soil is defined as the ratio of the weight of a given volume of solid to the weight of an equivalent volume of water at 4°C. Thus,

$${G_s}={{{W_s}} \over {{V_s}{\gamma _w}}}={{{\gamma _s}} \over {{\gamma _w}}} \qquad (2.10)$$

The specific gravity of the soil varies in between 2.65 to 2.80 (Ranjan and Rao, 2000). Specific gravity is also a unit less quantity.

2.3 Important Relationships

Followings are the important relationships between the various quantities defined in the previous section.

$$n={e \over {1 + e}} \qquad (2.11)$$

$$V_s={V \over {1 + e}} \qquad (2.12)$$

$$V_v={e \over {1 + e}}V \qquad (2.13)$$

$$e={{G_s}w \over S} \qquad (2.14)$$

$$\gamma_{bulk}=\gamma_t={{G_s + Se} \over {1 + e}}\gamma_w \qquad (2.15)$$

For completely saturated soil, $S=1$, Thus,

$$\gamma_{sat}={{G_s + e} \over {1 + e}}\gamma_w \qquad (2.16)$$

For completely dry soil, $S=0$, Thus,

$$\gamma_d={{G_s} \over {1 + e}}\gamma_w \qquad (2.17)$$

$$\gamma '={{G_s - 1} \over {1 + e}}\gamma_w \qquad (2.18)$$

$$\gamma_d={{\gamma_{bulk}} \over {1 + w}} \qquad (2.19)$$

Problem 1: In a partially saturated soil, moisture or water content is 20% and γ_{bulk} = 18 kN/m^3. Determine the degree of saturation and void ratio. G_s = 2.65. Take the unit weight of the water as 10 kN/m^3.

Solution:

$$\gamma_d={{\gamma_{bulk}} \over {1 + w}}={{18} \over {1 + 0.2}} = 15\;kN/{m^3}$$

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

Thus, $e={{G_s\gamma_w} \over {\gamma_d}}-1={{2.65 \times 10} \over {15}}-1=0.767$

Thus, the void ratio is 0.767.

$$e={{G_s}w \over S}$$

Thus, $S={{G_s}w \over e}={{2.65 \times 0.2} \over {0.77}}=0.691$

Thus, the degree of saturation is 69.1%.

LESSON 3. Index Properties of Soil

3.1 Various types of Soil Properties

The properties of soil are dependent on either individual grains of the soil (soil grain properties: such as mineralogical composition, size and shape of grains, specific gravity) or on the soil structure, mode of formation, stress history etc. (soil aggregate properties: such as Atterberg limits, permeability, relative density, unconfined compressive strength etc.).

The shape of soil grains is a very useful grain property incase of coarse grained soils. Soil grains can be Bulky, Flaky or plate-shaped, needle-shaped. For Bulky grains, all the dimensions of a grain are more or less same (sand or gravels). The bulky grains can be angular, sub-angular, rounded, sub-rounded. Figure 3.1 shows the various shapes of bulky particle. Soil having particles with high angularity posses higher shearing strength as compared to the soil having less amount of angular particles as particles with high angularity provide more resistance against displacement. Incase of flaky grains, thickness of the particles is vary less as compared to the other two dimensions of the grain (submicroscopic crystals of clay minerals). Incase of needle-shaped grain, one dimension is much longer than the other two dimensions (clay mineral like kaolinite).

3.1.1 Grain size distribution

Grain size distribution gives an idea about the percentage of various soil grains present in a dry soil sample. Grain size analysis can be carried out by sieve analysis incase of coarse-grained soil (such as sands, gravels) and hydrometer analysis incase of fine-grained soil (such as silt, clay). Incase of hydrometer analysis, Stokes's law is used to determine the grain size distribution of the soil.

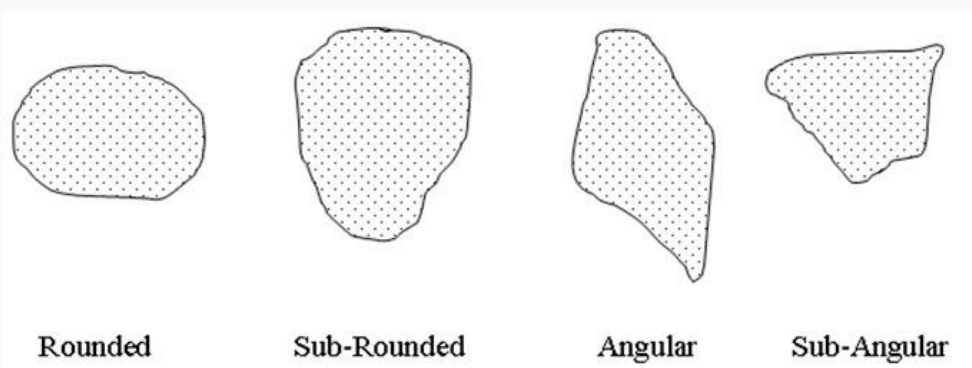


Figure 3.1. Shapes of Bulky Particles.

Based on the Sieve and Hydrometer analysis, grain-size distribution curves are plotted for different soils (as shown in Figure 3.2). In the graphs, X-axis represents the particle size (mm) (in log scale) and Y-axis represents percentage passing or percentage finer (in normal scale). In curve number 1, the percentage passing corresponding to the 0.5 mm particle size is 30%.

This means that the 30% particle size is less that 0.5mm. Based on the shape and slope of the grain-size distribution curve, the type and gradation of the soil can be identified. A well-graded soil has a good representation of grain or particle sizes over a wide range and has smooth gradation (Curve No.1). Poorly-graded soil has either an excess or a deficiency of certain particle sizes (Curve No. 2). In the curve number 2, particle sizes in between 0.3 mm to 1.0 mm are excessively present (almost 80%). The grain size distribution curve of a poorly-graded soil has uniform gradation. In the Gap-graded soil, some of the particles sizes are missing (Curve No. 3). In the Figure 3.2, for curve number 3, particle sizes in between 0.3 mm to 1.0 mm are missing.

In the grain size distribution curve, D_{10} (the diameter corresponding to 10% percentage finer or passing) is called the effective size. From the particle distribution curve, two shape parameters the (C_u) and Coefficient of Curvature (C_c) are determined.

The Coefficient of Uniformity (C_u) and Coefficient of Curvature (C_c) are defined as:

$$C_u=\frac{D_{60}}{D_{10}} \quad \text{and} \quad C_c=\frac{D_{30}^2}{D_{10}\times D_{60}} \qquad (3.1)$$

where D_{60} and D_{30} are the grain diameter corresponding 60% and 30% finer, respectively. For well-graded soil, C_c value must be in between 1 to 3 and C_u value must be greater than 4 for gravels and greater than 6 for sands.

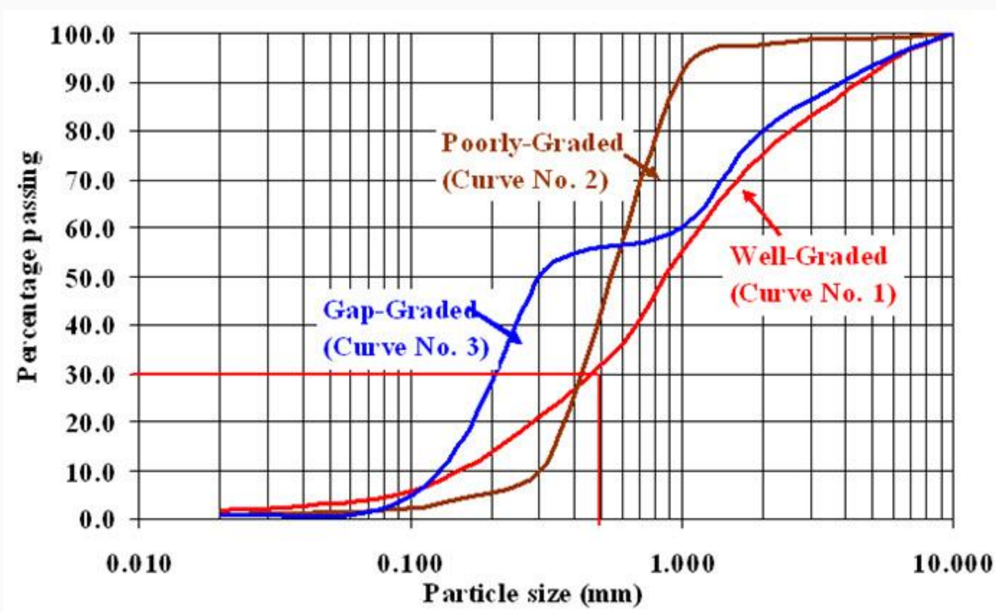


Figure 3.2. Grain-Size Distribute on Curves.

3.2 Atterberg limits

The physical properties of clays are significantly influenced by the amount of water present in the soil. Four states or stages are used to describe the consistency (the term used to describe whether soil is very soft, soft, medium, stiff or hard) of clayey soils depending upon the amount of water present in them, these are: **Liquid State**, **Plastic State**, **Semi-Solid State** and **Solid State**. The water contents at which soil changes its state from one to another is called consistency limits or **Atterberg limits**. If fine-grained soil is mixed with significant amount of water it is called Liquid State of the soil. In this state soil has practically no resistance to flow. If the amount of water is reduced from the soil then soil will change its state from liquid to plastic and from plastic to semi-solid and from semi-solid to solid. The water content at which soil changes it state from liquid to plastic is called **Liquid Limit** (w_L), from plastic to semi-solid is called **Plastic Limit** (w_P) and from semi-solid to solid is called **Shrinkage Limit** (w_S). Figure 3.3 shows the variation of volume of soil with water content at different states or stages of the soil. As the water content decreases the volume of soil also decreases upto shrinkage limit and after that no volume change is observed due to the reduction of water content. This is due to the fact that upto shrinkage limit, the soil is fully saturated and beyond shrinkage limit the degree of saturation is 100%-0%. Thus, before shrinkage limit as the water content is reduced an equal amount of volume reduction takes place in the soil. In liquid state, in terms of consistency soil is liquid (does not have any shear strength), in plastic state, the state of soil varies from soft to stiff and in semi-solid and solid states it varies from stiff to very hard. In plastic stage, soil can be moulded to any shapes due to its plasticity. In semi-solid state, soil becomes brittle (does not have plasticity) and the brittleness further increases as it goes to solid state. The strength of the soil increases as the amount of water reduces from it.

Based on the various limits, some index properties of the soil are determined. Plasticity index (I_P) of the soil is difference between liquid limit and plastic limit. Thus,

\[{I_P}={w_L} - {w_P}\] (3.2)

Plasticity index is used to indicate the degree of plasticity of the soil. If I_P = 0, the soil is non-plastic (sand). For low plastic soil, I_P <7, for medium plastic soil, I_P value lies in between 7 to 17 and for high plastic soil, I_P > 17 (Ranjan and Rao, 2000). When plastic limit is equal to or greater than liquid limit, I_p is considered as zero.

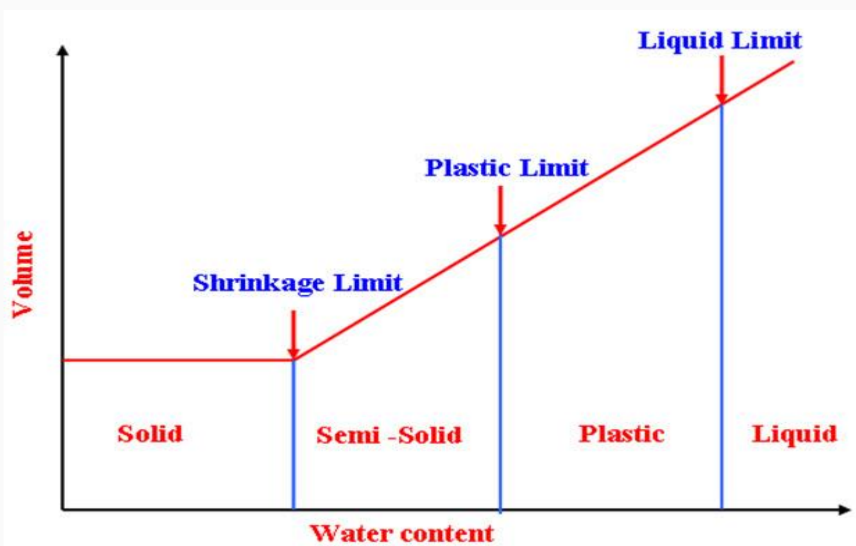


Figure 3.3. Atterberg limits

Consistency Index (I_C) is the ratio of the difference between the liquid limit and natural water content of the soil to its plasticity index. Thus,

$$ {I_C}={{{w_L} - {w_n}} \over {{I_P}}} \qquad (3.3) $$

If $I_C = 0$, *i.e* $w_n = w_L$, If $I_C = 1$, *i.e* $w_n = w_P$, if $I_C > 1$, the soil is in semi-solid or solid state and if $I_C < 0$, the soil is in liquid state.

Liquidity Index (I_L) is the ratio of the difference between the natural water content and plastic limit of the soil to its plasticity index. Thus,

$$ {I_L}={{{w_n} - {w_P}} \over {{I_P}}} \qquad (3.4) $$

From Eq. 3.3 and Eq. 3.4, one can say, $I_L = 1 - I_C$. If $I_L = 0$, *i.e* $w_n = w_P$, If $I_L = 1$, *i.e* $w_n = w_L$, if $I_L > 1$, the soil is in liquid state and if $I_L < 0$, the soil is in semi-solid or solid state. In plastic state, both I_L and I_C values lie in between 0 to 1. These index properties are very important aggregate properties for clayey soil.

Unconfined Compressive Strength (q_u) [load per unit area at which an unconfined cylindrical specimen of standard dimensions (as diameter 38mm and length 76mm) fails in simple compression] is a very important aggregate property of clay to describe its consistency. It is twice the undrained shear strength of clay. For very soft clay, q_u < 25 kN/m^2; for soft clay q_u value lies in between 25 – 50 kN/m^2; for medium clay q_u value lies in between 50 – 100 kN/m^2; for stiff clay q_u value lies in between 100 – 200 kN/m^2; for very stiff clay q_u value lies in between 200 – 400 kN/m^2; and for hard clay q_u > 400 kN/m^2 (Ranjan and Rao, 2000).

Relative Density (D_r or I_D) is the most important aggregate property of coarse-grained soil to describe its degree of denseness or looseness. Relative density is defined as:

$$ {I_D}={{{e_{\max }}-{e_{nat}}} \over {{e_{\max }}-{e_{\min }}}}\times 100\% \qquad (3.5) $$

where e_{max}, e_{min} and e_{nat} are the void ratio of the cohesionless soil in its loosest (maximum void ratio), densest (minimum void ratio) and natural (natural void ratio) state, respectively. In terms of unit weight of the soil the Eq.(3.5) can be written as:

$$ {I_D}=\left({{{\gamma _{d\max }}}\over{{\gamma _d}}}\right)\left({{{\gamma _d}-{\gamma _{d\min}}}\over{{\gamma_{d\max}}-{\gamma _{d\min }}}}\right) \times 100\% \qquad (3.6) $$

where g_{dmin}, g_{dmax} and g_d are the dry unit weight of cohesionless soil in its in its loosest (minimum dry unit weight), densest (maximum unit weight) and natural (dry unit weight at natural condition) state, respectively. For very loose granular soil I_D < 15%; for loose granular soil I_D lies in between 15 to 35%; for medium granular soil I_D lies in between 35 to 65%; for dense granular soil I_D lies in between 65 to 85%; for very dense granular soil I_D > 85% (Ranjan and Rao, 2000).

LESSON 4. Classification of Soil

4.1 Classification Based on Grain size

In general, soils may be classified as coarse-grained (cohesionless) and fine-grained (cohesive) soil. The both coarse-grained and fine-grained soils can be further sub-divided based on their grain size. According to Indian Standard Soil Classification System (ISSCS), soil with particle size > 300 mm is called **Boulder**. Soil with particle size in between 300 mm to 80 mm is called **Cobble.** The soil is called **Gravel** if particle size is in between 80 mm to 4.75 mm. The gravel is sub-divided as **Coarse Gravel** (80 mm to 20 mm) and **Fine Gravel** (20 mm to 4.75mm). Soil with particle size in between 4.75 mm to 0.075 mm is called **Sand**. The sand is sub-divided as **Coarse Sand** (4.75 mm to 2.0 mm), **Medium Sand** (2.0 mm to 0.425 mm) and **Fine Sand** (0.425 mm to 0.075 mm). Soil with particle size less than 0.075 mm is called **Fine-Grained** soil **(Silt or Clay)** and soil with particle size in between 80 mm to 0.075 mm is called **Coarse-Grained** soil. However, soil with particle size less than 0.002 mm is called **Clay** and soil with particle size in between 0.075mm to 0.002 mm is called **Silt**.

4.2 The Unified Soil Classification System

The Unified Soil Classification System (USCS) originally developed by Casagrande (1948). According to this system, the symbols of the various soils are as: Gravel (G), Sand (S), Silt or Silty (M), Clay or Clayey (C), Organic (O), Peat (Pt), Well graded (W), Poorly graded (P).

The soil is called coarse-grained soil if 50% or more soil is retained on the 0.075mm sieve. If the 50% or more of the coarse fraction is retained on the 4.75mm sieve, the soil is called Gravel. On the other hand if 50% or more of the coarse fraction is passed through the 4.75mm sieve, the soil is called Sand. Various types of coarse-grained soils are classified as: GW (Well graded Gravel), GP (Poorly graded Gravel), SW (Well graded Sand), SP (Poorly graded Sand), SM (Silty Sand), GM (Silty Gravel), SC (Clayey Sand), and GC (Clayey Gravel). In case of well graded gravels and well graded sands, less than 5% soils pass 75m sieve. However, In case of poorly graded gravels and poorly graded sands very little or no fines are present.

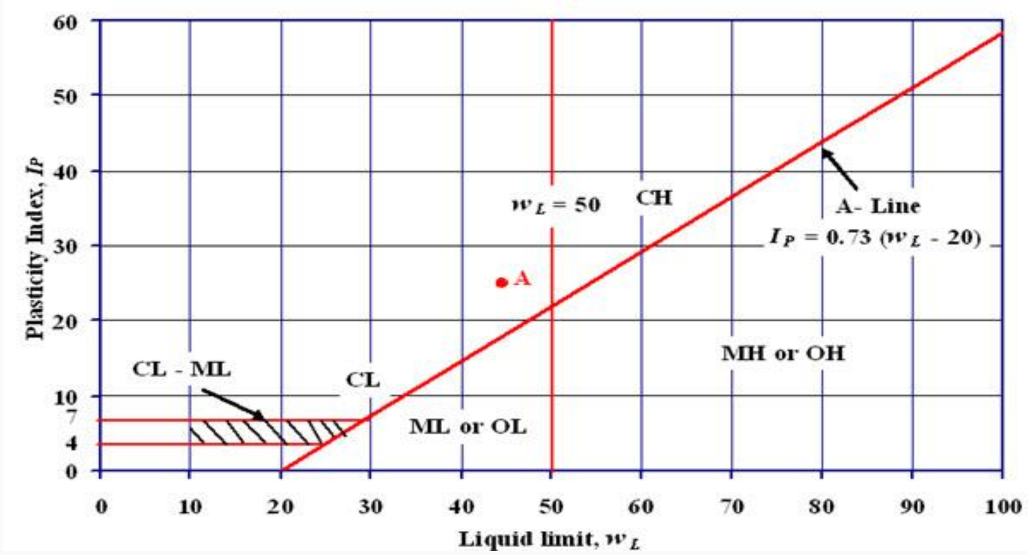


Fig. 4.1. Plasticity chart as per Unified Soil Classification System (USCS).

The soil is called fine-grained soil if 50% or more soil is passed through 0.075 mm sieve. The fine-grained soils are classified based on plasticity chart (as shown in Figure 4.1). The soil has low plasticity (CL: Clay with low plasticity, ML: Silt with low plasticity) if the liquid limit of the soil is less than 50% and if the liquid limit of the soil is greater than 50% the soil has high plasticity (CH: Clay with high plasticity, MH: Silt with high plasticity). However, more than one group can be termed as boundary soils (like GW-GM: Well graded gravel mixed with silt).

4.3 Aashto Soil Classification System

According to the AASHTO soil classification system, the soils are classified based on the Group Index (*GI*) value which can be calculated as:

$$GI = 0.2\,a + 0.005\,ac + 0.01\,bd$$

where

a is that part of the percent passing through the 75 m (0.075 mm) sieve greater than 35 and not exceeding 75, expressed as a positive whole number (range 1 to 40).

b is that part of the percent passing through the 75 m (0.075 mm) sieve greater than 15 and not exceeding 55, expressed as a positive whole number (range 1 to 40).

c is that part of liquid limit greater than 40 and not exceeding 60, expressed as a positive whole number (range 1 to 20).

d is that part of plasticity index greater than 10 and not exceeding 30, expressed as a positive whole number (range 1 to 20).

The group index should be rounded off to the nearest whole number. If the calculated group index value is negative, then it is taken as zero. A group index value equal to zero indicates a good subgrade material, whereas group index value equal to or greater than 20 indicates a very poor subgrade material.

4.4 Indian Standard Soil Classification System (ISSCS)

According to this system, the symbols of the various soils are as: Gravel (G), Sand (S), Silt or Silty (M), Clay or Clayey (C), Organic (O), Peat (Pt), Well graded (W), Poorly graded (P). To classify the fine-grained soil, plasticity chart (as shown in Figure 2) is used. The difference between the plasticity charts used for Unified Soil Classification System (USCS) and Indian Standard Soil Classification System (ISSCS) is that in USCS, the soil is classified as High Plasticity (if liquid limit >50%) or Low Plasticity (if liquid limit < 50%) soil, but in ISSCS, the soil is classified as High Plasticity (if liquid limit >50%) or Intermediate Plasticity (if liquid limit is in between 35% to 50%) or Low Plasticity (if liquid limit < 35%). For example, if a soil sample has liquid limit (w_L) 45% and plasticity index (I_P) 25, according to the Unified Soil Classification System (USCS) the point is above 'A' line (point A in Figure 4.1) and it is classified as CL. However, according to Indian Standard Soil Classification System (ISSCS) the point is also above 'A' line (point B in Figure 4.2), but it is classified as CI.

Figure 4.3 shows the flow chart to classify a soil according to the Indian Standard Soil Classification System. Figure 4.4 and Figure 4.5 show the classification of coarse-grained and fine-grained soil, respectively as per Indian Standard Soil Classification System.

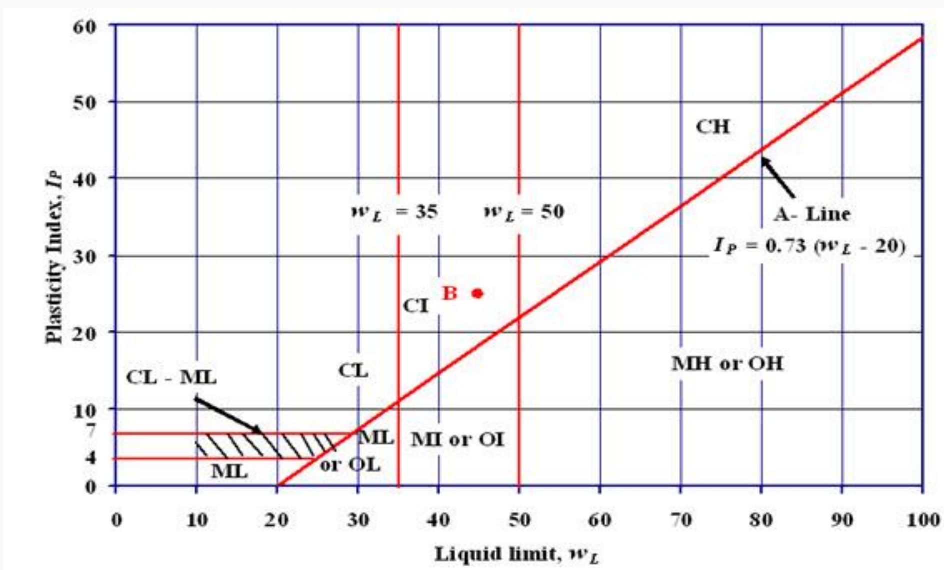


Fig. 4.2. Plasticity chart as per Indian Standard Soil Classification System (ISSCS).

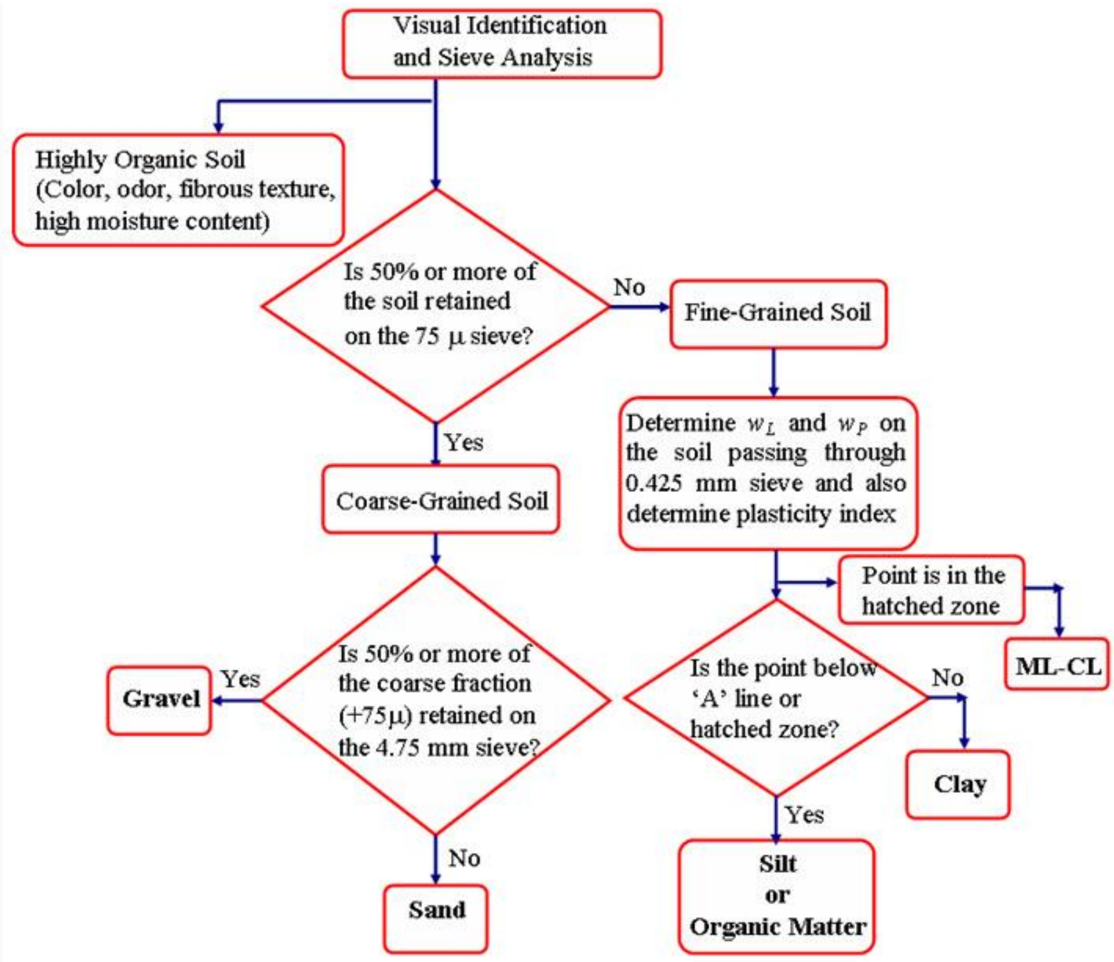
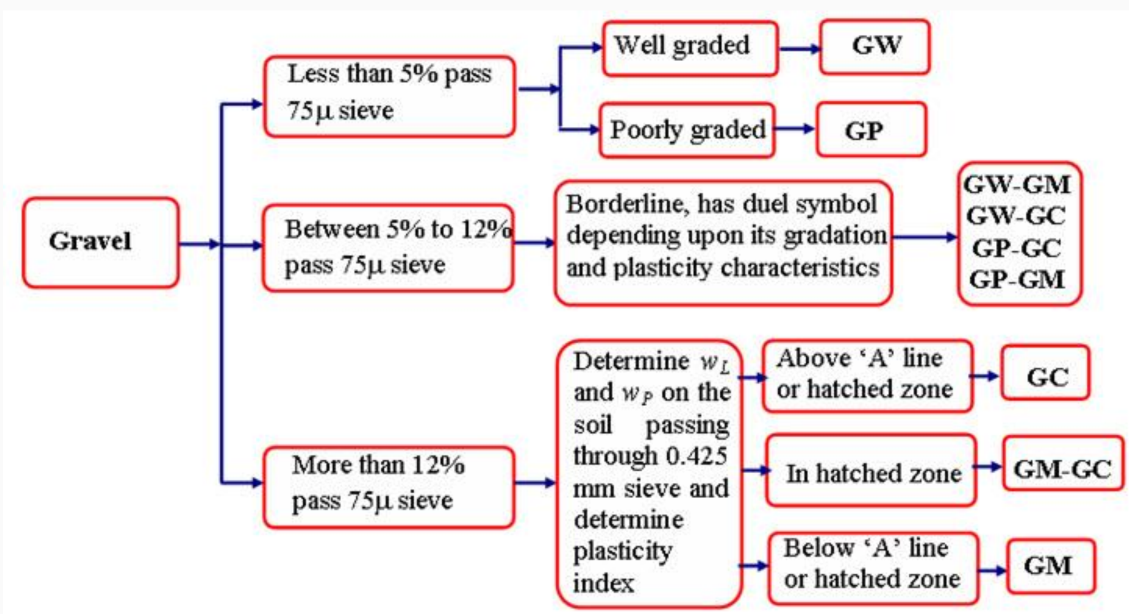
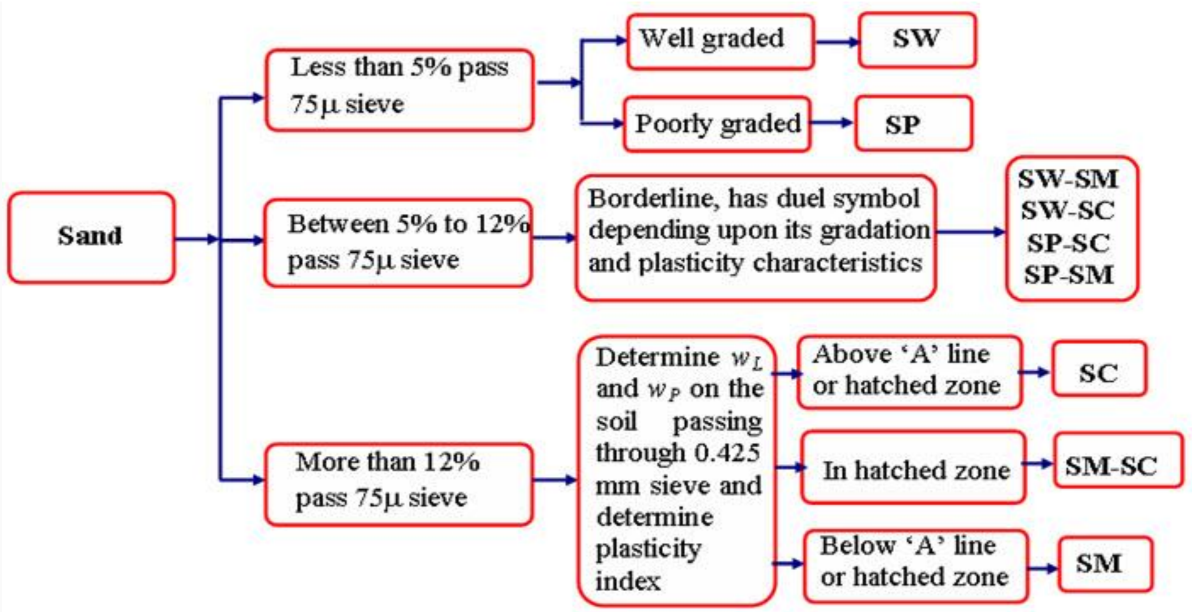


Fig. 4.3. Flow chart to classify soil (as per ISSCS).



(a)



(b)

Fig. 4.4. Classification of coarse-grained soil: (a) Gravel (b) Sand (as per ISSCS).

