LECTURE NOTE

COURE CODE BCE402

GEOTECHNICAL ENGINEERING – II

*Under revision

BCE402- Syllabus

Module – I (10 Hours)

Stress distribution in soil: Boussinesq equations, Stress isobar and pressure bulb concept, pressure distribution on horizontal and vertical planes, stresses due to point load, line load, strip load, uniformly loaded circular and rectangular areas. Use of newmark's chart. Westergaard's solution. Approximate methods (point load method, two-to-one load distribution method). Contact pressure distribution due to loaded areas. Concept of active zone.

Module –II (10 Hours)

Lateral earth pressure and retaining structures: Earth pressure at rest, active and passive earth pressure. Earth pressure theories, Rankine's theory, Coloumb's wedge theory, Rebhann's and Culmann's graphcal methods, stability conditions for retaining walls. Stability of earth slopes: Stability of infinite slopes, stability analysis of finite slopes, Swedish method of slices, fiction circle method, Bishop's method. Use of Taylor stability number. Fellnious metod for locating centre of critical slip circle.

Module – III (10 Hours)

Subsoil exploration: Methods, direct (test pits, trenches), semi-direct (borings), indirect (sounding, penetration tests, and geophysical methods).

Planning of exploration programme, spacing and depth of boring, soil sampling, types of samples, standard penetration test, static and dynamic cone penetration test, in-situ vane shear test. Seismic refraction method, electrical resistivity methods,

Module-IV (10 Hours)

Shallow foundation: Introduction, bearing capacity, methods and determination of bearing capacity, settlement of foundations.

Deep foundation: Classification of pile, pile driving methods, pile capacity (static and dynamic analysis) pile-group analysis, load test on piles.

Reference Books:

1. Geotechnical Engineering, C. Venkatramaiah, New Age International publishers.
2. Geotechnical Engineering, T.N. Ramamurthy & T.G. Sitharam, S. Chand & Co.
3. Soil Mechanics, T.W. Lambe & Whiteman, Wiley Eastern Ltd, Nw Delhi.
4. Foundation Engineering, P.C. Verghese, Prentice Hall of India.

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LECTURE 1

STRESS DISTRIBUTION IN SOIL

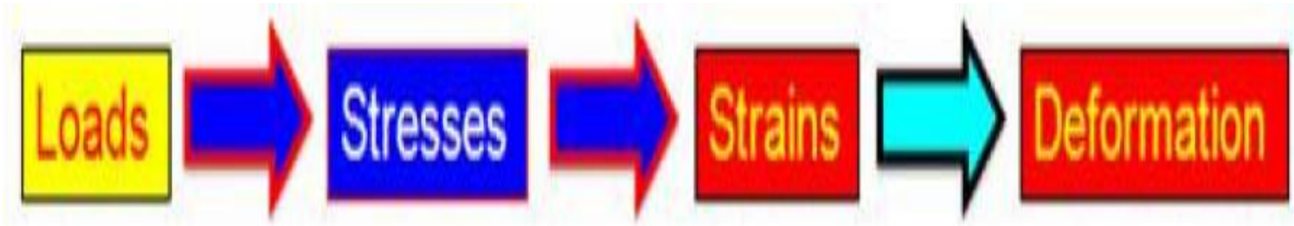

Fig. 1.1

Stress in soil is caused by the first or both of the following :-

(a) Self weight of Soil.

(b) Structural loads, applied at or below the surface.

The estimation of vertical stresses at any point in a soil mass due to external loading is essential to the prediction of settlements of buildings, bridges and pressure. The stresses induced in a soil due to applied loads depend upon its Stress – Strain characteristics. The stress strain behaviour of soils is extremely complex and it depend upon a large number of factors, such as drainage conditions, water content, void ratio, rate of loading, the load level, and the stress path. However simplifying assumptions are generally made in the analysis of soil behaviour to obtain stresses. It is generally assumed that the soil mass is homogeneous and isotropic. The stress strain relationship is assumed to be linear. The theory of elasticity is used to determine the stresses in the soil mass. Though it involves considerable simplification of real soil behaviour and the stresses computed are approximate, the results are good enough for soil problems usually encountered in the practice.

Geostatic stress:

Stresses due to self weight are known as geostatic stresses.

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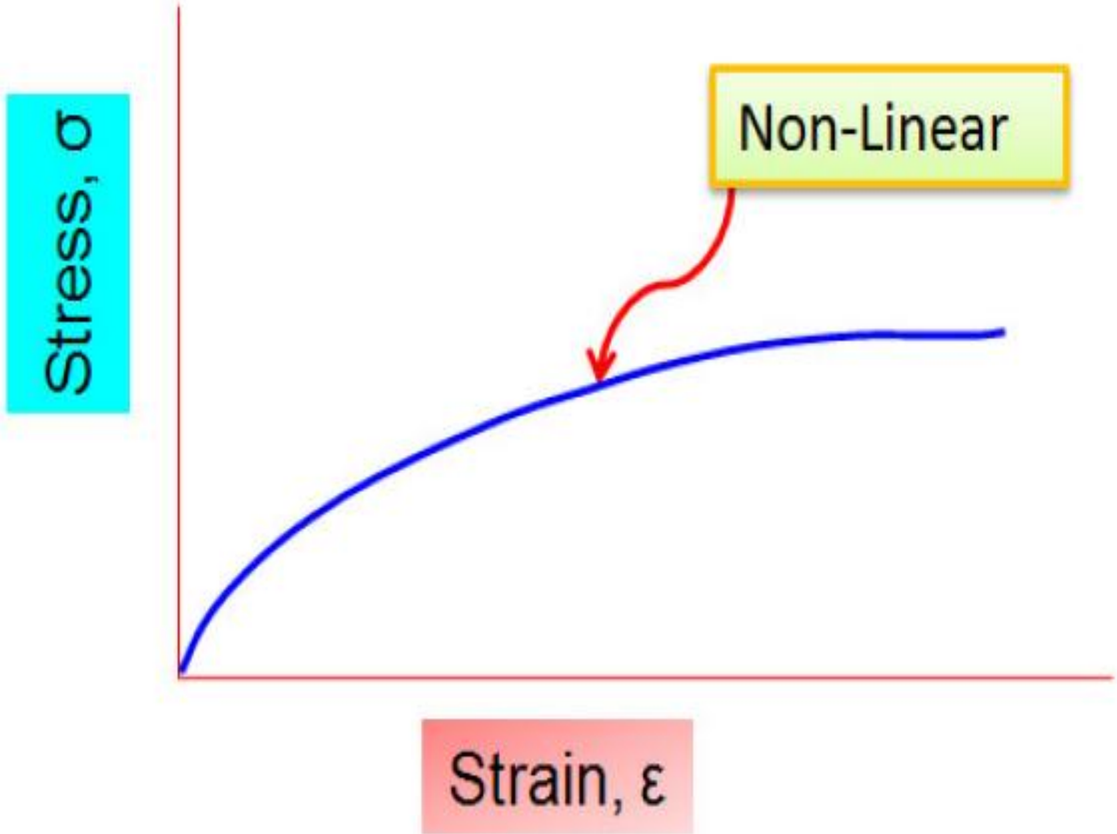

Fig. 1.2

Typical stress strain curve for soils

Many problems in foundation engineering require a study of the transmission and distribution of stresses in large and extensive masses of soil, some examples are wheel loads transmitted through embankments to culverts, foundation pressures transmitted to soil strata below footings, pressures from isolated footings transmitted to retaining walls and wheel loads transmitted through stabilized soil pavements to subgrades below. In such cases the stresses are transmitted in all downward and lateral directions. Estimation of vertical stresses at any point in a soil mass due to external loading is essential to the predication of settlements of building, bridges and embankments.

At a point within a soil mass, stresses will be developed as a result of the soil lying above the point (Geostatic stress) and by any structural or other loading imposed onto that soil mass .

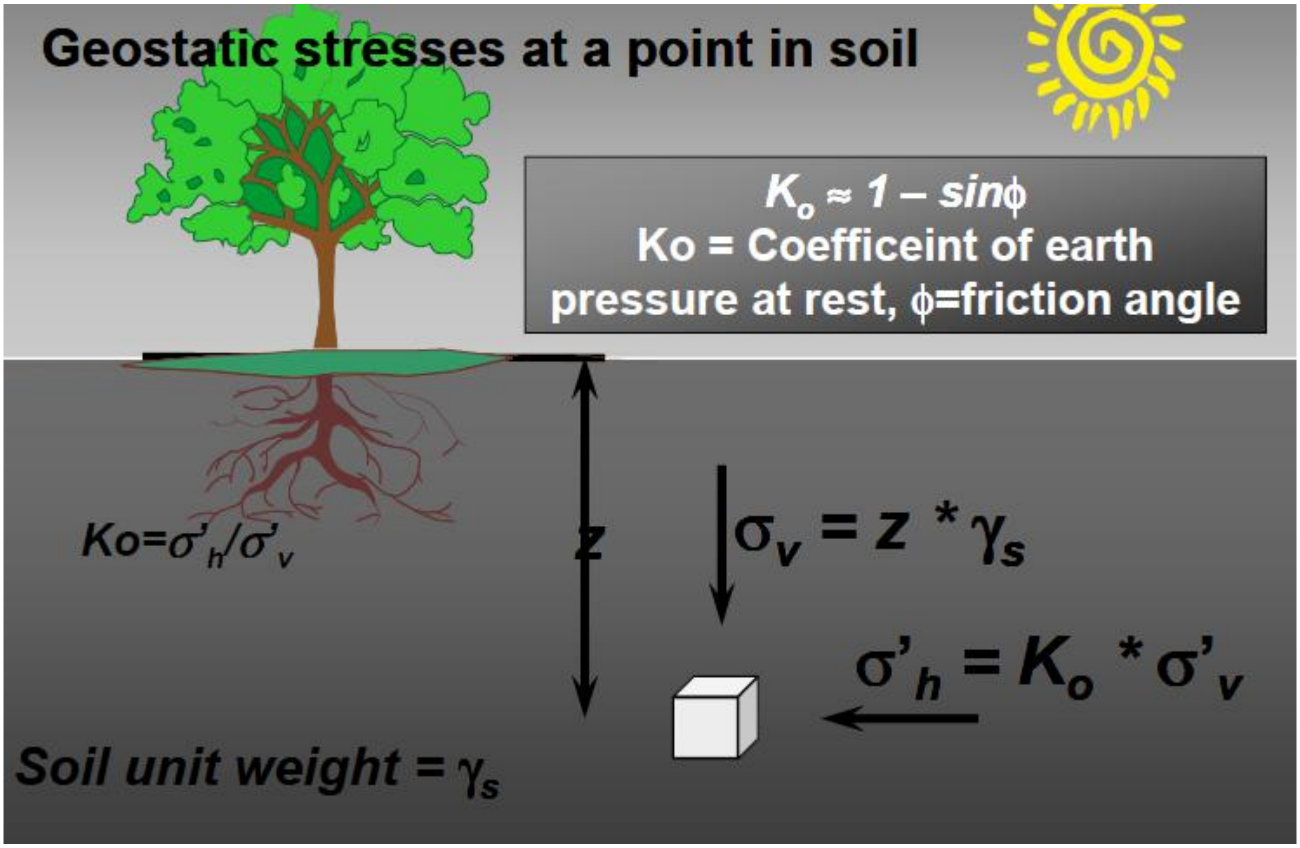

Fig. 1.3 Geostatic stresses at a point in soil

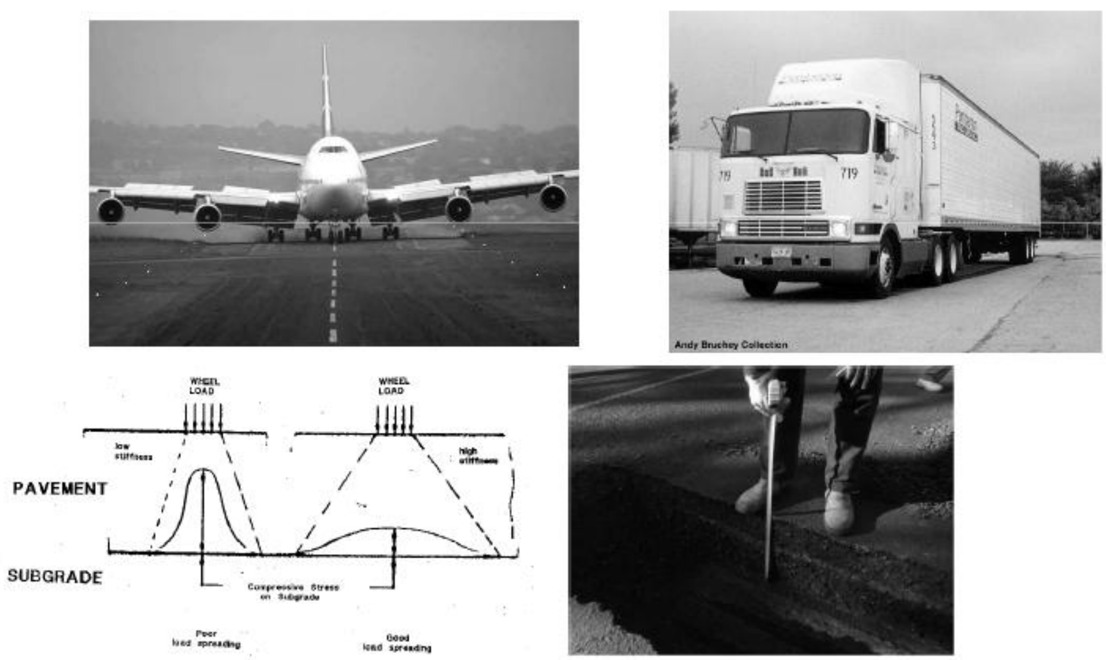

Fig. 1.4 Subsurface stresses in road pavements and airport runways are increased by wheel load on the surface

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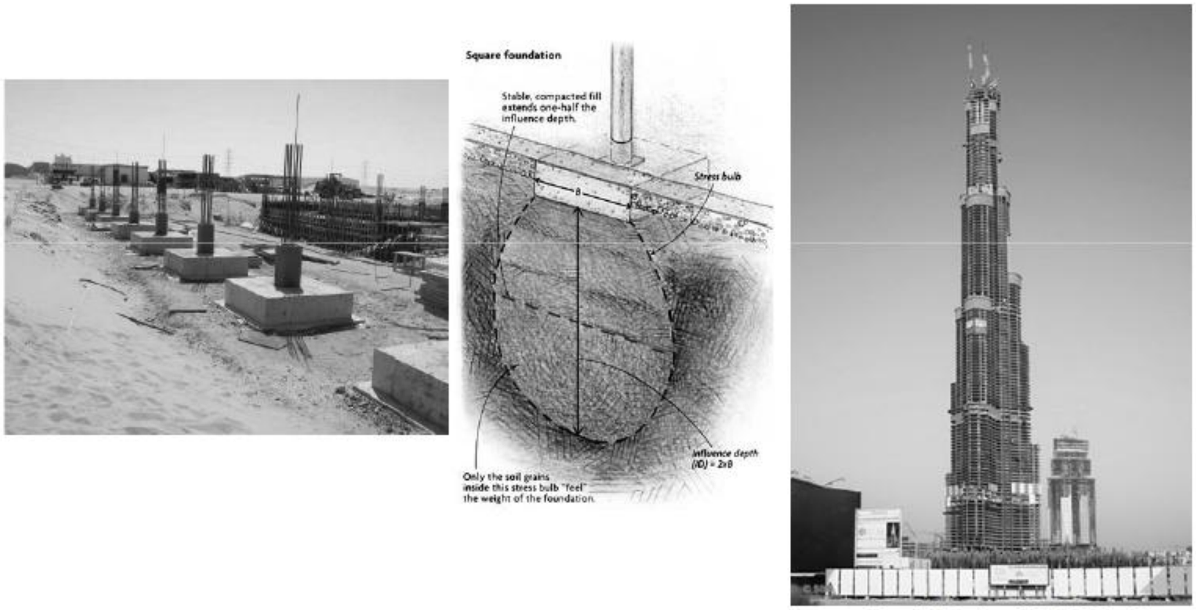

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Fig. 1.5 Subsurface stresses in soils are increased by foundation loads

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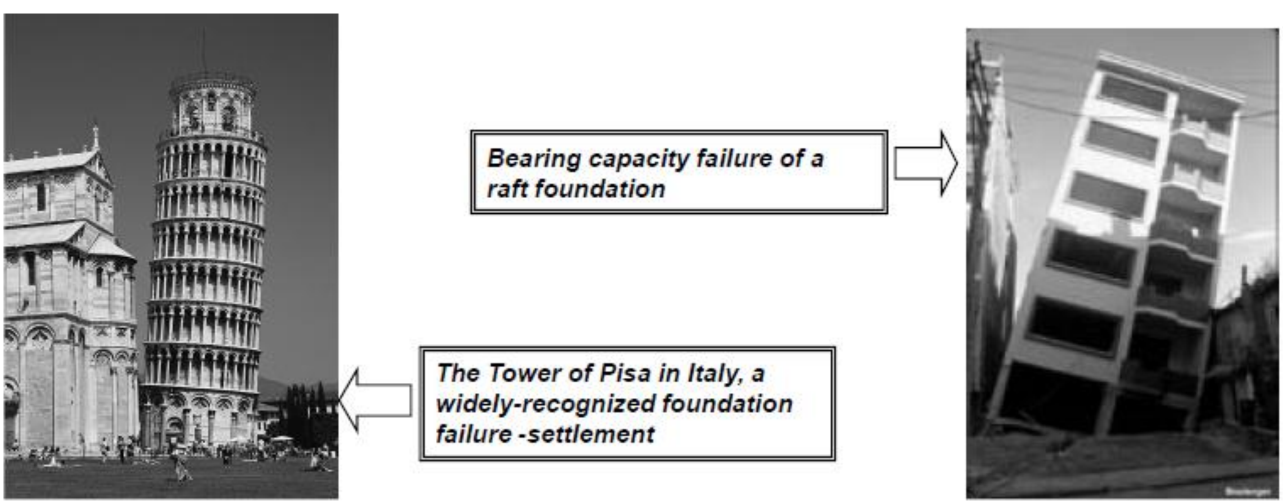
LECTURE 2

Fig. 2.1 Embankments and landfills cause to increase subsoil stresses

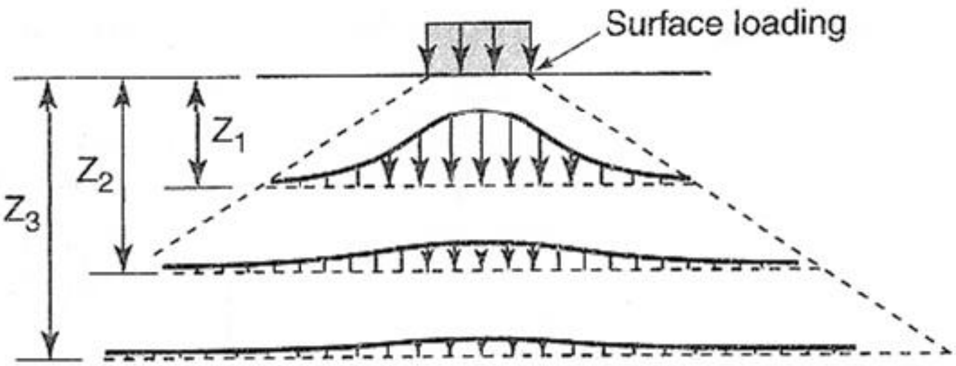


It is required to estimate the stress increase in the soil due to the applied loads on the surface.
The estimated subsoil stress increase is used:
- to estimate settlement of foundation
- to check the bearing capacity of the foundation

Fig. 2.2 ***The surface loading area is much larger than the depth of a point where vertical stress increment* ($\Delta\sigma$) *is calculated (e.g. land fills, preloading by soil deposition)***

Finitely loaded area

If the surface loading area is finite (point, circular, strip, rectangular, square), the vertical stress increment in the subsoil decreases with increase in the depth and the distance form the surface loading area.