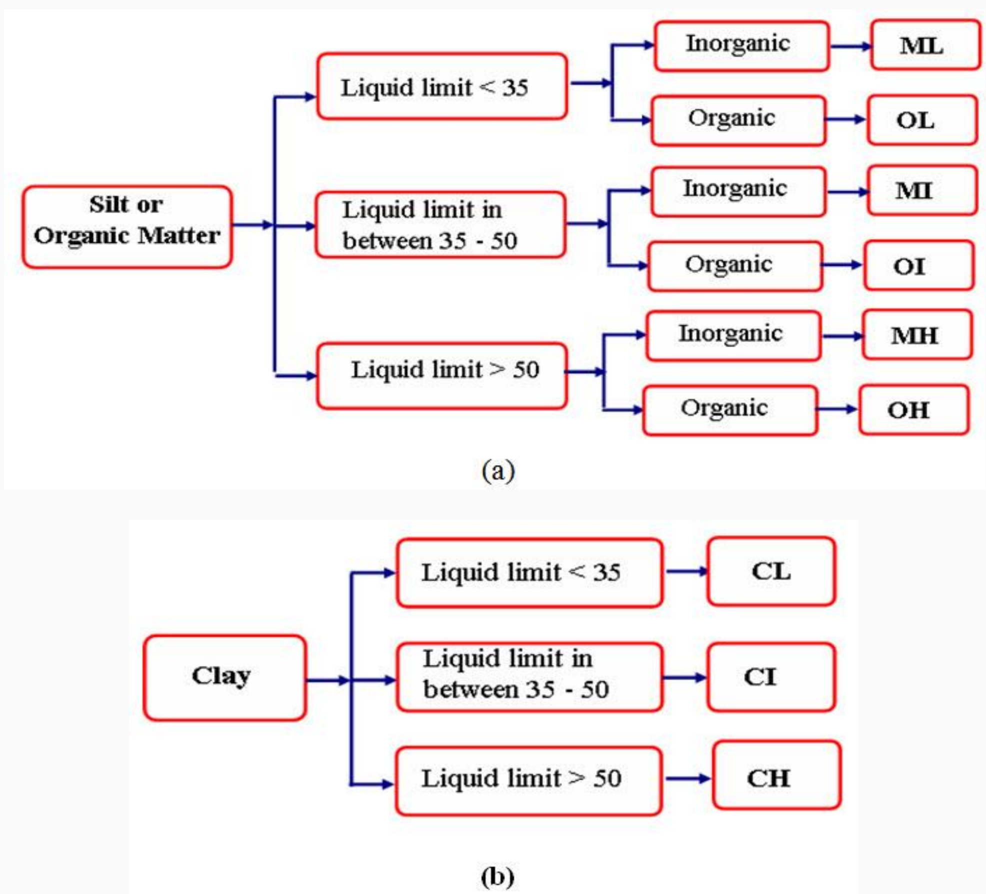


Fig. 4.5. Classification of fine-grained soil: (a) Silt or Organic matter (b) Clay (as per ISSCS).

MODULE 2. Stress and Strength

LESSON 5. Stress in Soil

5.1 Concept of Effective Stress

The effective stress concept is very useful in soil mechanics to calculate shear strength, settlement, earth pressure. Figure 1 shows a soil sample whose top portion is dry or above water table (W.T) and bottom portion is fully saturated or below the water table. A point at any depth within the soil mass (say point 'A'), the soil is subjected to stress due to the soil pressure above that level. The total stress (σ) acting at the point 'A' is:

$$\sigma={h_1}{\gamma _d} + {h_2}{\gamma _{sat}} \qquad (5.1)$$

where h_1 and h_2 are the thickness of the dry and saturated zone of the soil mass, respectively. γ_d and γ_{sat} are the dry and saturated unit weight of the soil. The unit of unit weight is 'kN/m^3' and unit of thickness of the soil layer is 'm'. Thus, unit of stress is 'kN/m^2'. The stress is defined as load per unit area. The stress at any point within the soil mass can be calculated by multiplying unit weight of the soil and thickness of the soil layer. The proper unit weight of the soil has to be used depending on the position of water table or nature of soil (*i.e* dry, partially saturated, fully saturated or submerged).

The total stress within the completely saturated soil has two parts: i) stress due to the pore water and called **pore water pressure** (as the pores of the soil solids are filled with water) and ii) stress due to the soil skeleton (or soil particles) and called **effective stress**. The pore water pressure (u) at a depth of h_2 below ground water table (point 'A' in Figure 5.1) is determined by multiplying the height of water above the point and unit weight of water. Thus:

$$u={h_2}{\gamma _w} \qquad (5.2)$$

where γ_w is the unit weight of the water (generally taken as 9.81 kN/m^3 or 10 kN/m^3). The effective stress ($\sigma '$) is equal to total stress minus pore water pressure. Thus,

$$\sigma=\sigma ' + u \qquad (5.3)$$

and

$$\sigma '=\sigma-u={h_1}{\gamma _d}+{h_2}{\gamma _{sat}}-{h_2}{\gamma _w}={h_1}{\gamma _d}+{h_2}({\gamma _{sat}}-{\gamma _w})= {h_1}{\gamma _d} + {h_2}\gamma ' \qquad (5.4)$$

where $\gamma '$ is the submerged unit weight of soil. Thus, for dry soil total stress and effective stress both are same (as for dry soil u =0) and can be determined by multiplying the thickness of soil layer and dry unit weight (γ_d) of the soil. In case of fully saturated soil, total stress can be determined by multiplying the thickness of soil layer and saturated unit weight (γ_{sat}) of the soil, whereas effective stress can be determined by multiplying the thickness of soil layer and submerged unit weight ($\gamma '$) of the soil. In case of partially saturated soil, total stress and effective stress both are same (as for partially saturated soil u =0) and can be determined by multiplying the thickness of soil layer and bulk unit weight (γ_t) of the soil. Generally soil below water table is considered as fully saturated soil and soil above water table is dry or partially saturated soil.

In case of partially saturated soil, soil pores are filled with water and air both. Thus, total stress has three component i) stress due to pore water and is called pore water pressure (u_w) ii) stress due to pore air and is called pore air pressure (u_a) and iii) stress due to the soil skeleton and called effective stress (s¢). The effective stress expression for partially saturated soil is:

$$\sigma '=\sigma-{u_a}+\chi ({u_a}-{u_w}) \qquad (5.5)$$

where χ is a constant depends on the unit cross-sectional area of the soil occupies by the water. The value of χ varies in between 0 to 1. For dry soil, χ=0 and for completely saturated soil, χ=1. However, in general, for calculation purpose, the pore water pressure and pore air pressure of the dry and partially saturated soil are neglected.

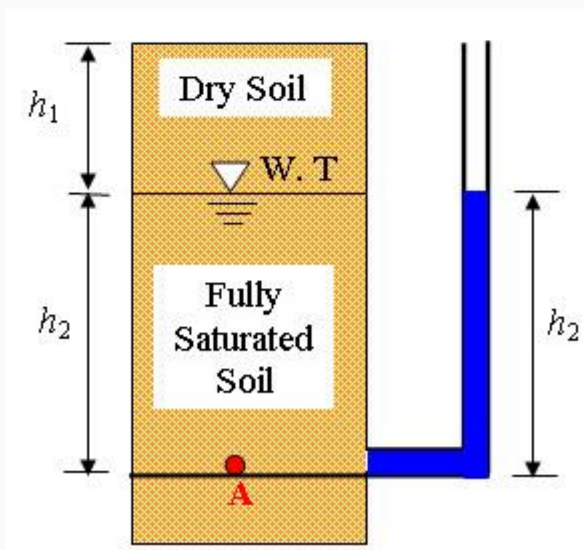


Fig. 5.1. Pore water pressure measurement by stand pipe.

The total stress can be measured by pressure cell or pressure sensor and the pore water pressure can be measured by standpipe or piezometer. However, the effective stress can not be measured. As shown in Figure 1, if a stand pipe is inserted at any level within the soil mass below water table, the water level can be determined by observing the height upto which the water raises in the stand pipe. If effective stress increases the soil particles become denser. Thus, the void ratio and compressibility of the soil decreases which caused increase in the shearing resistance of the soil. An equal increase in the total and pore water pressure keeps effective stress constant. Thus, there will be very little (or no effect) on the soil particles.

5.2. Stress calculation in Soil

The following example shows the stress calculation at different level within the soil.

Problem 1: Calculate total, effective stress and pore water pressure at different level of the soil as shown in Figure 5.2.

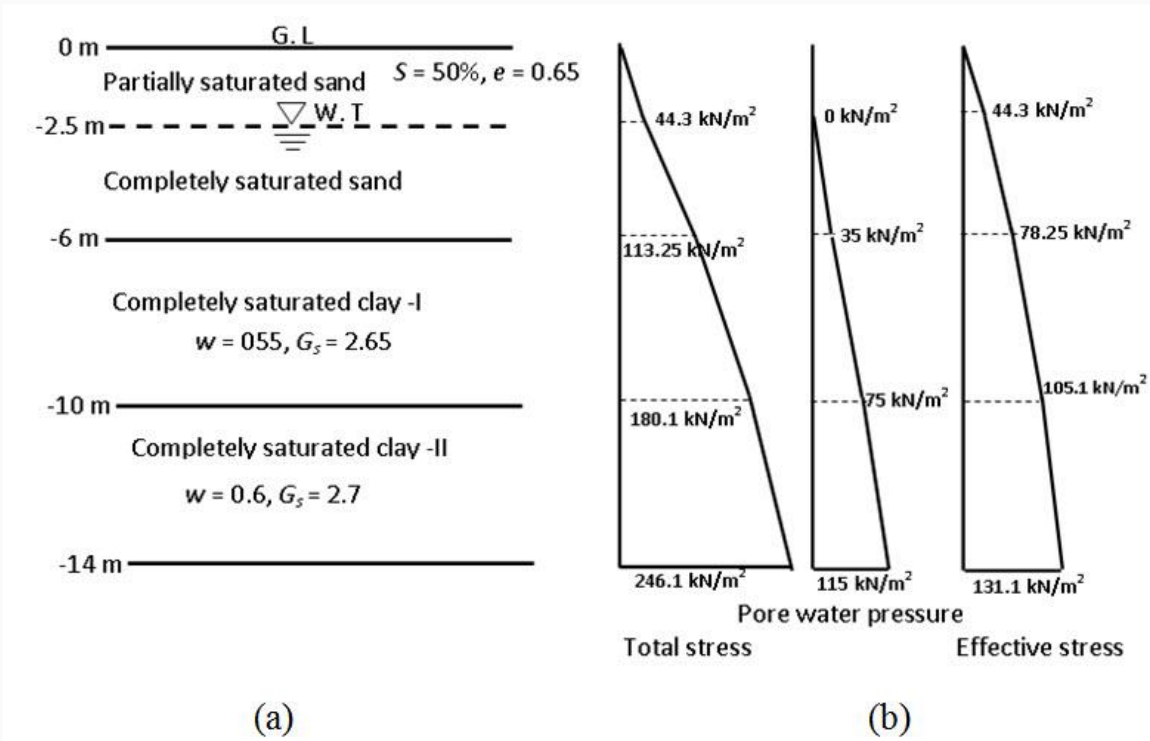


Fig. 5.2. (a) Soil profile (b) Stress diagrams

Solution:

The bulk unit weight of the partially saturated sand

$${\gamma _{t(sand)}}={{{G_s} + Se} \over {1 + e}}{\gamma _w}={{2.6 + 0.5 \times 0.65} \over {1 + 0.65}} \times 10=17.72\;kN/{m^3}$$

The unit weight of the water is taken as 10 kN/m^3. The saturated unit weight of sand is

$${\gamma _{sat(sand)}}={{{G_s} + e} \over {1 + e}}{\gamma _w}={{2.6 + 0.65} \over {1 + 0.65}} \times 10=19.7\;kN/{m^3}$$

The saturated unit weight of the clay-I is.

$${\gamma _{sat(clay - I)}}={{{G_s} + e} \over {1 + e}}{\gamma _w}={{{G_s} + {G_s}w} \over {1 + {G_s}w}}{\gamma _w}={{2.65 + 2.65 \times 0.55} \over {1 + 2.65 \times 0.55}} \times 10=16.71\;kN/{m^3}$$

[$Se = G_s w$ and for fully saturated soil S =1]

The saturated unit weight of the clay-II is

$${\gamma _{sat(clay - II)}}={{{G_s} + e} \over {1 + e}}{\gamma _w}={{{G_s} + {G_s}w} \over {1 + {G_s}w}}{\gamma _w}={{2.7 + 2.7 \times 0.6} \over {1 + 2.7 \times 0.6}} \times 10=16.49\;kN/{m^3}$$

<u>At elevation -2.5 m</u>

Total stress (σ) = 17.72 ´ 2.5 = 44.3 kN/m^2

Pore water pressure (u) = 0

Effective stress () = 44.3 - 0 = 44.3 kN/m^2

<u>At elevation -6 m</u>

Total stress (σ) = 17.72 ´ 2.5 + 19.7 ´ 3.5 = 113.25 kN/m^2

Pore water pressure (u) = 10 ´ 3.5 = 35 kN/m^2

Effective stress ($\sigma '$) = 113.25 - 35 = 78.25 kN/m^2

<u>At elevation -10 m</u>

Total stress (σ) = 17.72 ´ 2.5 + 19.7 ´ 3.5 + 16.71 ´ 4 = 180.1 kN/m^2

Pore water pressure (u) = 10 ´ 7.5 = 75 kN/m^2

Effective stress ($\sigma '$) = 180.1 - 75 = 105.1 kN/m^2

<u>At elevation -14 m</u>

Total stress (σ) = 17.72 ´ 2.5 + 19.7 ´ 3.5 + 16.71 ´ 4 + 16.49 ´ 4 = 246.1 kN/m^2

Pore water pressure (u) = 10 ´ 11.5 = 115 kN/m^2

Effective stress ($\sigma '$) = 246.1 - 115 = 131.1 kN/m^2

LESSON 6. Stress in Soil due to applied load

6.1 Concept of Boussinesq's Analysis

In the lesson 5, the stress calculation within the soil due to the overburden pressure (pressure due to the soil above any depth is called overburden pressure) has been discussed. In this lesson, the stress calculation within soil due to the applied load will be discussed. Boussinesq (1885) proposed equations to determine stresses in subgrade materials due to the applied loads. The subgrade material has been considered as weightless, unstressed, semi-infinite, elastic, homogeneous, and isotropic. The concentration load (*Q*) has been applied normally to the upper surface of the material (as shown in Figure 6.1).

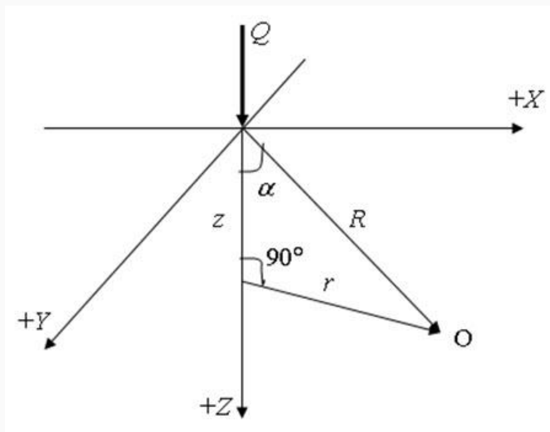


Fig. 6.1. Boussinesq's stresses.

The vertical stress (σ_z) at a point 'O' within the soil with radial distance '*R*' from the point of application of the concentration load *Q* can be determined as:

$${\sigma _Z}={{3Q} \over {2\pi {z^2}}}{\cos ^5}\alpha \qquad (6.1)$$

The shear stress (t_{rz}) at a point 'O' within the soil with radial distance '*R*' from the point of application of the concentration load *Q* can be determined as:

$${\tau _{rz}}={{3Q} \over {2\pi {z^2}}}{\cos ^4}\alpha \sin \alpha \qquad (6.2)$$

where *z* and *r* are the depth and horizontal distance of the point 'O' from the point of application of the concentration load *Q*, respectively.

Now,

$$\cos \alpha=\frac{z}{R}=\frac{z}{{{{\left({{r^2}+{z^2}}\right)}^{\frac{1}{2}}}}} \qquad (6.3)$$

Thus, Eq. (6.1) can be written as:

$${\sigma _Z}=\frac{{3Q}}{{2\pi }}\frac{{{z^3}}}{{{R^5}}}=\frac{{3Q}}{{2\pi }}\frac{{{z^3}}}{{{{\left({{r^2}+{z^2}}\right)}^{\frac{5}{2}}}}}=\frac{{3Q}}{{2\pi {z^2}}}{\left[{\frac{1}{{1+{{\left({\frac{r}{z}}\right)}^2}}}}\right]^{\frac{5}{2}}} \quad (6.4)$$

The Eq. (6.4) can be written as:

$${\sigma _Z}={Q \over {{z^2}}}{K_B} \quad (6.5)$$

where

$${K_B}=\frac{3}{{2\pi }}{\left[{\frac{1}{{1+{{\left({\frac{r}{z}}\right)}^2}}}}\right]^{\frac{5}{2}}} \quad (6.6)$$

The K_B is called Boussinesq influence factor which is a function of (r/z) ratio. If r = 0, K_B = 0.4775. Thus, vertical stress just below the point of application of load 'Q' on the axis (at any depth) can be expressed as:

$${\sigma _Z}=0.4775{Q \over {{z^2}}} \quad (6.7)$$

The shear stress can be expressed as:

$${\tau _{rz}} =\frac{{3Q}}{{2\pi }}\frac{{r{z^2}}}{{{{\left({{r^2}+{z^2}}\right)}^{\frac{5}{2}}}}} \quad (6.8)$$

6.2. Concept of Westergaard's Analysis

Westergaard (1938) proposed vertical stress equation within soil due to a point load by assuming zero Poisson's ratio value of the soil to prevent any lateral strain and allowing only vertical movement of the soil. The vertical stress can be calculated as:

$${\sigma _Z}=\frac{Q}{{{z^2}}}\frac{1}{{\pi{{\left[{1+2{{\left({\frac{r}{2}}\right)}^2}}\right]}^{\frac{3}{2}}}}} \quad (6.9)$$

or $${\sigma _Z}={Q \over {{z^2}}}{K_w} \quad (6.10)$$

where K_w is called Westergaard influence factor can be expressed as:

$${K_w}=\frac{1}{{\pi{{\left[{1+2{{\left({\frac{r}{2}}\right)}^2}}\right]}^{\frac{3}{2}}}}} \quad (6.11)$$

6.3 Approximate Stress Distribution method

In this method a 2:1 distribution of stress is assumed (as shown in Figure 6.2). If a rectangular area of B x L is loaded by uniformly distributed load q, the vertical stress a depth of z below the loaded area can be determined as:

$${\sigma _z}={{q(B \times L)} \over {(B + z) \times (L + z)}} \quad (6.12)$$

The maximum stress is equal to average stress at depth equal to 2B. The maximum stress is greater than average stress if z < 2B and the maximum stress is less than average stress if z > 2B.

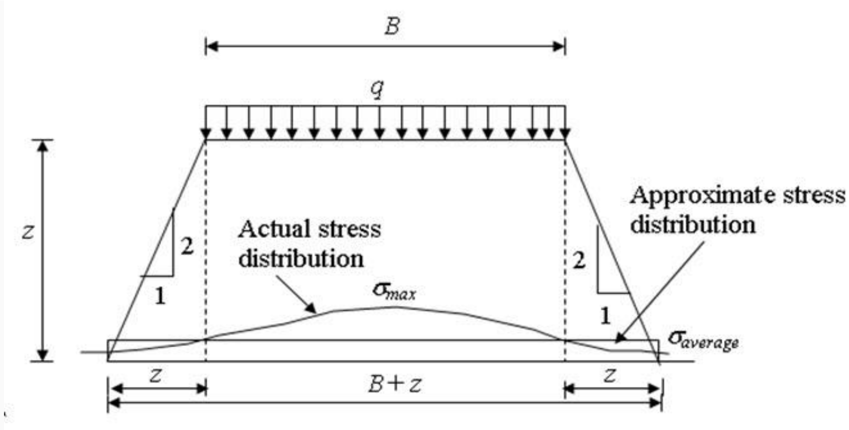


Fig. 6.2. Approximate stress calculation method.

Problem 1

Draw the vertical stress distribution due to a concentrated load (*Q*) of 50 kN acting on the surface of a soil for the following condition by using Boussinesq's equation:

i) Variation of stress with depth for horizontal distance, *r* = 0 (points just vertically below the load) and *r* = 1m.

ii) Variation of stress with horizontal distance (either side of the load) for depth, *z* = 2m and *z* = 3m.

Solution

Equation (6.4) is used to calculate the vertical stress at any depth at any horizontal distance from the point of application of the concentration load. Figure 6.3 shows the variation of vertical stress with depth at horizontal distance, *r* = 0 and *r* = 1m. Keeping *r* value constant, the stress is calculated for different *z* values upto *z* = 8m. For example, at (*r* = 1m, *z* = 2m), the vertical stress is 4.22 kN/m^2.

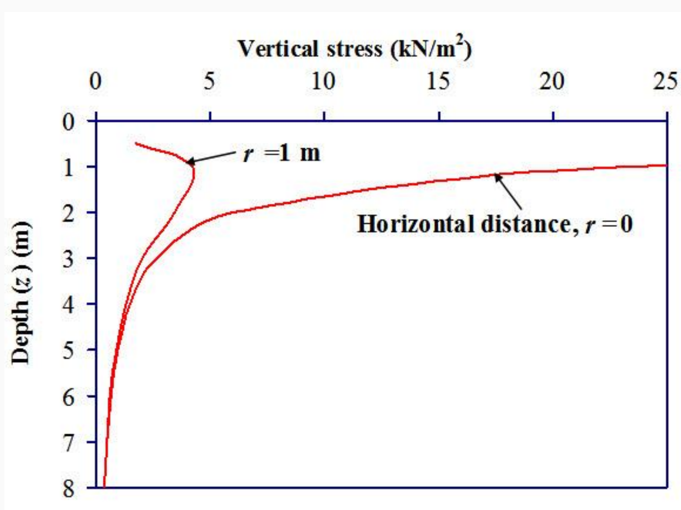


Fig. 6.3. Variation of vertical stress with depth.

Figure 6.4 shows variation of vertical stress with horizontal distance at depth z = 2m and z = 3m. It is observed that as the depth increases vertical stress decreases at the centre line. Same vertical stress is observed at different location with respect to depth and horizontal distance. For example, same vertical stress (2 kN/m^2) is observed at (r = ± 1.5m, z = 2m) as well as at (r = ± 1 m, z = 3m). The line joining points with same vertical stress below ground level is called

Isobar.

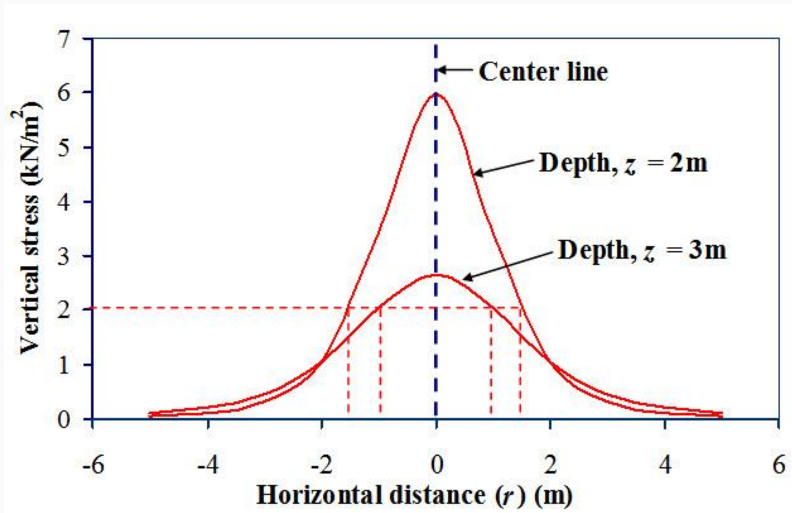


Fig.6.4. Variation of vertical stress with horizontal distance.

LESSON 7. Stress Under Loaded Area

7.1. Vertical stress under loaded area

Line Load: The vertical stress (σ_z) at point O (as shown in Figure 7.1) due to line load of intensity q/unit length can be determined as :

$${\sigma _Z}=\frac{q}{z}\frac{2}{\pi}{\left[{\frac{1}{{1+{{\left({\frac{x}{z}}\right)}^2}}}}\right]^2} \qquad (7.1)$$

where z is the depth of the point O and x is the horizontal distance of the point O along X-axis.

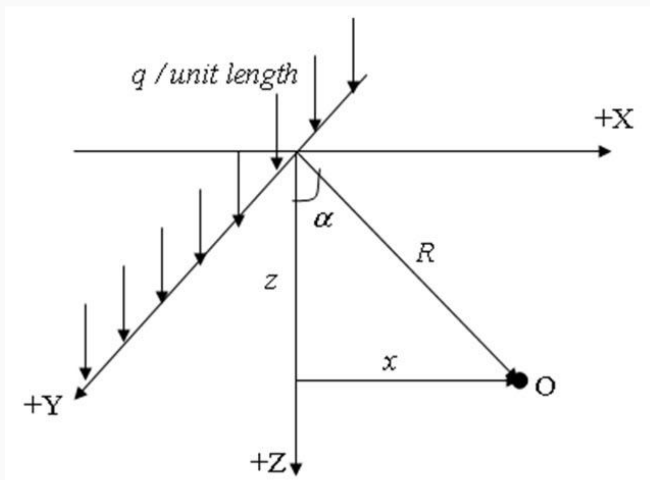


Fig. 7.1. Vertical stress due to line load.

Strip Load: The vertical stress (σ_z) at point O (as shown in Figure 7.2) due to uniform strip load of intensity q/unit length can be determined as:

$${\sigma _Z}={q \over \pi }(\alpha+\sin \alpha\cos 2\beta) \qquad (7.2)$$

where width of the load region is B. If b =0, *i.e.* vertical stress directly below the centre of the strip load can be determined as:

$${\sigma _Z}={q \over \pi }(\alpha+ \sin \alpha) \qquad (7.3)$$

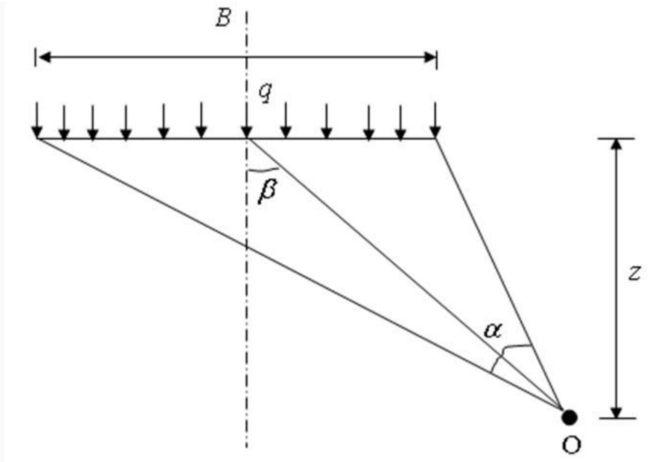


Fig. 7.2. Vertical stress due to strip load.

Uniformly loaded rectangular area: The vertical stress (σ_z) at any point O below the **corner** of a uniformly loaded rectangular area of intensity q (as shown in Figure 7.3) can be determined as:

$$\sigma_z = q\frac{1}{4\pi}\left[\frac{2mn\,(m^2 + n^2 + 1)^{\frac{1}{2}}}{m^2 + n^2 + 1 + m^2 n^2} \times \frac{m^2 + n^2 + 2}{m^2 + n^2 + 1} + \tan^{-1}\frac{2mn\,(m^2 + n^2 + 1)^{\frac{1}{2}}}{m^2 + n^2 + 1 - m^2 n^2} \right]$$

where $m = L/z$ and $n = B/z$, L is the length of the loaded area and B is the width of the loaded area. The vertical stress at any point below the loaded area can be determined by dividing the rectangular loaded area into number of parts such that there will be a common corner point (the point of interest) for all the rectangles or squares.

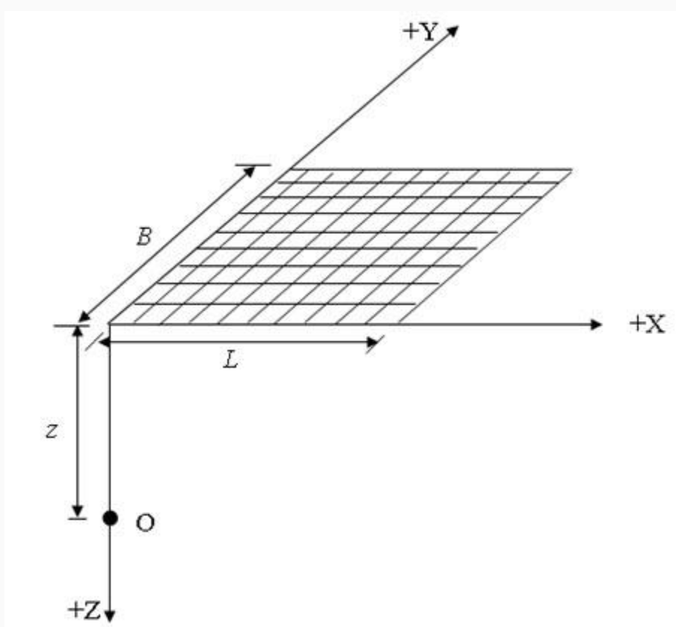


Fig. 7.3. Vertical stress under corner of a uniformly loaded rectangular area.

Uniformly loaded circular area: The vertical stress (σ_z) at any point O directly below the center of a uniformly loaded circular area of intensity *q* (as shown in Figure 7.4) can be determined as:

$$\sigma_Z=q\left[1-\left(\frac{1}{1+\left(\frac{R}{z}\right)^2}\right)^{\frac{3}{2}}\right] \qquad (7.5)$$

where *R* is the radius of the circular loaded area and *z* is the depth of the point O.

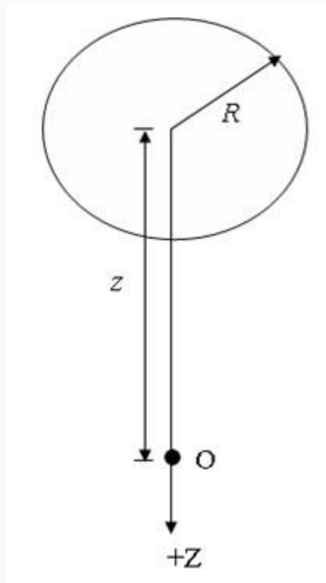


Fig. 7.4. Vertical stress under centre of a uniformly loaded circular area.

LESSON 8. Newmark's Influence Chart

8.1 Use of Newmark's Influence Chart

Newmark (1942) developed influence chart to determine the vertical stress due to loaded of area of any shape, irregular geometry at any point below the loaded area. A uniformly loaded circular area of radius r_1 is divided into 20 divisions (say the first circle in Figure 8.1). Now if q is the intensity of loading then each small unit of the first circle will produce a vertical stress equal to ${{\sigma_z}\over{20}}$ at any depth of z below the center of the loaded area (as first circle is divided into 20 small divisions). Now from Eq. 7.5 of lesson 7 one can write that at any depth z vertical stress due to each small unit of the circle as:

$$\frac{\sigma_Z}{20}=\frac{q}{20}\left[1-\left(\frac{1}{1+\left(\frac{r_1}{z}\right)^2}\right)^{\frac{3}{2}}\right] \qquad (8.1)$$

Let the right hand side of the Eq. 8.1 is equal to an arbitrary fixed value which is called influence value (say 0.005q). Thus,

$$\frac{q}{20}\left[1-\left(\frac{1}{1+\left(\frac{r_1}{z}\right)^2}\right)^{\frac{3}{2}}\right]=0.005q \qquad (8.2)$$

The influence value of the chart is equal to 0.005 and each small unit is producing a vertical stress equal to 0.005q. After solving Eq. 8.2, one can get ${{r_1}\over z}=0.27$. Thus, the radius of the first circle is 0.27z. In the influence chart (Figure 8.1), AB line representing the value of z (in the figure it is 2.5 cm). Thus, according to the chart shown in the Figure 8.1, any depth is represented by 2.5cm and based on that the scale has to be decided. According to that scale the loaded area has to be drawn for stress calculation. In the Figure 1, the radius of the first circle is 0.27 x 2.5 = 0.675 cm. Similarly, the second concentric circle of radius r_2 is also divided in to 20 divisions. Including first and second circles, there are 40 divisions and again each unit is producing a vertical stress equal to 0.005q at a depth of z below the centre of the loaded area. Therefore, the each small unit in the second circle has two units and each unit will produce a vertical stress equal to 0.005q a depth of z below the centre of the loaded area. Thus, each small unit in the second circle will produce 2 x 0.005q amount of vertical stress a depth of z below the centre of the loaded area. Vertical stress due to each unit of the second circle is:

$$\frac{q}{20}\left[1-\left(\frac{1}{1+\left(\frac{r_2}{z}\right)^2}\right)^{\frac{3}{2}}\right]=2\times 0.005q \qquad (8.3)$$

Again After solving Eq. 8.3, one can get ${{r_2}\over z}=0.40$. Thus, the radius of the second circle is 0.40z. The general expression of the vertical stress produce by each unit of the each circle at a depth of z below the centre of the loaded area can be written as:

$$\frac{q}{20}\left[1-\left(\frac{1}{1+\left(\frac{r_i}{z}\right)^2}\right)^{\frac{3}{2}}\right] = i\times 0.005q \qquad (8.4)$$

where i = 1, 2, ……..9. After solving Eq. 8.4, the radius of the third to ninth circle can be determined as: 0.52z, 0.64z, 0.77z, 0.92z, 1.11z, 1.39z, 1.91z, respectively. The radius of the tenth circle is infinity and cannot be drawn.

Problem 1

A raft foundation of dimension 11m x 6.2m is placed at a depth of 2m below the ground level. Determine the net stress due to the raft foundation (stress due to applied load only) at a depth of 7m below the ground level. Also determine the total stress due to raft (stress due to applied load) and stress due to soil (overburden pressure) at a depth of 7m below the ground level. The raft is subjected to total load of 10000 kN. The unit weight of the soil is 18 kN/m^3. Neglect the pore water pressure (assumed soil is completely dry).

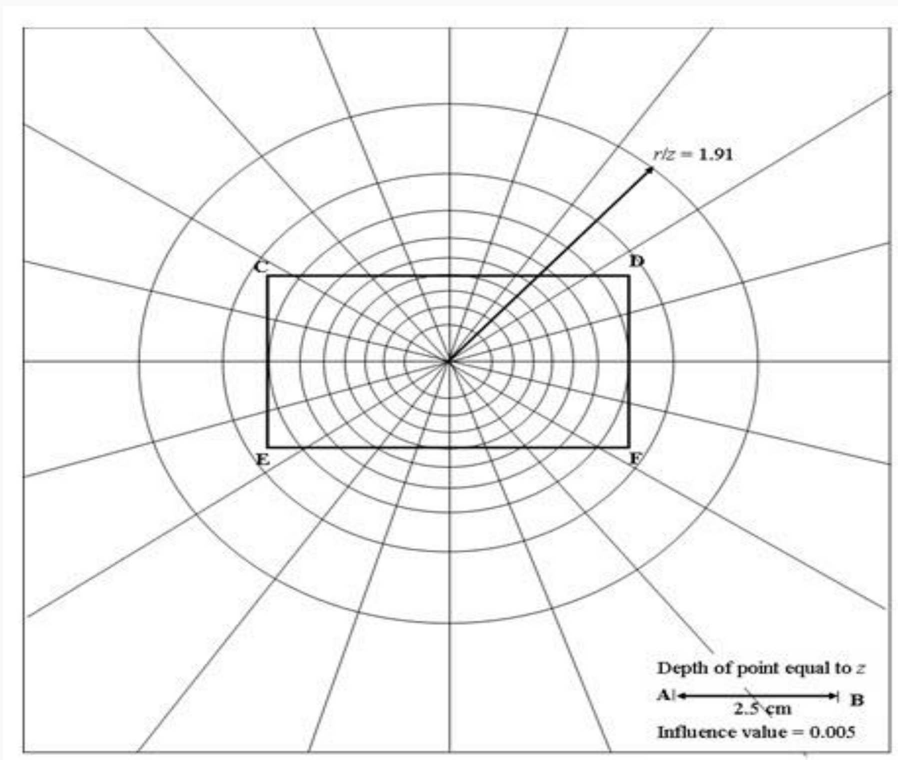


Fig. 8.1. Newmark's influence chart to calculate vertical stress

Solution:

The total stress acting at the base of the raft = $\frac{100000}{11 \times 6.2}=146.6$ kN/m^2. The net stress at the base of raft = 146.6 – 18 x 2 = 110.6 kN/m^2 (net stress means total stress minus the stress due to the soil above the base of the foundation as before the application of load soil was existing there. Thus, stress due to soil has to be deducted to calculate the net stress).

Now, depth of the point below the base of the raft is (z) = 7 – 2 = 5m.

Thus, according to the Newmark's chart (Figure 1), 2.5 cm = 5m. Scale is 1: 200. Now, the raft (CDEF) is drawn with a scale of 1: 200 and placed on the Newmark's chart (as shown in Figure 8.1) such that the centre of the raft is coincided with the centre of the Newmark's chart. This is to be noted that, here the stress below the centre of the raft is determined. Thus, centre of the raft is coincided with the centre of the Newmark's chart. If the vertical stress below the corner or any other point within the raft is to be determined than the corner or the point on interest has to be coincided with the centre of the Newmark's chart. The total number of influence area covered by the raft = 116 (as shown in Figure 8.1). The net stress at a depth of 7 m below the ground level or 5 m below the base of the raft = 110.6 x 0.005 x 116 = 64.2 kN/m^2. Thus, vertical stress due to applied load only is 64.2 kN/m^2.

The vertical stress due to the overburden pressure at a depth of 7 m below the ground level = 18 x 7 = 126 kN/m^2. Thus total vertical stress due to the applied load and overburden pressure at a depth of 7 m below the ground level = (64.2 + 126) = 190.2 kN/m^2.

LESSON 9. Shear Strength of Soil

9.1. Shear Resistance

Sliding between the particles during loading is a major factor for deformation of soil. The resistance that the soil offers during deformation is mainly due to the shear resistance between the particles at their contact points. Figure 9.1 shows the two particles in contact which is similar to the contact between two bodies. The normal force (*N*) is perpendicular the contact surface and shear force (*t*) is tangential force parallel to the contact surface. During sliding between two bodies, the maximum shear force can be written as:

$$\tau_{\max} = \mu N \qquad (9.1)$$

where μ is the coefficient of friction. In case of soil particles, the maximum shear force can be written as:

$$\tau_{\max} = \tan \phi N$$

where ø is the angle in internal friction of soil.

9.2 Mohr Circle

At any stressed point, three mutually perpendicular planes exist on which shear stress is zero. These planes are called principal planes. The normal stresses that act on these planes are called principal stress. The largest principal stress is called major principal stress (σ_1), the lowest principal stress is called minor principal stress (σ_3) and the third stress is called intermediate stress (σ_2). The corresponding planes are called major, minor and intermediate plane, respectively. The critical stress values generally occur on the plane normal to the intermediate plane. Thus, only σ_1 and σ_3 are considered. Figure 9.2 shows an element and direction of σ_1 and σ_3. The major and minor principle planes are also shown. The major and minor principle planes are horizontal and vertical direction, respectively. The normal stress and shear stress at any plane making and angle *q* with horizontal can be determined analytically as:

$$\sigma = {{\sigma_1 + \sigma_3} \over 2} + {{\sigma_1 - \sigma_3} \over 2}\cos 2\theta \qquad (9.3)$$

$$\tau = {{\sigma_1 - \sigma_3} \over 2}\sin 2\theta \qquad (9.4)$$

The stresses can be determined by graphically using Mohr Circle as shown in Figure 9.2. Mohr Circle is drawn in normal (σ) and shear (*t*) axis. The compressive normal stress is considered as positive. The shear stress that produces anti-clockwise couples on the element is considered as positive. The circle is drawn by taking O [$(\sigma_1 + \sigma_3)/2$, 0] as center and $(\sigma_1 - \sigma_3)/2$ as radius (as shown in Figure 9.2). Now from (σ_3, 0) point draw a line parallel to AB plane. The line intersects the Mohr Circle at a point whose coordinates represents the normal and shear stress acting on AB plane [D(σ, *t*)]. The A (σ_3, 0) point is called pole or the origin of plane.

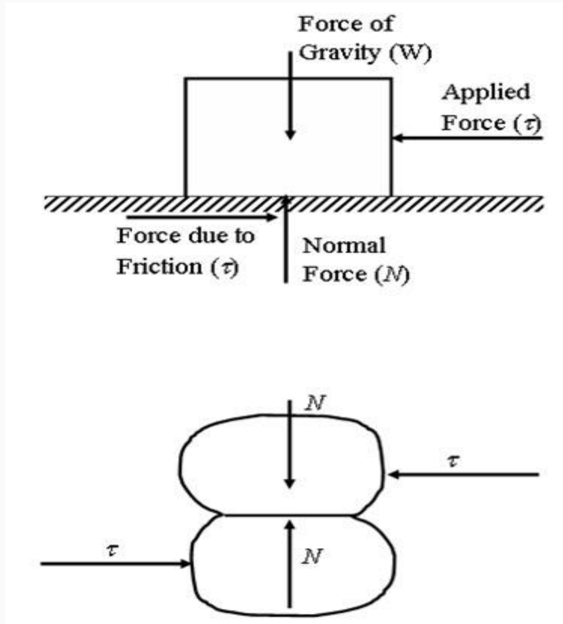


Fig. 9.1. Two particles/bodies in contact.

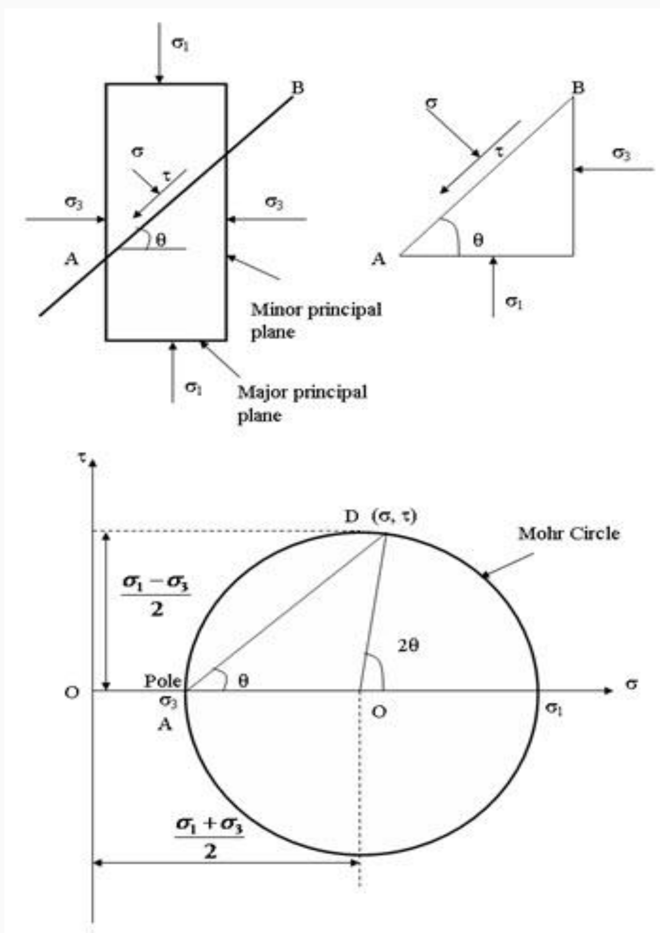


Fig. 9.2. Mohr Circle.

LESSON 10. Mohr-Coulomb Failure Theory

10.1. Mohr-Coulomb failure theory

Soil generally fails in shear. At the failure surface, shear stress reaches the shear strength (t_f) of the soil. It the failure surface, sliding between the particles takes place as shown in Figure 10.1. The resistance that the soil offers during deformation is mainly due to the shear resistance between the particles at their contact points at the failure surface. No crushing of individual particle takes place. According to Mohr-Coulomb failure criterion, the shear strength of the soil can be expressed as:

$$\tau_f = c + \sigma \tan \emptyset \qquad (10.1)$$

where *c* is the cohesion and Ø is the angle of internal friction of the soil. σ is the applied normal stress. The line satisfying the Eq. (10.1) is called the Mohr-Coulomb failure envelop (as shown in Figure 10.2 with red color). In Figure 10.2, it is shown that t_f is the maximum stress soil can take without failure under an applied vertical stress σ (with blue color). Figure 10.3 shows the Mohr circle of two soil elements one at the failure surface (red color) and one at any other location (blue color). The Mohr circle touches the failure envelop incase of soil element taken from location of failure surface, whereas Mohr circle of the soil element taken from other than the location of failure surface is situated below the failure envelop. Keeping σ_3 constant, if vertical stress (σ_1) increases the Mohr Circle becomes larger and finally it will touch the failure envelop and failure will take place (as shown in Figure 10.4). Figure 10.5 shows the Mohr circle for total stress and effective stress condition. The Eq. (10.1) represents the shear strength in terms of total stress (σ). In terms of effective stress (σ' = σ - u), the shear strength of the soil can be expressed as:

$$\tau_f = c' + \sigma' \tan\phi'$$

where *u* is the pore water pressure.

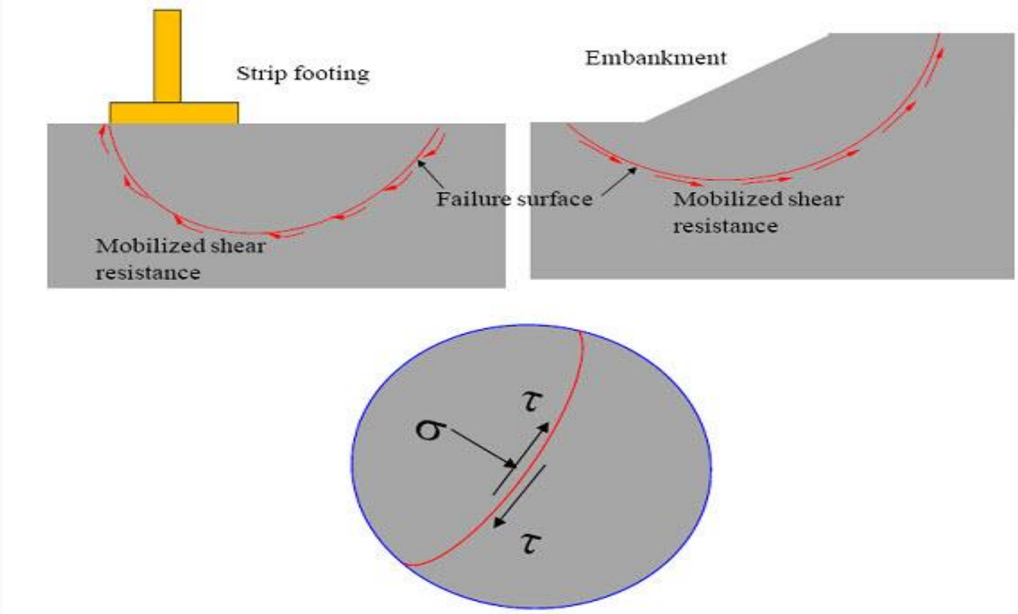


Fig. 10.1. Failure surface and shear resistance.

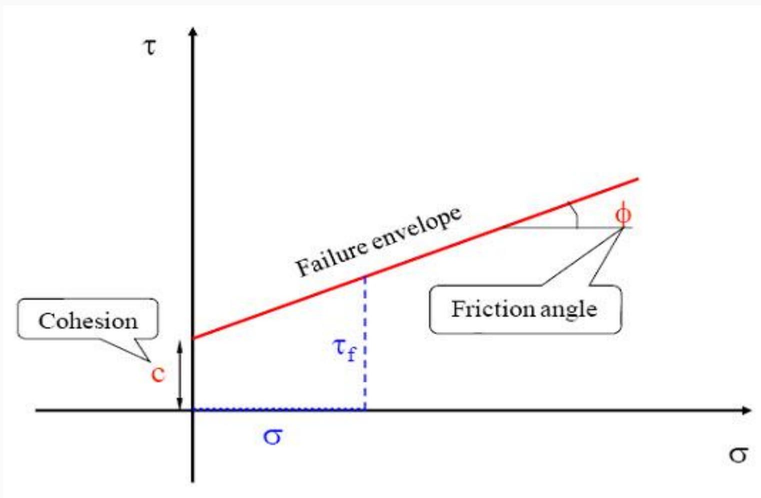


Fig. 10.2. Mohr-Coulomb failure criterion and failure envelop.

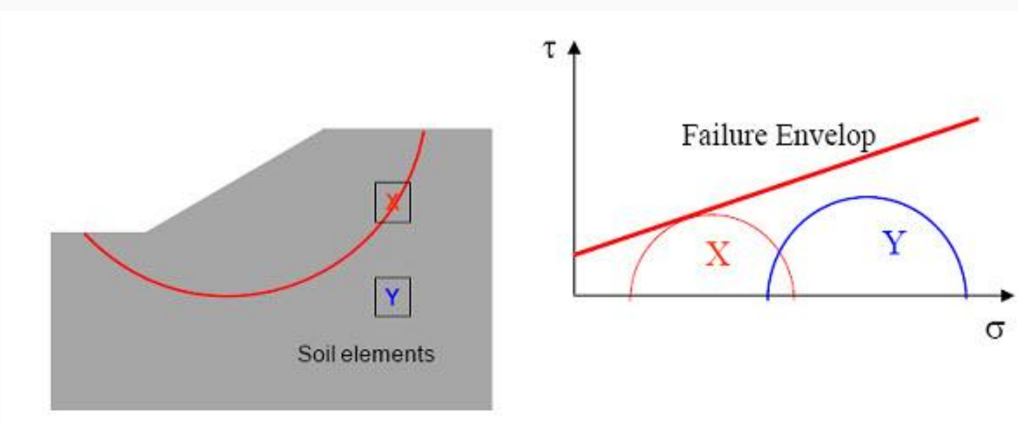


Fig.10.3. Mohr circles of different soil elements

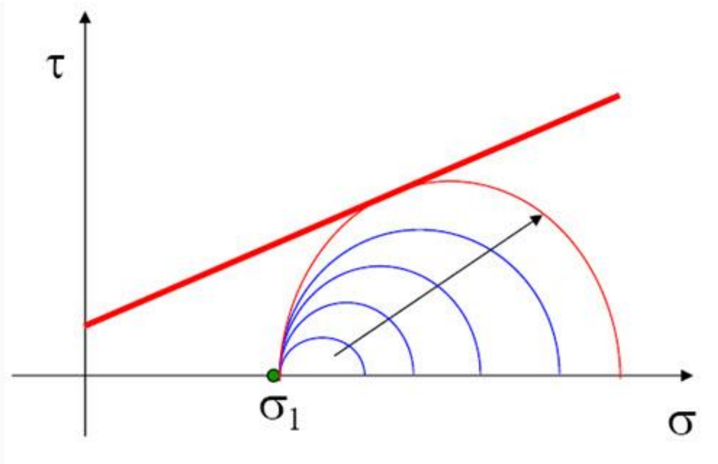


Fig. 10.4. Mohr circles for different stress condition

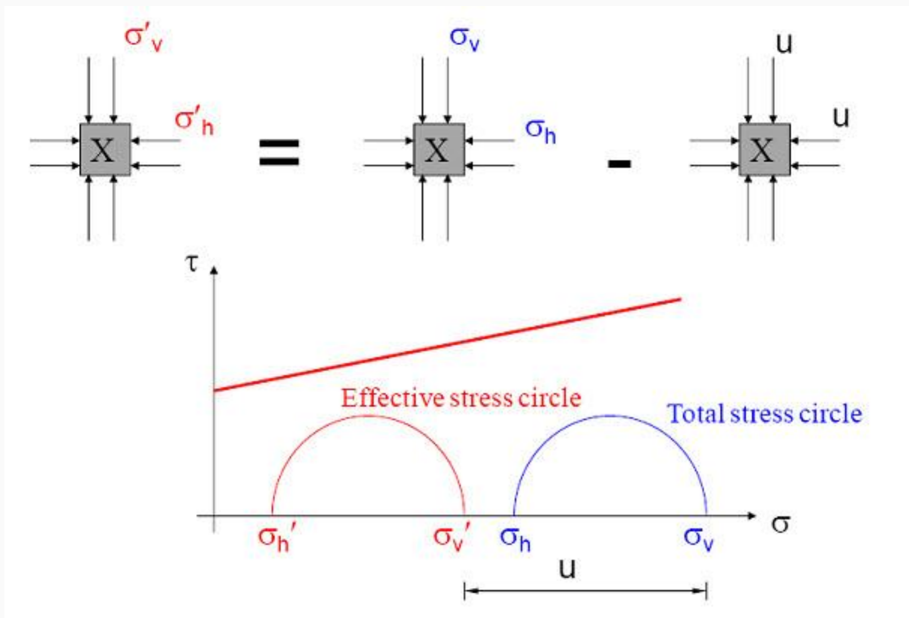


Fig. 10.5. Mohr circles for total stress and effective stress condition

Lesson. 11 Strength Parameters of Soil

11.1. Determination of Strength parameters

The shear strength parameters cohesion (*c*) and friction angle (Φ) can be determined by different laboratory tests for different types of soils.

Direct shear test:

The soil sample is tested in a confined metal box of square cross-section. The box has two halves horizontally and a small clearance is maintained between them. Figure 10.1 shows a direct shear test set-up. Upper half of the box is fixed and lower half of the box is pushed or pulled horizontally with respect to the fixed half. Thus, a shear is applied in the soil sample. A constant normal force (vertical) is applied on the sample throughout the test. Then horizontal force or shearing is applied till the failure. The shearing is normally applied at a constant rate of strain. The amount of shear load is measured with the help of proving ring. The vertical as well as horizontal deformation is measured with the help of dial gauges. The test procedure is repeated for different normal stresses (four to five normal stresses). The shear stress at failure is plotted against different normal stresses (as shown in Figure 11.2). The shear strength parameters are determined from the best-fit straight line passing through the test points (as shown in Figure 11.2). The test is suitable for sandy soils. If the sample is partially or fully saturated, porous stones are placed below and top of the sample to allow free drainage. Figure 11.3 shows typical shear stress-shear displacement and change in height of sample-shear displacement plot of soils obtained from direct shear tests.

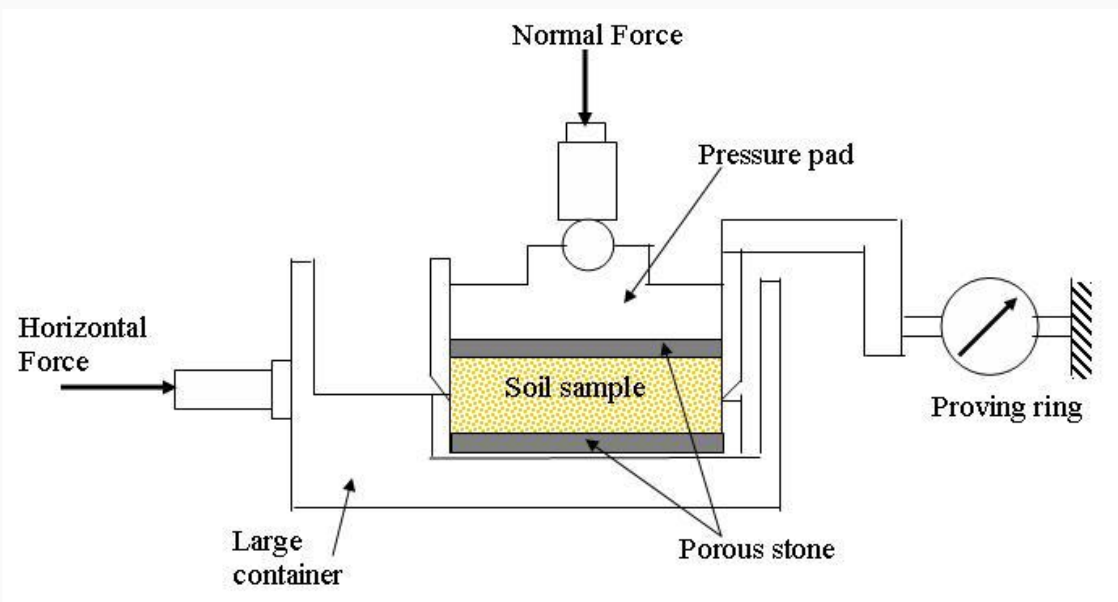


Fig. 11.1. Direct shear test

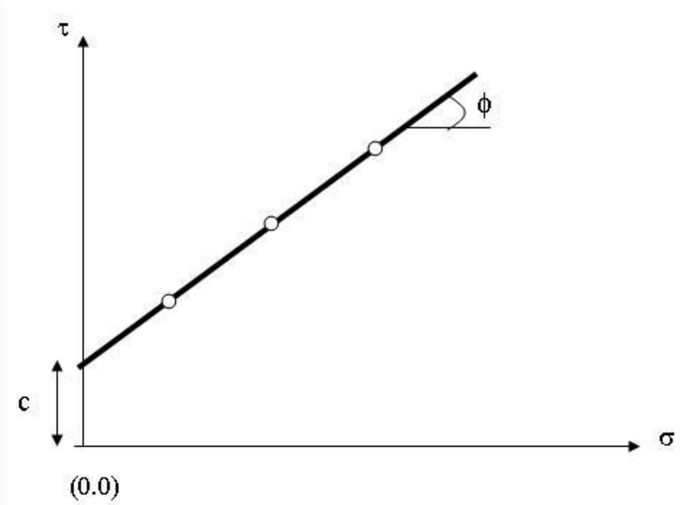


Fig. 11.2. Shear stress-normal stress plot

Fig. 11.3. Shear stress-shear displacement and change in height of sample-shear displacement plot of soils obtained from direct shear tests.

Unconfined compression test

The test is suitable for saturated clay ($\varnothing_u$ =0). The test is conducted under zero cell pressure. Thus, it a special case of triaxial test with $\sigma_3 = 0$ (triaxial test is described in lesson 12). A cylindrical specimen is subjected to axial stress until failure. Figure 11.4 shows the Mohr circle for unconfined compression test. For purely clayey soil, $\varnothing_u$ =0, the subscript u is used as the test is undrained test. The major principle stress (σ_1) is equal to the unconfined compressive strength of the soil (q_u).

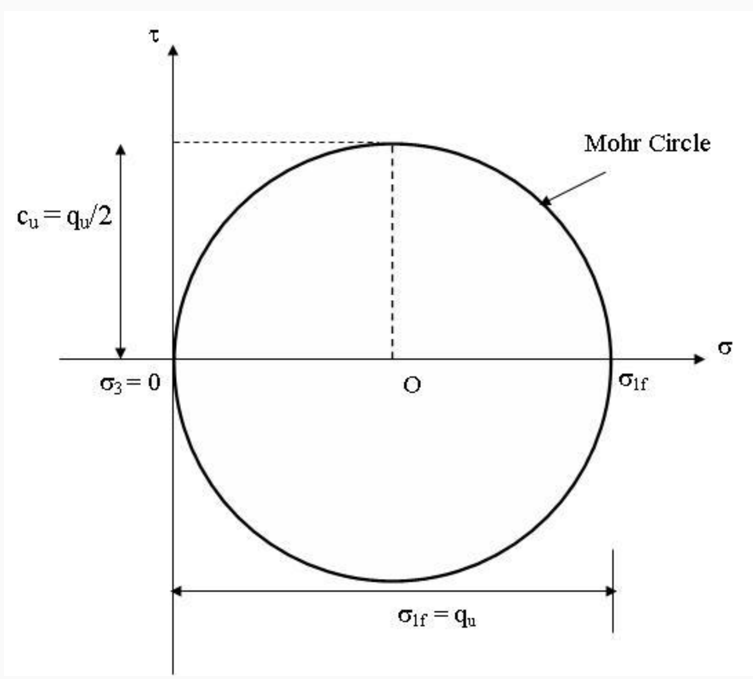


Fig. 11.4. Mohr-Coulomb plot for an unconfined compression test on saturated clay.

The undrained cohesion can be determined as:

$$c_u = \frac{q_u}{2}$$ ....(11.1)

For determining the unconfined compressive strength of the soil (q_u) (the applied load at failure divided by the cross-sectional area of the sample), the cross-section of the soil sample at failure load (A_f) is determined as:

$$A_f = \frac{A_0}{1-\varepsilon}$$ ....(11.2)

where A_0 is the initial cross-sectional area of the sample and e is the axial strain in the sample. The strain in the sample can be determined as:

\[\varepsilon=\frac{{Change\;in\;length\;(\Delta L)}}{{Initial\;length\;of\;the\;sample\;(L)}}\]..(11.3)

The usual sizes of the samples are: 76 mm (length) x 38 mm (diameter) or 100 mm (length) x 50 mm (diameter).

LESSON 12. Tri-Axial Test

12.1. Various Types of Tri-axial Tests

In triaxial test, tests are conducted in two stages, Stage I: under all round cell pressure (s_3) and Stage II: under shearing or loading (as shown in Figure 12.1). During all round cell pressure, if drainage is allowed the consolidation takes place in the sample. This type of sample is called consolidated sample. However, if drainage is not allowed then the sample is called unconsolidated sample. During shearing or loading (Deviatoric stress, $\Delta\sigma_d = \sigma_1 - \sigma_3$), if drainage is allowed the loading is called drained loading. However, if drainage is not allowed then the loading is called undrained loading. Figure 12.2 shows a triaxial test setup. The drainage in the sample is controlled by closing or opening the drainage valve. The all round cell pressure is applied by using water inside the triaxial cell.

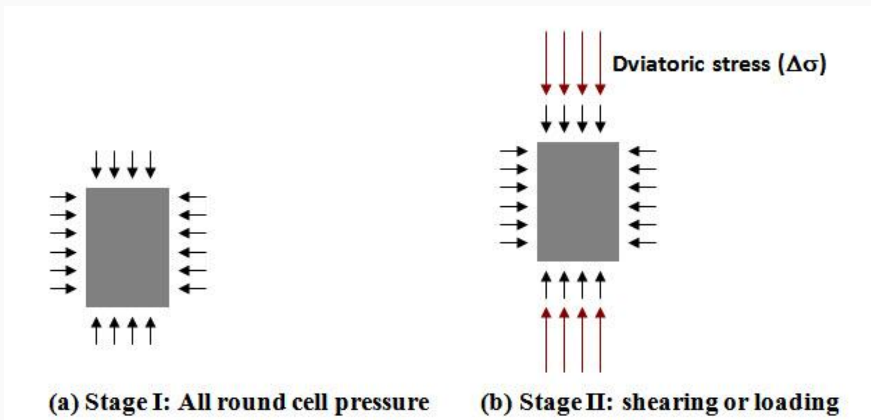


Fig. 12.1. Stages of Tri-axial tests.

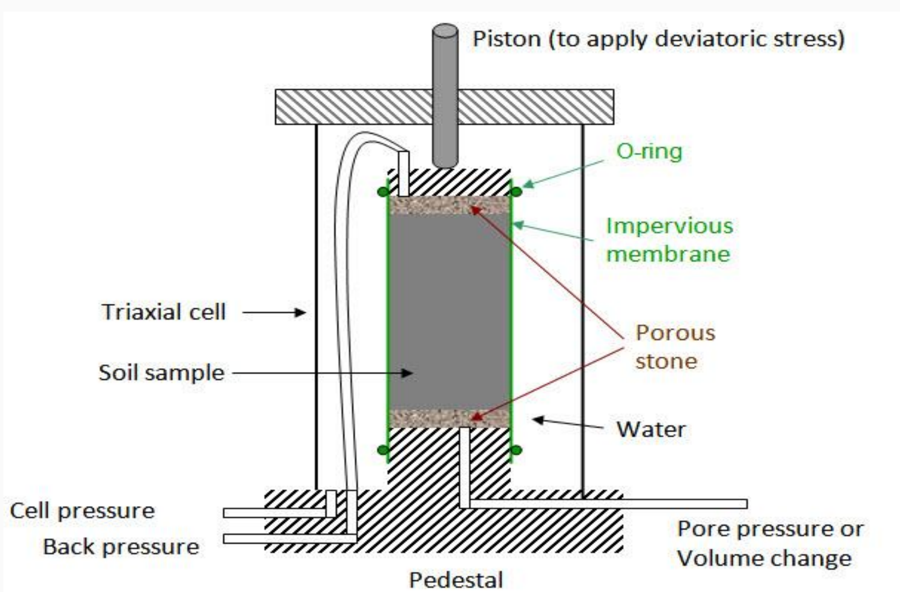


Fig. 12.2. Tri-axial tests test setup

The usual sizes of the samples are: 76 mm (length) x 38 mm (diameter) or 100 mm (length) x 50 mm (diameter). Thus, the length/diameter ratio of the cylindrical sample is 2. This test is suitable for both sand and clay. Depending on whether drainage is allowed or not during all round cell pressure and shearing, three types of triaxial tests are conducted:

(i) Unconsolidated Undrained (UU) test

(ii) Consolidated Undrained (CU) test

(iii) Consolidated Drained (CD) test

Unconsolidated Undrained (UU) test

In this type of test, pore pressure is developed during shearing. However, the pore water pressure is not measured. Thus, effective stress value is unknown. The parameters are determined in terms of total stress (c_u). It is a very quick test. The determined parameters are used for the analysis under undrained condition such as short term stability.

Consolidated Undrained (CU) test

In this type of test, pore pressure is developed during shearing and it is also measured. Thus, effective stress value is known. The parameters are determined in terms of effective stress (c' and Ø'). It is faster than CD test, but slower than the UU tests. This test is preferred to determine c' and Ø'.

Consolidated Drained (CD) test

In this type of test, no excess pore pressure is developed during the whole test. Shearing is done very slowly to avoid build-up the excess pore water pressure. The parameters are determined in terms of effective stress (c' and Ø'). It is a very slow test (can take few days). The determined parameters are used for the analysis under fully drained condition such as long term stability.