Methods have been developed to estimate the vertical stress increment in sub-soil considering the shape of the surface loading area.

Fig. 2.3 Subsurface stress increment due to a point load

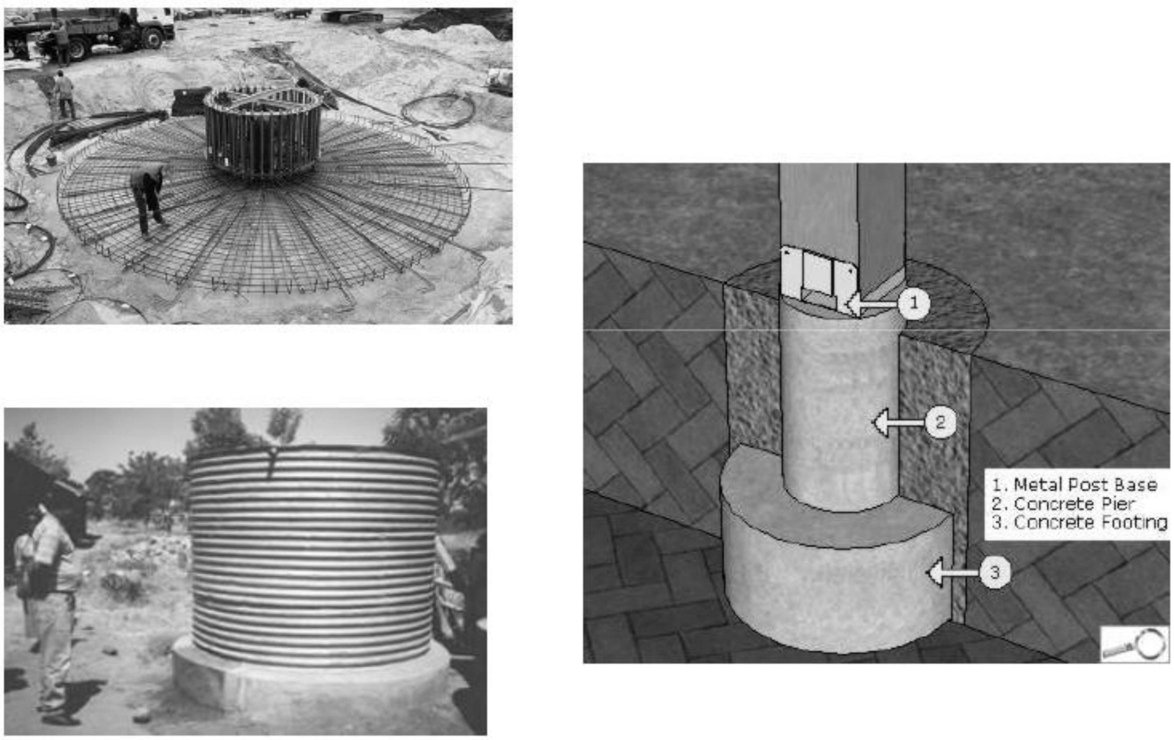


Fig. 2.4 Subsurface stress increment due to circular loaded Area

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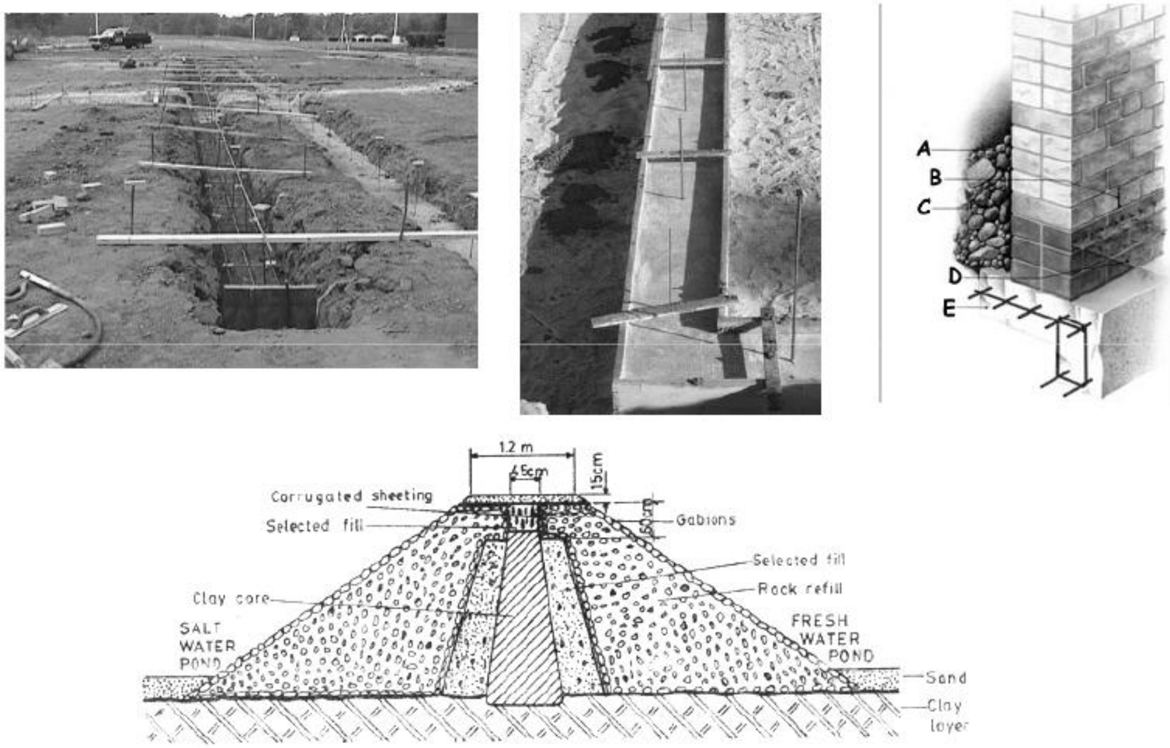

Fig. 2.5 Subsurface stress increment due to strip loading

Fig. 2.6 Subsurface stress increment due to rectangularly/ squarely loaded area

LECTURE 3

Point Load :-

Although a point load or a Concentrated load is, strictly speaking hypothetical in nature, consideration of it serves a useful purpose in arriving at the solutions for more complex loadings in practice.

Boussinesqs Solution

Assumptions made by **Boussinesq**.

(i) The soil medium is an elastic, homogeneous, isotropic and semi infinite medium, which extends infinitely in all directions from a level surface.

(ii) The medium obeys Hookes law.

(iii) The self weight of the soil is ignored.

(iv) The soil is initially unstressed

(v) The change in volume of the soil upon application of the loads on to it is neglected.

(vi) The top surface of the medium is free of shear stress and is subjected to only the point load at a specified location.

(vii) Continuity of stress is considered to exist in the medium.

(viii) The stresses are distributed symmetrically with respect to z axis.

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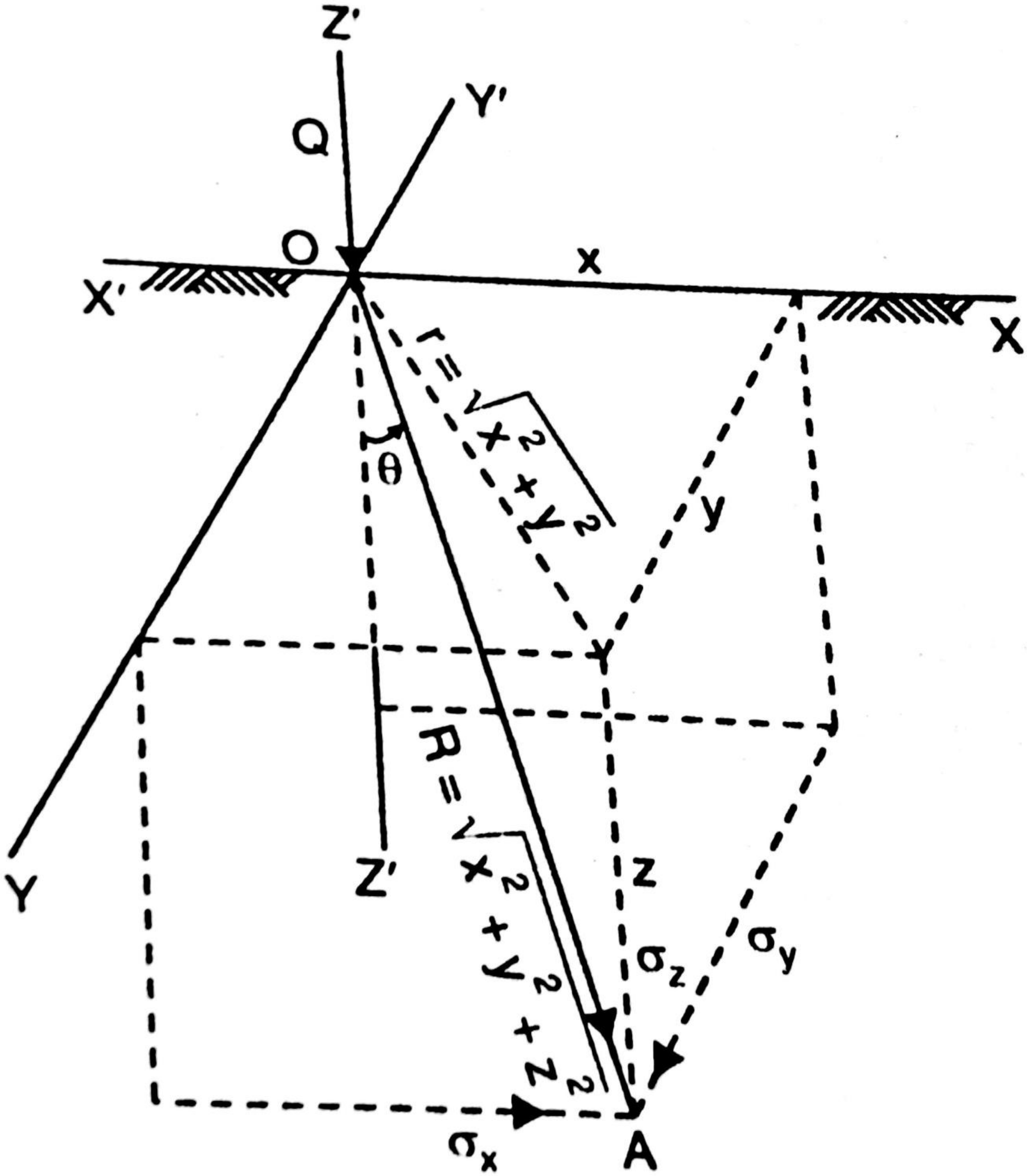

Fig. 3.1

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The Boussinesq equations are as follows :

$$\sigma_z = \frac{3Q}{2\pi}\frac{Z^3}{R^5}$$

$$= \frac{3Q}{2\pi}\frac{\cos^2\theta}{Z^2}$$

$$= \frac{3Q}{2\pi}\frac{Z^3}{(r^2+z^2)^{5/2}}$$

$$= \frac{3Q}{2\pi Z^2}\left[\frac{1}{1+(r/z)^2}\right]^{5/2} \text{ --------------------------------} \qquad (1)$$

$$\sigma_x = \frac{Q}{2\pi}\left[\frac{3x^2Z}{R^5} - (1-2v)\left\{\frac{x^2-y^2}{Rr^2(R+Z)} + \frac{y^2Z}{R^3r^2}\right\}\right]$$

$$\sigma_y = \frac{Q}{2\pi}\left[\frac{3y^2Z}{R^5} - (1-2v)\left\{\frac{y^2-x^2}{Rr^2(R+Z)} + \frac{x^2Z}{R^3r^2}\right\}\right]$$

$$\sigma_R = \frac{3Q}{2\pi}\frac{\cos\theta}{R^2}$$

$$\sigma_r = \frac{Q}{2\pi}\left[\frac{3zr^2}{R^2} - \frac{(1-2v)}{R(R+Z)}\right]$$

$$\tau_{rz} = (3QrZ^2)/(2\pi R^5)$$

$$= \frac{3Qr}{2\pi Z^3}\left[\frac{1}{1+(r/z)^2}\right]^{5/2}$$

Equations (1) may be rewritten as

$$\sigma_z = K_B\frac{Q}{Z^2}$$

where K_B, Boussinesq's influence factor is given by :

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

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This intensity of vertical stress directly below the point load, on its axis of loading is given by :

$$\sigma_z = \frac{0.4775Q}{Z^2}$$

Pressure Distribution :

It is possible to calculate the following pressure distributions by equation (1) of Boussinesq and present them graphically.

(i) Vertical stress distribution on a horizontal plane, at a depth 'z' below the ground surface.

(ii) Vertical stress distribution along a vertical line, at a distance 'r' from the line of action of the single concentrated load.

Vertical Stress Distribution on a Horizontal Plane :-

The vertical stress on a horizontal plane at depth 'Z' is given by:

$\sigma_z = K_B \frac{Q}{Z^2}$, Z being a specified depth.

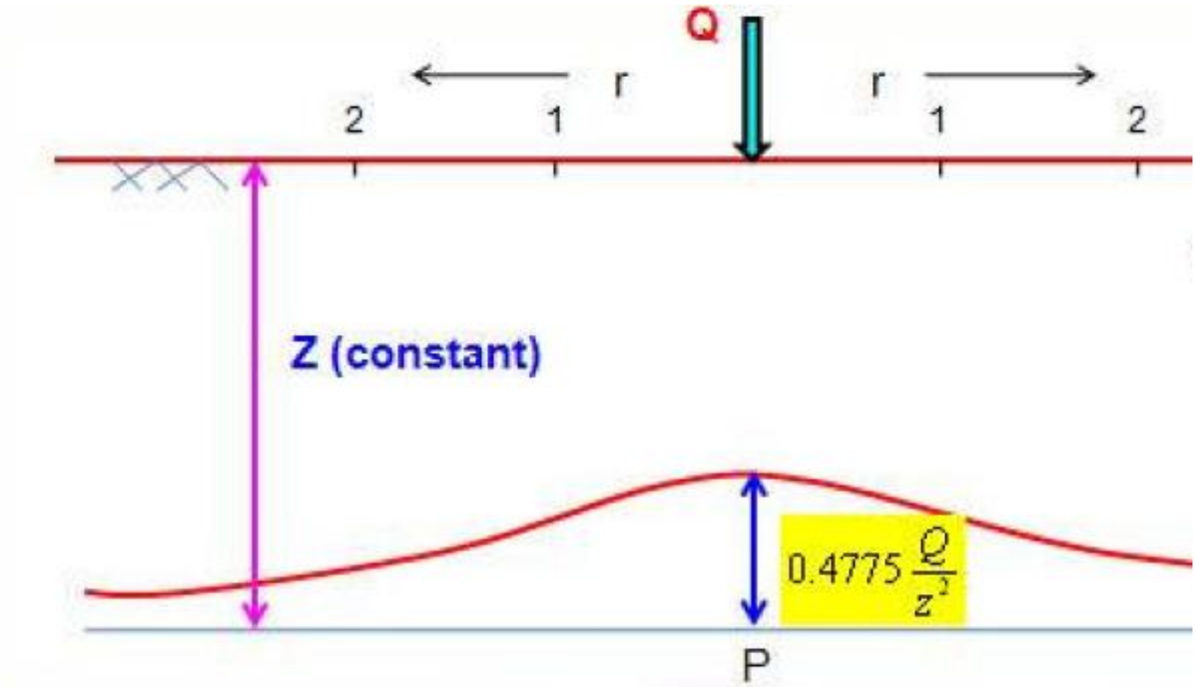

Fig. 3.2 Vertical stress distribution on a horizontal plane at depth 'z' (Boussinesq's)

LECTURE 4

Vertical stress distribution along a vertical line

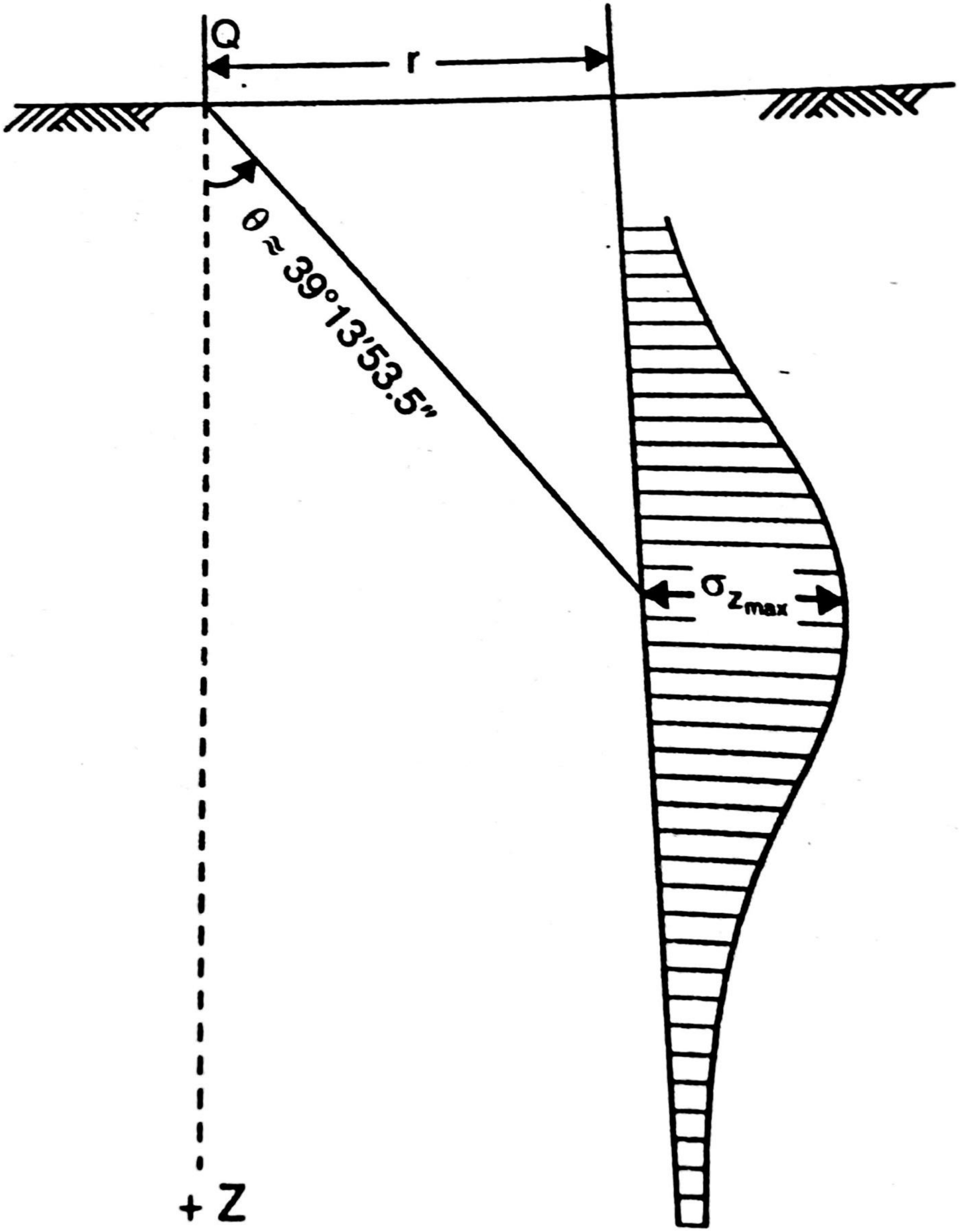

Fig. 3.3 Vertical stress distribution along a vertical line at radial distance 'r'

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Stress isobar or pressure Bulb concept An isobar is a stress contour or a line which connects all points below the ground surface at which the vertical pressure is the same in fact an isobar is a spatial curved, surface and resembles a bulb in shape, this is because the vertical pressure at all points in a horizontal plane at equal radial distances from the load is the same. Thus, the stress isobar is also called the bulb of pressure or simply the pressure bulb. The vertical pressure at each point on the pressure bulb is the same.