In general, during the test, deviator stress ($\sigma_1 - \sigma_3$), all round pressure (σ_3) and pore water pressure is measured (for example in CU test). From the test data, $(\sigma_1)_f$ value at failure is determined for different cell pressure (σ_3). After subtracting the pore water pressure, $(\sigma_1)'_f$ and σ'_3 are determined. Mohr-Circles are drawn for different $(\sigma_1)'_f$ and σ'_3 values. Failure envelop is drawn by drawing a common tangential line for all the circles (as shown in Figure 12.3). From that line, strength parameters c' and Ø are determined. Figure 12.4 shows typical deviator stress-axial strain and volume change-axial strain plot of soils obtained from CD trai-axial tests.

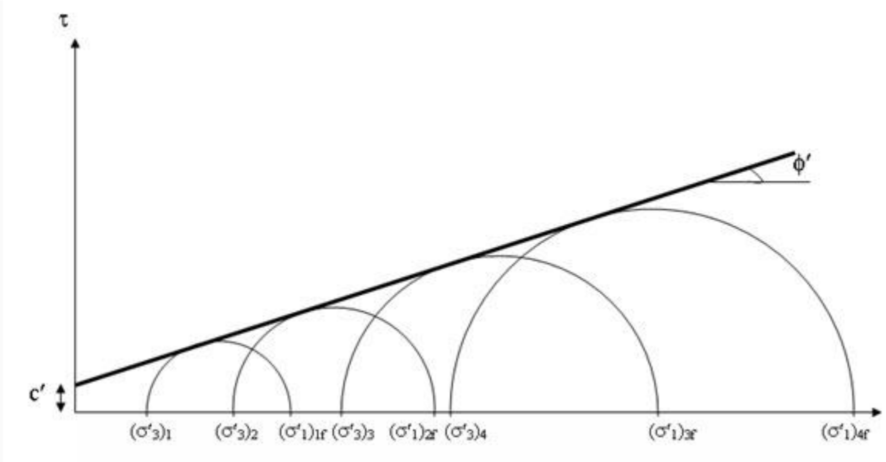


Fig. 12.3. Shear stress-normal stress plot.

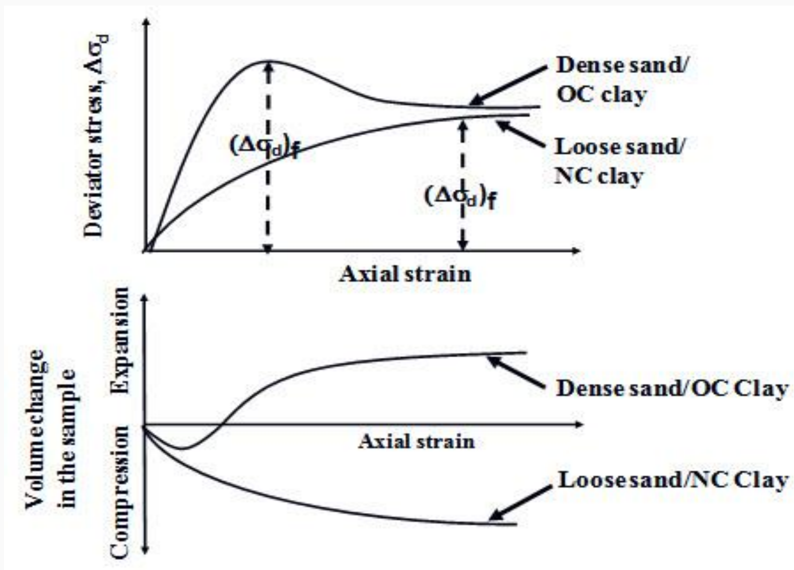


Fig.12.4. Deviator stress-axial strain and volume change-axial strain plot of soils obtained from CD t Tri-axial tests.

LESSON 13. Stress Path

13.1. What is Stress path?

Stress path is used to represent the successive states of stress in a test specimen of soil during loading or unloading. Series of Mohr circles can be drawn to represent the successive states of stress, but it is difficult to represent number of circles in one diagram. Figure 13.1 shows number of Mohr circles by keeping σ_3 and increasing σ_1 on σ - t plane. The successive states of stress can be represented by a series of stress points and a locus of these points (in the form of straight or curve) is obtained. The locus is called stress path. The stress points on σ - t plane can be transferred to p-q plane (as shown in Figure 13.1). The coordinates of the stress points on p-q plane can be obtained as:

$$p={{{\sigma _v} + {\sigma _h}} \over 2} \qquad (13.1)$$

$$q={{{\sigma _v} - {\sigma _h}} \over 2} \qquad (13.2)$$

The stress path can be drawn as:

(a) Total stress path (TSP)

(b) Effective stress path (ESP)

(c) Stress path of total stress minus static pore water pressure (TSSP)

If in a field situation, static ground water table exists, initial pore water pressure u_0 will act on the sample. Thus, the static pore water pressure will be equal to u_0. The effective stress coordinates of the stress points on p'-q' plane can be obtained as:

$$p'={{{\sigma _v} - u + {\sigma _h} - u} \over 2} = {{{{\sigma '}_v} + {{\sigma '}_h}} \over 2} \qquad (13.3)$$

$$q'={{{\sigma _v} - u - ({\sigma _h} - u)} \over 2} = {{{{\sigma '}_v} - {{\sigma '}_h}} \over 2} = {{{\sigma _v} - {\sigma _h}} \over 2}=q \qquad (13.4)$$

Figure 13.2 shows different stress paths for normally consolidated clay obtained from CU (effective) test.

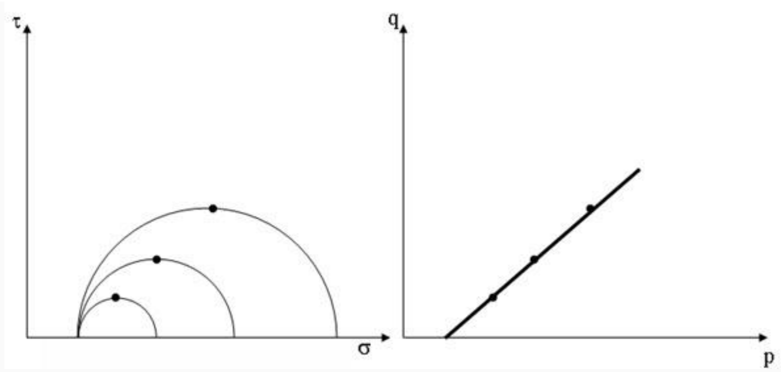


Fig. 13.1. Stress point on Mohr circle and on p-q plane.

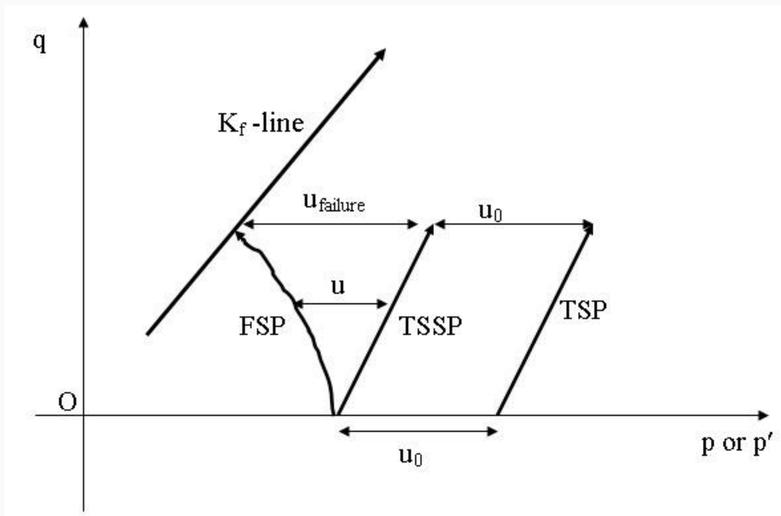


Fig.13.2. Different stress paths.

For initial condition, $\sigma_v = \sigma_h = 0$ and if σ_v and σ_h are increased in such as way that ratio σ_h/σ_v is constant. This ratio is called lateral stress ratio, *K*. Thus,

$$K={{{\sigma _h}} \over {{\sigma _v}}} \qquad (13.5)$$

Similarly, coefficient of lateral earth pressure at rest (K_0) and at failure (K_f) can be expressed as:

$$K_0={{{{\sigma '}_h}} \over {{{\sigma '}_v}}} \qquad (13.6)$$

$$K_f={{{{\sigma '}_{hf}}} \over {{{\sigma '}_{vf}}}} \qquad (13.7)$$

On *p*-*q* plane, the stress ratios are representing straight lines (as shown in Fig. 13.3).

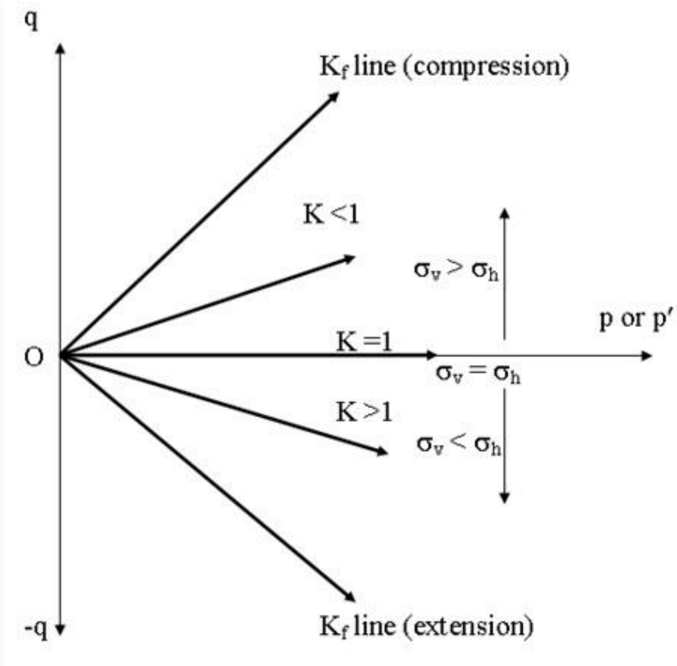


Fig. 13.3. Variation of Stress Rations.

Figure 13.4 shows the relationship between K_f line (on *p*-*q* plane) and Mohr-Colomb failure envelope (on $s - t$ plane). The shear strength parameters can also be determined from K_f line on *p*-*q* plane as:

$$\phi={\sin ^{ - 1}}(\tan \psi) \qquad (13.8)$$

$$c={a \over {\cos \phi }} \qquad (13.9)$$

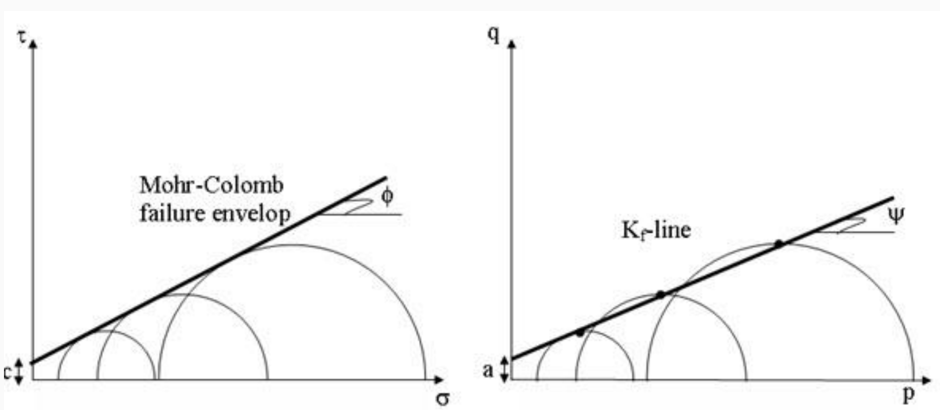


Fig. 13.4. Relationship between K_f line and Mohr-Colomb failure envelope.

MODULE 3. Compaction, Seepage and Consolidation of Soil

LESSON 14. Compaction

14.1 Introduction

Densification of soils by applying mechanical energy is called compaction. Due to compaction the volume of air voids reduces, density of the soil increases. This is one of the common methods of soil improvement. The compressibility and permeability of the soil decrease due to compaction. Compaction decreases the settlement of soil and increases the bearing capacity of the soil.

14.2 Laboratory Tests

The compaction can be done in laboratory by Standard Proctor Test or Modified Proctor Test. In case of Standard Proctor Test, a mould of standard of volume 944 cc is filled with soil in three layers. Each layer is compacted by 25 blows of a hammer of weight 2.495 (or 2.5 kg) kg. The falling height of the hammer is fixed at 304.8 mm (or 30 cm) each time. This process is done for different water content. The compaction mould with hammer is shown in Figure 14.1. The wet unit weight (γ_t) of the soil is calculated each time by knowing the weight of the wet soil and volume of the mould (944 cc). The amount of water content is measured. Once the bulk or wet unit weight and water content (w) of the soil are calculated, the dry unit weight (γ_d) of the soil is calculated as:

$$\gamma_d = \frac{\gamma_t}{1 + w} \qquad (14.1)$$

The dry unit weight is plotted at different water content. This curve is called as 'compaction curve'. Figure 14.2 shows a typical compaction curve for clay. The compaction curve has significant importance in case of clayey soil. From the compaction curve, maximum dry unit weight (γ_{dmax}) and optimum water content (OMC) are obtained (as shown is Figure 14.2). The specification of compaction in the field is decided based on maximum dry unit weight or/and optimum water content. The typical range of maximum dry unit weight of soil is 16 to 20 kN/m^3 with a wide range of 13 to 24 kN/m^3. Similarly, the typical range of OMC is 10 to 20% with a wide range of 5 to 30% (Ranjan and Rao, 2003).

In case of Modified Standard Proctor Test, a mould of standard of volume 944 cc is filled with soil in five layers. Each layer is compacted by 25 blows of a hammer of weight 4.54 kg (or 4.5 kg). The falling height of the hammer is fixed at 457.2 mm (or 45cm) each time. This process is done for different water content.

The Indian standard equivalent, the Standard Proctor Test is called the light compaction test (IS: 2720, Part VII, 1974) and Modified Standard Proctor Test is called the heavy compaction test (IS: 2720, Part VIII, 1983). In case of light compaction test, a mould of standard of volume 1000 cc is filled with soil in three layers. Each layer is compacted by 25 blows of a hammer of weight 2.6 kg. The falling height of the hammer is fixed at 310 mm each time. In case of heavy compaction test, a mould of standard of volume 1000 cc is filled with soil in five layers. Each layer is compacted by 25 blows of a hammer of weight 4.9 kg. The falling height of the hammer is fixed at 450 mm each time.

The dry unit weight can be written in terms of water content and degree of saturation as:

$${\gamma _d}={G \over {1 + e}}{\gamma _w}$$

$$=\frac{G}{{1+\frac{{wG}}{S}}}{\gamma _w} \qquad (14.2)$$

where G is the specific gravity of the soil, w is the water content, S is the degree of saturation, e is the void ratio and g_w is the unit weight of water generally taken as 10 kN/m^3. For completely saturated soil, the dry unit weight of the soil is calculated from Eq. (14.2) by putting S =1. The line (unit weight vs water content for a particular specific gravity) corresponding to the 100% degree of saturated is called Zero Air-Void Line (as shown in Figure 14.2). For a particular specific gravity and water content the obtained dry unit weight by putting S = 1 is called Zero Air Void Density or Zero Air Void Unit Weight. The Zero Air Void Unit Weight can be achieved only theoretically, in field it can not be possible to achieve it. What ever compaction energy is applied in the soil, some air will still remain within the soil voids.

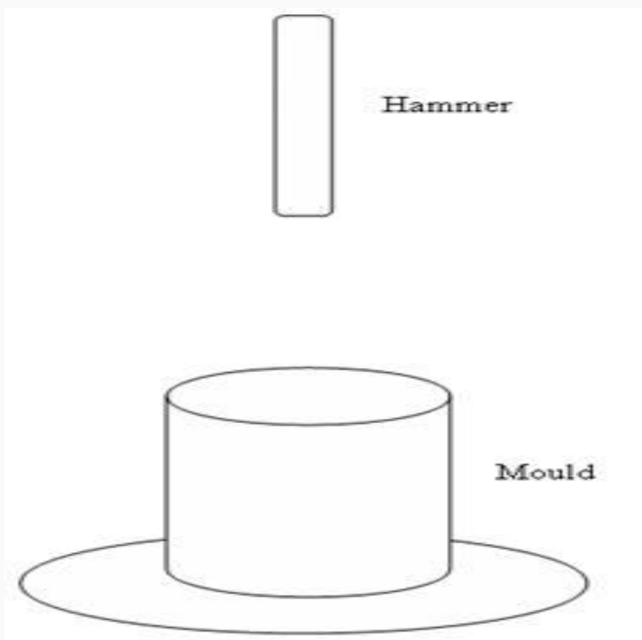


Fig. 14.1. Compaction mould with hammer.

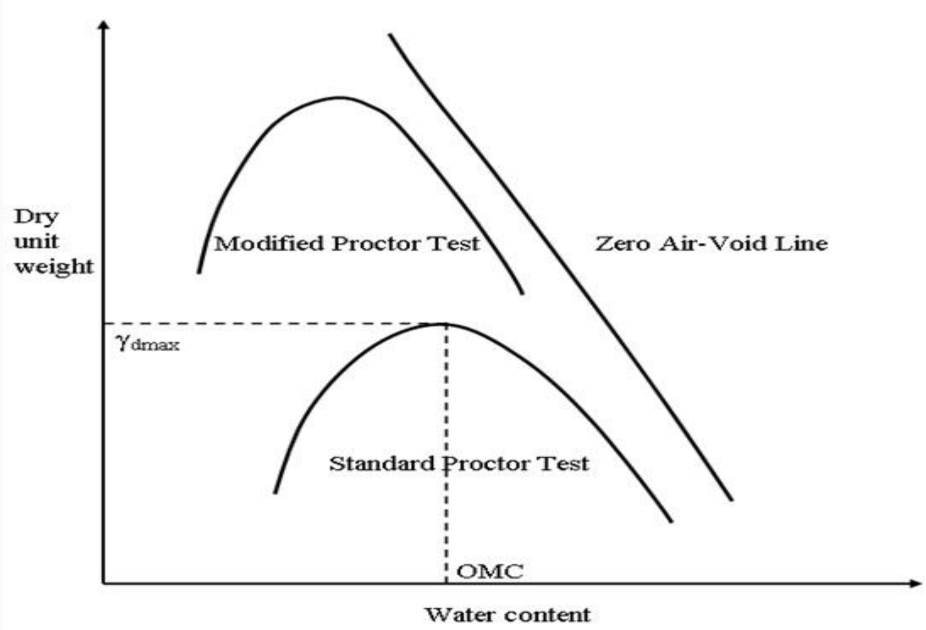


Fig. 14.2. Typical compaction curves of clay.

From the Figure 14.2 it is observed that in case of modified proctor test, the maximum dry unit weight increases as compared to the standard proctor test. However, OMC is more in case of standard proctor test as compared to the modified proctor test. Thus, as the compaction energy increases maximum dry unit weight increases with a decreasing rate.

LESSON 15. Factors Affecting Compaction

15.1 Factors affecting compaction

The compaction of soil depends on the following factors:

- Water content
- Compactive effort
- Type of soil
- Method of compaction

Water content: Water content has significant effect on compaction characteristics of soil. At low water content, soil is stiff and soil grains offer more resistance to compaction. As the water content increases, water films are formed around the soil grains and water around the soil grains act as lubricant. Due to this, soil grains come close to each other and make a dense configuration. At optimum moisture content, soil reaches the maximum unit weight as lubrication effect is the maximum at this stage. Further addition of more water replaces the soil grains. Thus, addition of more water after optimum moisture content reduces the unit weight as unit weight of water is less than the unit weight of soil grains. Because of the due to the addition of water dry unit weight of soil increases upto a certain water content value (OMC) and beyond that addition of more water decreases the dry unit weight of soil (as shown in Figure 15.1). This can be also explained by concept of soil structure and electrical double layer theory.

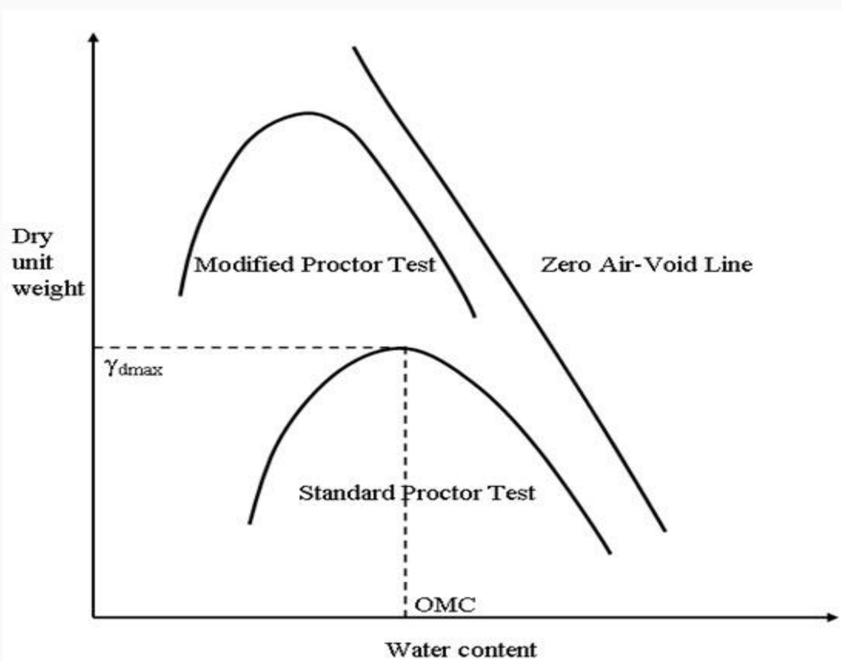


Fig.15.1. Compaction curves for clay.

Compactive effort: As the compactive effort increases maximum dry unit weight increases in a decreasing rate. However, the OMC value decreases as the compactive effort increases. Thus, if the compactive effort increases the compaction curve is shifted to the top and to the left side as shown in Figure 15.2. However, as the water content increases the effect of compactive effort on dry unit weight decreases. If the peaks of the compaction curves for different compactive efforts are joined the obtained line is called line of optimum (as shown in Figure 15.2). The line of optimum is nearly parallel to line of zero air voids. Thus, the efficiency of the compaction does not increase as the compactive effort increases.

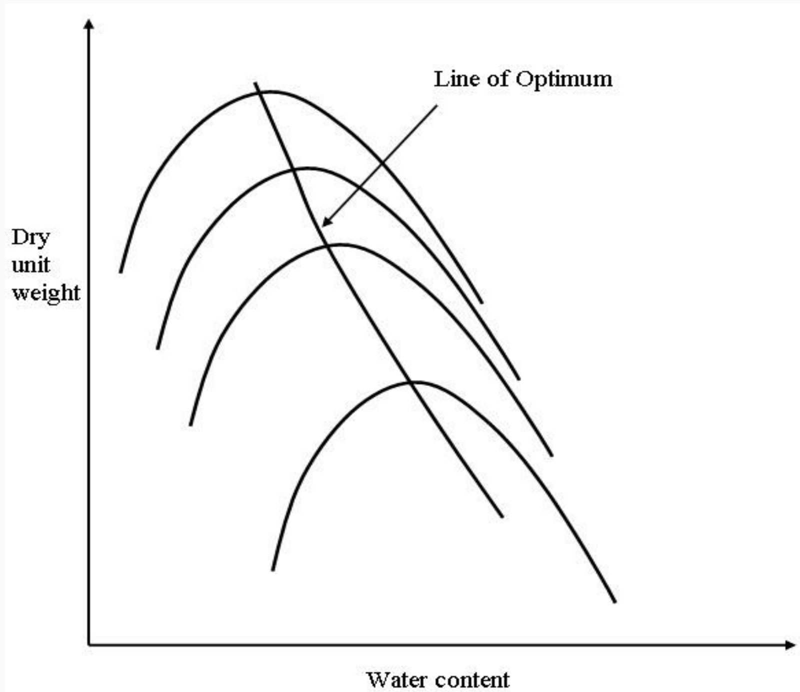


Fig.15.2. Effect of compactive effort.

Type of soil: Soil type has significant effect on compaction. It is observed that cohesive soil has higher OMC value and poorly graded sands have lower dry unit weight as compared to the coarse-grained, well graded soil with some percentage of fines. However, excessive amount of fines reduces the maximum dry unit weight value of the soil. Maximum dry unit weight of sand is obtained either completely dry or saturated condition. Compaction curve has very little importance in case of sandy soils.

Method of compaction: It is observed that mode of compaction influences the shape of compaction curves. The shape of compaction curves for standard proctor test and modified proctor test are some what different. The shape of the compaction curve obtained from laboratory tests and obtained from field by various compactive methods are also different.

LESSON 16. Field Compaction

16.1. Introduction

The laboratory compaction test gives an ideal that what compacted dry unit weight to be achieved in field. Generally in the field, 90 % - 95% of laboratory obtained dry unit weight is attained. The amount of compaction achieved in the field depends on the thickness of the layer, type of roller used, pressure intensity applied on the soil, number of passes of the roller. Depending upon the type of soils, compaction can be done by vibration, rolling or ramming. Roller can be smooth wheel roller, sheepsfoot rollers and pneumatic rubber rollers.

16.2. Compaction of Cohesion less Soil

In case of cohesionless soils, vibration is the most effective method of compaction. Best results can be obtained when the frequency of vibration is near to the natural frequency of the soil to be compacted. The vibrating equipments can be hydraulic type or dropping weight type.

16.3. Compaction of Cohesive Soil

In case of moderately cohesive soils, compaction can be done in layers to get best possible results. The compaction is done by roller. For silts of low plasticity, pneumatic rollers are preferred. In case of soils with moderate plasticity, sheepsfoot rollers are preferred.

The types of equipment used for compaction of various soils are reported in Table 16.1 (Ranjan and Rao, 2000):

Table 16.1: Type of equipment used for compaction of different soils

Soil type	Equipment	Use
Sands	Vibratory rollers	Embankments for oil storage tanks
Sand, silts, clay	Pneumatic rubber rollers	Base, Sub-base, embankments for highway, airfield
Clay	Sheepsfoot rollers	Core of the earth dam
Crushed rock, gravel, sand	Smooth wheeled rollers	Road construction
All soils	Rammer	Fills behind the retaining walls, trench fills

The field compaction can be expressed by relative compaction which is the ratio of dry unit weight [$\gamma_{d(field)}$] of soil in filed and maximum dry unit weight [γ_{dmax}] of soil in laboratory. Thus,

$$\text{Relative compaction} = {{\gamma _{d(field)}} \over {\gamma _{d\max }}} \qquad (16.1)$$

Problem 1

The in situ void ratio of a granular soil is 0.45. The maximum (e_{max}) and minimum (e_{min}) void ration of the soil is 0.7 and 0.4, respectively. The specific gravity (G_s) of the soil is 2.7. Determine the relative compaction.

Solution:

$${\gamma _{d(\max)}}={{G_s} \over {1 + {e_{\min }}}}{\gamma _w}={{2.7} \over {1 + 0.4}} \times 10=19.29\,kN/{m^3}$$

$${\gamma _{d(\min)}}={{G_s} \over {1 + {e_{\max }}}}{\gamma _w}={{2.7} \over {1 + 0.7}} \times 10=15.88\,kN/{m^3}$$

$${\gamma _{d(in\,situ)}}={{G_s} \over {1 + {e_{in\;situ}}\;}}{\gamma _w}={{2.7} \over {1 + 0.45}} \times 10=18.62\,kN/{m^3}$$

Thus, the relative compaction is:

$${{{\gamma _{d(in\;situ)}}} \over {{\gamma _{d\max }}}}={{18.62} \over {19.29}} \times 100 = 96.5\,\%$$

Problem 2

An embankment of volume 50,000 m^3 is constructed (compacted) with maximum dry density of 18 kN/m^3 and optimum moisture content of 10%. Determine how much weight of soil (wet weight) will be required to construct the embankment.

Solution:

The unit weight of wet soil can be written as:

$${\gamma _{wet}}=(1 + w){\gamma _d}$$

where w is the water content, γ_d is the dry unit weight of the soil and γ_w is the wet unit weight of the soil. Thus, wet unit weight can be written as:

$${\gamma _{wet}}=(1 + 0.1) \times 18 = 19.8\;kN/{m^3}$$

Thus, the weight of the dry and wet soil can be written as:

Weight of dry soil required = 50,000 × 18 = 9 × 10^5 kN.

Weight of wet soil required = 50,000 × 19.8 = 9.9 × 10^5 kN.

LESSON 17. Permeability of Soil

17.1 Introduction

Permeability of the soil quantitatively describes how easily water can flow through it. In loose soil, amount of pores within the soil grains is more. Water can flow easily through loose soils. However, in case of dense soil, amount of pores within the soil grains is less. Water can not flow easily through dense soils (as shown in Figure 17.1). Thus, permeability is high in case of loose soil whereas, it is low in case of dense soil.

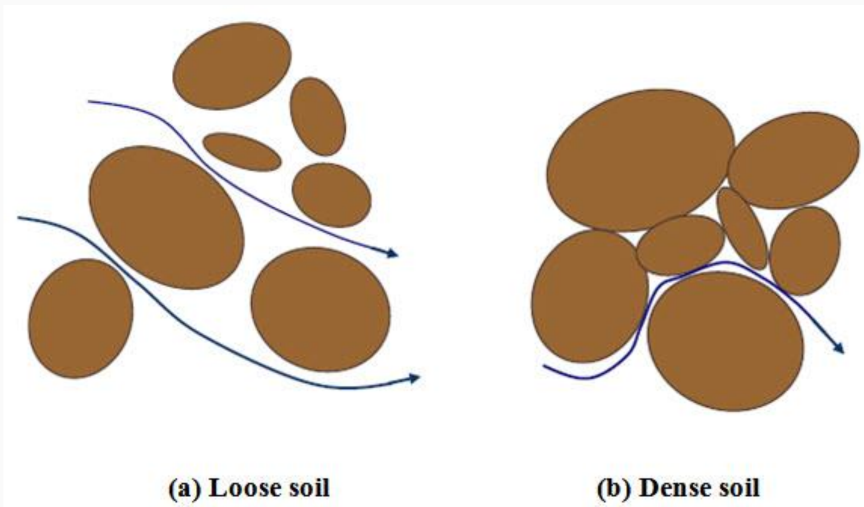


Fig.17.1. Flow of water a different type of soils

Darcy's Law

Velocity (*v*) of flow is proportional to the hydraulic gradient (***i***). Thus,

V = Ki (17.1)

where *k* is the coefficient of permeability (cm/sec) or hydraulic conductivity. If *i* is equal to one, the $v = k$. Thus, coefficient of permeability is velocity of water for unit hydraulic gradient. Hydraulic gradient can be expressed as: $i = \Delta h / L$, where Δh is the head loss and *L* is the length between two points (as shown in Figure 17.2).

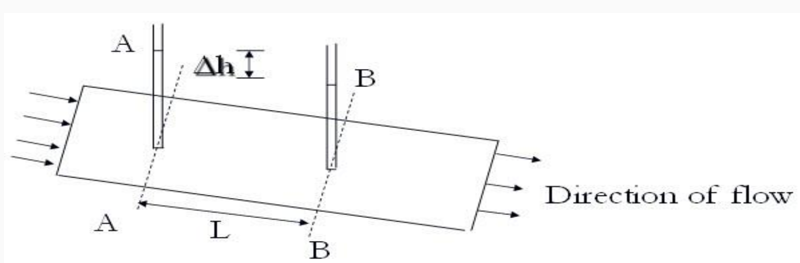


Fig.17.2. Flow of water

17.2. Permeability in layered soil

Figure 17.3 shows a layered soil system. The thickness of each layer is H_1, H_2, H_3 ………. H_n. The coefficient of permeability is k_1, k_2, k_3 …………… k_n. In case of horizontal flow, it takes place through all the layers at the same time. Thus, hydraulic gradient is same for all the layers, i.e $i_1 = i_2 = i_3 =$ …… $i_n = i$, where n is the number of layers. However, the velocity of flow is different in different layer.

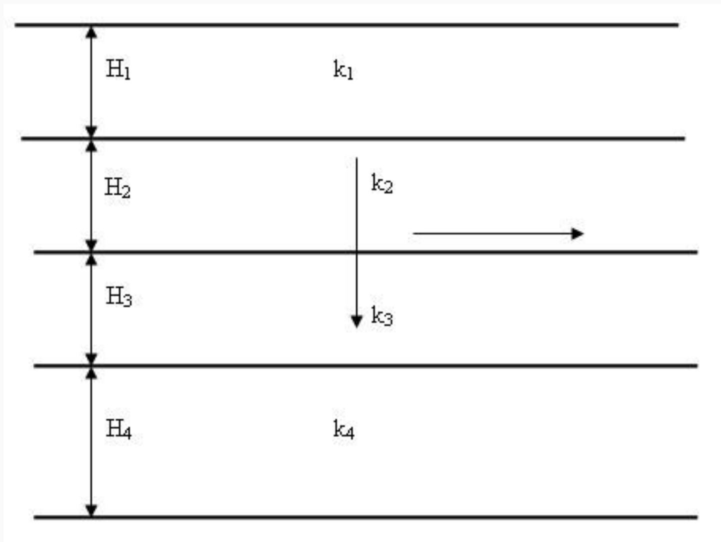


Fig. 17.3. Permeability of layered soil

According to Darcy's law, the average discharge velocity (v_{avg}) can be written as:

$$v_{avg} = k_H i = \frac{1}{H}(v_1 H_1 + v_2 H_2 + \ldots + v_n H_n) \qquad (17.2)$$

where k_H is the coefficient of horizontal permeability, v_1, v_2 …….v_n is the velocity of flow in different layers. H is the total thickness of the layers, i.e H_1+H_2+………$+H_n = H$. The Eq. (17.2) can be written as:

$$k_H i = \frac{1}{H}(k_1 i H_1 + k_2 i H_2 + \ldots + k_n i H_n) \qquad (17.3)$$

$$k_H = \frac{1}{H}(k_1 H_1 + k_2 H_2 + \ldots + k_n H_n) \qquad (17.4)$$

In case of vertical flow, the hydraulic gradient is different in each layer. However, the velocity of flow is same in all the layers. The total head loss is h and the head losses in each layer is h_1, h_2, ………h_n.

Head loss in each layer, $h_1 = H_1 i_1$, $h_2 = H_2 i_2$ …………………… $h_n = H_n i_n$ (17.5)

Thus, the total head loss $h = H_1 i_1 + H_2 i_2 + \ldots + H_n i_n$ (17.6)

The velocity of flow is same in all the layers. Thus,

$$v={k_v}{h \over H}={k_1}{i_1} \qquad (17.7)$$

where k_v is the coefficient of vertical permeability.

Thus,

$${k_v}={{H{k_1}{i_1}} \over {{H_1}{i_1} + {H_2}{i_2} + ...{H_n}{i_n}}} \qquad (17.8)$$

$${k_v} = \frac{{H{k_1}{i_1}}}{{\frac{{{H_1}}}{{{k_1}}} + \frac{{{H_2}}}{{{k_2}}} + ... + \frac{{{H_n}}}{{{k_n}}}}} \qquad (17.9)$$

For more information see the Appendix 17.1

17.3. Factors affecting permeability

Some of the factors affecting permeability are:

- Grain Size
- Viscosity and temperature
- Void ratio
- Soil fabric of clay

Typical values of coefficient of permeability of various soils are presented as (Das, 1999):

Type of soil	Hydraulic conductivity (cm/sec)
Medium to coarse gravel	$>10^{-1}$
Coarse to fine sand	10^{-1} to 10^{-3}
Fine sand, silty sand	10^{-3} to 10^{-5}
Silt, clayey silt, silty clay	10^{-5} to 10^{-6}
Clays	10^{-7} or less

Appendix 17.1

Head loss in each layer, ${h_1}={H_1}{i_1}$, ${h_2}={H_2}{i_2}$..................${h_n}={H_n}{i_n}$ (17.10)

The total head loss $h={h_1} + {h_2} + ... + {h_n}$ (17.11)

The velocity of flow is same in all the layers. Thus,

$$v={k_v}{h \over H}={k_1}{{{h_1}} \over {{H_1}}}={k_2}{{{h_2}} \over {{H_2}}}=\cdots={k_i}{{{h_i}}\over {{H_i}}}=\cdots={k_n}{{{h_n}} \over {{H_n}}} \qquad (17.12)$$

where k_v is the coefficient of vertical permeability. Rearranging,

$$h=v{H \over {{k_v}}}\quad {h_1}=v{{{H_1}}\over{{k_1}}}\quad {h_2}=v{{{H_2}}\over{{k_2}}}\quad\cdots{h_i}=v{{{H_i}}\over{{k_i}}}\quad\cdots{h_n}=v{{{H_n}}\over{{k_n}}} \qquad (17.13)$$

The total head loss

$$h=v{H \over{{k_v}}}=v{{{H_1}}\over {{k_1}}}+v{{{H_2}}\over{{k_2}}}+\cdots v{{{H_i}}\over{{k_i}}}+\quad\cdots v{{{H_n}}\over {{k_n}}} \qquad (17.14)$$

$${H \over{{k_v}}}={{{H_1}}\over{{k_1}}}+{{{H_2}}\over {{k_2}}}+\cdots{{{H_i}} \over {{k_i}}}+\quad\cdots{{{H_n}}\over {{k_n}}} \qquad (17.15)$$

$${k_v} = \frac{H}{\frac{{{H_1}}}{{{k_1}}}+\frac{{{H_2}}}{{{k_2}}}+\cdots\frac{{{H_i}}}{{{k_i}}}+\quad\cdots\frac{{{H_n}}}{{{k_n}}}} \qquad (17.16)$$

LESSON 18. Measurement of Permeability

18.1. Introduction

The coefficient of permeability can be determined by laboratory as well as field tests and by empirical approach. In the laboratory, coefficient of permeability is determined by constant head and falling head test. In the field, permeability is determined by unconfined and confined flow pumping tests.

Constant Head Test

Constant head test is suitable for coarse grained soil as sufficient discharge is required to determine the coefficient of permeability. Figure 18.1 shows the schematic diagram of constant head permeameter. The water is allowed to flow through the soil sample from a reservoir such a way that a constant water level is maintained in the reservoir by overflow. The quantity of water (*Q*) flowing through the soil for a particular time (*t*) or discharge is measured. The coefficient of permeability is determined as:

$$k={{QL} \over {Aht}}cm/s \qquad (18.1)$$

where

k = coefficient of permeability (cm/s)

Q=discharge collected in time t (cm^3/s)

A = c/s area of the sample (cm^2)

h = head drop (distance) in manometers levels (cm)

L = distance between manometer tapping point (cm)

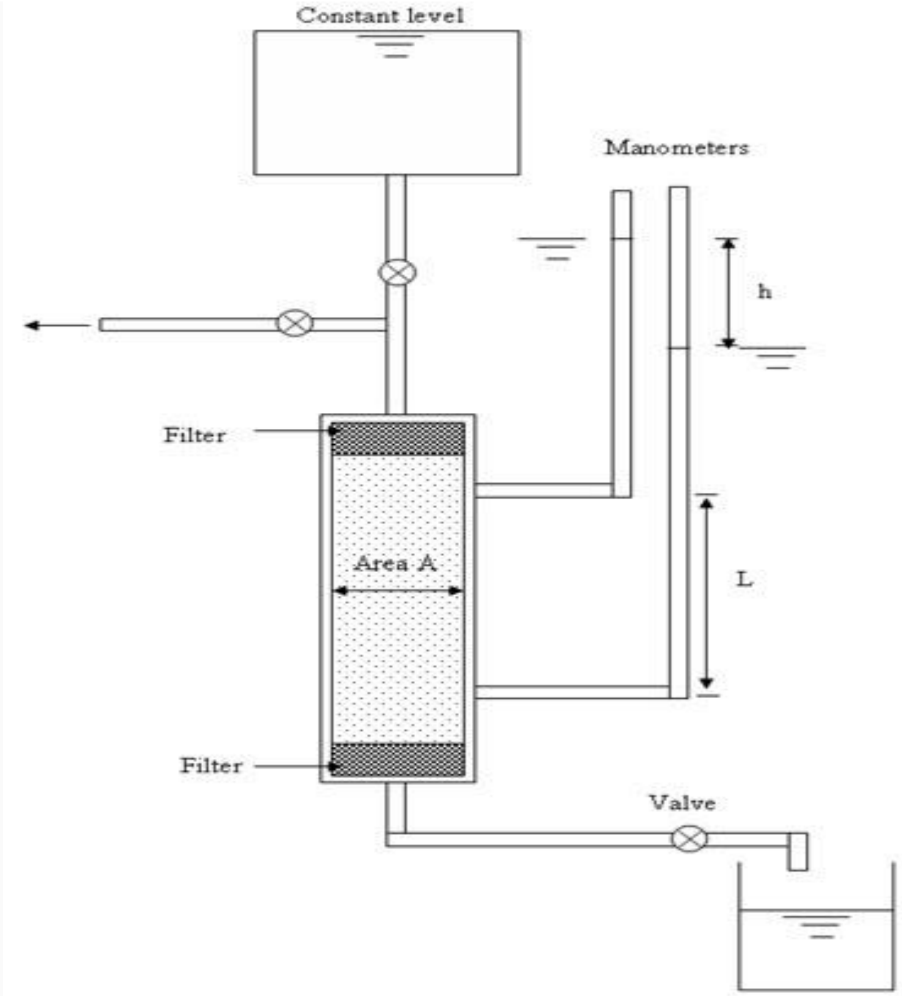


Fig.18.1. Constant head permeameter

Falling Head Test

Falling head test is suitable for fine sands, silts. Figure 18.2 shows the schematic diagram of falling head permeameter. The water is allowed to flow through the soil sample and the height difference of the water level in the stand pipe for a particular time interval is measured. The coefficient of permeability is determined as:

$$k=2.3{{aL} \over {At}}{\log _{10}}{{{h_1}} \over {{h_2}}} \qquad (18.2)$$

where

k = coefficient of permeability (cm/s)

A = c/s area of the sample (cm^2)

a= area of the stand pipe (cm^2)

h = distance in manometers levels (cm)

h_1 (cm) and h_2 (cm) = height of the water level in stand pipe in time difference t (s)

Unconfined Pumping Tests

Figure 18.3 shows the unconfined pumping test procedure. Here aquifer is underlain by an impermeable layer and pumping well is extended up to the bottom of the permeable layer as shown in Figure 18.3. Two observation wells are inserted in the aquifer. The permeability is determined as:

$$k=2.3{q \over \pi }{{{{\log }_{10}}({r_2}/{r_1})} \over {(h_2^2 - h_1^{2})}} \quad (18.3)$$

where q is discharge, k is coefficient of permeability, h_1 and h_2 can be determined by measuring the drawdown at the observation wells (as shown in the Figure 18.3). r_2 and r_1 are the distance of the observation well from the pumping well.

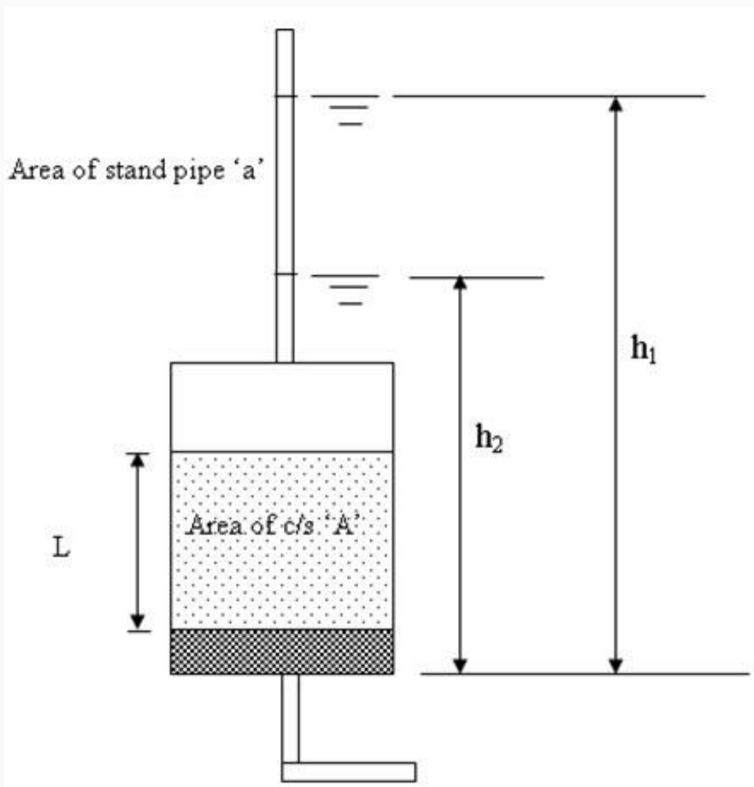


Fig. 18.2. Falling head permeameter

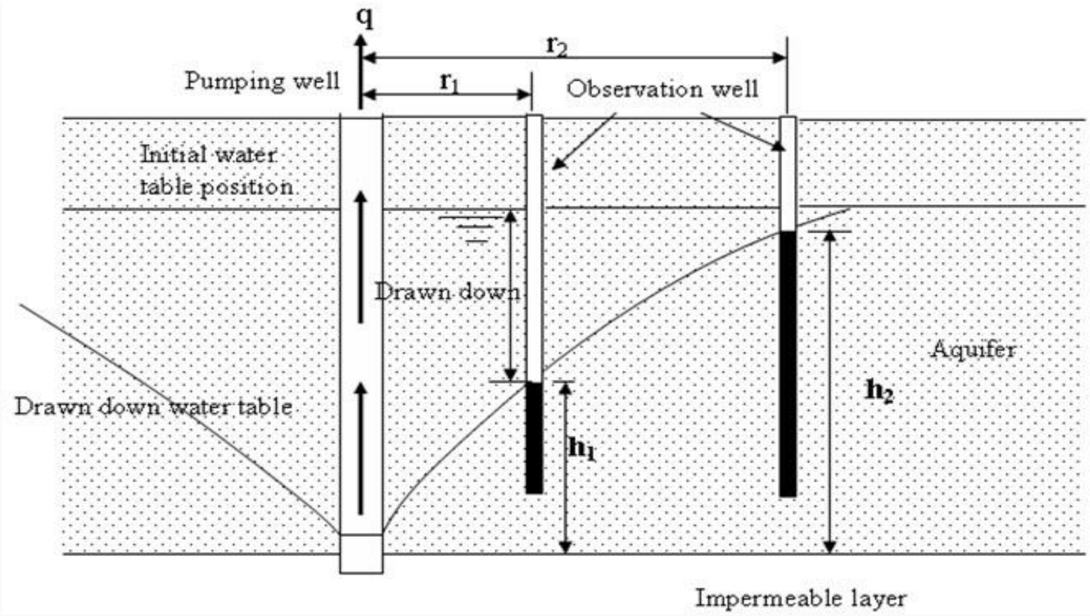


Fig. 18.3. Pumping test in unconfined aquifer

Confined Pumping Tests

Figure 18.4 shows the confined pumping test procedure. Here aquifer is confined in top and bottom by impermeable layers as shown in Figure 18.4. Two observation wells are inserted in the aquifer. The permeability is determined as:

$$k=2.3{q \over {2\pi D}}{{{{\log }_{10}}({r_2}/{r_1})} \over {(h_2 - h_1)}} \qquad (18.4)$$

where q is discharge, k is coefficient of permeability, h_1 and h_2 can be determined by measuring the drawdown at the observation wells (as shown in the Figure 18.3). r_2 and r_1 are the distance of the observation well from the pumping well. D is the thickness of the aquifer.

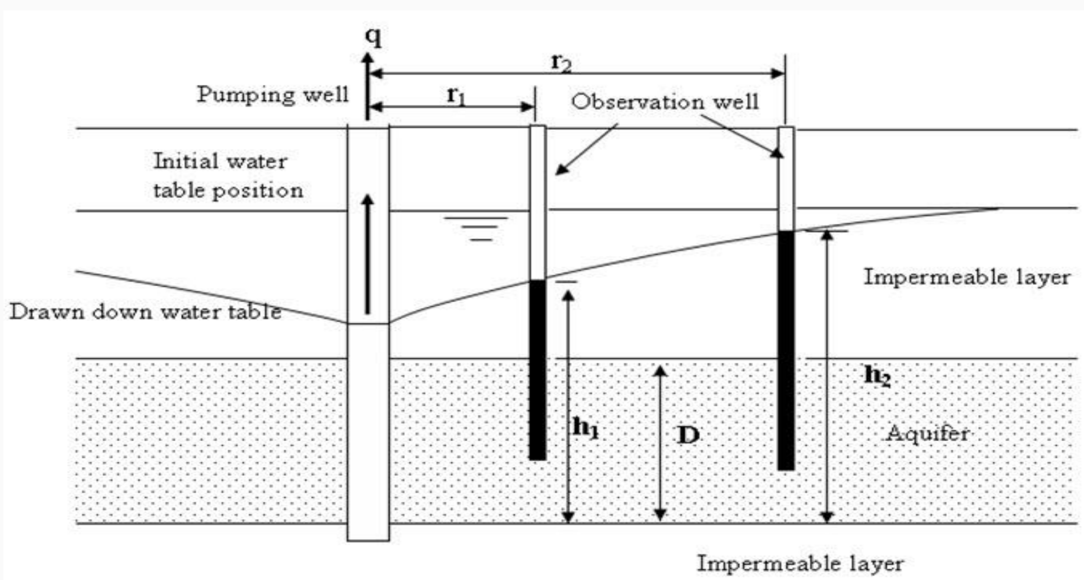


Fig. 18.4. Pumping test in confined aquifer

LESSON 19. Seepage in Soil

19.1 Introduction

When water flows through the soil a drag force called seepage force is generated on the individual soil grains. Figure 19.1 shows the stresses within soil under static condition.

The total vertical stress σ_v at a point X within the soil is: $\sigma_v = \gamma_w h_w + \gamma_{sat} z$ (19.1)

The stress due to water or pore water pressure is: $u = \gamma_w (h_w + z)$ (19.2)

Thus, effective stress is: $\sigma_{v'} = \sigma_v - u = \gamma' z$ (19.3)

where γ_w is the unit weight of water, γ_{sat} the saturated unit weight of soil and γ' is the submerged unit weight of soil ($\gamma' = \gamma_{sat} - \gamma_w$).

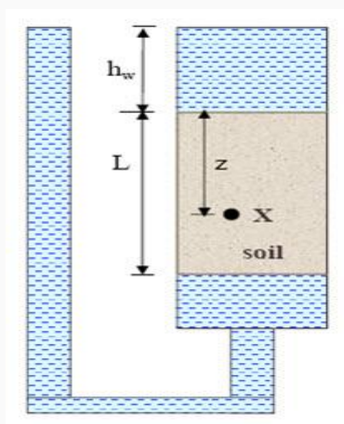


Fig. 19.1. Stresses within the soil under static condition.

Figure 19.2 shows the stresses within soil due to downward flow of the water. The total vertical stress (σ_v) at a point X within the soil is:

$\sigma v = \gamma_w h_w + \gamma_{sat} z$ (19.4)

The pore water pressure at point A is: $u_A = \gamma_w h_w$ (19.5)

The pore water pressure at point B is: $u_B = \gamma_w (h_w + L - h_L)$ (19.6)

Thus, the pore water pressure at point X is:

$u = \gamma_w h_w + \gamma_w (L-h_L)(z/L) = \gamma_w h_w + \gamma_w (z - iz)$
$= \gamma_w (h_w + z) - \gamma_w iz$ (19.7)

Thus, effective stress is: $\sigma_v' = \sigma_v - u = \gamma' z + \gamma_w iz$
(19.8)

where i is the hydraulic gradient. $\gamma_w iz$ is the amount of reduction in pore water pressure and amount of increment in effective stress due to downward water flow.

Figure 19.3 shows the stresses within soil due to upward flow of the water.

The total vertical stress (σ_v) at appoint X with the soil is:
$\sigma_v = \gamma_w h_w + \gamma_{sat} z$ (19.9)

The pore water pressure at point A is: $u_A = \gamma_w h_w$ (19.10)

The pore water pressure at point B is: $u_B = \gamma_w (h_w + L + h_L)$ (19.11)

Thus, The pore water pressure at point X is:

$u = \gamma_w h_w + \gamma_w (L + h_L)(z/L) = \gamma_w h_w + \gamma_w (z + iz)$
$= \gamma_w (h_w + z) + \gamma_w iz$ (19.12)

Thus, effective stress is: $\sigma v' = \sigma_v - u = \gamma' z - \gamma g_w iz$
(19.13)

where i is the hydraulic gradient. $\gamma_w iz$ is the amount of increment in pore water pressure and amount of reduction in effective stress due to upward water flow.

Eq. (19.13) can be written as:

$${\sigma '_v}={\gamma _w}z\left({{{\gamma '} \over {{\gamma _w}}} - i} \right)$$
(19.14)

In Eq. (19.14), γ' / γ is called "Critical Hydraulic Gradient (i_c). If $i > i_c$, the effective stresses become negative. Thus, there is no inter granular contact between the soil grains and failure takes place. This is called **"Quick Condition"** for granular soil.

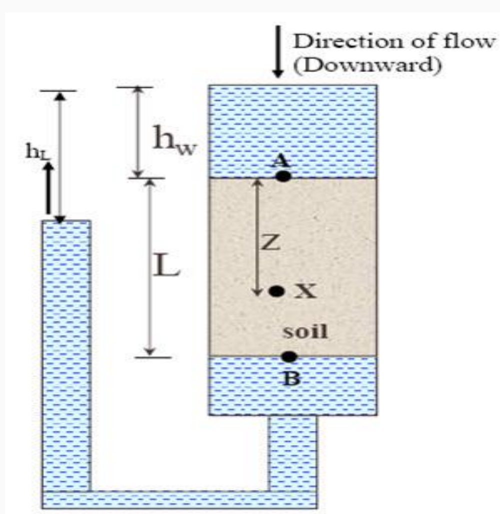


Fig. 19.2. Stresses within the soil due to downward water flow.

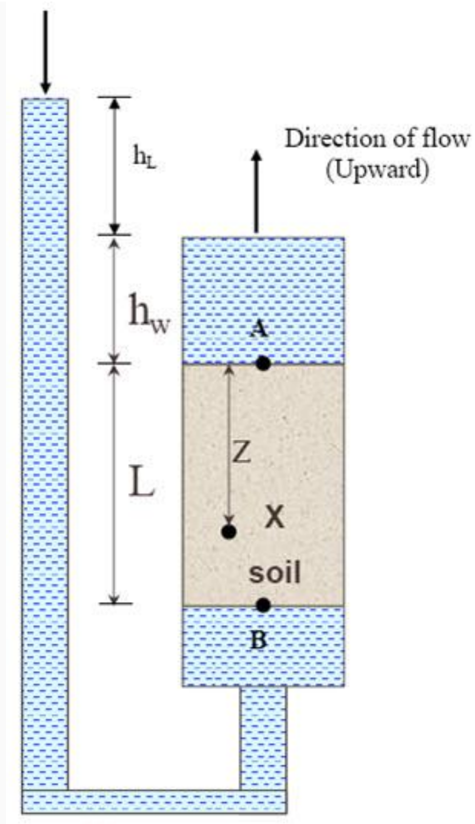


Fig.19.3. Stresses within the soil due to upward water flow.

19.2 Laplace's Equation

This equation is valid for two-dimensional flow when soil mass is fully saturated and Darcy's law is valid. The soil mass is homogeneous and isotropic, soil grains and pore fluid are assumed to be incompressible. Flow condition does not change with time i.e. steady state condition exists. The equation can be written as (under the assumed conditions):

$${{\partial^2 h} \over {\partial x^2}} + {{\partial^2 h} \over {\partial z^2}} = 0 \qquad (19.15)$$

where *h* is the head loss in *x* and *z* direction. The solution of Laplace equation gives two sets of curves perpendicular to each other. One set is known as flow lines and other set is known as equipotential lines. The flow lines indicate the direction of flow and equipotential lines are the lines joining the points with same total potential or elevation head.

LESSON 20. Flow Net

20.1 Introduction

The flow lines indicate the direction of flow and equipotential lines are the lines joining the points with same total potential or elevation head. From upstream to downstream, total head steadily decreases along the flow line. A network of selected flow lines and equipotential lines is called flow net (as shown in Figure 20.1)

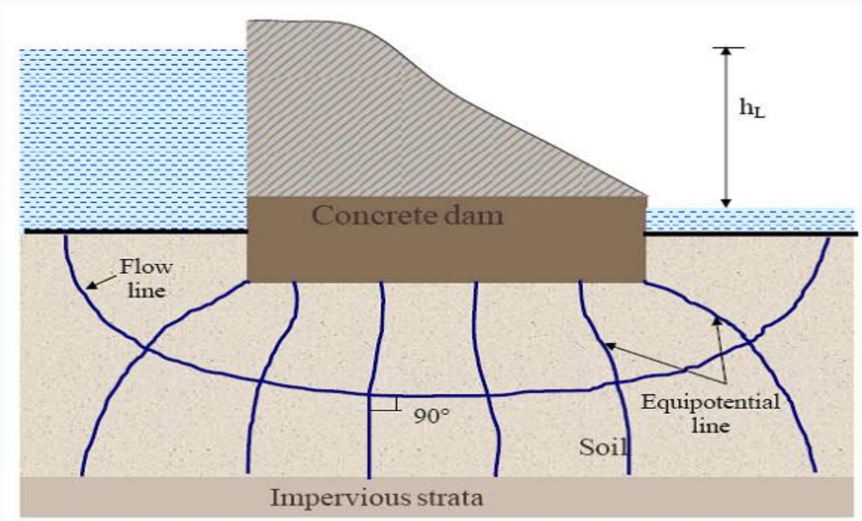


Fig.20.1. Flow net.

From the flow net, the quantity of seepage (Q) is calculated as:

\[Q=k{h_L}{{{N_f}} \over {{N_d}}}\]   (20.1)

where k is the coefficient of permeability of soil, h_L is head loss from upstream to downstream, N_f is number of flow channels per unit length normal to the plane (in the Figure 20.2, N_f = 2), N_d is the number of equipotential drops (in the Figure 20.2, N_d = 7). If h_L is the total head loss during the flow and N_d is the number of equipotential drops, then $\Delta h = h_L / N_d$ (as shown in Figure 20.2).

Total head at a point X = h_L - number of drops from upstream × Δh (20.2)

Elevation head = $-z$ (20.3)

Pressure head = Total head – Elevation head   (20.4)

It is not necessary to make all the filed elementary squares in a flow net, but a/b ratio (as shown in Figure 20.2) should be same for all the filed elements. If $a/b = n$, then Eq. (20.1) can be written as:

$$Q=k{h_L}{{{N_f}} \over {{N_d}}}n \qquad (20.5)$$

At the downstream, near the dam (as shown in Figure 20.3), the exit hydraulic gradient (i_{exit}) can be determined as:

$${i_{exit}} = {{\Delta h} \over {\Delta L}} \qquad (20.6)$$

If i_{exit} is greater than the critical hydraulic gradient (i_c), the soil grains at exit get washed away. This phenomenon progresses towards the upstream and forming a free passage of water. This is called Piping in granular soils. The safety factor against piping can be determined as:

$${F_{piping}}={{{i_c}} \over {{i_{exit}}}} \qquad (20.7)$$

The typical range of F_{piping} is 5 to 6.

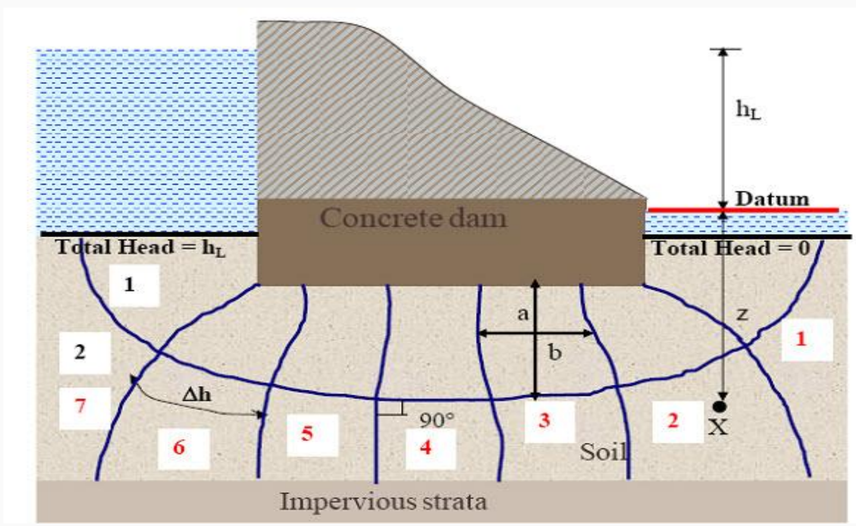


Fig. 20.2. Use of flow net.

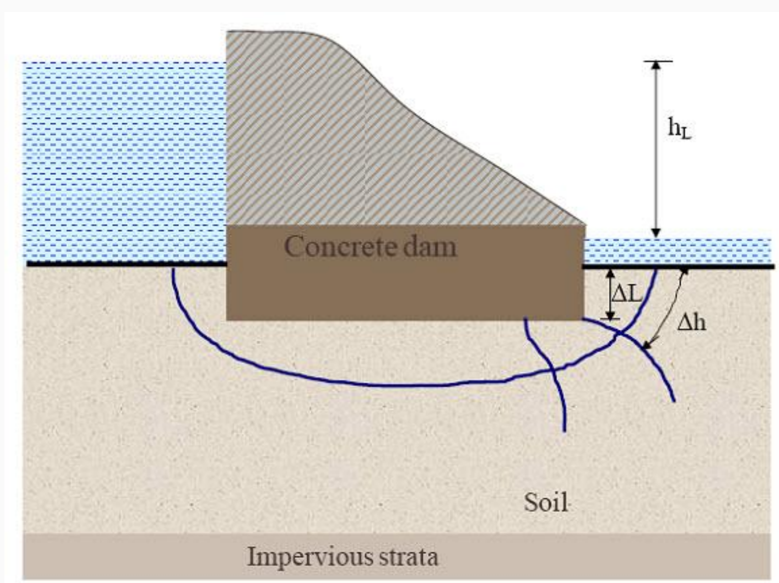


Fig. 20.3 Piping in granular soil.

20.2 Seepage in Anisotropic soil

The Laplace equation presented in previous lesson (lesson 19) is valid for isotropic soil. If soil is anisotropic and coefficient of permeability in *x* and *z* direction is not same, the Laplace equation is modified as:

$${k_x}{{{\partial ^2}h} \over {\partial {x^2}}} + {k_z}{{{\partial ^2}h} \over {\partial {z^2}}}=0 \qquad (20.8)$$

where k_x and k_z are the coefficient of permeability in *x* and *z* direction, respectively. The Eq. (20.8) can be written as:

$$\frac{{{\partial ^2}h}}{{\frac{{{k_z}}}{{{k_x}}}\partial{x^2}}}+\frac{{{\partial^2}h}}{{\partial {z^2}}}=0 \qquad (20.9)$$

Convert the x a new coordinate system x' such that

$$x'=x\sqrt {{{{k_z}} \over {{k_x}}}} \qquad (20.10)$$

and $\partial {x'^2}=\partial {x^2}{{{k_z}} \over {{k_x}}}$, Thus, Eq.(20.9) can be written as:

$${{{\partial ^2}h} \over {\partial {{x'}^2}}} + {{{\partial ^2}h} \over {\partial {z^2}}}=0 \qquad (20.11)$$

The Eq.(20.11) is Laplace equation for isotropic soil w.r.t x' and z coordinates. Here x coordinate is transformed to x' coordinate [as per Eq. (20.10)] for converting anisotropic soil medium into a fictitious isotropic medium (by keeping z coordinate unchanged). Thus, during the coordinate transformation horizontal dimension (x dimension) is multiplied by $\sqrt {{{{k_z}} \over {{k_x}}}}$. The value of coefficient of permeability for transformed section is taken as:

$$k'=\sqrt {{k_x}{k_z}} \qquad (20.12)$$

Thus, in this case the quantity of seepage (Q) is calculated as:

$$Q=\sqrt {{k_x}{k_z}} {h_L}{{{N_f}} \over {{N_d}}} \qquad (20.13)$$

20.3 Seepage in Non-Homogeneous Section

Figure 20.4 shows the Change in direction of flow lines at intersection of two soil layers with different permeability. In such situation, the condition is:

$${{{k_1}} \over {{k_2}}}={{\tan {\alpha _1}} \over {\tan {\alpha _2}}} \qquad (20.14)$$

If $k_1 > k_2$, the flow lines get defected towards the normal after the intersection (i.e. $a_1 > a_2$). Similarly, If $k_1 < k_2$, the flow lines get defected away from the normal after the intersection (i.e. $a_1 < a_2$). If the permeability of one layer is 10 times more than the permeability of other layer, then it is assumed that no resistance for lowing of water is offered from more pervious layer, thus, no deflection correction is necessary.