Pressure at points inside the bulb are greater than that at a point on the surface of the bulb and pressures at points outside the bulb are smaller than that value.

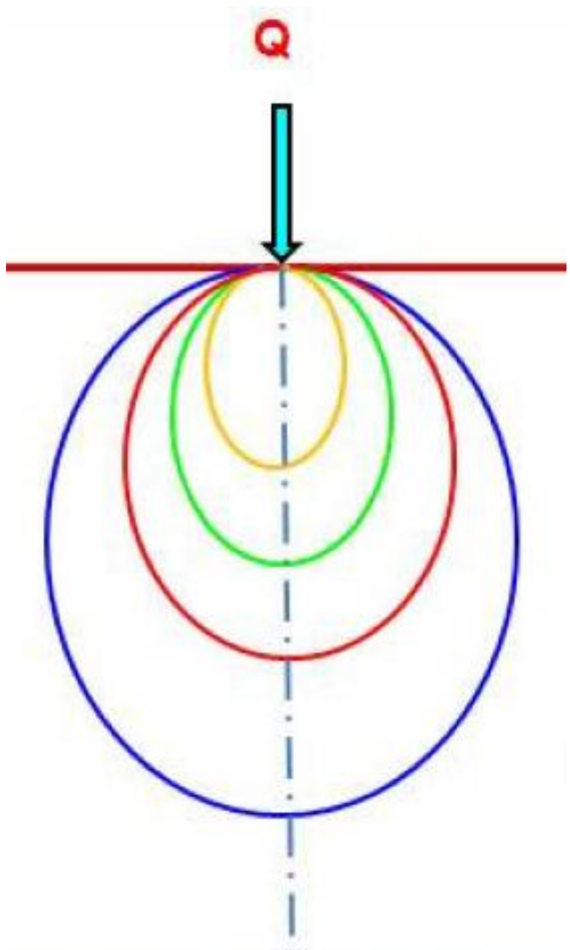

Fig. 4.1 Isobar diagram (A system of pressure bulbs for a point load- Boussinesq's)

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Westergaards Solution:-

Westergaard (1938) has obtained an elastic solution for stress distribution in soil under a point load based on conditions analogous to the extreme condition of this type. The material is assumed to be laterally reinforced by numerous, closely spaced horizontal sheets of negligible thickness but of infinite rigidity, which prevent the medium from undergoing lateral strain , this may be viewed as representative of an extreme case of non isotropic condition.

The vertical stress σ_z caused by a point load, as obtained by the Westergaard is given by :

$$\sigma_z = \frac{Q}{z^2} \frac{\frac{1}{2\pi}\sqrt{\frac{1-2\upsilon}{2-2\upsilon}}}{\left[\left(\frac{1-2\upsilon}{2-2\upsilon}\right)+\left(\frac{r}{z}\right)^2\right]^{3/2}}$$

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Line Load :-

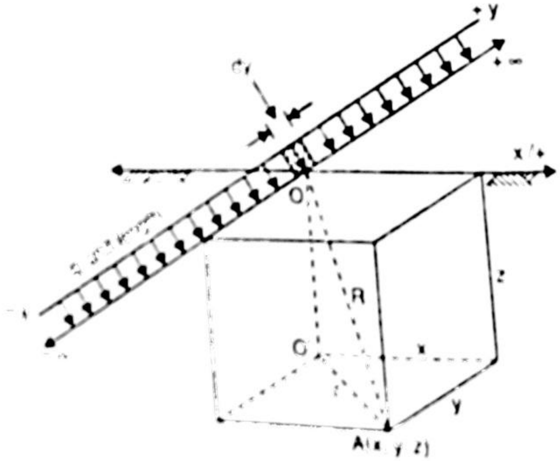

Fig 4.2 Line load acting on the surface of semi infinite elastic soil medium.

Let a load, uniformly distributed along a line,of intensity $q`$ per unit length of a straight line of infinite extension, act on the surface of a semi infinite elastic medium. Such a loading produces a state of plane strain , that is the stains and stresses in all planes normal to the line of the loading are identical and it is adequate to consider the conditions in one such plane, Let the y axis be directed along the line of loading as shown in Fig.4.2

Let us consider a small length dy of the line loads as shown: the equivalent point load is q`dy and, the vertical stress at A due to this load is given by :

$$d\sigma_z = \frac{3(q`dy)z^3}{2\pi r^5} = \frac{3q`z^3 dy}{2\pi(x^2+y^2+z^2)^{5/2}}$$

The vertical stress σ_z at ‘A’ due to the infinite length of line load may be obtained by integrating the equation for $d\sigma_z$ with respect to the variable ‘y’ within the limits $-\infty$ and $+\infty$.

$$\therefore \sigma_z = \int_{-\infty}^{\infty} \frac{3q`z^3 dy}{2\pi(x^2+y^2+z^2)^{5/2}}$$

$$= 2\int_{0}^{\infty} \frac{3q`z^3 dy}{2\pi(x^2+y^2+z^2)^{5/2}}$$

$$\sigma_z = \frac{2q`z^3}{\pi(x^2+z^2)^2} = \frac{2q`}{\pi z}\frac{1}{\left[1+\left(x/z\right)^2\right]^2}$$

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LECTURE 5

Vertical pressure under strip load :

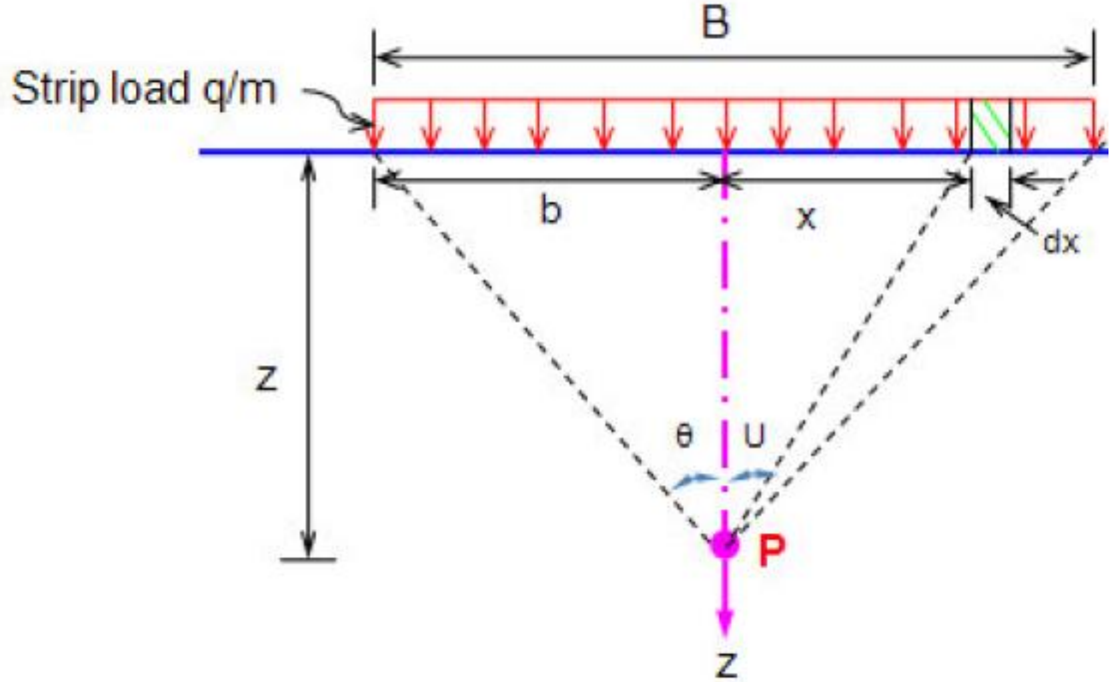

Fig 5.1 Vertical stress due to strip load point 'P' below the centre

Fig. 5.1 shows an infinite strip of width B, loaded with uniformly distributed load intensity 'q' per unit area. Let us find the vertical pressure at a point 'p' situated below a depth 'z', on a vertical axis passing through the centre of the strip. Consider a strip load of width dx, at distance 'x' from the centre. The elementary line load intensity along this elementary strip of width 'dx' will be 'qdx'. The vertical pressure at 'p' due to this elementary line load is given by :

$$\Delta\sigma_z = \frac{2(qdx)}{\pi z}\frac{1}{\left[1+\left(x/z\right)^2\right]^2}$$

Total vertical pressure due to the whole strip load is given by :

$$\sigma_Z = \frac{q}{\pi}(2\theta + \sin 2\theta)$$

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Uniform Load on Circular Area :

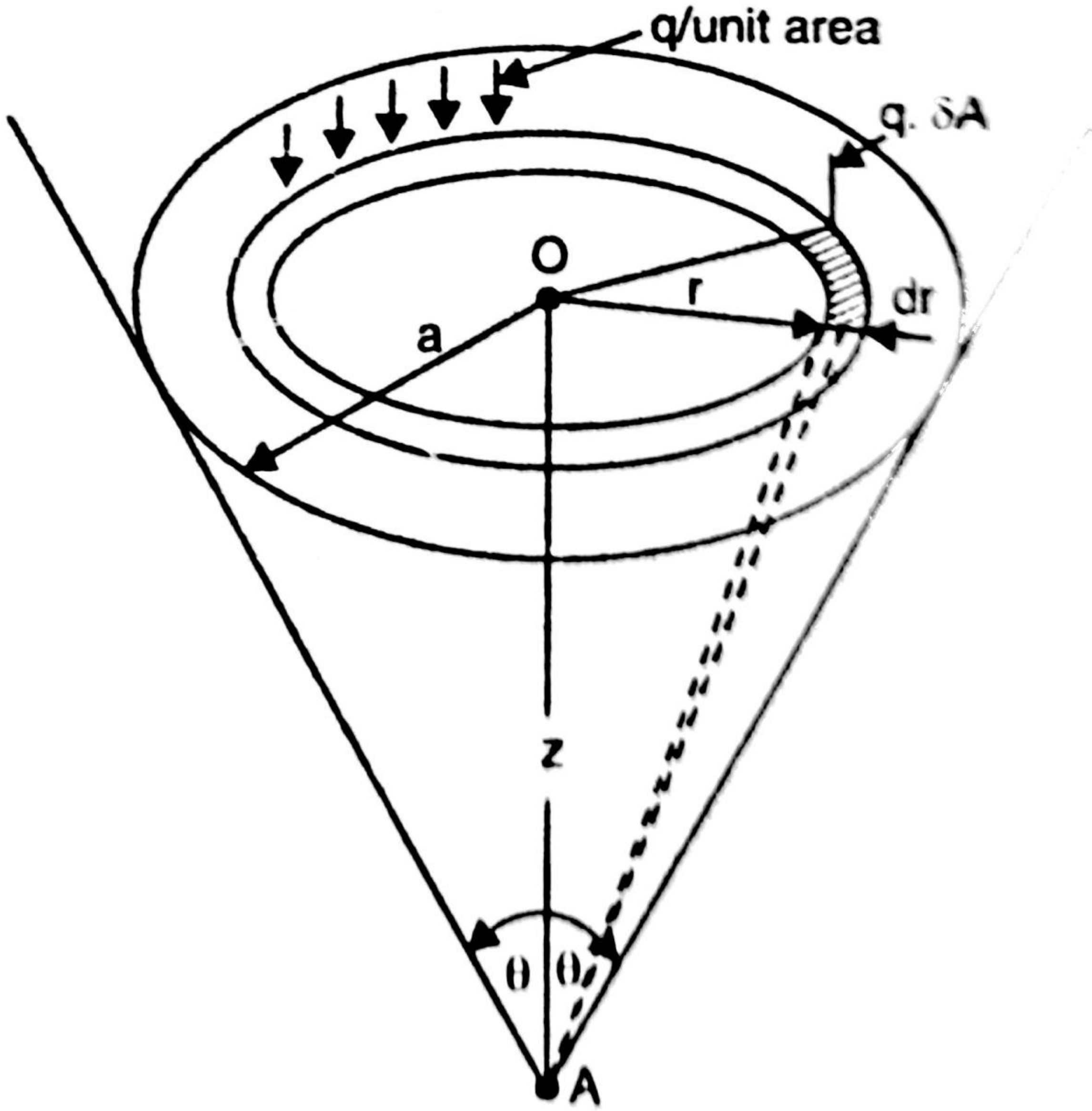

Fig 5.2 Uniform load over circular area.

Let the circular area of radius a be loaded uniformly with ‘q’ per unit area as shown in Fig 5.2.

Let us consider an elementary ring of radius ‘r’ and thickness ‘dr’ of the loaded area. This ring may be imagined to be further divided into elemental areas each δA. The load from such an elemental area is $q\delta A$. The vertical stress $\delta\sigma_z$ at point ‘A’ at a depth ‘z’ below the centre of the loaded area is given by :

$$\delta\sigma_z = \frac{3(q\delta A)}{2\pi}\frac{z^3}{(r^2+z^2)^{5/2}}$$

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The stress $d\sigma z$ due to the entire ring is given by :

$$d\sigma z = \frac{3q}{2\pi}\left(\sum \delta A\right)\frac{z^3}{(r^2+z^2)^{5/2}}$$

$$= \frac{3q}{2\pi}(2\pi r dr)\frac{z^3}{(r^2+z^2)^{5/2}}$$

$$\therefore d\sigma z = \frac{3qZ^3 r dr}{(r^2+z^2)^{5/2}}$$

The total vertical stress σz at 'A' due to entire loaded area is obtained by integrating $d\sigma z$ within the limits r = 0 to r = a

$$\therefore \sigma z = 3qz^3 \int_{r=0}^{r=a} \frac{r dr}{(r^2+z^2)^{5/2}}$$

Setting $r^2 + z^2 = R^2$, rdr = RdR, the limits for 'R' will be 'z' and $\left(a^2+z^2\right)^{1/2}$.

$$\therefore \sigma z = 3qz^3 \int_{R=Z}^{R=(a^2+z^2)^{\frac{1}{2}}} \frac{dR}{R^4}$$

$$= qz^3\left[\frac{1}{z^3} - \frac{1}{(a^2+z^2)^{3/2}}\right]$$

$$\therefore \sigma_z = q\left[1 - \frac{1}{\left[1+\left(a/z\right)^2\right]^{3/2}}\right]$$

This may be written as : $\sigma_z = qK_{BC}$

Where K_{BC} = Boussinesq influence coefficient for uniform load on circular area and

$$K_{BC} = \left[1 - \frac{1}{\left\{1+\left(a/2\right)^2\right\}^{3/2}}\right]$$

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LECTURE 6

Uniform load on Rectangular Area : -

Based on Baussinesq's Theory :-

The following are the two popular forms of Newmark's equation for σz :-

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

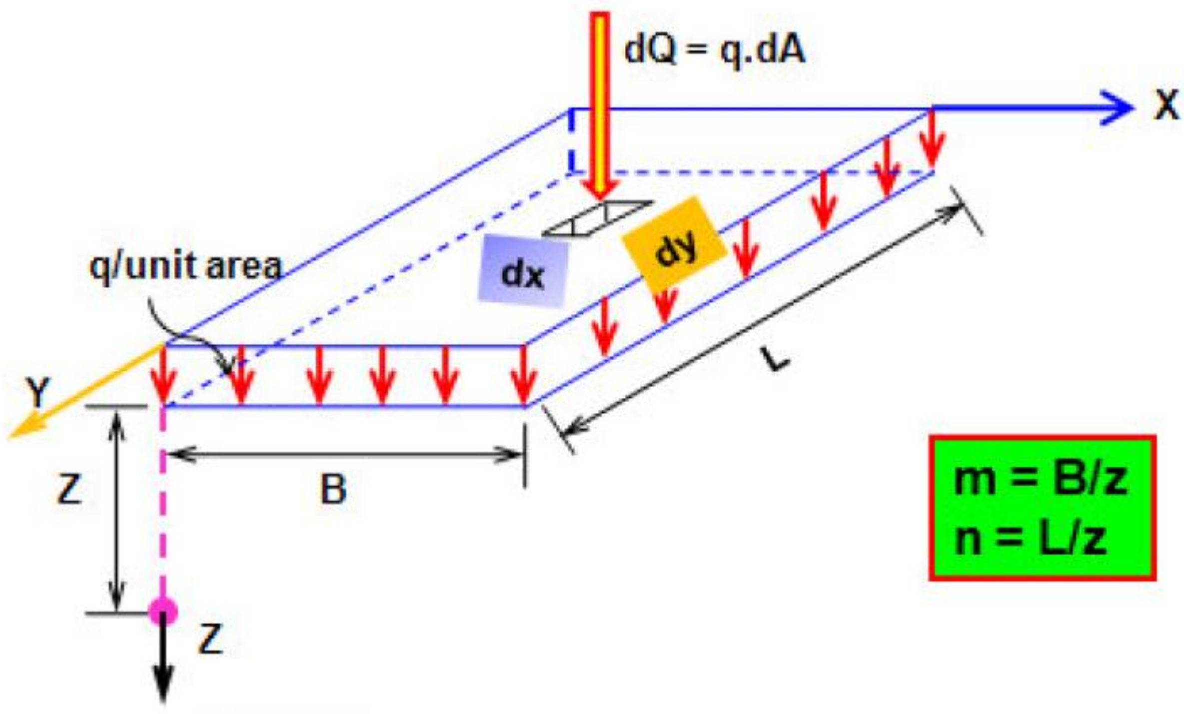

Fig 6.1 Vertical stress of a uniformly loaded rectangular area

$$\sigma_z = \frac{q}{4\pi}\left[\frac{\left(2mn\sqrt{m^2+n^2+1}\right)}{(m^2+n^2+1+m^2n^2)}\left(\frac{m^2+n^2+2}{m^2+n^2+1}\right)+\tan^{-1}\left(\frac{2mn\sqrt{m^2+n^2+1}}{m^2+n^2+1-m^2n^2}\right)\right] \text{--------1.5}$$

where $m = \frac{B}{z}$ and $n = \frac{L}{z}$

Equation 1.5 may be written in the form :

$\sigma_z = qI_\sigma$ where

$I_\sigma =$ Influence value

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

Uniform Load on irregular Areas :

Newmark's chart

Influence chart for vertical stress increase

•Newmark (1942) constructed influence chart , based on the Boussinesq solution to determine the vertical stress increase at any point below an area of any shape carrying uniform pressure.

•Chart consists of influence areas which has a influence value of 0.005 per unit pressure

• The loaded area is drawn on tracing paper to a scale such that the length of the scale line on the chart is equal to the depth z

•Position the loaded area on the chart such that the point at which the vertical stress required is at the centre of the chart.

•Then count the number of influence areas covered by the scale drawing, N

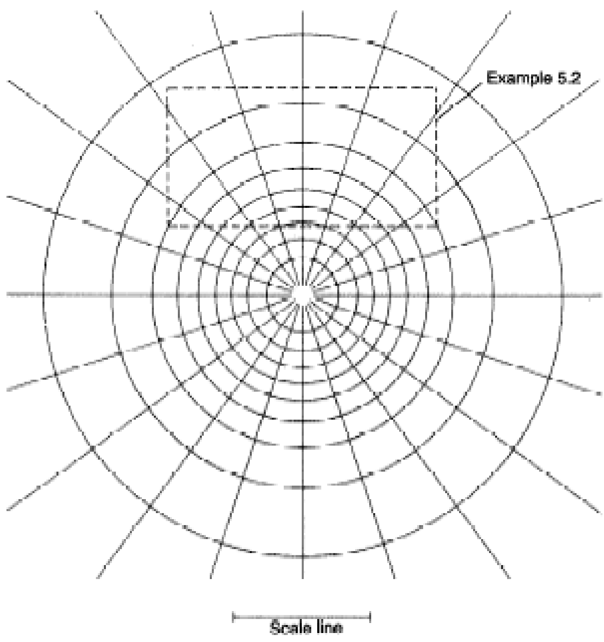

Fig 6.2 ***vertical stress increase at z***

$$\Delta\sigma_Z = \boldsymbol{0.005\, q_o\, N}$$

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Approximate Methods :

1. Equivalent point load method

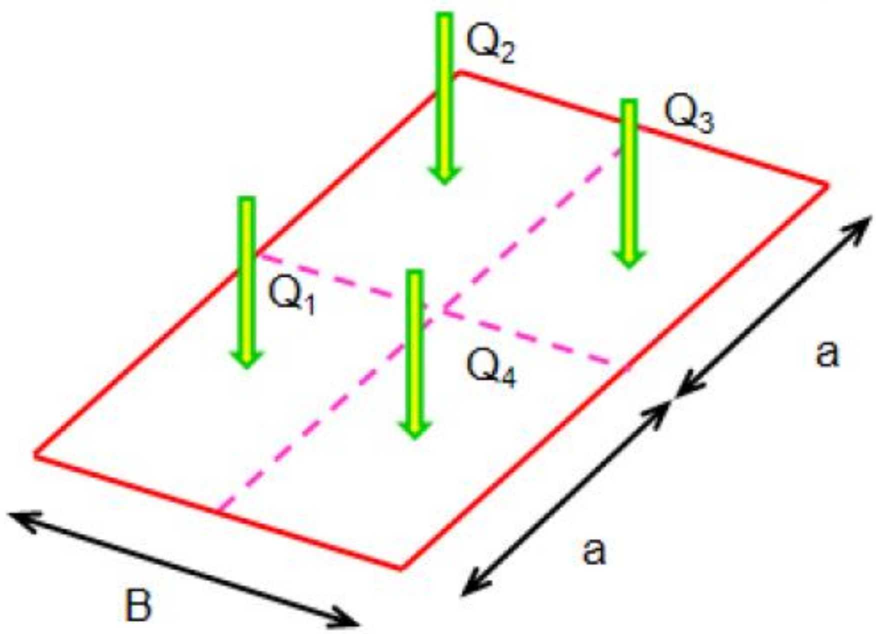

Fig 6.3 Equivalent point load method

$\sigma_Z = (Q_1K_{B1} + Q_2K_{B2} + \ldots\ldots..)$

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2. Two is to one method.

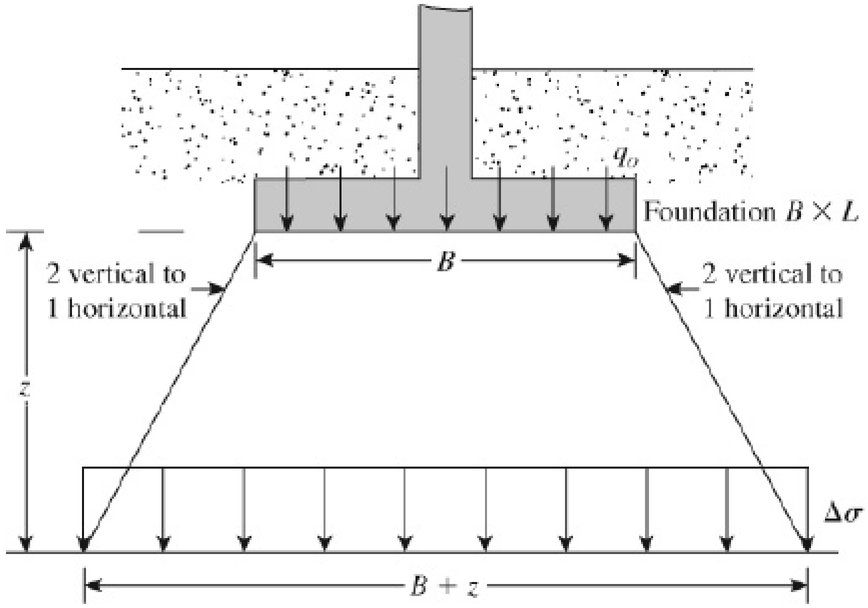

Fig 6.4 Two is to one method

$$\Delta\sigma = \frac{q_o BL}{(B+Z)(L+Z)}$$

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Q. A concentrated load of 22.5 KN acts on the surface of a homogeneous soil mass of large extent. Find the stress intensity at a depth of 15 metres and (i) directly under the load, and (ii) at a horizontal distance of 7.5 metres. Use Boussinesq's equations.

A : According to Boussinesq's theory,

$$\sigma_Z = \frac{Q}{Z^2} \frac{\left(3/2\pi\right)}{\left[1+\left(r/z\right)^2\right]^{5/2}}$$

(i) Directly under the load :

r = 0; $\therefore r/z = 0$

z = 15m, Q = 22.5 KN

$$\therefore \sigma_z = \frac{22.5}{15\times 15}.\frac{\left(3/2\pi\right)}{\left((1+0)^{5/2}\right)}$$

= 47.75 N/m^2

(ii) At a horizontal distance of 7.5 metres :-

r = 7.5 m, z = 15 m

r/z = 7.5/15 = 0.5

$$\sigma_z = \frac{22.5}{15\times 15}.\frac{\left(3/2\pi\right)}{\left[\left(1+(0.5)^2\right)\right]^{5/2}}$$

= 27.33 N/m^2

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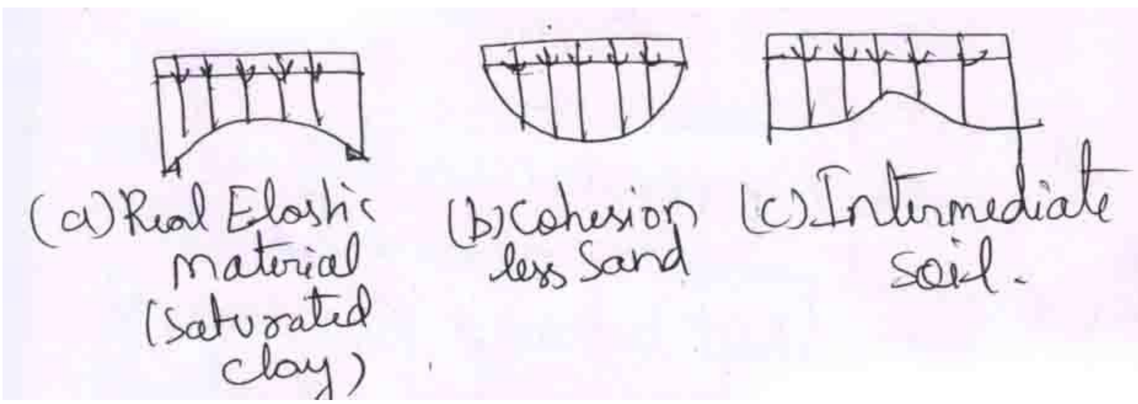
LECTURE 7

Contact pressure :-

Contact pressure is defined as the vertical pressure acting at the surface of the contact between the base of a footing and the underlying soil mass. To simplify design, the computation of the bending moments etc. in the footings is commonly based on the assumption that the footings rest on a uniformly spaced bed of springs so that the distribution of contact pressure is uniform. The actual contact pressure distribution, however, depends upon the flextural rigidity of the footing and the elastic properties of the sub-grade. If the footing is flexible, the distribution of contact pressure is uniform irrespective of the type of the subgrade or under soil material. If the footing is perfectly rigid, the contact pressure distribution depends upon the type of the subgrade.



Contact pressure distribution under Rigid Footings

(Fig. 7.1)

Fig. 7.1 shows the pressure distribution under rigid footing resting over (a) real, elastic material and, (b) conhesionless sand and (c) soil having intermediate characteristics.

In the case of a real elastic material, theoretical intensity of contact pressure at the centre is q/z, and infinite at the outer edges. However local yielding causes redistribution of pressure, making it finite at the edges. When the loading approaches a value sufficient to cause failure of soil, the contact pressure distribution may probably be very nearly uniform. In the case of sand, no resistance to deformation is offered at the outer edges of the footing, making the contact pressure zero there. The pressure distribution is parabolic with maximum value at the centre, though it tends to become more uniform with increasing footing width. When a footing is neither perfectly flexible, nor perfectly rigid, and the underlying soil possesses both cohesion and friction, the contact pressure lies between the extreme conditions for uniform and non-uniform distribution for flexible and rigid footings.

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Factors affecting contact pressure distribution

The factors are:

1. Flexural rigidity of base of footing
2. Type of soil
3. Confinement

Flexural rigidity of base of footing

Uniform loading on a flexible base induces uniform contact pressure on any type of soil, while a rigid base induces non-uniform pressure. Foundation bases are usually thick massive concrete structures, which cannot be treated as ideally flexible.

Type of soil

The contact pressure distribution also depends on the elastic properties of the soil. The elastic properties of soil depends on the type of soil.

a. Sandy soil

b. Clayey soils

c. C-ϕ soil

Confinement

For surface loading in sand contact pressure is zero and for clayey soils, it is very high. When the footings are confined then the edge stresses and the contact pressure distribution changes. In sand, if the foundation is embedded or confined, then there would be some finite contact pressure at the edges. In clayey soil the contact pressure at the edges slightly reduces as confinement increase at the edges to surface loading The more the foundation is below the surface of the sand, the more the shear resistance developed at the edges due to increase in the overburden pressure and as a consequence, the contact pressure distribution tends to be more uniform as compared to being parabolic to surface loading.

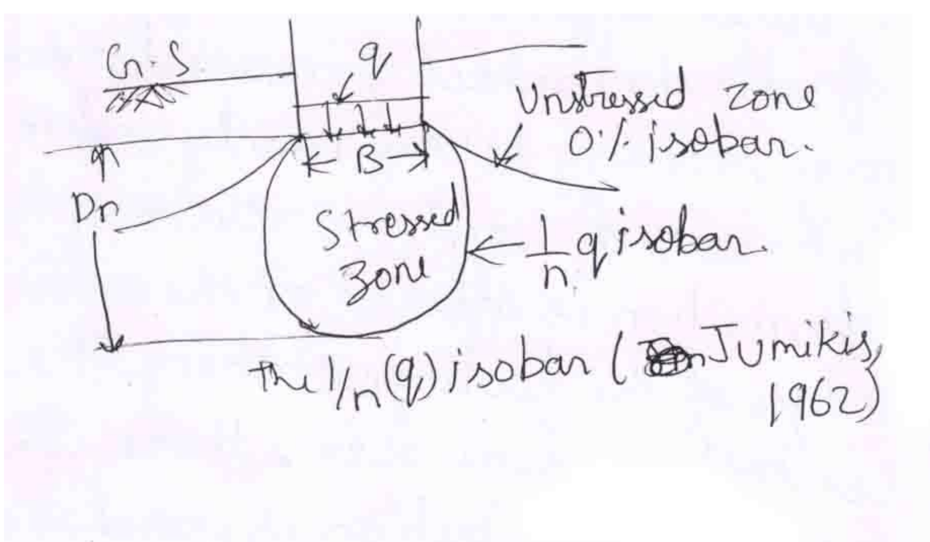


Fig. 7.2 Active zone

Lateral Earth Pressure

At – Rest, Active and Passive Pressures :-

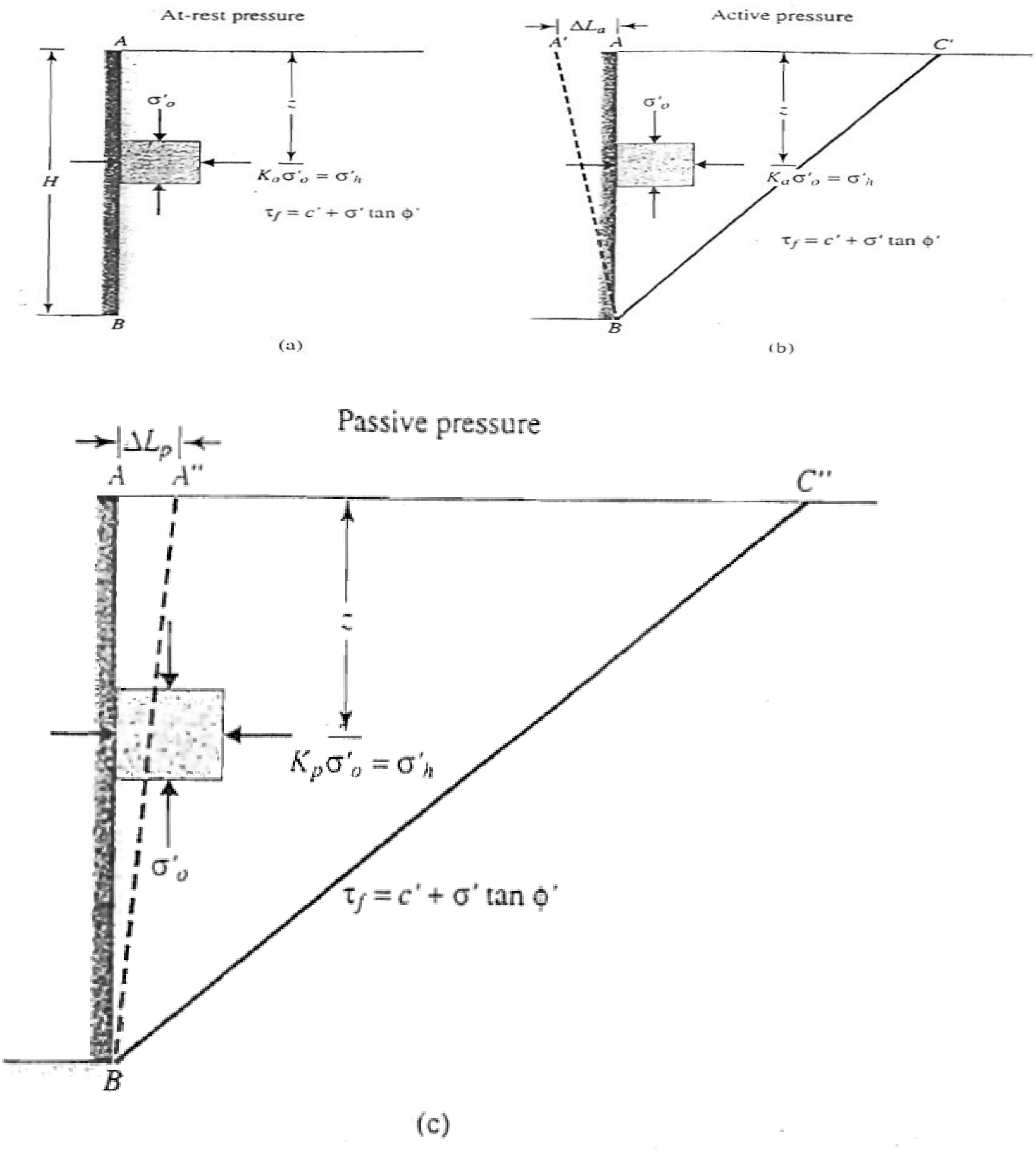


Fig. 7.3Definition of at – rest, and passive pressures (Note : Wall AB is frictionless)

Consider a mass of soil shown in Fig. 7.3(a). The mass, is bounded by a frictionless wall of height AB. A soil element located at a depth 'z' is subjected to a vertical effective pressure σ_0' and a horizontal effective pressure σ_n'. There are no shear stresses on the vertical and horizontal planes of the soil element. Let us define the ratio of σ_n' to σ_0' as a non-dimensional quantity k, or $K = \frac{\sigma_n'}{\sigma_0'}$.

Now, three possible cases may arise concerning the retaining wall and they are described.