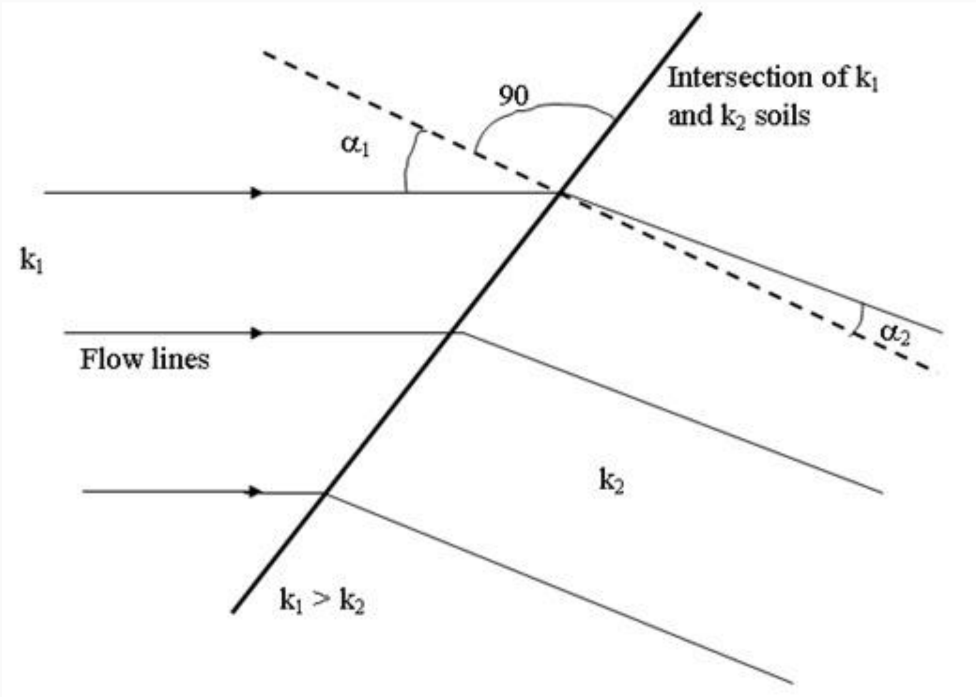


Fig.20.4. Change in direction of flow lines at intersection of two soil layers with different permeability.

LESSON 21. Consolidation of Soil

21.1 Introduction

When soil is loaded due to the load coming from superstructure, the soil volume will decrease due to the change of particle arrangement in the soil. If both soil particle and water within the soil voids are assumed to be incompressible and soil is completely saturated, then volume change will occur due to the removal of water from the soil voids as a result of externally applied loading. Due to the volume change a downward deformation will take place which causes settlement of the superstructure (as shown in Figure 21.1 and Figure 21.2). The rate of volume change depends on the permeability of soil. Thus, consolidation is a major issue in case of clay due to its low permeability. Consolidation is a time-dependent phenomenon of soil. In the Figure 21.1, the vertical strain e can be written as:

$$\varepsilon={{\Delta H} \over {{H_0}}} \qquad (21.1)$$

where H_0 is the initial thickness of the soil and H is the change in thickness. From the phase diagram (as shown in Figure 21.1), the vertical strain can be written as:

$$\varepsilon={{\Delta e} \over {1 + {e_0}}} \qquad (21.2)$$

where Δe is the change in volume of void (or change in void ratio) due to the removal of water from soil pores, e_0 is the initial volume of voids (or initial void ratio). The volume of solid is considered as 1. Thus, volume of voids is e (void ratio, e = volume of voids/volume of solids). The vertical settlement of the soil due to consolidation can be written as:

$$\Delta H={H_0}{{\Delta e} \over {1 + {e_0}}} \qquad (21.3)$$

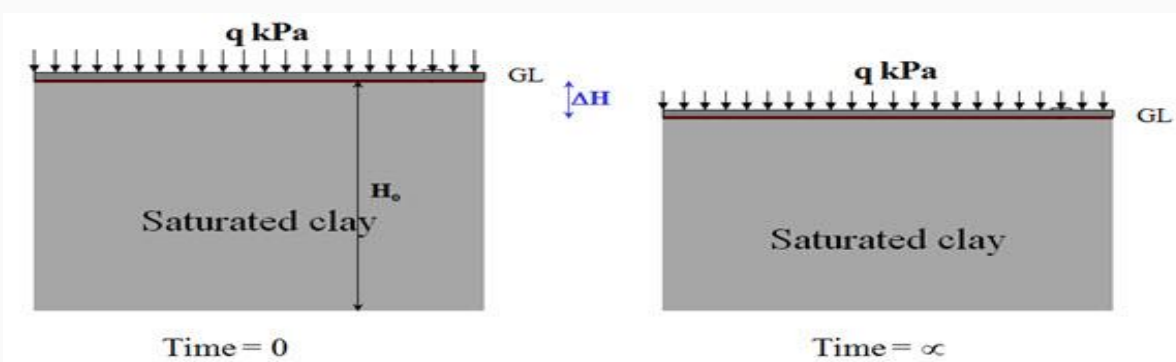


Fig. 21.1 Consolidation of saturated clay

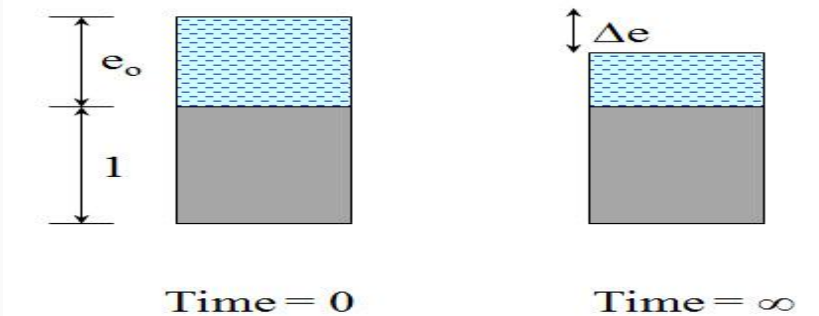


Fig. 21.2 Phase diagram of consolidation of saturated clay

21.2 Settlement of Soil

The total settlement of the soil is the summation of three settlements (i) immediate settlement (ii) primary consolidation settlement and (iii) secondary consolidation settlement or settlement due to creep. Immediate (or elastic) settlement occurs almost immediately after the loading is applied due to the distortion of the soil without any volume change due to removal of water. The time–dependent settlement due to the removal of water from a loaded saturated soil is known as primary consolidation settlement. The primary consolidation depends on the permeability and compressibility of the soil. Some soil (such as peat or soft organic clay) shows time-dependent settlement under constant effective stress during the post primary consolidation period. The settlement during post primary consolidation period is known as secondary consolidation settlement or creep. Thus, the total settlement (S_t) can be written as:

$$S_t = S_i + S_c + S_s \qquad (21.3)$$

where S_i is the immediate settlement, S_c is the settlement due to the primary consolidation and S_s is the settlement due to the secondary consolidation. In general for granular soil, $S_t = S_i$.

21.3 Factors Affecting Consolidation

The following factors affect the consolidation:

- Type of soil
- Stress history
- Effective stress

21.4 Comparisons between Consolidation and Compaction

Compaction is almost an instantaneous phenomenon, whereas consolidation is a time-dependent phenomenon. In case of consolidation soil is always saturated, whereas in case of compaction soil is always unsaturated. Consolidation is the reduction of water voids, whereas compaction is the reduction of air voids. For compaction specified compaction techniques are used, whereas consolidation occurs due to application load on the soil.

LESSON 22. One Dimensional Consolidation

22.1 Terzaghi's Theory

By using the Terzaghi's theory the rate of consolidation can be determined. The assumptions are:

- The compression and flow are 1-D only (in the vertical direction only)
- The soil is homogeneous
- The soil is completely saturated
- Dercy's law is valid
- Both water inside the pore and soil grains are incompressible
- Strains are small
- Void ratio decreases with the increase in applied stress. Hence change in void ratio, $de = -a_v \, d\bar \sigma$, where a_v is the coefficient of compressibility.

Figure 22.1 shows a clay layer in between two sand layers (drainage layers). Clay layer is subjected a uniformly distributed stress σ. A small soil element of dimension *dx*, *dy* and *dz* is chosen at a depth of '*z*' from the top of the clay layer or consolidating layer. The continuity equation of the element can be written as:

$$\left({{\partial {v_x}} \over {\partial x}} + {{\partial {v_y}} \over {\partial y}} + {{\partial {v_z}} \over {\partial z}} \right)dx\,dy\,dz={{\partial V} \over {\partial t}} \qquad (22.1)$$

where v_x, v_y and v_z are the velocity in x, y and z direction, respectively. δV is the change in volume. For 1-D consolidation, Eq. (22.1) can be written as:

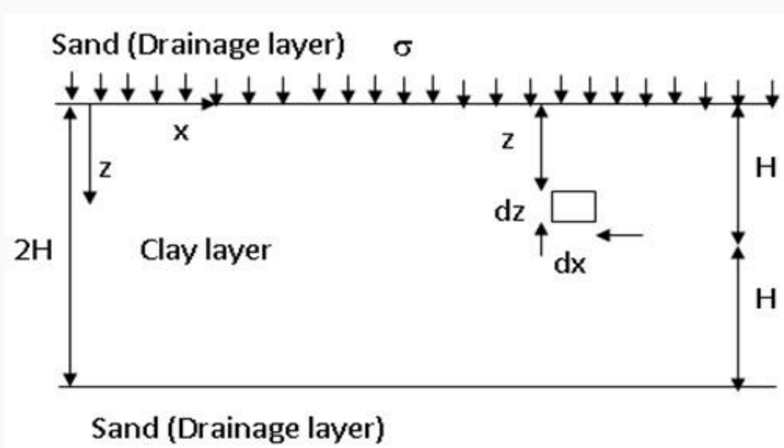


Fig. 22.1. 1-D consolidation.

$$\left({{{\partial {v_z}} \over {\partial z}}} \right)dx\,dy\,dz={{\partial V} \over {\partial t}} \qquad (22.2)$$

Now according to Dercy's law,

$${v_z}={k_z}{{\partial h} \over {\partial z}} \qquad (22.3)$$

where k_z is the coefficient of permeability, h is the head which causes flow during consolidation. The Eq. (22.3) can be written as:

$${{\partial {v_z}} \over {\partial z}}={k_z}{{{\partial ^2}h} \over {\partial {z^2}}} \qquad (22.4)$$

The head h can be expressed as:

$$h={u \over {{\gamma _w}}} \qquad (22.5)$$

where 'u' is the excess pore water pressure and γ_w is the unit weight of water. Combining Eq. (22.2) and Eq. (22.4), one can write

$${{{k_z}} \over {{\gamma _w}}}{{{\partial ^2}u} \over {\partial {z^2}}}dx\,dy\,dz={{\partial V} \over {\partial t}} \qquad (22.6)$$

Now rate of change of volume can be written as:

$${{\partial V} \over {\partial t}}={\partial\over {\partial t}}\left({dx\,dy\,dz} \right) \qquad (22.7)$$

If volume of soil element $V = dx\,dy\,dz$ and e_0 is the initial void ratio, then the volume of voids (V_v) can be written as:

$${V_v}={{{e_0}}\over {1 + {e_0}}}dx\,dy\,dz \qquad (22.8)$$

When V_v is to experience change, e will be a variable. Thus, Eq. (22.8) can be written as:

$${V_v}={e \over {1 + {e_0}}}dx\,dy\,dz \qquad (22.9)$$

The change in volume is due to the change in volume of voids. Thus, combining Eq. (22.7) and Eq. (22.9) one can write

$${{\partial V}\over{\partial t}}={\partial\over{\partial t}}\left({{e\over{1 + {e_0}}}dx\,dy\,dz}\right) \qquad (22.10)$$

Now, volume of solid soil (V_s) grains is a constant and can be expressed as:

$${V_s}={1 \over {1 + {e_0}}}dx\,dy\,dz \qquad (22.11)$$

Thus, Eq. (22.10) can be written as:

$$ {{\partial V}\over{\partial t}}={\partial\over{\partial t}}\left({{e\over{1 + {e_0}}}dx\,dy\,dz}\right)={{dx\,dy\,dz}\over{1 + {e_0}}}{{\partial e}\over {\partial t}} \qquad (22.12)$$

A time t =0, applied stress is equal to excess pore water pressure. Thus, $du = -d\sigma$, the negative sign means as the excess pore water pressure decreases effective stress increases. Again, $de = -a_v \, d\bar \sigma$. Thus, $de = a_v \, du$. Eq. (22.12) can be written as:

$$ {{\partial V}\over{\partial t}}={{dx\,dy\,dz}\over{1+{e_0}}}{{\partial e}\over{\partial t}}={{dz\,dy\,dz}\over{1 + {e_0}}}{a_v}{{\partial u}\over {\partial t}} \qquad (22.13)$$

Combining Eq. 22.6) and Eq. (22.13), one can get

$$ {{{k_z}}\over{{\gamma _w}}}{{{\partial ^2}u}\over{\partial{z^2}}}dx\,dy\,dz={{dz\,dy\,dz}\over {1 + {e_0}}}{a_v}{{\partial u}\over{\partial t}} \qquad (22.14)$$

Thus,

$$ {{\partial u}\over{\partial t}}={{{k_z}}\over {{\gamma _w}}}{{1 + {e_0}}\over {{a_v}}}{{{\partial ^2}u}\over{\partial {z^2}}} \qquad (22.15)$$

$$ {{\partial u} \over {\partial t}}={c_v}{{{\partial ^2}u} \over {\partial {z^2}}} \qquad (22.16)$$

where c_v is the coefficient of consolidation and can be expressed as:

$$ {c_v}={{{k_z}}\over{{\gamma _w}}}{{1 + {e_0}}\over {{a_v}}}={{{k_z}}\over{{\gamma _w}{m_v}}} \qquad (22.17)$$

where m_v is the compressibility of soil and can be expressed as:

$$ {m_v}={{{a_v}} \over {1 + {e_0}}} \qquad (22.18)$$

The boundary and initial conditions are:

- At $t = 0$, $u = u_i$ and $u_i = \sigma$
- At $t \rightarrow \propto$, $u = 0$ for all z
- $t > 0$, $z = 0$, $u = 0$
- $t > 0$, $z = 2H$, $u = 0$ (if drainage layer is present in top and bottom of the consolidating layer).

In Terzaghi's solution, three non-dimensional factors are presented as:

Drainage depth ratio, $Z={z \over H}$ (22.19)

Tine factor, $T_v = \frac{c_v t}{H^2}$ (if drainage layer is present in top and bottom of the consolidating layer) (22.20)

Time factor, $T_v = \frac{c_v t}{(2H)^2}$ (22.21)

Thus, $t \propto \frac{H^2}{c_v}$ (22.22)

Degree of consolidation, $U_z = \frac{u_i - u_z}{u_i} = 1 - \frac{u_z}{u_i}$ (22.23)

In Terzaghi's solution, U_z is expressed in terms of Fourier series as:

$U_z = 1 - \sum\limits_{n=0}^{\propto} f_1(Z) f_2(T_v)$ (22.24)

- At $t = 0$, $T_v = 0$ and $U_z = 0$ for all values of Z
- At $t \rightarrow \propto$, $T_v \rightarrow \mu$ and $U_z = 1$ or 100% for all values of Z

Thus, degree of consolidation is a function of T_v. For example, U = 50% if T_v = 0.197 and U = 90% if T_v = 0.848. Keeping soil parameters constant, as the time increases degree of consolidation also increases.

LESSON 23. Consolidation Test

23.1 Introduction

In the laboratory, the consolidation of a saturated soil specimen is conducted in a device called oedometer as shown in Figure 23.1. The test is done for undisturbed soil sample collected from field. The test is done by applying incremental external loading as shown in Figure 23.2. After allowing full consolidation, the next increment of loading is applied [as shown in Figure 23.2 (a)]. The change in void ratio (Δe_1) due to application of loading increment, Δq_1 can be determined as:

$$\Delta {e_1} = {{\Delta {H_1}} \over H}(1 + {e_0}) \qquad (23.1)$$

where H is the initial thickness of the soil sample, ΔH_1 is the change in thickness or height (compression) of the sample due to the application of Δq_1, e_0 is the initial void ratio of the sample. After completing the loading test, unloading test is also conducted as shown in Figure 23.2(b). In case of loading, as the stress increases void ratio decreases and for unloading case, as the stress decreases void ratio increases. After determining the void ratio (e) due to application of incremental pressure (p), e – log σ_v' is plotted (as shown in Figure 23.3) for both loading and unloading conditions. The σ'_v is the effective vertical stress which is determined from the applied incremental loading. The slope of the straight portion of the loading curve is compression index (C_c) and slope of the unloading curve is swelling index. Thus, the Compression index can be determined as: $C_c = (e_1 - e_2) / (\log \sigma'_2 - \log \sigma'_1)$. According to Skempton (1944): $C_c = 0.009\ (w_L - 10)$, where w_L is the liquid limit.

The coefficient of compressible (a_v) can be determined as:

$$ {a_v}={{\Delta e} \over {\Delta {{\sigma '}_v}}} \qquad (23.2)$$

where Δe is the change in void ratio due to the change in effective vertical stress $\Delta\sigma'_v$. Similarly, the coefficient of volume change or coefficient of volume compressibility (m_v) can be determined as:

$${m_v}={{{a_v}} \over {1 + {e_0}}} = {{\Delta e} \over {\Delta {{\sigma '}_v}}}\left({{1 \over {1 + {e_0}}}} \right) \qquad (23.3)$$

where e_0 is the initial void ratio of the soil sample.

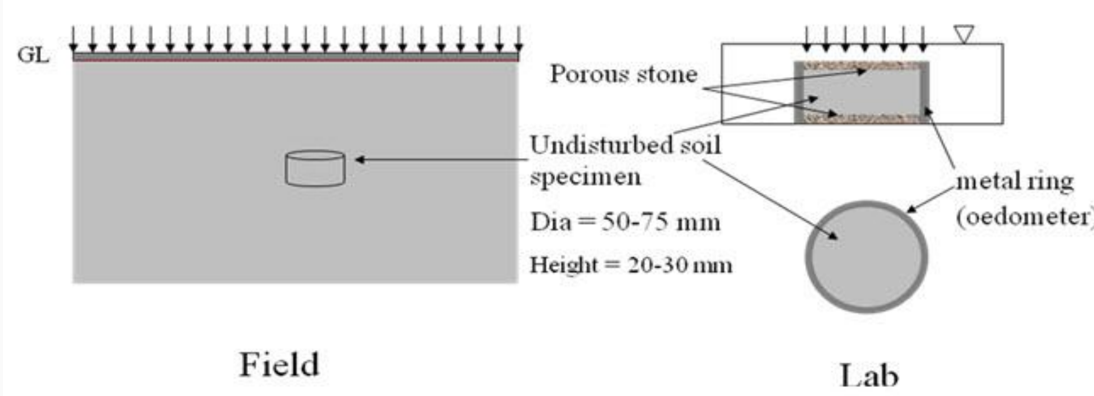


Fig. 23.1. Laboratory consolidation test.

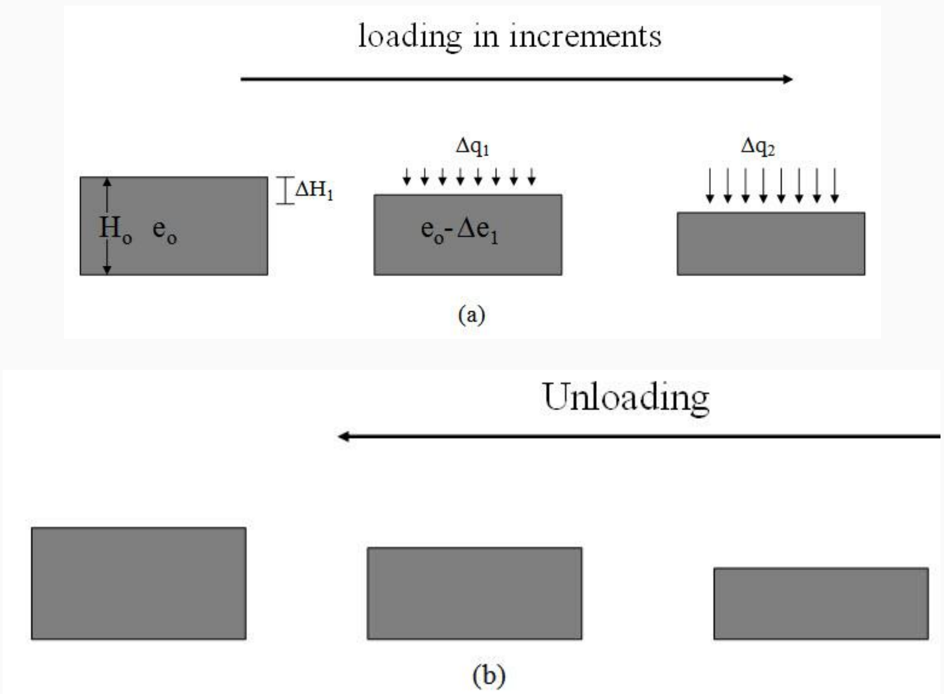


Fig. 23.2. Loading arrangement (a) Loading (b) Unloading.

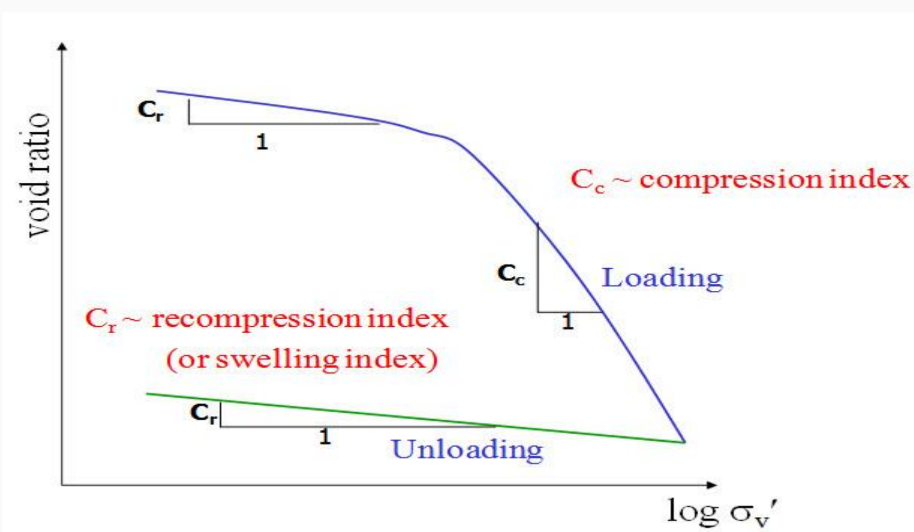


Fig. 23.3. e – log σ_v' plot.

23.2 Over Consolidation Ratio

Casagrande (1936) proposed to determine the pre-consolidation stress from e – log σ_v' plot as shown in Figure 23.4. Locate the point O where there is smallest radius of curvature. Draw a line OA parallel to stress or x axis. Draw a tangent OB at point O. Draw the line OC such that it bisects the angle AOB. Extend the straight portion of the e – log σ_v' plot. The stress corresponding to the point of intersection of extended line and OC line is the pre-consolidation pressure the soil was subjected. If the present effective overburden pressure σ'_{v0} is equal to the preconsolidated pressure σ_c' (maximum past effective overburden pressure), the soil is normally consolidated soil. However, if $\sigma'_{v0} < \sigma_c'$, the soil is overconsolidated. Thus, overconsolidation ratio (OCR) is defined as: OCR = σ_c'/σ'_{v0}.

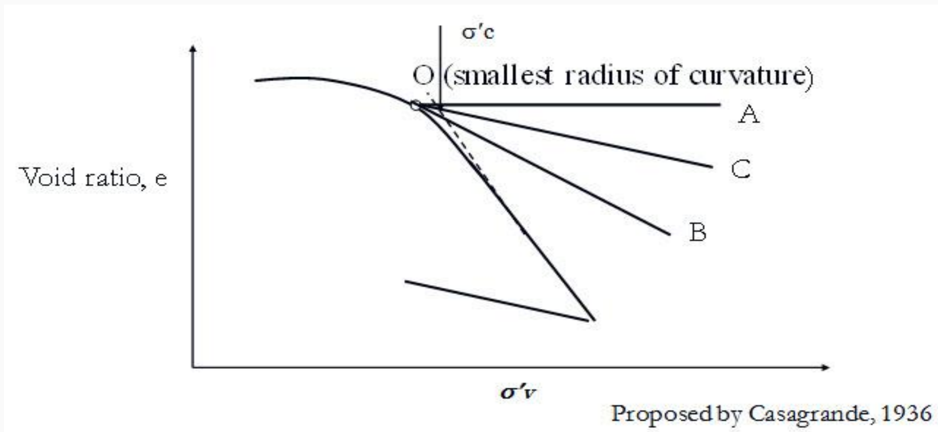


Fig. 23.4. Determination of preconsolidation stress .

23.3 Settlement Calculation

Figure 23.5 shows the e – log σ_v' plots for normally consolidated and overconsolidated clay for settlement calculation. For normally consolidated clay the settlement (S) can be determined as:

$$S={{C_c} \over {1 + {e_0}}}H{\log _{10}}\left({{{{\sigma '}_{v0}} + \Delta {{\sigma '}_v}} \over {{{\sigma '}_{v0}}}} \right) \qquad (23.4)$$

where H is total thickness of the soil layer.

For over-consolidated clay: Case 1: $\sigma'_{v0}+\Delta\sigma'_v < \sigma'_c$

$$S={{C_s} \over {1 + {e_0}}}H{\log _{10}}\left({{{{\sigma '}_{v0}} + \Delta {{\sigma '}_v}} \over {{{\sigma '}_{v0}}}} \right) \qquad (23.5)$$

Case 2: $\sigma'_{v0} < \sigma'_c < \sigma'_{v0}+\Delta\sigma'_v$

$$S={{C_s} \over {1 + {e_0}}}H{\log _{10}}\left({{{{\sigma '}_c}} \over {{{\sigma '}_{v0}}}} \right) + {{C_c} \over {1 + {e_0}}}H{\log _{10}}\left({{{{\sigma '}_{v0}} + \Delta {{\sigma '}_v}} \over {{{\sigma '}_c}}} \right) \qquad (23.6)$$

where C_s is the swelling index.

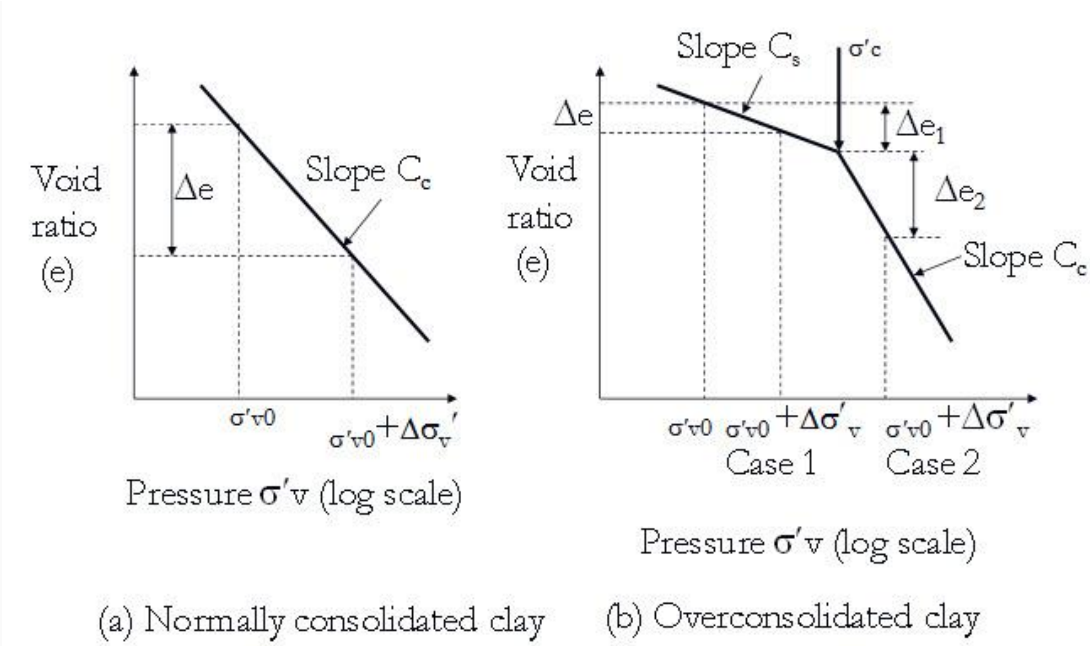


Fig. 23.5. Settlement calculation (a) Normally consolidated clay

(b) Overconsolidated clay .

LESSON 24. Determination of Coefficient of Consolidation

24.1 Taylor's Square Root of Time Fitting Method

From the oedometer test (explained in Lesson 23) the dial reading (settlement) corresponding to a particular time is measured. From the measured data, dial reading vs $\sqrt {Time}$ graph can be drawn (as shown in Figure 24.1). A straight line can be drawn passing through the points on initial straight portion of the curve (as shown in Figure 24.1). The intersection point between the straight line and the dial reading axis is denoted as R_0 which is corrected zero reading i.e U = 0%. Starting from R_0, draw another straight line such that its abscissa is 1.15 times the abscissa of first straight line. The intersection point between the second straight line and experimental curve represents the R_{90} and corresponding $\sqrt {{t_{90}}}$ is determined. Thus, the time required (t_{90}) for 90% consolidation is calculated. The Coefficient of consolidation (c_v) is determined as:

$${c_v}={{{T_v}{H^2}} \over t} \qquad (24.1)$$

where H is the thickness of the soil sample, t is required time. T_v is the vertical time factor and can be determine as:

$${T_v}=\left({{\pi\over 4}}\right){U^2}\quad \quad if\;U \le 60\% \quad \% \qquad (24.2)$$

$${T_v}=1.781 - 0.933{\log _{10}}(100 - U)\quad \quad if\;U > 60\% \quad \% \qquad (24.3)$$

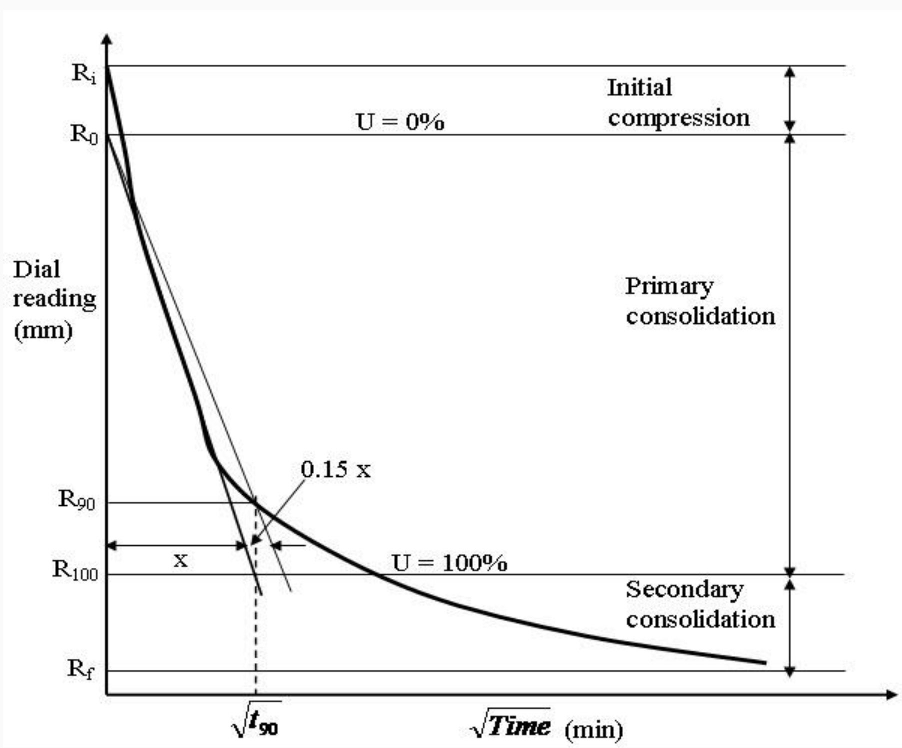


Fig. 24.1. Taylor's Square Root of Time Fitting Method.

24.2 Casagrande's Logarithm of Time Fitting Method

From the oedometer test (explained in Lesson 23) the dial reading (settlement) corresponding to a particular time is measured. From the measured data, dial reading vs time graph can be drawn (as shown in Figure 24.2). Select two points t_1 and t_2 in initial part of the curve such that $t_2 = 4\ t_1$. The points corresponding to the chosen times are marked on the curve. The vertical distance (z) between the two points on the curve is measured. Select another point R_0 such that the vertical distance between that point and point on the curve corresponding to the t_1 time is also z. R_0 is corrected zero reading i.e U = 0%. Determine the U=100% line by drawing two tangents form the straight portion of the curve as shown in Figure 24.2. Once U=0% and 100% lines are identified, U= 50% line is also determined by choosing the middle point between the U=0% and 100% lines. Time (t_{50}) corresponding to the 50% degree of consolidation is determined from the curve. The Coefficient of consolidation (c_v) is determined from Eq.(24.1).