Case 1 :-

If the wall AB is static – that is, if it does not move either to the right or to the left of its initial position- the soil mass will be in a state of static equilibrium. In that case, σ_n' is referred to as the at rest earth pressure or $K = K_0 = \frac{\sigma_n'}{\sigma_0'}$.

where K_0 = at rest earth pressure coefficient.

Case 2 :-

If the frictionless wall rotates sufficiently about its bottom to a position of A`B (Fig. 7.3 b), then a triangular soil mass ABC` adjacent to the wall will reach a state of plastic equilibrium and will fail sliding down the plane BC`. At this time, the horizontal effective stress, $\sigma_n' = \sigma_a'$, will be referred to as active pressure. Now,

$$K = \frac{\sigma_n'}{\sigma_0'} = \frac{\sigma_a'}{\sigma_0'} = K_a$$

where K_a = active earth pressure coefficient.

*Under revision

LECTURE 8

Case 3 :-

If the frictionless wall rotates sufficiently about its bottom to a position $A''B$ (Fig.7.3(c)), then a triangular soil mass ABC'' will reach a state of plastic equilibrium and will fail sliding upward along the plane BC''. The horizontal effective stress at this time will be $\sigma_n{}' = \sigma_p{}'$, the so called passive pressure. In this case.

$$K = \frac{\sigma_n{}'}{\sigma_0{}'} = \frac{\sigma_p{}'}{\sigma_0{}'} = \mathrm{K_p}$$

Where K_p = passive earth pressure coefficient.

Rankine's Theory of Active Pressure

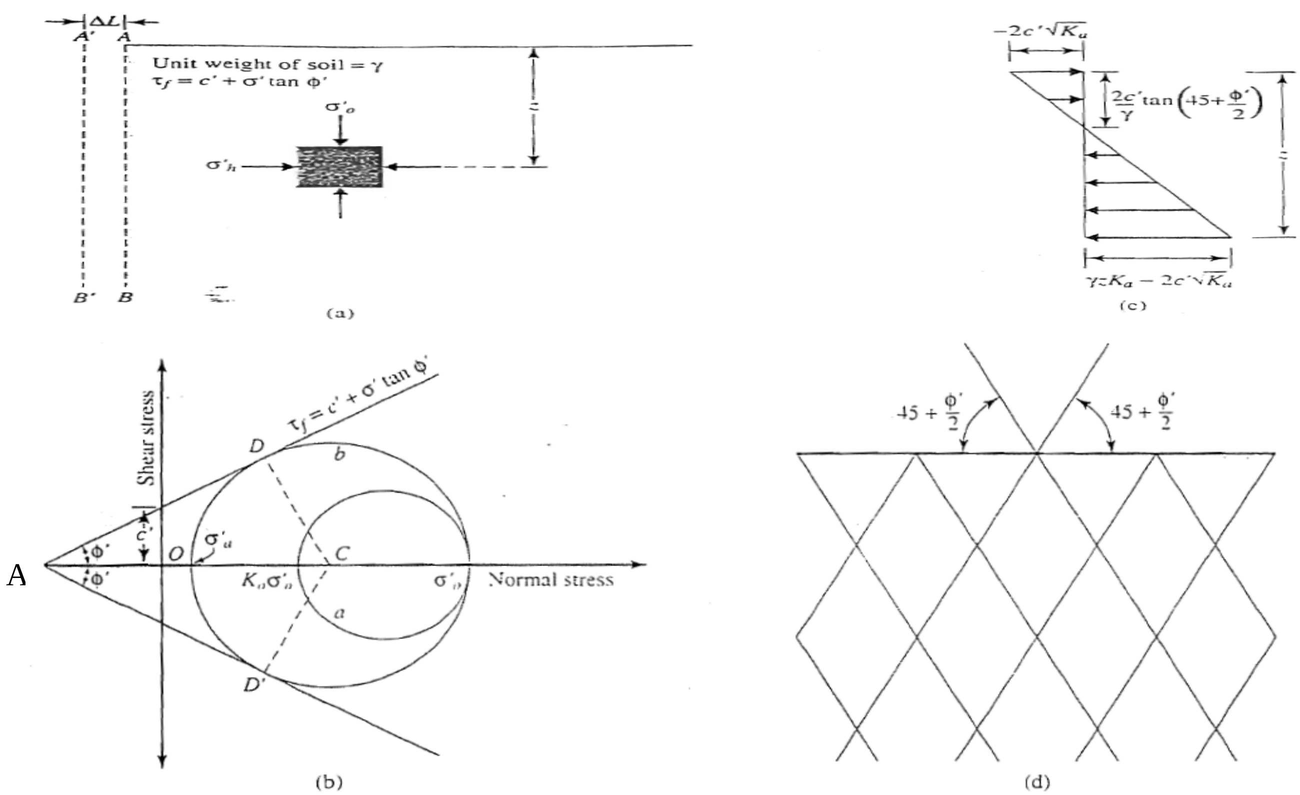


Fig. 8.1 Rankines active earth pressure

$$\sin\phi' = \frac{CD}{AC} = \frac{CD}{AO + OC}$$

*Under revision

But CD = radius of the failure circle.

$$= \frac{\sigma_0' - \sigma_a'}{2}$$

AO = $c' \cot\phi'$ and OC = $\frac{\sigma_0' + \sigma_a'}{2}$

So $\sin\phi' = \dfrac{\dfrac{\sigma_0' - \sigma_a'}{2}}{c' \cot\phi' + \dfrac{\sigma_0' + \sigma_a'}{2}}$

Or $c' \cos\phi' + \dfrac{\sigma_0' + \sigma_a'}{2} \sin\phi' = \dfrac{\sigma_0' - \sigma_a'}{2}$

$$\sigma_a' = \sigma_0' \frac{1 - \sin\phi'}{1 + \sin\phi'} - 2c' \frac{\cos\phi'}{1 + \sin\phi'} \quad \text{-----------------------------} \quad 2.1$$

But σ_0' = Vertical effective overburden pressure = rz

$\dfrac{1 - \sin\phi'}{1 + \sin\phi'} = \tan^2\left(45 - \phi'/2\right)$ and $\dfrac{\cos\phi'}{1 + \sin\phi'} = \tan\left(45 - \phi'/2\right)$

Substituting the preceding values into equation 2.1 we get

$$\sigma_a' = rz \tan^2\left(45 - \frac{\phi'}{2}\right) - 2c' \tan\left(45 - \frac{\phi'}{2}\right)$$

$$K_a = \frac{\sigma_a'}{\sigma_0'} = \tan^2\left(45 - \frac{\phi'}{2}\right)$$

Theory of Rankine’s passive pressure :-

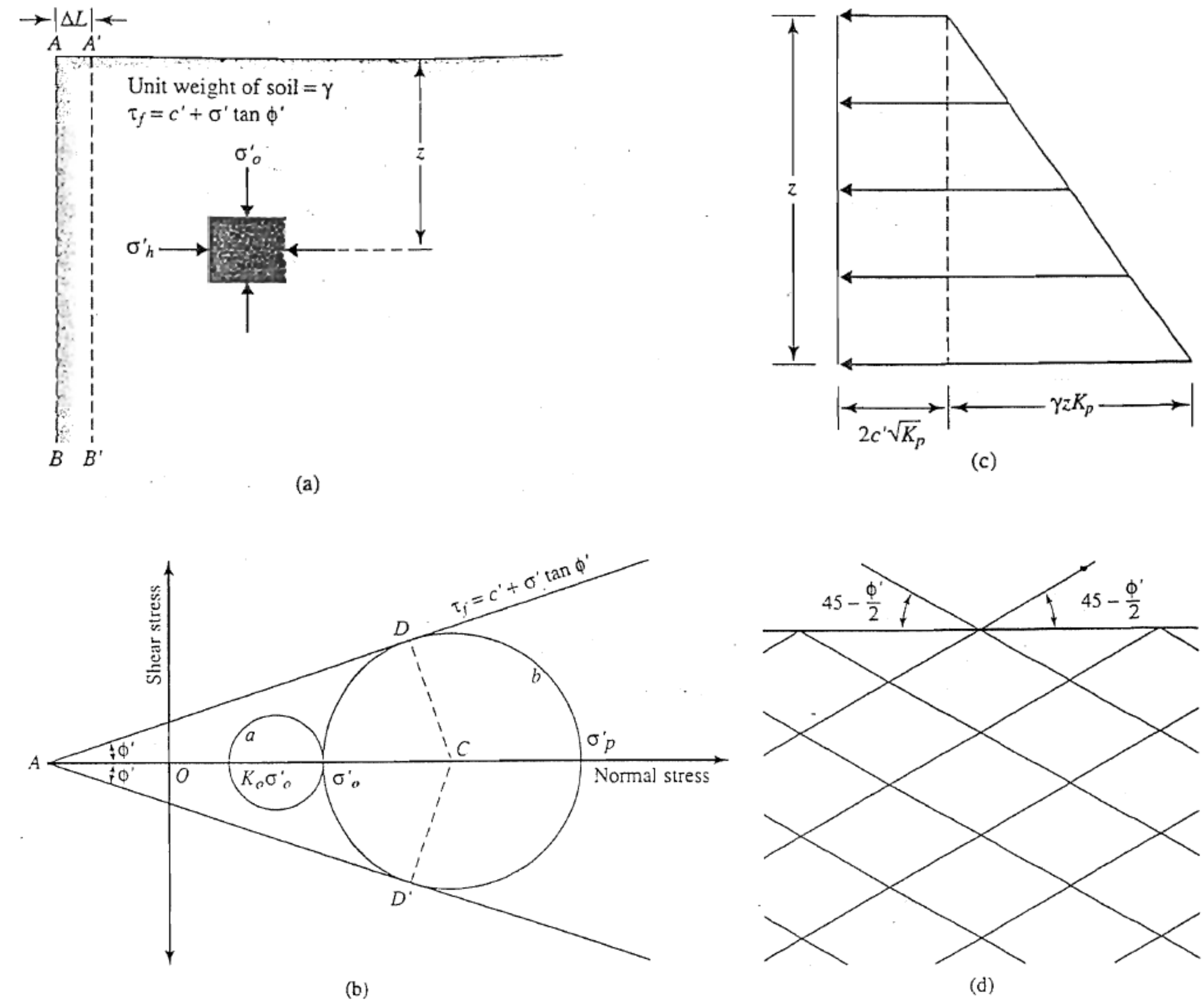


Fig. 8.2 Rankine’s passive earth pressure

$$\sigma_p' = \sigma_0' \tan^2\left(45 + \phi'/2\right) + 2C' \tan\left(45 + \phi'/2\right)$$

$$= rz \tan^2\left(45 + \phi'/2\right) + 2C' \tan\left(45 + \phi'/2\right)$$

$$\frac{\sigma_p'}{\sigma_0'} = \tan^2\left(45 + \frac{\phi'}{2}\right) = \mathrm{K_p}$$

Backfill – Cohesionless soil with horizontal ground surface.

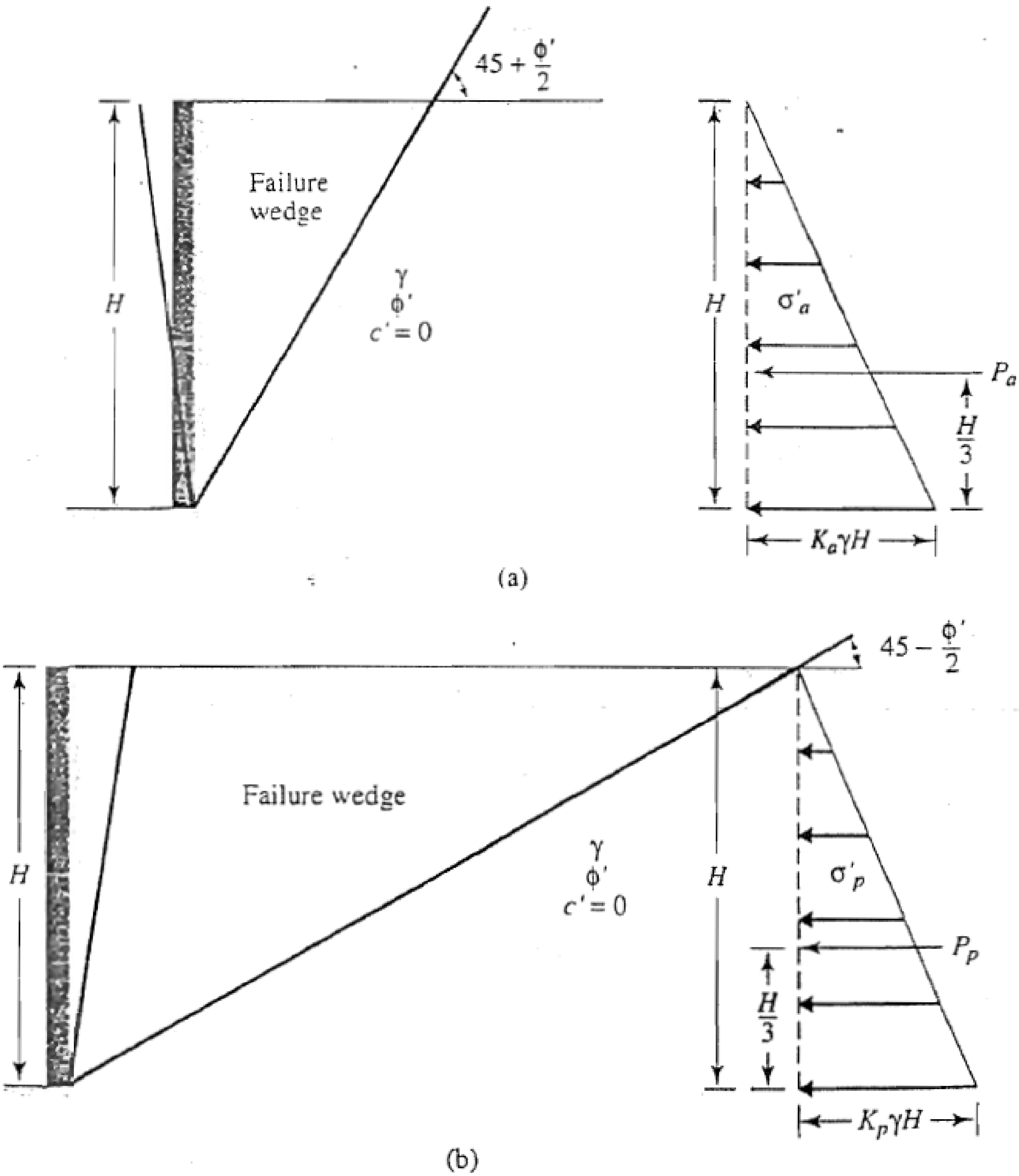


Fig. 8.3 Pressure distribution against a retaining wall for cohesionless soil backfill with horizontal ground surface (a) Rankines active state (b) Rankines passive state

*Under revision

$$\sigma_a{}' = K_a \, \gamma \, z = K_a \, \gamma \, H \quad (Note \; c' = 0)$$

$$P_a = (1/2) K_a \, \gamma \, H^2$$

Passive case

$$\sigma_p{}' = K_p \, \gamma \, H$$

$$P_p = (1/2) K_p \, \gamma \, H^2$$

*Under revision

LECTURE 9

Backfill – cohesive soil with horizontal backfill.

Active Case

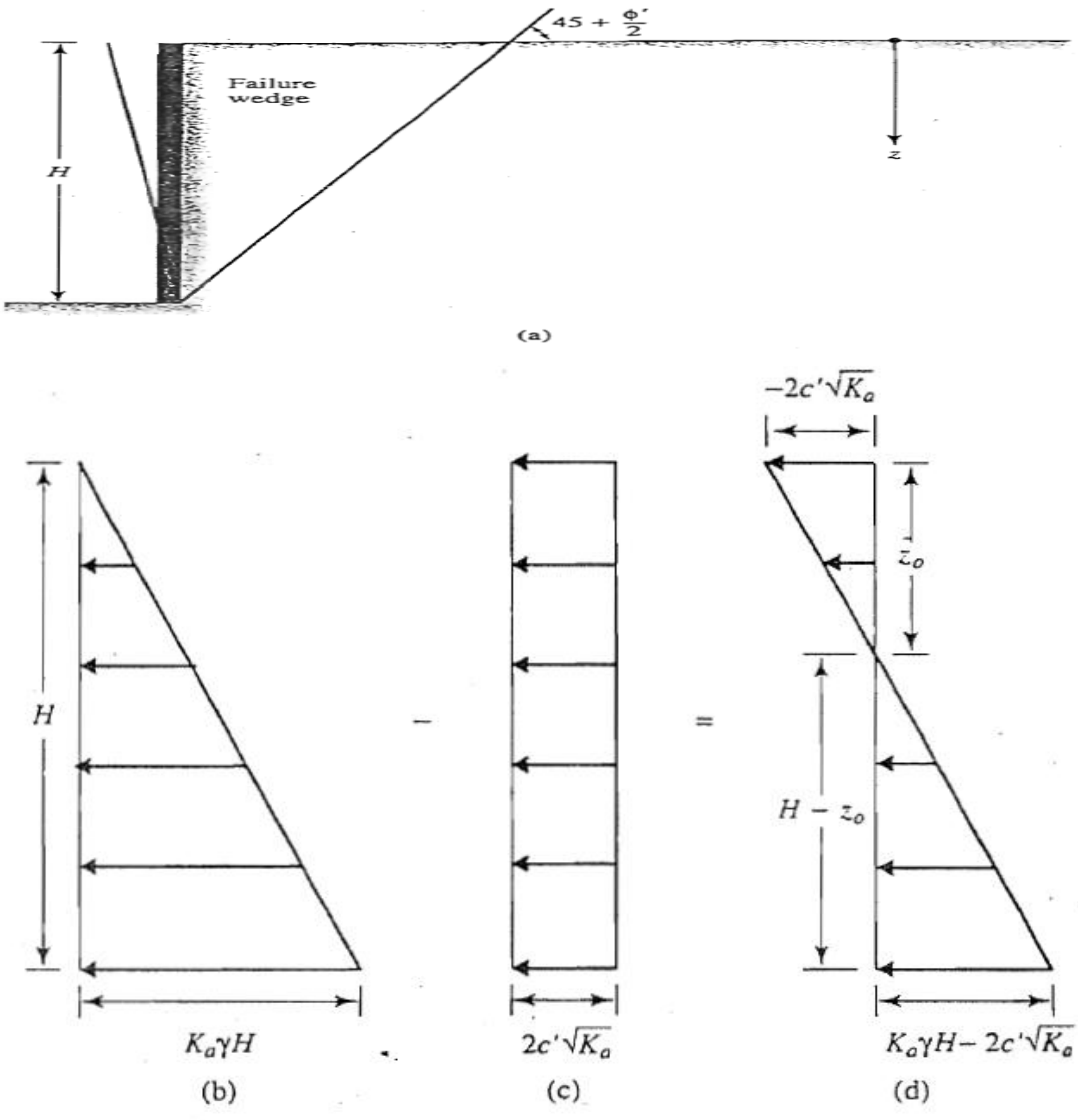


Fig. 9.1

$$\sigma_a' = K_a \gamma z - 2\sqrt{K_a}c'$$

$$K_a \gamma z_o - 2\sqrt{K_a}c' = 0$$

*Under revision

$Z_0 = 2c'/(\gamma \sqrt{ka})$

Passive case

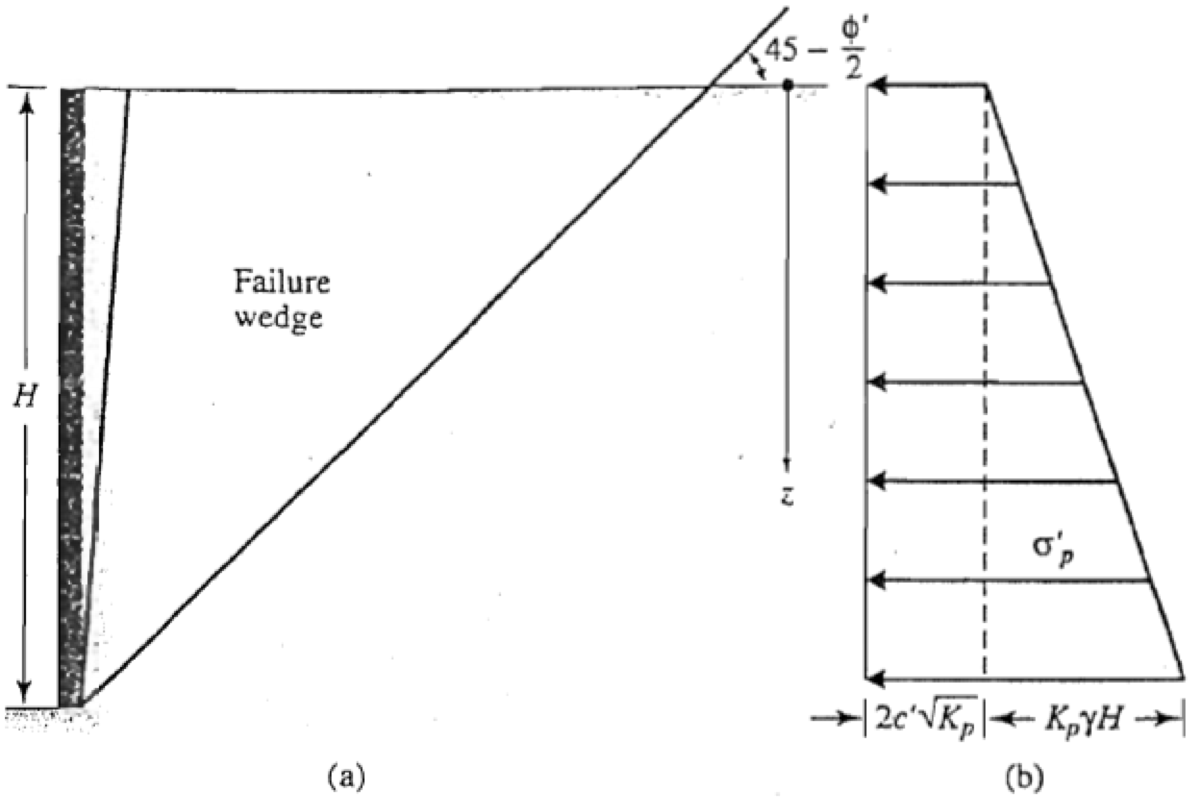


Fig. 9.2

$\sigma_p' = K_p \gamma z + 2\sqrt{K_p}\, c'$

At Z = H

$\sigma_p' = K_p \gamma H + 2\sqrt{K_p}\, c'$

$P_p = (1/2) K_p \gamma H^2 + 2\sqrt{K_p}\, c' H$

*Under revision

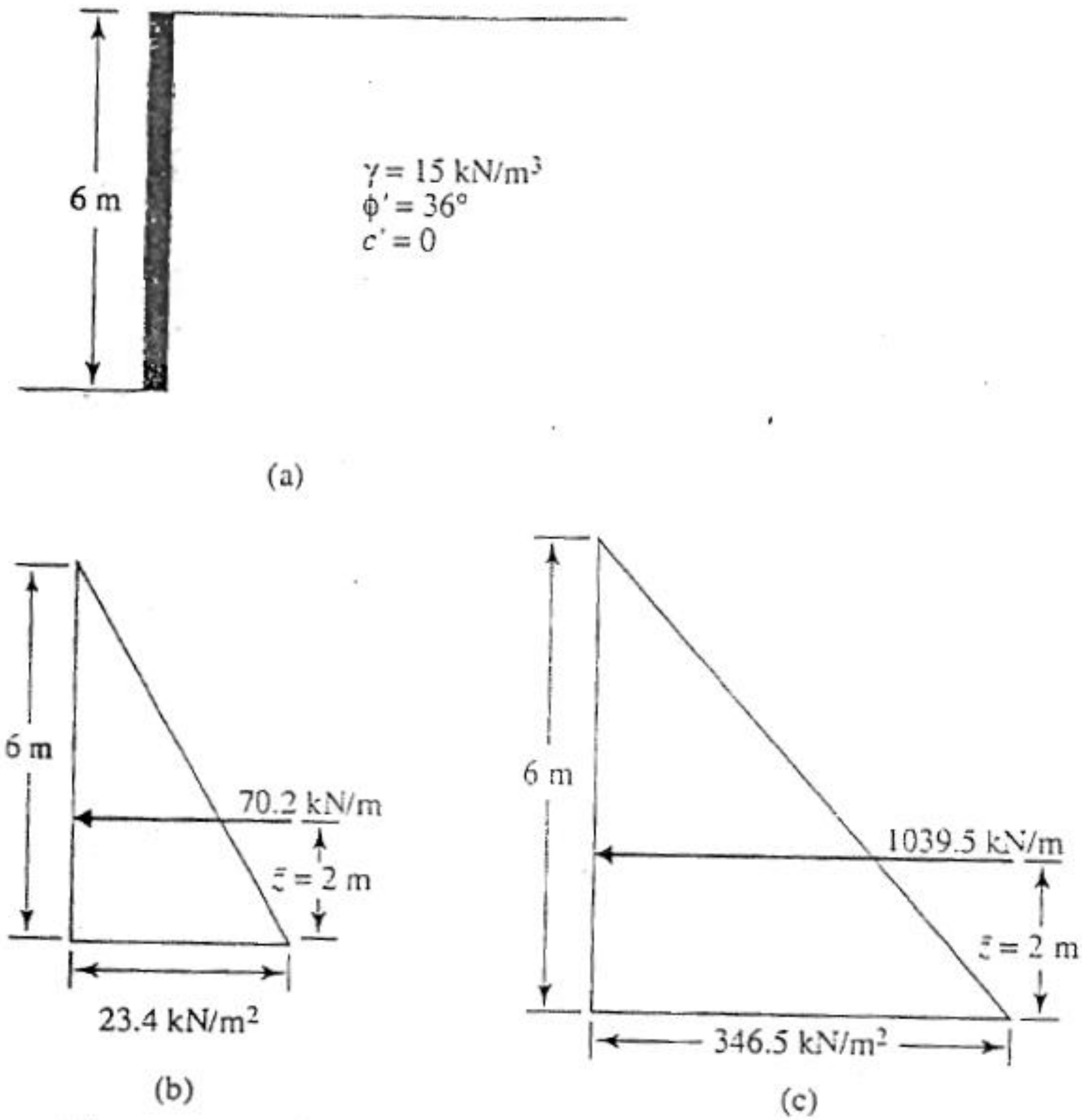


Fig. 9.3

Q.An 6m high retaining wall is shown in above figure determine.

(a) The Rankine active force per unit length of the wall and the location of the resultant.

(b) The Rankine passive force per unit length of the wall and the location of the resultant.

Solution :

As $c' = 0$, so

$\sigma_a{}' = K_a \sigma_0{}' = K_a \gamma z$

$$K_a = \frac{1-\sin\phi'}{1+\sin\phi'} = \frac{1-\sin 36}{1+\sin 36} = 0.26$$

At $Z = 0, \sigma_a{}' = 0; at\ z = 6m$

$$\sigma_a{}' = 0.26 \times 15 \times 6 = 23.4\ kN/m^2$$

$$Pa = \frac{1}{2} \times 6 \times 23.4 = 70.2 kN/m$$

Also,

Z = $\frac{6}{3} = 2m$

$\sigma_p{}' = K_p \sigma_0{}' = K_p \gamma z$

$$K_p = \frac{1+\sin\phi'}{1-\sin\phi'} = \frac{1+\sin 36}{1-\sin 36} = 3.85$$

At z = 0, $\sigma_p{}' = 0$, at z = 6m

$$\sigma_p{}' = 3.85 \times 15 \times 6 = 346.5 kN/m^2$$

$$P_p = \frac{1}{2} \times 6 \times 346.5 = 1039.5 kN/m$$

Also $\overline{Z} = \frac{6}{3} = 2m.$

*Under revision

LECTURE 10

Coulomb's Active Pressure

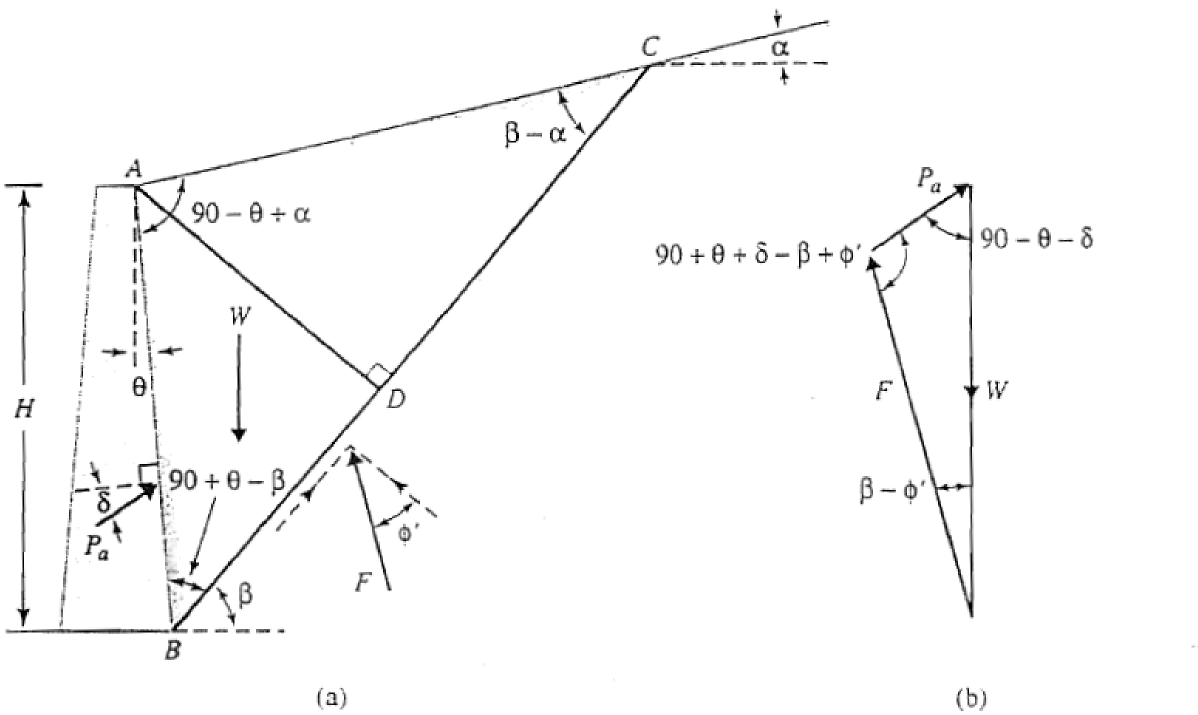


Fig. 10.1 Coulomb's active pressure (a) Trial Failure wedge (b) Force polygon

Forces :-

1. W, the weight of the soil wedge.
2. F, the resultant of the shear and normal forces on the surface of failure BC. This is inclined at an angle of ϕ' to the normal drawn to the plane BC.
3. P_a, the active force per unit length of the wall. The direction of P_a is inclined at an angle to δ to the normal drawn to the face of the wall that supports the soil δ is the angle of friction between the soil and the wall.

The force triangle for the wedge is shown in Fig. 10.1 (b) from the law of sines, we have

$$\frac{W}{\sin(90+\theta+\delta-\beta+\phi')}=\frac{P_a}{\sin(\beta-\phi')}$$

$$P_a = (1/2)\gamma H^2 \left[\frac{\cos(\theta-\beta)\cos(\theta-\alpha)\sin(\beta-\phi')}{\cos^2\theta\sin(\beta-\alpha)\sin(90+\theta+\delta-\beta+\phi')} \right]$$

$$\frac{dp_a}{d\beta} = 0$$

$$P_a = (1/2)K_a\gamma H^2$$

$$K_a = \frac{\cos^2(\phi'-\theta)}{\cos^2\theta\cos(\delta+\theta)\left[1+\sqrt{\frac{\sin(\delta+\phi')\sin(\phi'-\alpha)}{\cos(\delta+\theta)\cos(\theta-\alpha)}}\right]^2}$$

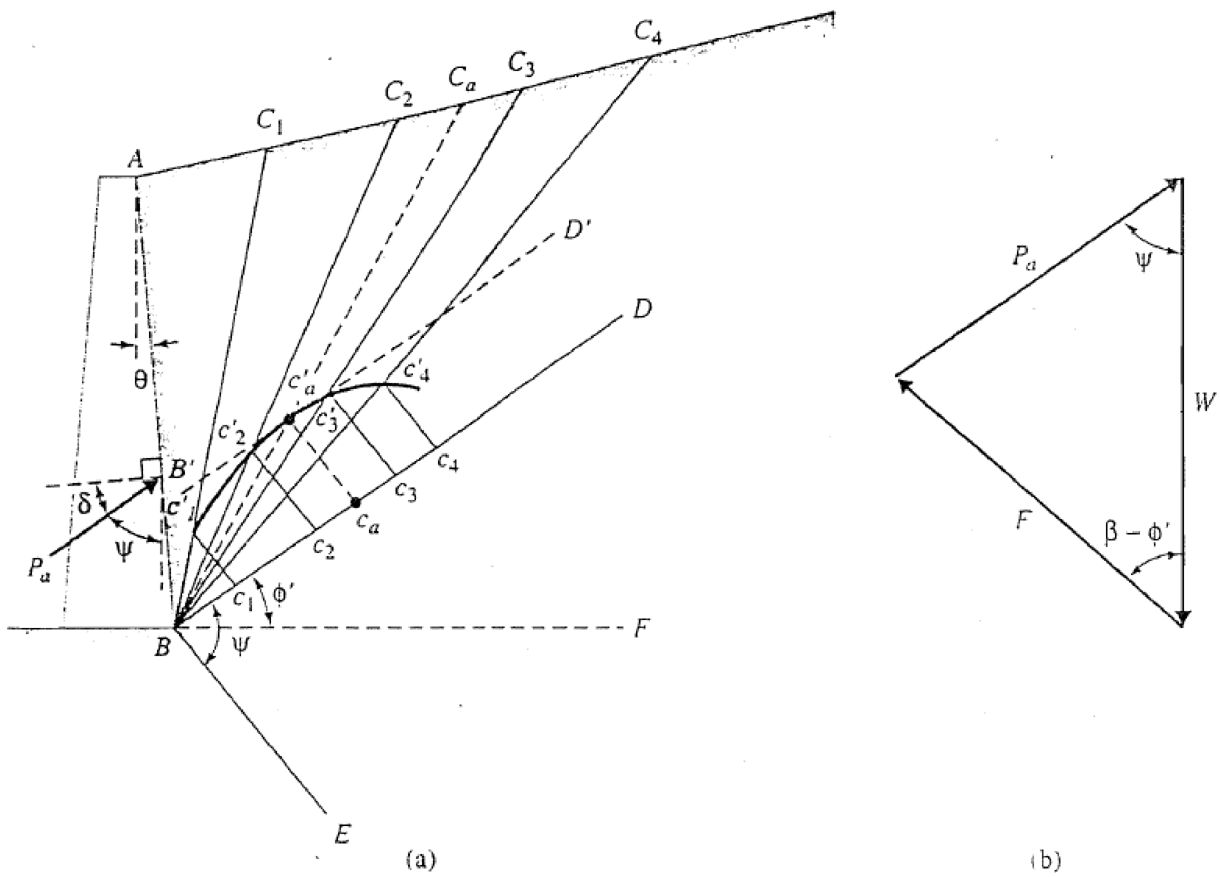


Fig. 10.2 Culmann's solution for active earth pressure.

1. Draw the features of the retaining wall and the backfill to a convenient scale.
2. Determine the value of φ (degrees) = $90-\theta-\delta$, where θ = the inclination of the back face of the retaining wall with the vertical and δ = angle of wall friction.
3. Draw a line BD that makes an angle ϕ' with the horizontal.

4. Draw a line BE that makes an angle ψ with line BD.
5. To consider some trial failure wedges draw lines BC_1, BC_2, BC_3,……….BC_n.
6. Find the areas of ABC_1, ABC_2, ABC_3…..ABC_n.
7. Determine the weight of soil, W, per unit length of the retaining wall in each of the trial failure wedges as follows :

   W_1 = (Area of ABC_1) x γ x (1)

   W_2 = (Area of ABC_2) x γ x 1

   W_3 = (Area of ABC_3) x γ x 1

   .

   .

   .

   .