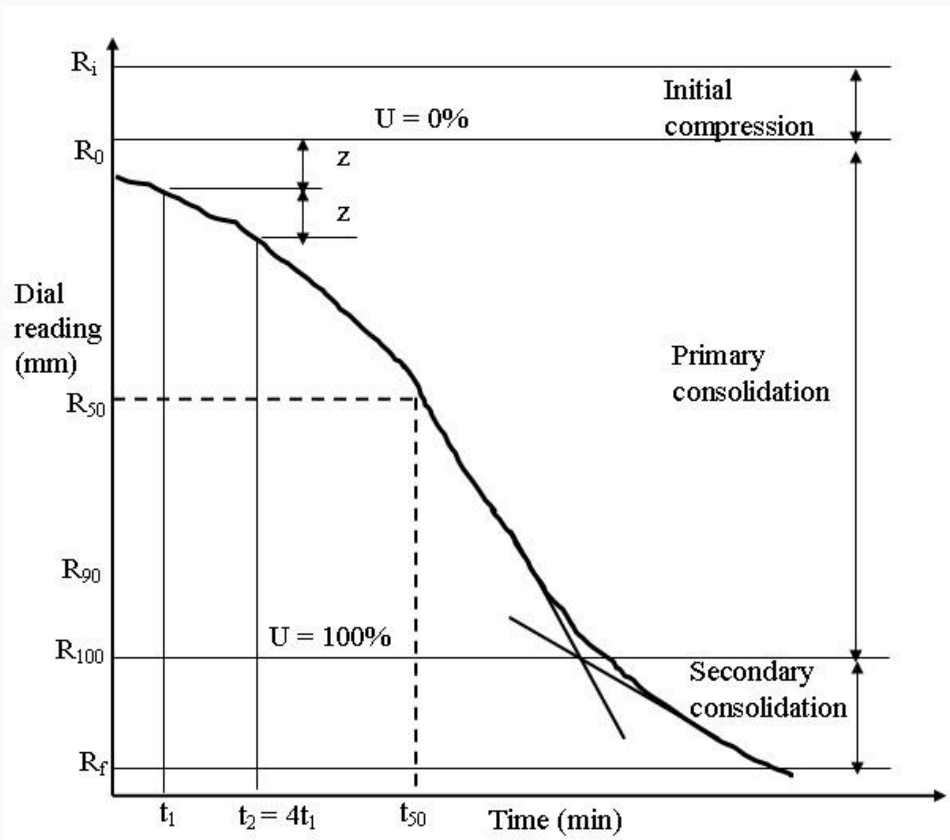


Fig. 24.2. Casagrande's Logarithm of Time Fitting Method.

Problem 1

Determine the total settlement of two layered soil system as shown in Figure 24.3. A strip loading of intensity 50 kN/m[3] of width 3 m is applied at the ground surface. The water table is at a depth of 1.0 m below ground level.

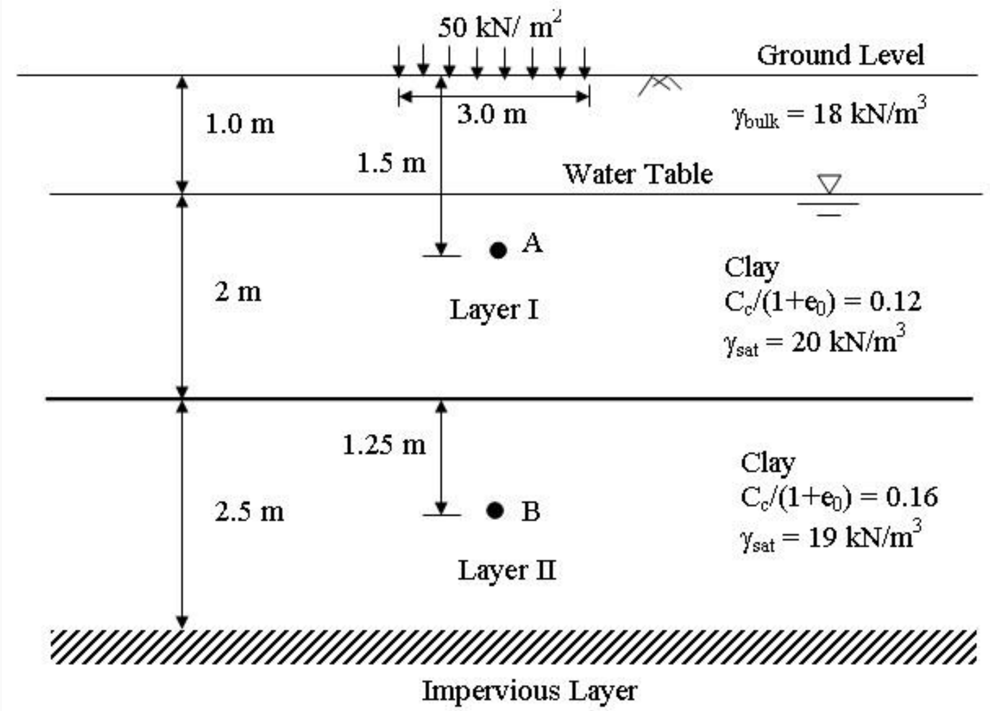


Fig. 24.3. Settlement calculation of two layered soil system.

Solution:

The thickness of the Layer I and II is 3m and 2.5m, respectively. Choose one point at the middle of each normally consolidated clay layer. Here point A is chosen at a depth of 1.5 m from the ground level (if loading is applied at any depth from the ground level, then the point in the layer I will be chosen as middle of the soil layer in between the loading depth and top of the layer II. In layer II, point will be chosen at the middle of the layer II). The point B is chosen at a depth of 1.25m from the top of the layer II (or 4.25 m below the ground level).

Layer I

At point A,

σ'_{v0} = 18 × 1 + (1.5 – 1) × (20 – 10) = 23 kN/m²

$\Delta\sigma'_v$ = 50 × 3 / (3+1.5) = 33.33 kN/m² (taking 1:2 distribution of loading)

H_1 = 3m (if loading is applied at a depth, then H_1 will be taken as total thickness of layer I minus the depth of the loading).

$${S_1}={{{C_c}} \over {1 + {e_0}}}{H_1}{\log _{10}}\left({{{{{\sigma '}_{v0}} + \Delta {{\sigma '}_v}} \over {{{\sigma '}_{v0}}}}} \right) = 0.12 \times 3 \times {\log _{10}}\left({{{23 + 33.33} \over {23}}} \right)=140mm$$

Layer II

At point B,

σ'_{v0} = 18 × 1 + 2 × (20 – 10) + 1.25 (19-10) = 49.25 kN/m^2

$\Delta\sigma'_v$ = 50 × 3 /(3+4.25) = 20.69 kN/m^2 (taking 1:2 distribution of loading)

H_2 = 2.5m (if loading is applied at a depth, then H_2 will be taken as total thickness of layer II).

$${S_2}={{{C_c}} \over {1 + {e_0}}}{H_2}{\log _{10}}\left({{{{{\sigma '}_{v0}} + \Delta {{\sigma '}_v}} \over {{{\sigma '}_{v0}}}}} \right) = 0.16x2.5x{\log _{10}}\left({{{49.25 + 20.69} \over {49.25}}} \right)=60.93mm$$

Total settlement = S_1+S_2= 140 +60.93 = 200.93 mm

MODULE 4. Earth pressure, Slope Stability and Soil Exploration

LESSON 25. Earth Pressure

25.1 Introduction

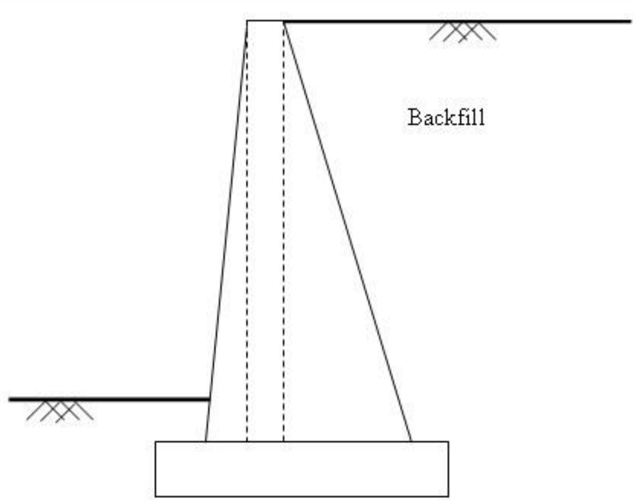
The soil that is retained by various structures like retaining walls, sheet piles (as shown in Figure 25.1) exerts a force on those structures. This force is called the earth pressure and the material that is retained by the structures is referred as backfill. Based on the wall movement, the earth pressure is classified as:

(i) Earth pressure at rest

(ii) Active earth pressure

(iii) Passive earth pressure

In case of a rigid and unyielding retaining wall, soil mass is in rest condition and no deformation or displacement is occurred in the wall. The earth pressure under this condition is called **earth pressure at rest**. If a retaining wall rotates about its toe and moving way from the backfill (as shown in Figure 25.2), the soil mass expends which causes reduction of the earth pressure on the wall. The earth pressure under this condition is called **active earth pressure**. On the other hand if a retaining wall moves towards the backfill (as shown in Figure 25.2), the soil is compressed and the earth pressure on the wall increases. The earth pressure under this condition is called **passive earth pressure**. Thus, the passive earth pressure is more than the earth pressure at rest and the earth pressure at rest is more than the active earth pressure.



(a)

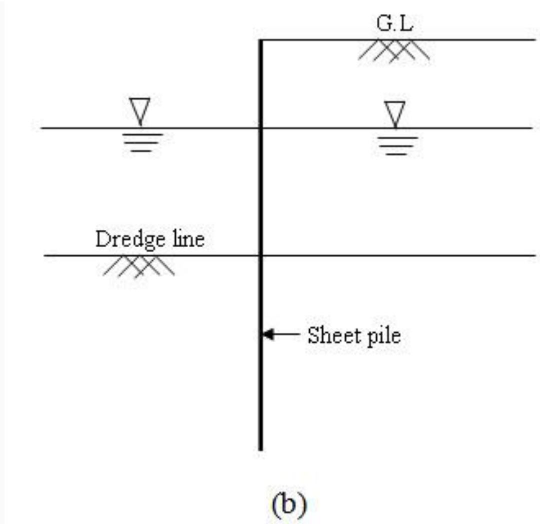


Fig. 25.1. Earth pressure acting on structure like (a) retaining wall (b) sheet Pile.

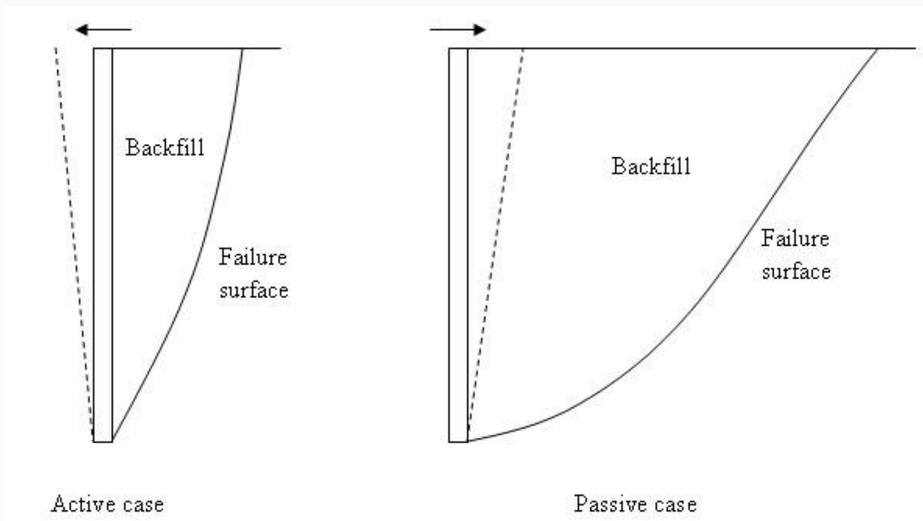


Fig. 25.2. Active and Passive earth pressure.

25.2 Earth Pressure at rest

The earth pressure at rest per unit length at a depth of z from the ground surface is expressed as:

$$p_0 = K_0\,\gamma\,z \qquad (25.1)$$

where γ is the unit weight of the soil, K_0 is the coefficient of earth pressure at rest. Thus, the earth pressure is zero at the top of a retaining wall and at the base of the wall is $K_0\gamma H$ (if H is the height of the wall). The total force per unit length at rest condition acting on a retaining wall of height H is expressed as:

$$P_0 = {1 \over 2}K_0\,\gamma\,H^2 \qquad (25.2)$$

The force (P_0) will act at a height of $H/3$ from the base of the wall.

For sand and normally consolidated clays, the value of K_0 is determined as (Jaky, 1944):

$$K_0 = 1 - \sin \varphi \quad (25.3)$$

where ø is the angle of friction of the soil. For over consolidated soil the K_0 is expressed as:

$$K_0 = K_{0(nc)}\sqrt{OCR} \quad (25.4)$$

where OCR is the over consolidation ratio.

LESSON 26. Rankine's Theory of Earth Pressure

26.1 Introduction

The assumptions of Rankine theory are:

(i) Semi-infinite mass of soil bound by a horizontal surface

(ii) Wall surface is vertical and smooth

(iii) Soil homogeneous, dry and cohesionless

However, the theory is extended for cohesive and submerged soils also. The failure surface for active case and passive case makes an angle of $(45^\circ+\phi/2)$ and $(45^\circ-\phi/2)$, respectively (as shown in Figure 26.1).

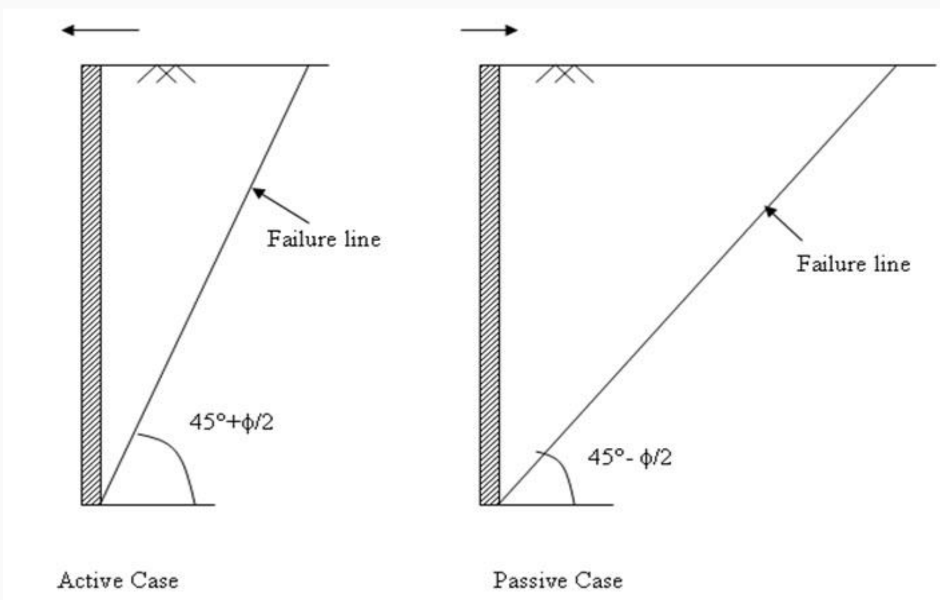


Fig. 26.1. Failure surface of active and passive case.

26.2 Earth Pressure on Retaining Wall

26.2.1. Cohesionless Backfill

Figure 26.2 shows a retaining wall with horizontal backfill with angle of internal friction ϕ and effective unit weight γ'. The height of the wall is H. From Rankine's theory, the active earth pressure (p_A) at a depth of Z can be written as:

$$p_A = K_A \gamma' Z \qquad (26.1)$$

where K_A is the active earth pressure coefficient and can be expressed as:

$$K_A = \frac{1 - \sin\phi}{1 + \sin\phi} \qquad (26.2)$$

Thus, the active earth pressure acting at the base of the wall is $K_A\gamma'H$. The total active forces/unit length can be expressed as:

$$P_A = \frac{1}{2} K_A \gamma' H^2 \qquad (26.3)$$

The force is acting at a height of $H/3$ from the base of the wall.

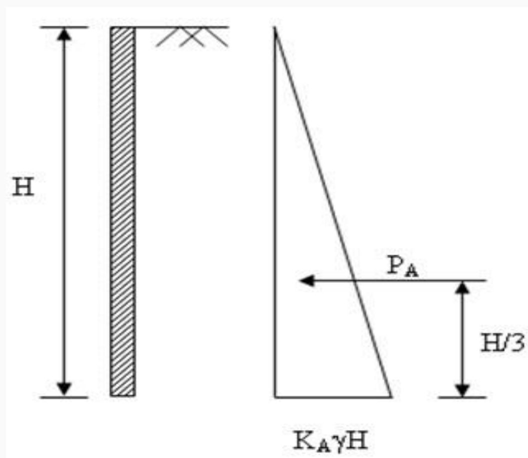


Fig. 26.2. Active pressure acting on retaining wall with horizontal backfill.

For passive case, the passive force/unit length can be expressed as:

$$P_P = \frac{1}{2} K_P \gamma' H^2 \qquad (26.4)$$

where K_P is the coefficient of passive earth pressure and can be expressed as:

$$K_P = \frac{1 + \sin\phi}{1 - \sin\phi} \qquad (26.5)$$

The passive force is also acting at a height of $H/3$ from the base of the wall. Figure 26.3 shows the effect of water table on the active earth pressure. The height of the wall is H and the position of the water table is at a depth of H_1 from the ground surface. The active stress acting at the base of the wall is

$$p_A = K_A\gamma H_1 + K_A\gamma'(H-H_1) + \gamma_w(H-H_1) \qquad (26.6)$$

$$P_{A1} = \frac{1}{2} K_A \gamma' H_1^2 \qquad (26.7)$$

$$P_{A2} = K_A \gamma H_1 (H - H_1) \qquad (26.8)$$

$$P_{A3} = \frac{1}{2}\left\{ K_A\gamma'(H - H_1) + \gamma_w (H - H_1) \right\}(H - H_1) \qquad (26.9)$$

The resultant fore will act at a distance of Z_c from the base of the wall. Thus,

$$Z_c=\frac{P_{A1}\left(H - H_1 + \frac{H_1}{3}\right) + P_{A2}\left(\frac{H - H_1}{2}\right) + P_{A3}\left(\frac{H - H_1}{3}\right)}{P_{A1} + P_{A2} + P_{A3}} \qquad (26.10)$$

Figure 26.4 shows the active earth pressure for surcharge of intensity q/unit area on the ground surface. In such case earth pressure is constant through out the retaining wall. The active force/unit length can be expressed as:

$$P_A = K_A q H \qquad (26.11)$$

In this case P_A is acting at a height of $H/2$ from the base of the wall.

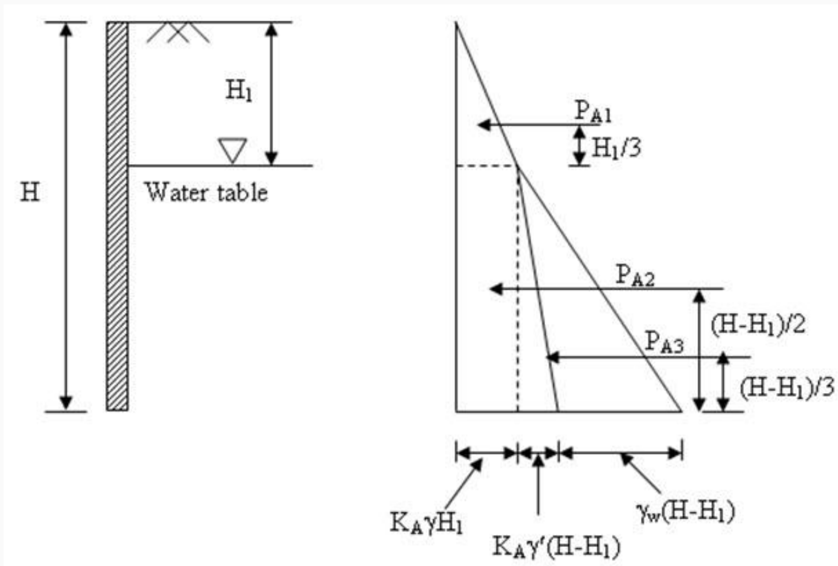


Fig. 26.3. Effect of water table on active earth pressure.

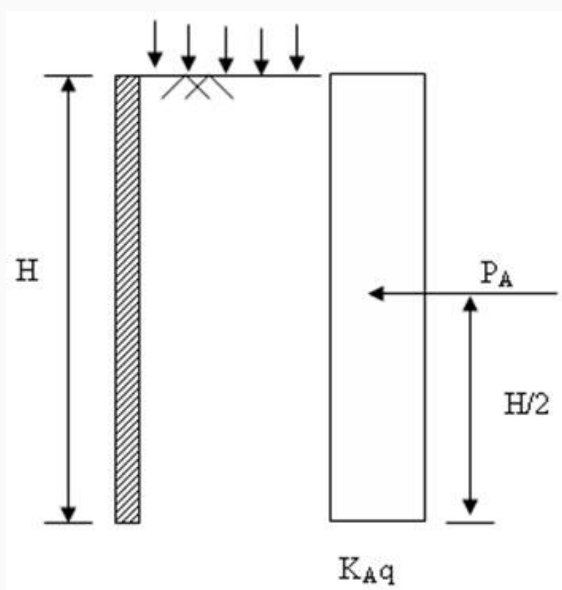


Fig. 26.4. Effect of surcharge.

Figure 26.5 shows a retaining wall with inclined backfill. The active pressure at a depth of *Z* can be written as:

$p_A = K_A \gamma Z \cos i$ (26.12)

where $${K_P}={{\cos i - \sqrt {{{\cos }^2}i - {{\cos }^2}\phi } } \over {\cos i + \sqrt {{{\cos }^2}i - {{\cos }^2}\phi } }} \qquad (26.13)$$

The total active force/unit length can be expressed as:

$${P_A}={1 \over 2}{K_A}\gamma \,{H^2}\cos i \qquad (26.14)$$

Similarly for inclined backfill, K_p can be expressed as:

$${K_P}={{\cos i + \sqrt {{{\cos }^2}i - {{\cos }^2}\phi } } \over {\cos i - \sqrt {{{\cos }^2}i - {{\cos }^2}\phi } }} \qquad (26.15)$$

The total passive force/unit length can be expressed as:

$${P_P}={1 \over 2}{K_P}\gamma \,{H^2}\cos i \qquad (26.16)$$

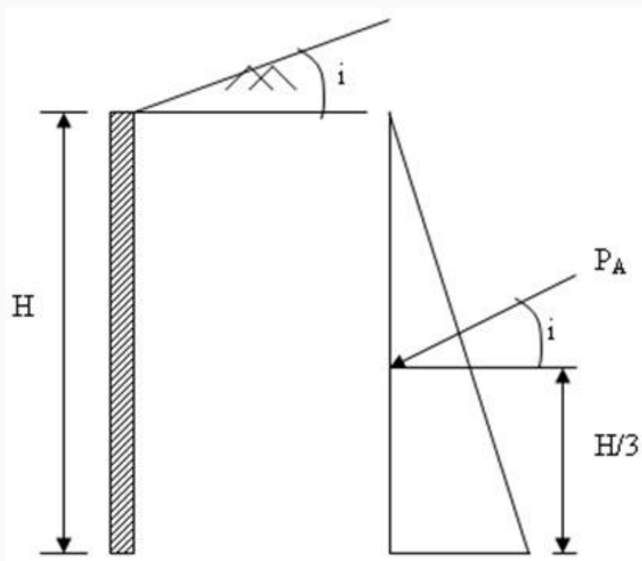


Fig. 26.5. Active earth pressure for inclined backfill.

LESSON 27. Rankine's Theory of Earth Pressure and Numerical Exercise

27.1 Earth Pressure on Retaining Wall

27.1.1. *c-ø* backfill

In case of *c-f* backfill, negative active earth pressure is developed upto the Z_0 depth from the ground level as shown in Figure 27.1. It is clear that the net active pressure is zero upto a depth of $2Z_0$. Thus, in case of cohesive soil, vertical cut can be made with out any lateral supported upto the depth equal to $2Z_0$. The depth $2Z_0$ is called as critical depth of vertical cut H_c in a cohesive soil and can be expressed as:

$$H_c = 2Z_0 = {4c \over \gamma \sqrt{K_A}} \qquad (27.1)$$

where c is the cohesion of the soil, γ is the effective unit weight of the soil and K_A is the coefficient of active earth pressure and can be expressed as:

$$K_A = {1 - \sin \phi \over 1 + \sin \phi} \qquad (27.2)$$

During the calculation of total active force (P_A) in case of *c-ø* backfill, negative zone is neglected and only the active earth pressure due to the positive zone is considered. Thus,

$$P_A = {1 \over 2} K_A \gamma \, \left(H - Z_0\right)^2 = {1 \over 2} K_A \gamma \, H^2 - 2cH\sqrt{K_A} + {2c^2 \over \gamma} \qquad (27.3)$$

P_A acts at the height of $(H-Z_0)/3$ from the base of the wall.

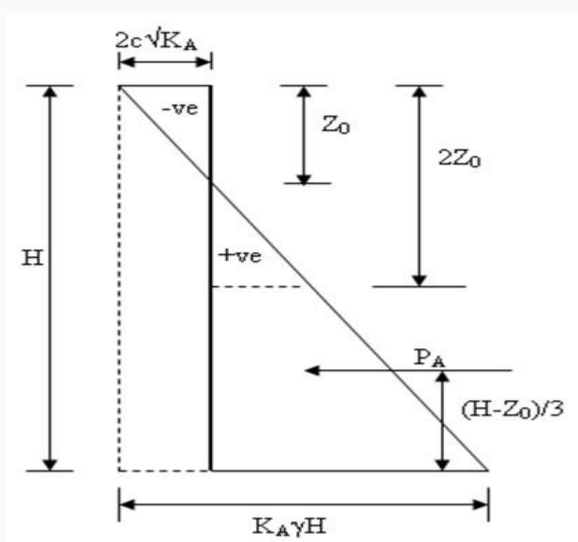


Fig. 27.1. Active earth pressure for *c*-ø backfill.

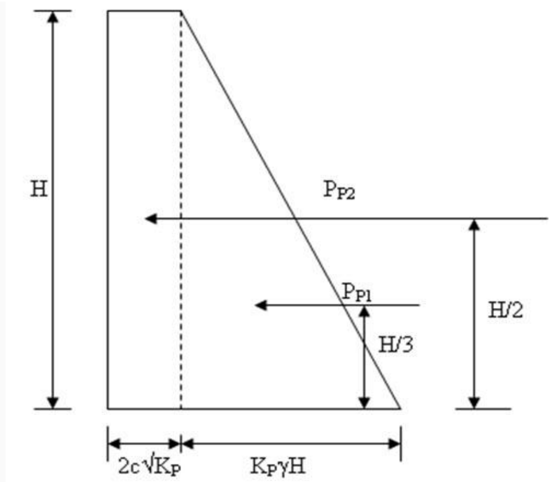


Fig. 27.2. Passive earth pressure for *c*-ø backfill.

Figure 27.2 shows the passive earth pressure distribution for *c*-ø backfill. The passive force P_{P1} and P_{P2} can be determined as:

The passive force P_{P1} and P_{P2} can be determined as:

$${P_{P1}}={1 \over 2}\gamma\,{K_P}{H^2}$$ acts at a height of $H/3$ from the base (27.4)

$${P_{P2}}=2cH\sqrt{{K_P}}$$ acts at a height of $H/2$ from the base (27.5)

Thus the total force P_P can be determined as:

$${P_P}={1\over 2}\gamma {K_P}{H^2} + 2cH\sqrt{{K_P}}$$ (27.6)

where $${K_P}={{1 + \sin \phi}\over{1-\sin\phi}}$$

Problem

Determine the active earth pressure distribution on the retaining as shown in Figure 27.3. Also determine the total active force and point of application of the force.

Solution:

K_{A1} for layer I = $${K_A}={{1-\sin\phi}\over{1+\sin\phi}}$$ = $${{1-\sin 28^\circ}\over{1+\sin 28^\circ}}$$ = 0.36

K_{A2} for layer II = $${K_A}={{1-\sin\phi}\over{1+\sin\phi}}$$ = $${{1-\sin 32^\circ}\over{1+\sin 32^\circ}}$$ = 0.31

Active Pressure distribution for layer I

At Z = 0m, P_A = 0 kN/m^2

At Z = 1.5m, P_A = 0.36 x 18 x 1.5 = 9.72 kN/m^2

At Z = 6 m, P_A (due to soil) = 9.72 + 0.36 x (20 – 10) x 4.5 = 9.72 + 16.2 =25.92 kN/m^2 (take unit weight of the water is 10 kN/m^3).

At Z = 6 m, P_A (due to water) = 4.5 x 10 = 45 kN/m^2

Active Pressure distribution for layer II.

At Z = 6 m, P_A (due to surcharge of the layer I) = 0.31 (1.5 x 18 + 4.5 x 10) = 22.32 kN/m^2

At Z = 6 m, P_A (due to the water Pressure above this level) = 45 kN/m^2

At Z = 11 m, P_A (due to soil) = 67.32 + 0.31 x (20 – 10) x 5 = 67.32 + 15.5 =82.82 kN/m^2

At Z = 6 m, P_A (due to water) = 5 x 10 = 50 kN/m^2

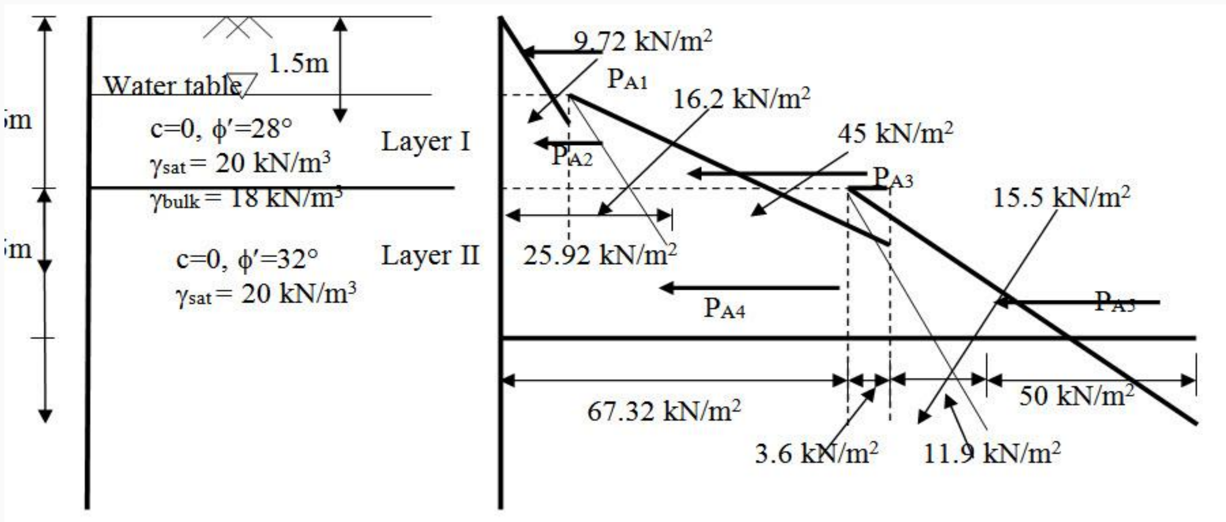


Fig. 27.3. Active earth Pressure distribution of problem 1.

The active force of the various levels is calculated as:

P_{A1} = 0.5 x 9.72 x 1.5 = 7.29 kN/m acts at a height of 10 (=1.5/3+4.5+5) m from the base

P_{A2} = 9.72 x 4.5 = 43.74 kN/m acts at a height of 7.25 (=4.5/2+5) m from the base

P_{A3} = 0.5 x (16.2 + 45) x 4.5 = 137.7 kN/m acts at a height of 6.5 (=4.5/3+5) m from the base

P_{A4} = 67.32 x 5 = 336.6 kN/m acts at a height of 2.5 (=5/2) m from the base

P_{A5} = 0.5 x (15.5 + 50) x 5 = 163.75 kN/m acts at a height of 1.67 (=5/3) m from the base

P_A = P_{A1}+ P_{A2} + P_{A3}+ P_{A4}+ P_{A5} = 7.29 + 43.74 + 137.7 + 336.6 + 163.75 = 689.08 kN/m

Point of aPPlication of the resultant force P_A is

$$= \frac{7.29 \times 10 + 43.74 \times 7.25 + 137.7 \times 6.5 + 336.6 \times 2.5 + 163.75 \times 1.67}{689.08} = 3.48 \text{ m from the base.}$$

LESSON 28. Slope Stability

28.1 Introduction

Slopes can be natural (in hillside and valley, coastal or river cliffs) or man-made (embankments for highways and railways, earth dams, excavation). In case of slopes existing forces cause the soil movements from higher point to lower point and slopes become more stable. If soil mass moves in a slope it is referred as slope failure. The slope failure may occur due to earthquake also. Slopes can be infinite slopes (as shown in Figure 28.1a) and finite slopes (as shown in Figure 28.1b).