   W_n = (Area of ABC_n) x γ x (1)
8. Adopt a conventional load scale and plot the weights W_1, W_2, W_3….W_n determined from step 7 on line BD.
9. Draw c_1c_1\`, $c_2$ $c_2$\`, c $_3c_3$\`………$C_nc_n$\` parallel to the line BE.
10. Draw a smooth curve through points c_1\`, $c_2$\`, c_3\`…..$c_n$\`. This curve is called the Culmann line.
11. Draw a tangent B\`D\` to the smooth curve drawn in step 10. B\`D\` is parallel to line BD. Let c_a\` be the point of tangency.
12. Draw a line c_aC_a\` parallel to the line BE.
13. Determine the active force per unit length of wall as

$$P_a = (\text{Length of } C_aC_a\text{`}) \times (\text{Load scale})$$

14. Draw a line Bc_a\`C_a. ABC_a is the desired failure wedge.

*Under revision

Rebhann’s graphical method :

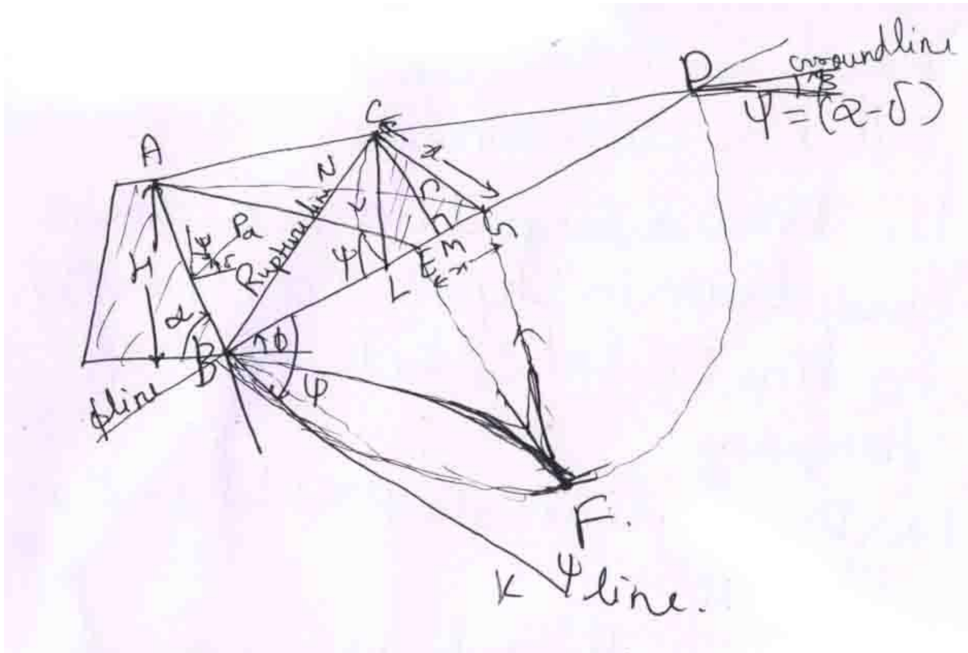


Fig. 10.3 The steps involved in the graphical method are as follows :

1. Let AB represent the back face of the wall and AD the backfill surface.
2. Draw BD inclined at φ with the horizontal from the heel B of the wall to meet the backfill surface in D.
3. Draw BK inclined at $\varphi = (\alpha - \delta\ )$with BD, which is the φ–line.
4. Through A, draw AE parallel to the ψ line to meet BD in E. Alternatively draw AE at $(\varphi + \delta)$ with AB to meet BD in E.
5. Describe a semicircle on BD as diameter.
6. Erect a perpendicular to BD at E to meet the semi-circle in F.
7. With B as centre and BF as radius draw an arc to meet BD in G.
8. Through G, draw a parallel line which is parallel to the ψ line to meet AD in C.
9. With G as centre and GC as radius draw an arc to cut BD in L, join CL and also draw a perpendicular CM from C on to LG. BC is the required rupture surface.

LECTURE 11

Stability considerations for gravity

Retaining walls :

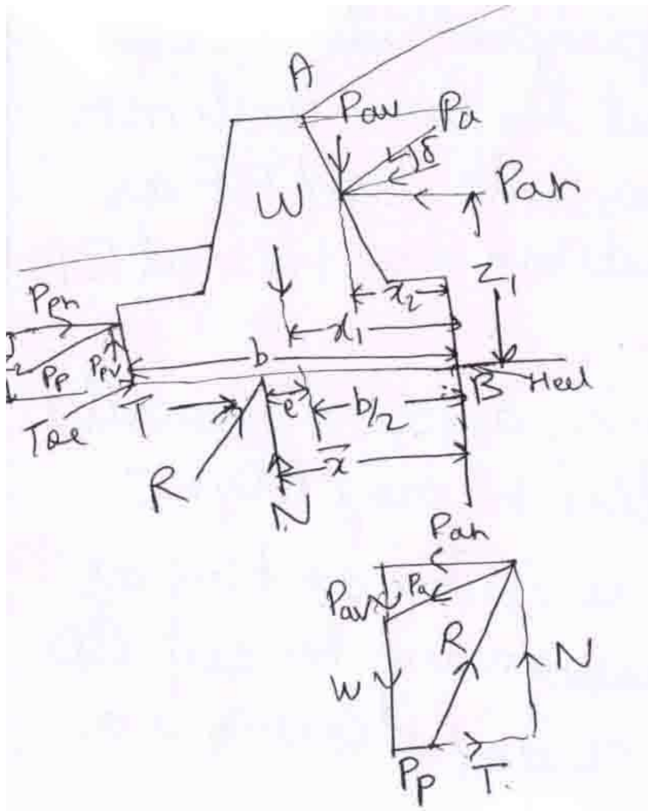


Fig. 11.1 Force diagram

$N = W + P_{av} - P_{pv}$

$T = P_{ah} - P_{ph}$

Taking moments about B.

$$N\bar{x} = Wx_1 + P_{av}x_2 + P_{ah}z_1 - P_{pv}b - P_{ph}z_2$$

$$x = \frac{(wx_1 + p_{av}x_2 + p_{ah}z_1 - p_{pv}b - p_{ph}z_2)}{N}$$

$\Rightarrow \sum M / \sum V$

*e = $\left(\bar{x} \sim b/2\right)$

$\sum M$ = Algebraic sum of the moments of all the actuating forces, other than that of reaction N.

$\sum V$ = Algebraic sum of all the vertical forces, other than T.

*Under revision

The criteria for a satisfactory design of a gravity retaining wall may be enunciated as follows :

(a) The base width of the wall must be such that the maximum pressure exerted on the foundation soil does not exceed the safe bearing capacity of the soil.

(b) Tension should not develop anywhere in the wall.

(c) The wall must be safe against sliding, that is, the factor of safety against sliding should be adequate.

(d) The wall must be safe against overturning, that is the factor of safety against overturning should be adequate.

$$\sigma_{\max} = \frac{N}{b}\left(1+\frac{6e}{b}\right)$$

$$\sigma_{\min} = \frac{N}{b}\left(1-\frac{6e}{b}\right)$$

Stability analysis of infinite slopes :-

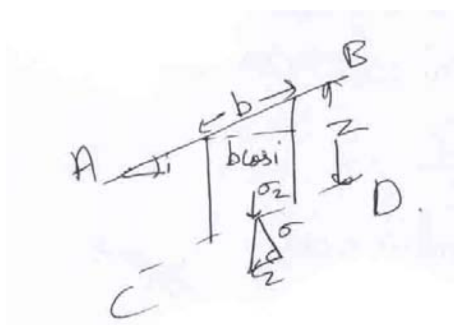


Fig. 11.2 Infinite slope

Considering a prism of soil of inclined length ‘b’ along the slope and depth ‘z’ upto the critical surface. The horizontal length of prism is *bcosi* , and its volume per unit length of prism is zbcosi.

$\therefore$ Weight of prism = w = γzb cos i

$\therefore$ Vertical stress σ_z on the surface CD is given by

$$\sigma_z = \frac{W}{b} = \gamma z \cos i$$

*Under revision

If σ and τ are the stress components normal and tangential to the surface CD, we have $\sigma = \sigma_z \cos i = \gamma z \cos^2 i$.

and $\tau = \sigma_z \sin i = \gamma z \cos i \sin i$

$$F = \frac{\tau_F}{\tau}$$

Case (i) cohesionless soil.

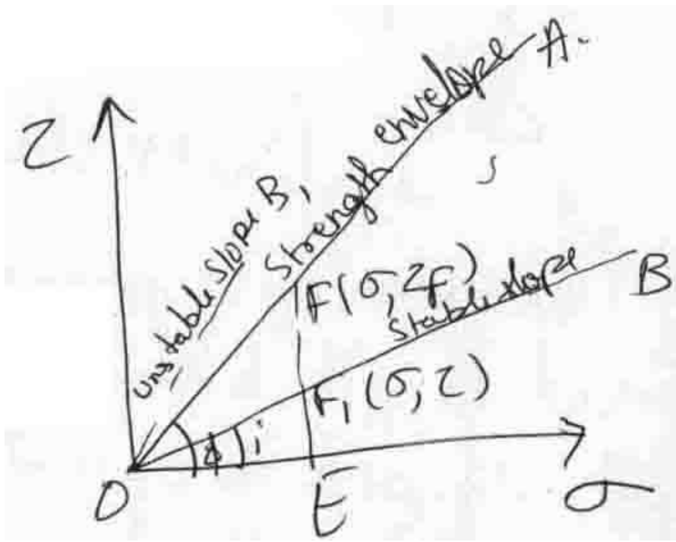


Fig. 11.3

$\tau_F = \sigma \tan \phi$

$\frac{\sigma}{z} = \frac{\cos i}{\sin i} = \cot i = \text{constant.}$

$\sigma = \tau \cot i$

$\tau = \sigma \tan i$

$$F = \frac{\tau_F}{\tau} = \frac{\tan \phi}{\tan i}$$

*Under revision

Case (ii) cohesive soil :-

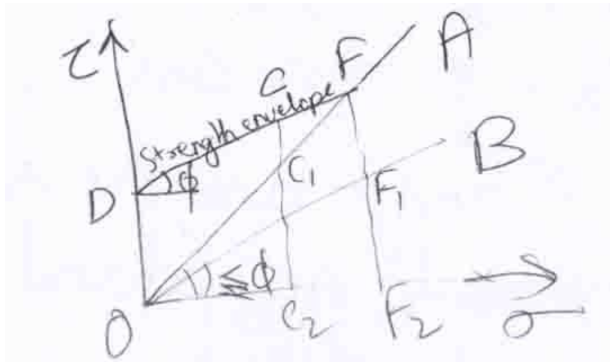


Fig. 11.4

$$\tau_F = c + \sigma \tan\phi$$

$$F = \frac{\tau_F}{\tau} = \frac{c + \sigma\tan\phi}{\tau}$$

Putting $\sigma = \gamma z \cos^2 i$

and $\tau = \gamma z \cos i \sin i$ we get

$$F = \frac{c + \gamma z \cos^2 i \tan\phi}{\gamma z \cos i \sin i} = \frac{C}{\gamma z \sin i \cos i} + \frac{\tan\phi}{\tan i}$$

For $z = H_c$ and $F = 1$

$\gamma H_c \cos i \sin i = c + \gamma H_c \cos^2 i \tan\phi$

$$H_c = \frac{c}{\gamma}\frac{1}{(\tan i - \tan\phi)\cos^2 i}$$

$$\frac{C}{\gamma H_c} = (\tan i - \tan\phi)\cos^2 i$$

$$S_n = \frac{C}{\gamma H_c} = \text{Stability number.}$$

If F_c represents the factor of safety with respect to cohesion and let C_m be the mobilized cohesion, at depth H, given by $\tau_m = \frac{\tau}{F_c}$

$$S_n = \frac{C}{\gamma H_c} = \frac{C_m}{\gamma H} = \frac{C}{F_c \gamma H}$$

*Under revision

$F_c = \frac{H_c}{H} =$ factor of safety with respect to cohesion.

*Under revision

LECTURE 12

Stability analysis of finite slopes.

Two basic types of failure of a finite slope may occur (i) slope failure

(ii) Base Failure.

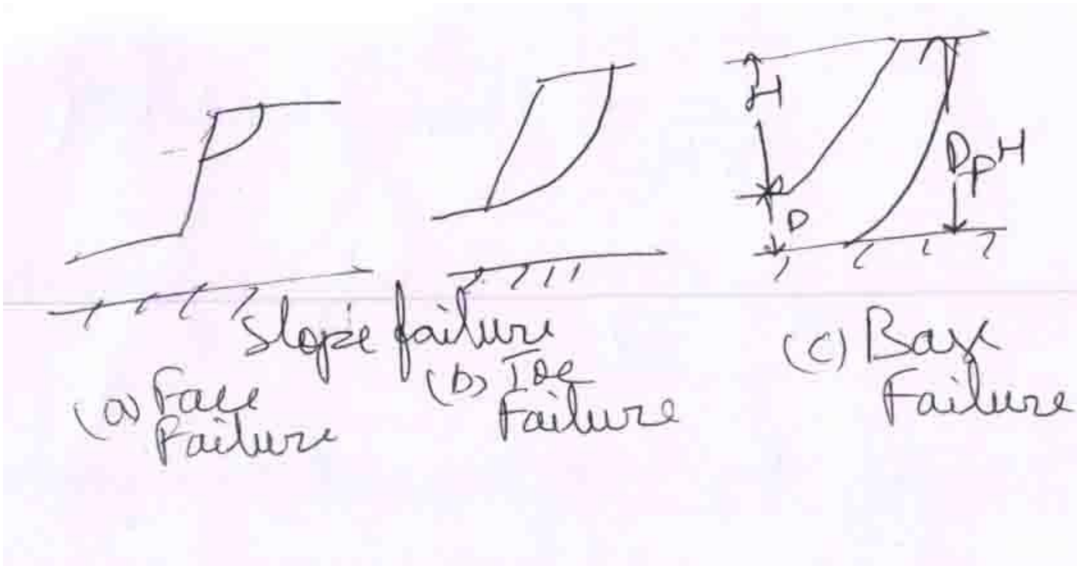


Fig. 12.1

Swedish slip circle method :

(i) Analysis of purely cohesive soil (ϕ_u =0 analysis)

(ii) Analysis of a soil possessing both cohesion and friction (c-ϕ analysis)

(i) ϕ_u = 0 analysis.

*Under revision

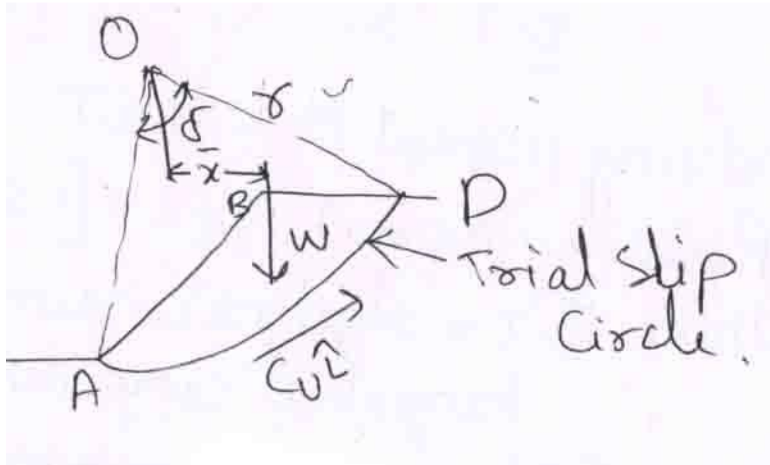


Fig. 12.2

$$F = \frac{M_R}{M_D} = \frac{C_u \hat{L} r}{W\bar{x}}$$

$$W\bar{x} = c_m \hat{L} r \;\text{ or }\; c_m = \frac{W\bar{x}}{\hat{L}} r$$

$$F = \frac{c_u}{c_m} = \frac{C_u \hat{L} r}{W\bar{x}}$$

(ii) C - ϕ analysis

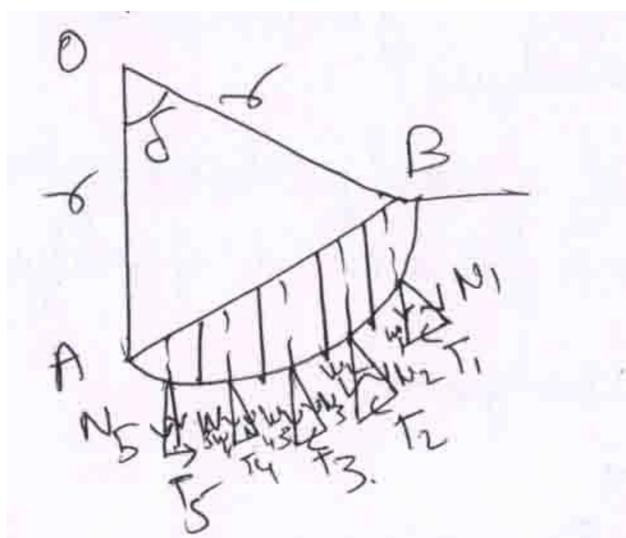


Fig. 12.3

*Under revision

Driving moment $M_D = r\sum T$

Resisting moment $M_R = r\left[c\sum \Delta L + \tan\phi \sum N\right]$

where $\sum T$ = algebraic sum of all tangential components

$\sum N$ = sum of all normal components

$\sum \Delta L = \widehat{L} = \dfrac{2\pi r\delta}{360^0}$ = length AB of slip circle.

$$F = \frac{M_R}{M_D} = \frac{c\widehat{L} + \tan\phi \sum N}{\sum T}$$

*Under revision

Friction Circle Method :-

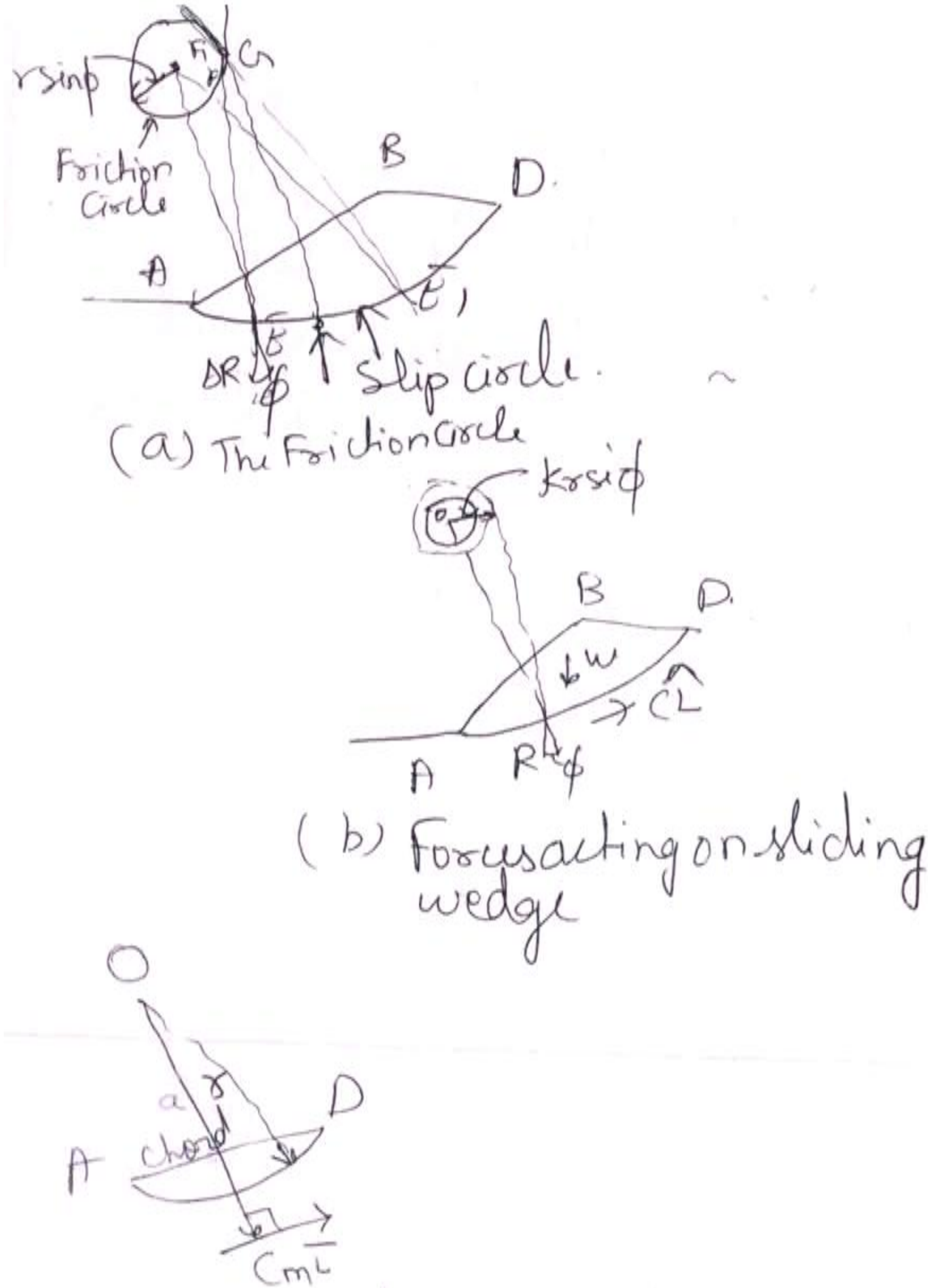


Fig. 12.4

(c) Position of mobilized resultant cohesion.