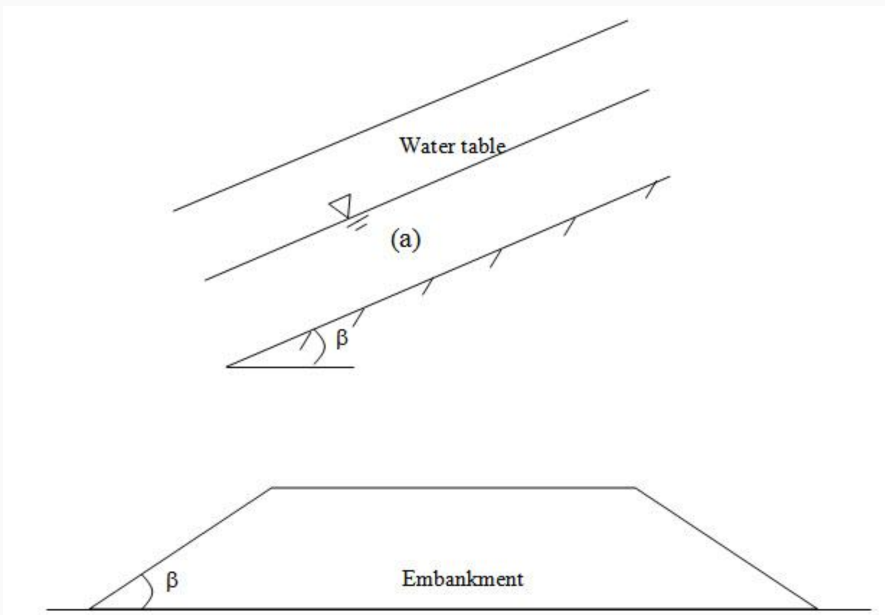


Fig. 28.1. (a) Infinite slope and (b) finite slope.

28.1.1. Infinite Slope

In case infinite slopes failure takes place due to the sliding of soil mass along a plane parallel to the slope at a certain depth and this type of failure is referred as translational slides. Figure 28.2 shows the forces acting on an element in the infinite slope. The soil is assumed to be homogeneous. In the Figure 28.2, *W* is the weight of the element. From the figure it is written as:

$$W=\gamma \,zb\cos \beta \qquad (28.1)$$

$$\sigma _n={{W\cos \beta } \over b}=\gamma \,z{\cos ^2}\beta \qquad (28.2)$$

$$\tau={{W\sin \beta } \over b}=\gamma \,z\cos \beta \sin \beta \qquad (29.3)$$

where σ_n is the normal stress, t is the shear stress and γ is the unit weight of the soil.

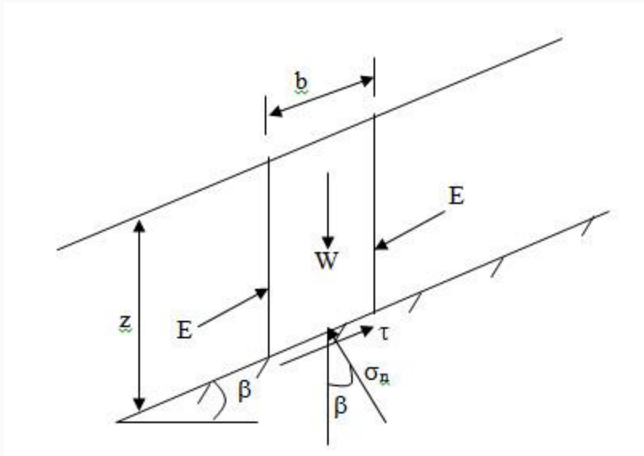


Fig. 28.2. Forces acting on an element in the infinite slope.

Cohesionless Soil

For cohesionless soil, the shear strength can be written as:

$$\tau_f = \sigma_n \tan \phi' = \gamma \, z \cos^2 \beta \tan \phi' \qquad (28.4)$$

where t_f is the failure shear strength and ø' is the angle of shearing resistance or angle of internal friction.

Thus, factor safety (F) can be written as:

$$F = {Shear\;Strength \over Shear\;Stress} = {\gamma \, z {\cos}^2 \beta \tan \phi \over \gamma \, z \cos \beta \sin \beta} = {\tan \phi' \over \tan \beta} \qquad (28.5)$$

Figure 28.3 shows the effective forces acting on an element in the infinite slope. The position of water table is also shown in the figure. The effective normal stress can be written as:

$$\sigma'_n = {(W - \gamma_w hb \cos\beta)\cos\beta \over b} = (\gamma \, z \cos\beta - \gamma_w h \cos\beta)\cos\beta$$

$$= (\gamma \, z - \gamma_w h) \cos^2 \beta \qquad (28.6)$$

$$\tau = {W \sin \beta \over b} = \gamma \, z \cos \beta \sin \beta \qquad (28.7)$$

where Y_w is the unit weight of water.

$$F = {Shear\;Strength \over Shear\;Stress} = {{\sigma'}_n \tan \phi \over \tau} = {(\gamma \, z - \gamma_w h){\cos}^2 \beta \tan\phi \over \gamma \, z \cos \beta \sin \beta} = \left(1 - {\gamma_w h \over \gamma \, z}\right){\tan\phi' \over \tan\beta} \qquad (28.8)$$

If water table is at the top *i.e* $z=h$ then,

$$F=\left({1 - {{{\gamma _w}} \over \gamma }} \right){{\tan \phi '}\over{\tan \beta }}$$ (28.9)

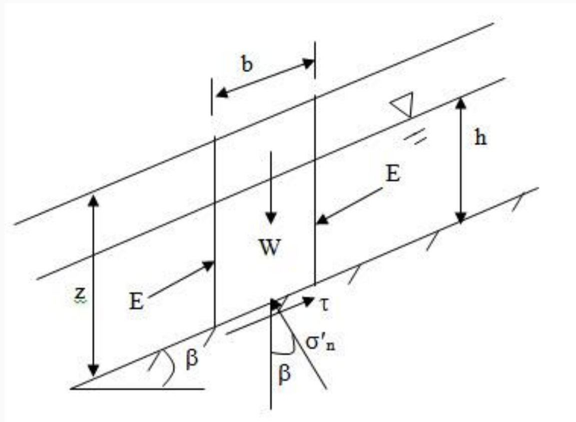


Fig. 28.3. . Effective forces acting on an element in the infinite slope.

Cohesive Soil

In case of *c*-ø soil, the shear strength can be written as:

$${\tau _f}=c' + {\sigma _n}\tan \phi '=c' + \gamma \,z{\cos ^2}\beta \tan \phi '$$ (28.10)

$$F=={{c' + \gamma \,z{{\cos }^2}\beta \tan \phi '} \over {\gamma z\cos \beta \sin \beta }}$$ (28.11)

For F = 1, the depth z is called as critical depth (h_c). Thus, putting F=1 and $z=h_c$ in Eq. (28.11), one can get

$${h_c}={{c'} \over {\gamma (\tan \beta-\tan \phi '){{\cos }^2}\beta }}$$ (28.12)

For seepage parallel to the slope,

$$F={{c' + (\gamma z - {\gamma _w}h){{\cos }^2}\beta \tan \phi'}\over{\gamma z\cos\beta\sin\beta }}$$ (28.13)

$${h_c} = \frac{{c'}}{{\gamma (\tan\beta-\frac{{{\gamma _w}}}{\gamma}\tan\varphi '){{\cos}^2}\beta}}$$ (28.14)

The stability number (S_n) can be written as:

$${S_n}={{c'} \over {\gamma {h_c}}}=(\tan \beta-\tan \phi '){\cos ^2}\beta$$ (28.15)

LESSON 29. Method for Stability Analysis

29.1 Finite Slope

There are three types of slope failure such as (as shown in Figure 29.1):

Slope Failure: In this type of failure, failure surface intersects the slope above the toe. It occurs when slope angle is high and soil close to toe is strong and soil in the upper part of the slope is strong.

Toe Failure: Most common type of failure. In this type of failure, failure surface passes trough the toe. It occurs for steep slopes and when soil above and below the base is homogeneous.

Base failure: In this type of failure, failure surface passes below the toe. It occurs for flat slopes and when soil below the base is relatively week.

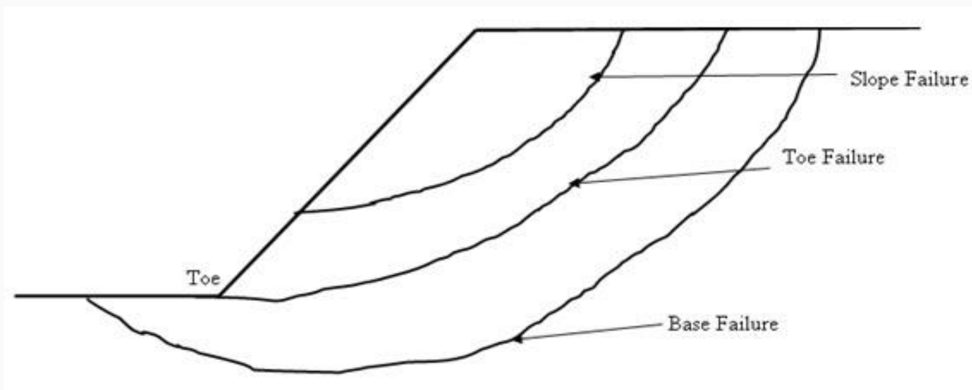


Fig. 29.1. Various failure surfaces.

29.2 Method of Slices

The stability of a slope can be analyzed by method of slices or Swedish Circle Method. In this method, the mass above the failure surface is divided into number of vertical slices of equal width (as shown in Figure 29.2). Considering a particular slice (slice number 5):

Driving force = $T = W \sin a$ (29.1)

Resisting force = $c'l + N \tan ø'$

$= c'l + W \cos a \tan ø'$ (29.2)

where l is the length of the failure surface in each slice, c' is the effective cohesion of the soil, ø' is the effective internal friction angle of the soil, N is the normal force acting on each slice (as shown in Figure 29.2), a is the angle each slice is making with horizontal, W is the weight of each slice ($W = \gamma\, b\, h$), γ is the unit weight of the soil, *b* is the width of each slice, *h* is the average height of each slice.

Thus, driving moment for slice number 5 = $R\, W \sin a$ (29.3)

Resisting moment = $Rc'l + RW \cos a \tan ø'$ (29.4)

he overall driving moment = $R\sum\limits_{i = 1}^n {{W_i}\sin {\alpha _i}}$ (29.5)

The overall resisting moment = $Rc'\sum\limits_{i=1}^n{{l_i}}+R\tan \phi'\sum\limits_{i = 1}^n {{W_i}\cos{\alpha _i}}$

= $Rc'L + R\tan \phi '\sum\limits_{i = 1}^n {{W_i}\cos {\alpha _i}}$ (29.6)

where *n* is the number of slice, *L* is the total length of the failure surface. Thus, the factor of safety against sliding (*F*) can be written as:

$$F={{Overall\;{{\rm Re}\nolimits} sisting\;Moment} \over {Overall\;Driving\;Moment}}$$

$$F={{c'L + \tan \phi '\sum\limits_{i = 1}^n {{W_i}\cos {\alpha _i}} } \over {\sum\limits_{i = 1}^n {{W_i}\sin {\alpha _i}} }} \qquad (29.7)$$

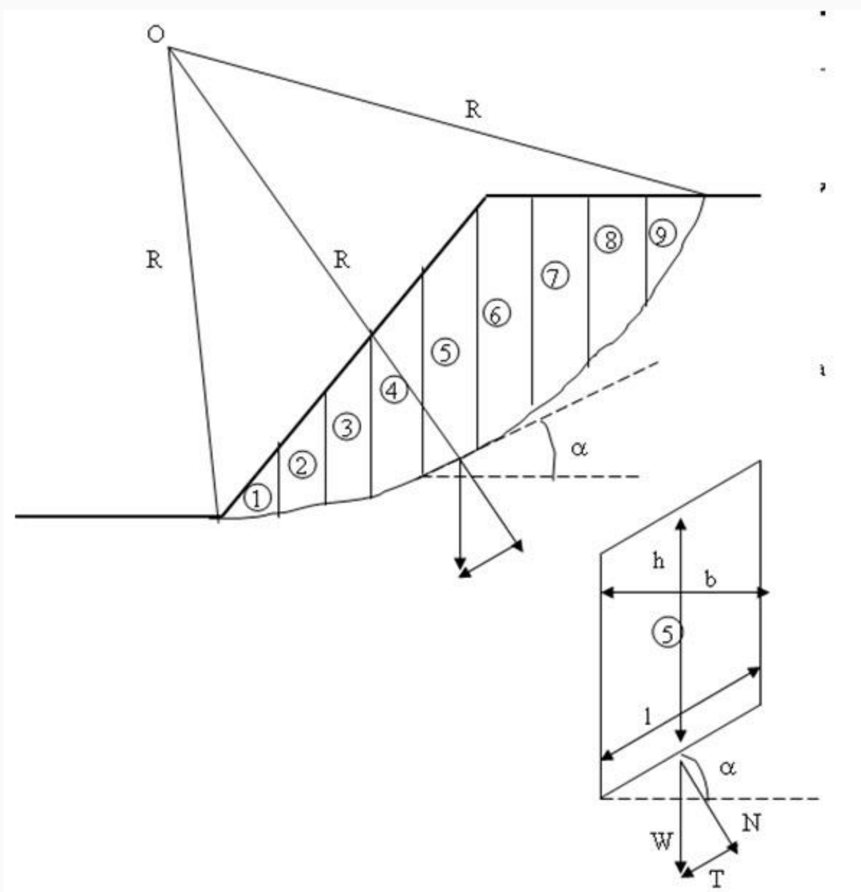


Fig. 29.2. Method of slices.

Considering effect of pore water pressure, the factor of safety can be written as:

$$F={{c'L + \tan \phi '\sum\limits_{i = 1}^n {({N_i} - {u_i})} } \over {\sum\limits_{i = 1}^n {{T_i}} }} \qquad (29.8)$$

where *u* is the resultant pore water pressure at the base of the slice.

LESSON 30. Soil Exploration

30.1 Introduction

The field and laboratory investigations required to obtain the necessary data of soils for proper design and successful construction of any structure are called soil exploration. The choice of the appropriate foundation and its depth depend on the bearing capacity, settlement of the foundation soil under the working loads. The bearing capacity and settlement depend on the various engineering properties of the foundation soils. The main objectives of the soil exploration are: determination of the nature of the soil deposits, depth and thickness of the different soil strata, location of the ground water table, collection of soil and rock samples for testing, determination of engineering properties of the soil and rock strata by laboratory or field tests that affect the performance of the structure placed on it. The soil index properties (like water content, Atterberg limits, etc.) and strength & compressibility characteristics (like cohesion, angle of internal friction, coefficient of consolidation, compression index) are very important parameters those are to be determined by testing. In the first stage of soil exploration, the aerial photographs, topographical maps, existing site investigation report (for nearby sites) are to be collected.

30.2 Different Methods

The methods available for soil exploration are:

1. Direct methods (like Test pits,trial pits or trenches)
2. Semi-direct methods (like Borings)
3. Indirect methods (like penetration test and geophysical methods)

30.2.1. Direct Methods

Test pits or trenches are open type or accessible exploratory methods where soils can be inspected in their natural condition. The necessary soils samples may be obtained by sampling techniques. The obtained soil samples can be used for finding strength and other engineering properties through laboratory tests. Test pits are considered suitable only for small depths - up to 3m. The cost of the soil exploration increases rapidly with depth. For greater depths, lateral supports or bracing of the excavations will be necessary. Test pits are usually made only for supplementing other methods or for minor structures.

30.2.2. Semi Direct Methods-Boring

Making or drilling bore holes into the ground for obtaining soil or rock samples from specific depths is called boring. The different types of borings are:

(i) Auger boring

(ii) Wash boring

(iii) Rotary drilling

(iv) Percussion drilling

The auger is useful for advancing a bore hole into the ground. It may be hand-operated or power-driven. The hand-driven augers are used for relatively small depths (less than 3 to 5 m), while the power-driven augers are used for greater depths (upto 60 to 70 m). The soil auger is advanced by rotating it while pressing it into the soil. As soon as the auger gets filled with soil, it is taken out and the soil sample collected. The soil samples obtained from this type of borings are highly disturbed. The Augur boring is suitable for partially saturated sands, silts and medium to stiff cohesive soils.Wash boring is commonly used for exploration below ground water table for which the auger method is not suitable. This method may be used in all kinds of soils except those mixed with gravel and boulders. A casing pipe is pushed in and driven with a drop weight. A hollow drill bit is screwed to a hollow drill rod connected to a rope passing over a pulley and supported by a tripod. Water jet under pressure is forced through the rod and the bit into the hole. This loosens the soil at the lower end and forces the soil-water suspension upwards along the annular surface between the rod and the side of the hole. This suspension is led to a settling tank where the soil particles settle while the water overflows into a sump. The water collected in the sump is used for circulation again. The soil particles collected are very disturbed sample and is not very useful for the evaluation of the engineering properties. Wash borings are primarily used for advancing bore holes; whenever a soil sample is required, the chopping bit is to be replaced by a sampler. The change of the rate of progress and change of color of wash water indicate changes in soil strata.

Rotary drilling can be used in sand, clay and rocks. A drill bit, fixed to the lower end of a drill rod, is rotated by power while being kept in firm contact with the hole. Drilling fluid or bentonite slurry is forced under pressure through the drill rod and it comes up bringing the cuttings to the surface. Even rock cores may be obtained by using suitable diamond drill bits. When soil samples are required, the drilling rod is raised and drilling bit is replaced by a sampler.

In case of Percussion drilling, a heavy drill bit is suspended from a drill rod or a cable and is driven by repeated blows. The water is added to facilitate the breaking of stiff soil or rock. The slurry of the pulverized material is bailed out at intervals. The method cannot be used in loose sand and is slow in plastic clay. The formation gets badly disturbed by impact.

The soil samples taken out of natural deposits for testing may be classified as:

(i) Disturbed sample

(ii) Undisturbed sample

A disturbed sample is that in which the natural structure of the soil gets modified partly or fully during sampling. An undisturbed sample is that in which the natural structure and other physical properties remain preserved. Disturbed but representative samples can generally be used for determination of the following purposes:

(i) Grain-size analysis

(ii) Determination of liquid and plastic limits

(iii) Determination of specific gravity of soil solids

(iv) Organic content determination

(v) Soil classification

The Undisturbed samples must be used for

(i) Consolidation test

(ii) Hydraulic conductivity test

(iii) Shear strength test (to determine the cohesion and angle of friction)

For collecting good quality undisturbed soil samples the area ratio [$A_r = (O.D^2 - I.D^2)/I.D^2 \times 100$ (%), where O.D and I.D are the outside and inside diameter of the sample tube] of the sample tube should be less than 10%. Thicker the wall, greater is the disturbance. Proper care has to be taken for transport and handling of the soil samples. The soil samplers are classified as:

(i) Thick wall samplers (Split spoon sampler)

(ii) Thin wall samplers (Shelby tubes)

The standard size of the spoon sampler is of 35 mm internal and 50.8 mm external diameter. Thus, the area ratio is 112% and the obtained samples are highly disturbed sample. The sampler is lowered to the bottom of the bore hole by attaching it to the drill rod. The sampler is then driven by forcing it into the soil by blows from a hammer. The assembly of the sampler is then extracted from the hole and the cutting edge and coupling at the top are unscrewed. The two halves of the barrel are separated and the sample is thus exposed. Samples are generally taken at intervals of about 1.53 m (5 ft). When the material encountered in the filed is sand (particularly fine sand below the water table), a spring core catcher is placed inside the split spoon. Shelby tubes are commonly used to obtain undisturbed clayey samples. The Shelby tube has outside diameter: 50.8 mm (2 in) and 76.3 mm (3 in).

LESSON 31. Field Tests: Indirect Methods

31.1 Standard Penetration Test (SPT)

The Standard Penetration Test (SPT) is widely used to determine the in-situ parameters of the soil. The test consists of driving a split-spoon sampler into the soil through a bore hole at the desired depth. The split-spoon sampler is driven into the soil a distance of 450 mm at the bottom of the boring. A hammer of 63.5 kg weight with a free fall of 760 mm is used to drive the sampler. The number of blows for a penetration of last 300 mm is designated as the Standard Penetration Value or Number *N* (ASTM D1586). The test is usually performed in three stages. The blow count is found for every 150 mm penetration. The blows for the first 150 mm are ignored as the top soil may be of disturbed nature due to advancement of borehole and hence considered as those required for the seating drive. The refusal of test when

- 50 blows are required for any 150 mm increment.
- 100 blows are obtained for required 300 mm penetration.
- 10 successive blows produce no advance.

The standard blow count $N¢_{70}$ can be computed as (ASTM D 1586):

$$N'_{70} = C_N \times N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4 \qquad (31.1)$$

where

η_i = correction factors

N'_{70} = corrected *N* using the subscript for the E_{rb} and the ' to indicate it has been corrected

E_{rb} = standard energy ratio value

C_N = correction for effective overburden pressure p'_0 (kPa) computed as [Liao and Whitman, 1986]:

$$C_N = \left(\frac{95.76}{p'_0} \right)^{\frac{1}{2}} \qquad (31.2)$$

SPT is standardized to some energy ratio (E_r) as:

$$E_r = \frac{Actual\;hammer\;energy\;to\;sampler,\;E_a}{Input\;energy,\;E_{in}} \times 100 \qquad (31.3)$$

Now $E_{in}={1 \over 2}m{v^2}={1 \over 2}{W \over g}{v^2}$ and $v={(2gh)^{1 \over 2}}$

Thus, $E_{in}={1 \over 2}{W \over g}(2gh)=Wh$ (31.4)

where W = weight of hammer and h = height of fall

The correction factor η_1 for hammer efficiency can be expressed as (Bowles, 996):

$$\eta_1={E_r \over E_{rb}} \quad (31.5)$$

Different types of hammers are in use for driving the drill rods. Two types are normally used. They are (Bowles, 1996):

(i) Donut hammer with E_r = 45 to 67

(ii) Safety hammer with E_r as follows:

- Rope-pulley or cathead = 70 to 80
- Trip or automatic hammer = 80 to 100

Now if E_r = 80 and standard energy ratio value (E_{rb}) = 70, then η_1 = 80/70 = 1.14

Correction factor η_2 for rod length (Bowles, 1996):

Length	>10 m	η_2 = 1.00
	6 – 10 m	= 0.95
	4 – 6 m	= 0.85
	0 – 4 m	= 0.75

Note: N is too high for Length < 10 m

Correction factor η_3 for sampler (Bowles, 1996):

Without liner		η_3 = 1.00
With liner:	Dense sand, clay	= 0.80
	Loose sand	= 0.90

Correction factor η_4 for borehole diameter

Hole diameter:	60 – 120 mm	η_4 = 1.00
	150 mm	= 1.05

200 mm = 1.15

Note: η_4 = 1.00 for all diameter hollow-stem augers where SPT is taken through the stem

Problem 1

Given: N = 21, rod length= 13 m, hole diameter = 100 mm, p'_0 = 200 kPa, E_r= 80; loose sand without liner. What are the standard N'_{70} and N'_{60} values?

Solution: For E_{rb}= 70: $N'_{70} = C_N \times N \times \eta_1 \times \eta_2 \times \eta_3 \times \eta_4$

Now, $C_N = \left(\frac{95.76}{200} \right)^{\frac{1}{2}} = 0.69$; η_1 = 80/70 = 1.14; η_2 = 1.0; η_3 = 1.0; η_4 = 1.0

Thus, $N'_{70} = 0.69 \times 21 \times 1.14 \times 1.0 \times 1.0 \times 1.0 = 17$

Now $E_{r1} \times N_1 = E_{r2} \times N_2$; Thus, $N'_{60} = \left(\frac{70}{60} \right) \times 17 = 20$

SPT Correlations in Clays (N. Sivakugan)

N'_{60}	c_u (kPa)	Consistency	Visual identification
0-2	0 - 12	very soft	Thumb can penetrate > 25 mm
2-4	12-25	soft	Thumb can penetrate 25 mm
4-8	25-50	medium	Thumb penetrates with moderate effort
8-15	50-100	stiff	Thumb will indent 8 mm
15-30	100-200	very stiff	Can indent with thumb nail; not thumb
>30	>200	hard	Cannot indent even with thumb nail

Note: N'_{60} is not corrected for overburden and c_u is the undrained cohesion of the clay.

SPT Correlations in Granular Soils (N. Sivakugan)

$(N')_{60}$	D_r (%)	Consistency	
0-4	0-15	very loose	
4-10	15-35	loose	
10-30	35-65	medium	
30-50	65-85	dense	

>50	85-100	very dense

Note: N'_{60} is not corrected for overburden

31.2 Static Cone Penetration Test (SCPT)

The Static cone penetration test has been standardized by "IS: 4968 (Part-III)-1976: Method for subsurface sounding for soils - Part III Static cone penetration test". The equipment consists of a steel cone, a friction jacket, sounding rod, mantle tube, a driving mechanism and measuring equipment. The cone has an apex angle of 60° ± 15′ and overall base diameter of 35.7 mm giving a cross-sectional area of 10 cm^2. The friction sleeve should have an area of 150 cm^2 as per standard practice. The sounding rod is a steel rod of 15 mm diameter which can be extended with additional rods of 1 m each in length. The driving mechanism should have a capacity of 20 to 30 kN for manually operated equipment and 100 kN for the mechanically operated equipment. With help of this test, the friction and tip resistance can be determined separately which is very useful information for pile foundation.

SCPT Correlations

In Clays: ${c_u}={{{q_c} - {\sigma _v}} \over {{N_k}}}$; where s_v = total vertical stress and N_k = cone factor (15-20). For Electric cone, N_k = 15 and for mechanical cone, N_k = 20.

In Sands: the modulus of elasticity can be correlated as: E = (2.5-3.5) q_c (for young normally consolidated sands), where q_c the tip or cone resistance.

31.3. Dynamic Cone Penetration Test (DCPT)

The dynamic cone penetration test is standardized by "IS: 4968 (Part I) – 1976:Method for Subsurface Sounding for Soils-Part I Dynamic method using 50 mm cone without bentonite slurry". The equipment consists of a cone, driving rods, driving head, hoisting equipment and a hammer. The hammer used for driving the cone shall be of mild steel or cast-iron with a base of mild steel and the weight of the hammer shall be 640 N (65 kg). The cone shall be driven into the soil by allowing the hammer to fall freely through 750 mm each time. The number of blows for every 100 mm penetration of the cone shall be recorded and total number of blows for each 300 mm penetration is considered as DCPT N value. The process shall be repeated till the cone is driven to the required depth. DCPT is better than SPT or SCPT in hard soils such as dense gravels. In case of SPT samples are collected for testing whereas in case of SCPT or DCPT samples can not be collected. Hammer is used in case of SPT and DCPT, but for SCPT no hammer is used, the cone is pushed inside the soil.

LESSON 32. Bore Hole Spacing and Depth

32.1 How many Bore Holes?

The number of bore holes depends on:

(i) Type and size of the project

(ii) Budget for site investigation

(iii) Soil variability

The bore holes have to be located where the loads are expected.

32.2 SPACING OF BORINGS (Das, 1999)

Type of project	Spacing (m)
Multistory buildings	10 – 30
One-story industrial plants	20 – 60
Highways	250-500
Residential subdivision	250-500
Dams and dikes	40 - 80

32.3 Minimum depth of boring (ASCE, 1972)

- Determine the net increase of stress, $\Delta\sigma$, under the foundation (as shown in Figure 1)
- Estimate the variation of the vertical effective stress, σ'_v, with depth
- Determine the depth $D = D_1$, at which stress increase $\Delta\sigma = q/10$, where q = estimated net stress on the foundation
- Determine the depth $D = D_2$, at which $\Delta\sigma/\sigma'_v = 0.05$.
- Unless bedrock is encountered, the smaller of the two depths, D_1 and D_2 will be the approximate minimum depth of boring required.

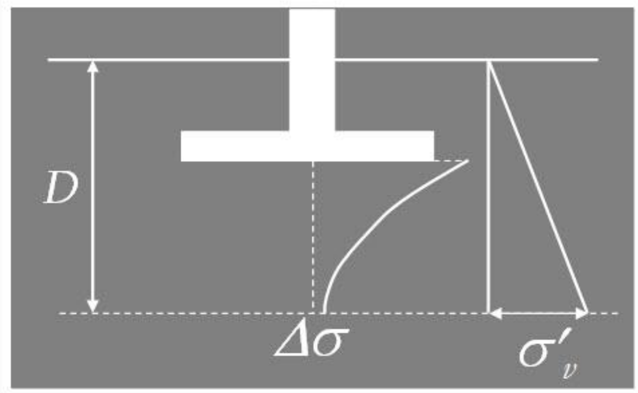


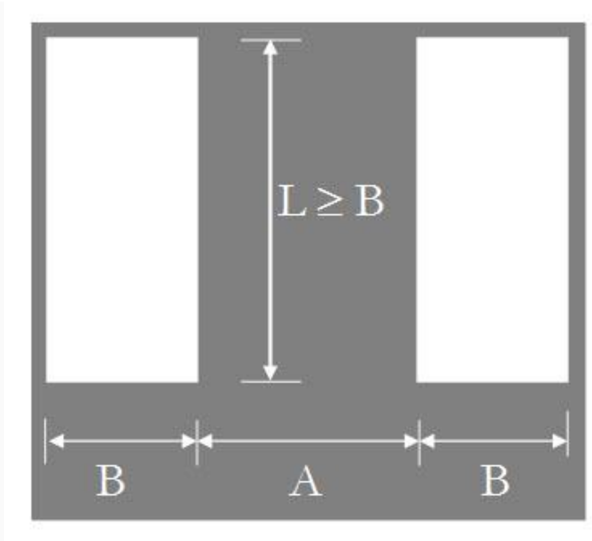
Fig. 32.1.

The minimum depth of boring for a building with a width of 30.5 m (100 ft) will be as follows (Sowers and Sowers, 1970)

No of stories	Boring depth
1	3.5 m
2	6.0 m
3	10 m
4	16 m
5	24 m

32.3.1. Depth of Borings (according to IS 1892-1979)

Type of foundation	Depth of boring
1. Isolated spread footing or raft	One and half times the width (*B*) of the foundation
2. Adjacent footings with clear spacing less than twice the width	One and half times the length (*L*) of footing
3. Pile and well foundation	To a depth of one and half times the width of structure from the bearing level (toe of pile or bottom of well)
4. (a) road cut	Equal to the bottom width of the cut
(b) Fill	Two meters below ground level or equal to the height of the fill whichever is greater.



D = 1.5 *B* for A ³ 4B and *D* = 1.5 *L* for A < 2B

D = 3 *B* for A >2*B* and <4*B*

D = 4.5 *B* for A < 2*B*

D = 1.5 *B* for A ³ 4*B*

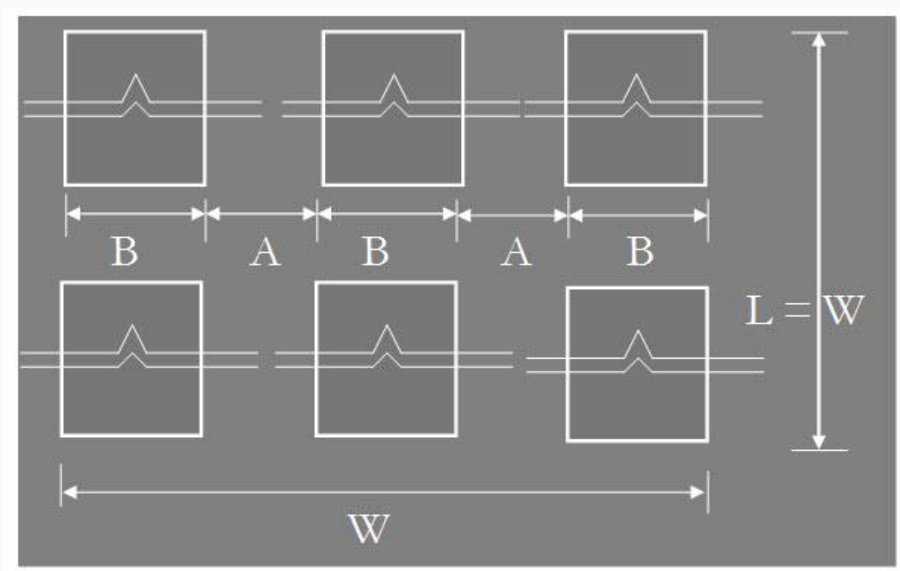


Fig. 32.2 Depth of boring according to IS 1892-1979.