Mobilized cohesion on elementary arc of length $\Delta L = C_m \Delta L$.

*Under revision

Total cohesive resistance = $c_m \widehat{L} = c_m \sum \Delta L$

$\overline{L}$ = Length of chord AD

Total cohesive force represented by AD = $c_m \overline{L}$

If ‘a’ is the perpendicular distance of direction of $c_m \overline{L}$ from 0, we have

$$(c_m \overline{L})a = \left(c_m \sum \Delta L\right) r = c_m \widehat{L}.r$$

$$\therefore a = r \frac{\widehat{L}}{L}$$

$$F_c = \frac{c}{c_m}$$

LECTURE 13

Taylor's stability Number

$$\frac{C \times H}{F_c \times \gamma H^2} = \frac{C}{F_c \gamma H} = S_n$$

The dimensionless quantity $\frac{C}{F_c \gamma H}$ is called Taylor's Stability Number (S_n)

$$c_m = \frac{c}{F_c}$$

$$S_n = \frac{C}{F_c \gamma H} = \frac{c_m}{\gamma H}$$

$$F_c = \frac{H_c}{H}$$

$$S_n = \frac{C}{F_c \gamma H} = \frac{C}{\gamma H_C} = \frac{c_m}{\gamma H}$$

Table 13.1 Taylor's Stability Number :

$\phi \rightarrow$ / $i \downarrow$	0^0	5^0	10^0	15^0
90^0	0.261	0.239	0.218	0.199
75^0	0.219	0.195	0.173	0.152
60^0	0.191	0.162	0.138	0.116

Q. A slope is to be constructed at an inclination of 30^0 with the horizontal determine the safe height of the slope at factor of safety of 1.5. The soil has the following properties :

$c = 15$ kN/m^2, $\phi = 22.5^0$ and $\gamma = 19$ kN/m^3.

Solution :

The mobilized friction angle ϕ_m is given by $\phi_m = \frac{\phi}{F} = \frac{22.5}{1.5} = 15^0$

For i = 30^0 and $\phi_m = 15^0$, $S_n = 0.046$

Now $S_n = \frac{c}{F\gamma H} \therefore H = \frac{C}{S_n F \gamma}$

$$H = \frac{15}{0.046 \times 1.5 \times 19} = 11.44m$$

*Under revision

Bishop's simplified method :

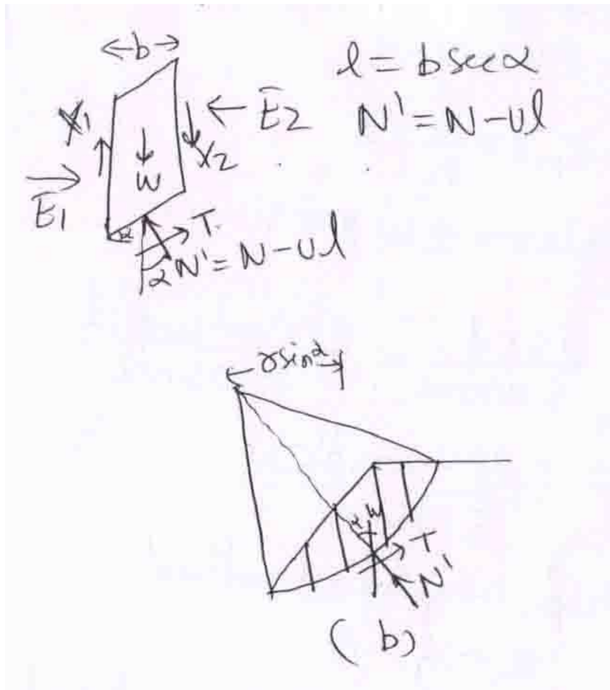


Fig. 13.1 Forces acting on the slices are :

(1) Weight of slice W = γhb, where 'h' is the average height.

(2) Normal force on the base $N' = N - ul$ where 'u' is the pore pressure and 'l' is the length of the base , l = b sec α.

(3) Shear force on the base, T = τ_m l.

(4) Normal forces on the sides E_1 and E_2.

(5) Shear forces on the sides X_1 and X_2.

(6) Any external force acting on the slice.

Taking moments about 0

$$\sum Tr = \sum Wr\sin\alpha$$

But $T = \tau_m l = \frac{S}{F_s} \times l$.

Therefore $\sum \frac{S}{F_S} \times l \times r = \sum W \times r \sin\alpha$

$$F_S = \frac{\sum s \times l}{\sum w \sin\alpha} = \frac{\sum (c' + \sigma' \tan\phi')l}{\sum w \sin\alpha}$$

$$F_S = \frac{\sum c'l + \tan\phi' \sum N'}{\sum w \sin\alpha}$$

In Bishop's simplified method $X_1 - X_2 = 0$.

Resolving the forces in the vertical direction,

$$W = N'\cos\alpha + ul\cos\alpha + T\sin\alpha + X_1 - X_2$$

Substituting $T = \frac{S}{F_s} \times l$ and $X_1 - X_2 = 0$. we have

$$W = N'\cos\alpha + ul\cos\alpha + \frac{(c'l + N'\tan\phi')}{F_s}\sin\alpha$$

$$N' = \frac{W - ul\cos\alpha - \frac{c'l\sin\alpha}{F_s}}{\cos\alpha + \frac{\tan\phi'}{F_s}\sin\alpha}$$

$$F_s = \frac{\sum \frac{1}{m_\alpha}\left[c'b + (w - ub)\tan\phi'\right]}{\sum W\sin\alpha}$$

here $m_\alpha = (1 + \tan\phi' \tan\alpha / F_s)\cos\alpha$

Soil Exploration :-

Primary objectives

(i) Determination of the nature of the deposits of soil.

(ii) Determination of the depth and thickness of the various soil strata and their extent in the horizontal direction.

(iii) The location of ground water and fluctuations in GWT.

(iv) Obtaining soil and rock samples from the various strata.

Soil Exploration :-

The subsoil exploration should enable the engineer to draw the soil profile indicating the sequence of the strata and the properties of the soils involved.

In general the methods available for soil exploration may be classified as follows:

1. Direct methods – Test pits, trial pits or Trenches.
2. Semi direct methods…… Borings.
3. Indirect methods…… Soundings or penetration tests and geophysical methods.

Test Pits :

Test pits or trenches are open type or accessible exploratory methods. Soils can be inspected in their natural condition. The necessary soil samples may be obtained by sampling techniques and used for ascertaining strength and other engineering properties by appropriate laboratory tests.

*Under revision

LECTURE 14

Boring :-

Making or drilling boreholes into the ground with a view to obtain soil or rock samples from specified or known depths is called ‘boring’.

The common methods of advancing bore holes are :-

1. Auger boring.
2. Auger and shell boring.
3. Wash boring
4. Percussion drilling.
5. Rotary drilling.

} More commonly employed for sampling in Rock Strata.

Table 14.1 Spacing of Borings :

	Nature of the project	Spacing of borings (metres)
1	Highway (subgrade survey)	300 to 600
2	Earth dam	30 to 60
3	Borrow pits	30 to 120
4	Multistory buildings	15 to 30
5	Single story factories	30 to 90

Depth of Borings :-

In order to furnish adequate information for settlement predictions, the borings should penetrate all strata that could consolidate significantly under the load of the structure. This necessarily means that, for important and heavy structures such as bridges and tall buildings, the borings should extend to rock. Recommended depths of borings for building are about 3.5 m and 6.5 m for single and two story buildings.

Soil sampling :-

Soil sampling is the process of obtaining samples of soil from the desired depth at the desired location in a natural soil deposit, with a view to assess the engineering properties of the soil for ensuring a proper design of the foundation.

*Under revision

Types of samples :-

Samples of soil taken out of natural deposits for testing may be classified as disturbed samples and undisturbed samples, depending upon the degree of disturbance caused during sampling operations.

Sounding and penetration tests :-

Methods of sounding normally consist of driving or pushing a standard sampling tube or a cone.

If a sampling tube is used to penetrate the soil, the test is referred to as the Standard Penetration Test (SPT). If a cone is used to penetrate the soil, the test is called a cone penetration test. Static and dynamic cone penetration tests are used depending upon the mode of penetration- static or dynamic.

Standard Penetration Test (SPT) :-

SPT is widely used to determine the parameters of the soil in-situ. The test is especially suited for cohessionless soils as a correlation has been established between the SPT value and the angle of internal friction of the soil.

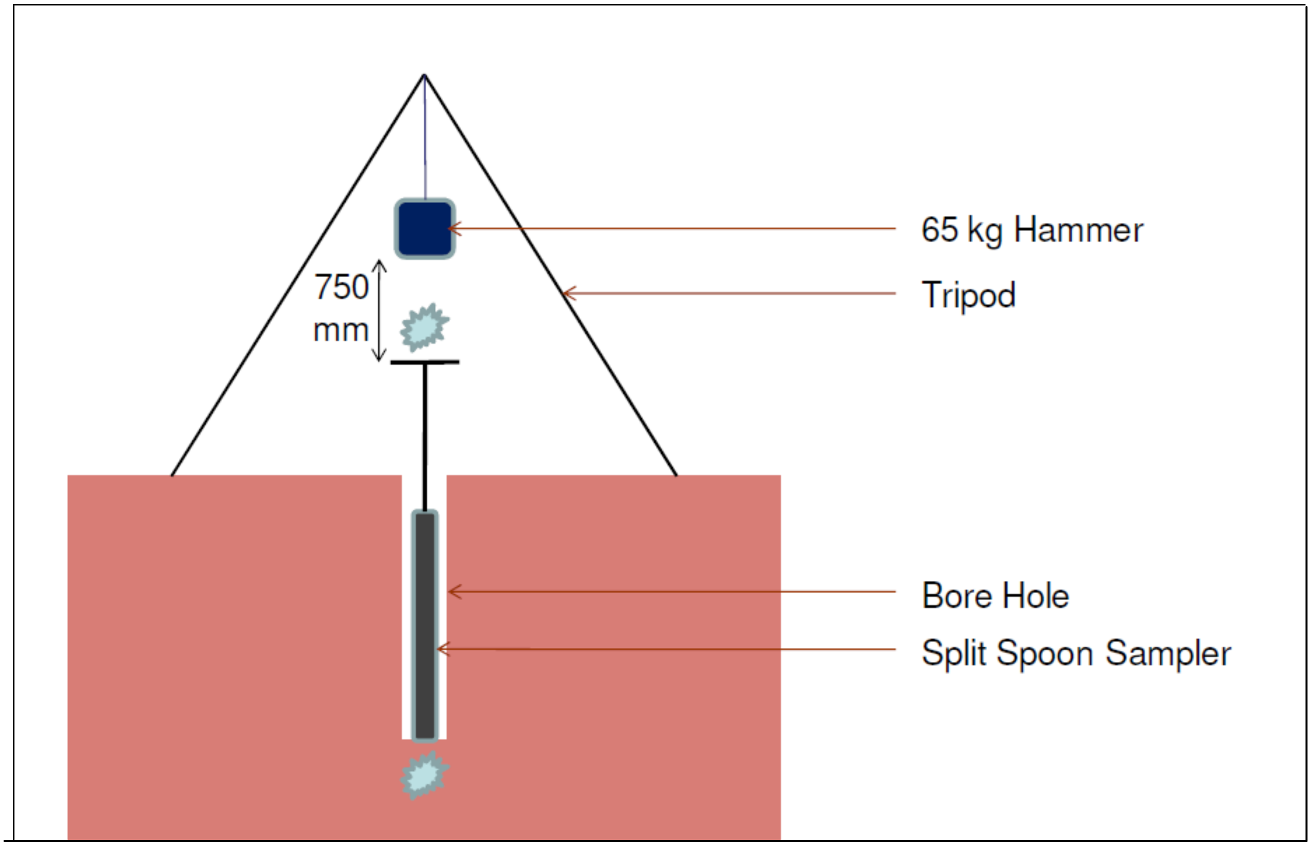


Fig. 14.1 Typical set up for Standard Penetration test assembly

1. Reference can be made to IS 2131 ± 1981 for details on Standard Penetration Test.
2. It is a field test to estimate the penetration resistance of soil.
3. It consists of a split spoon sampler 50.8 mm OD, 35 mm ID, min 600 mm long and 63.5 kg hammer freely dropped from a height of 750 mm.
4. Test is performed on a clean hole 50 mm to 150 mm in diameter.
5. Split spoon sampler is placed vertically in the hole, allowed to freely settle under its own weight or with blows for first 150 mm which is called seating drive.
6. The number of blows required for the next 300 mm penetration into the ground is the standard penetration number N
7. Apply the desired corrections (such as corrections for overburden pressure, saturated fine silt and energy).

*Under revision

8. N is correlated with most properties of soil such as friction angle, undrained cohesion, density etc.

Advantages of Standard Penetration Test are

1. Relatively quick & simple to perform
2. Equipment & expertise for test is widely available
3. Provides representative soil sample
4. Provides useful index for relative strength & compressibility of soil
5. Able to penetrate dense & stiff layers
6. Results reflect soil density, fabric, stress strain behavior
7. Numerous case histories available

Disadvantages of Standard Penetration Test are

1. Requires the preparation of bore hole.
2. Dynamic effort is related to mostly static performance
3. If hard stone is encountered, difficult to obtain reliable result.
4. Test procedure is tedious and requires heavy equipment.
5. Not possible to obtain properties continuously with depth.

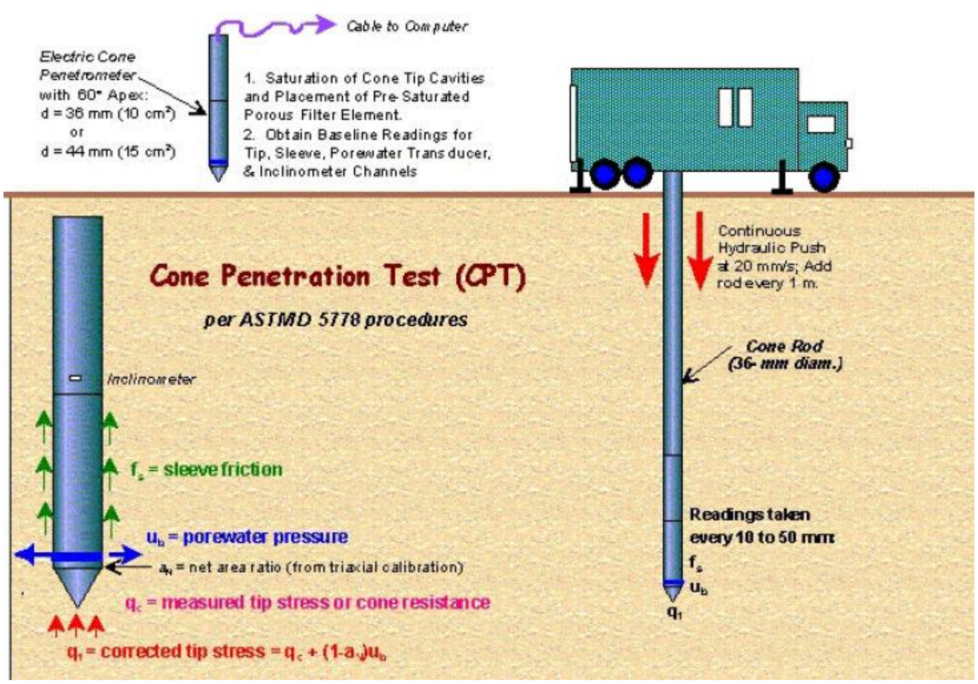
The number of blows for a penetration of 300 mm is designated as the standard penetration value or number N.

Corrections for the observed N values :-

1. Due to overburden
2. Due to dilatancy.

LECTURE 15

Cone Penetration Test



Typical set up for Static Cone Penetration test assembly

Cone Penetration Test can either be Static Cone Penetration Test or Dynamic Cone Penetration Test. Continuous record of penetration resistance with depth is achieved. Consists of a cone 36 mm dia (1000 mm^2) and 60^o vertex angle. Cone is carried at the lower end of steel rod that passes through steel tube of 36 mm dia. Either the cone, or the tube or both can be forced in to the soil by jacks. Cone is pushed 80 mm in to the ground and resistance is recorded , steel tube is recorded, steel tube is pushed up to the cone and resistance is recorded .Further ,both cone and tube are penetrated 200 mm and resistance is recorded. Total resistance (q_c) gives the CPT value expressed in kPa.

Cone resistance represents bearing resistance at the base and tube

*Under revision

resistance gives the skin frictional resistance. Total resistance can be correlated with strength properties, density and deformation characteristics of soil. Correction for overburden pressure is applied. Approximately,

$$N = 10q_c \text{ (kPa)}$$

Advantages of SCPT are

1. Continuous resistance with depth is recorded.
2. Static resistance is more appropriate to determine static properties of soil.
3. Can be correlated with most properties of soil.

Disadvantages of SCPT are

1. Not very popular in India.
2. If a small rock piece is encountered, resistance shown is erratic & incorrect.
3. Involves handling heavy equipment.

Static Cone Penetration Test :-

The static cone test is most successful in soft or loose soils like silty sands, loose sands, layered deposits of sands, silts and clays as well as in clayey deposits.

Basically the test procedure for determining the static cone and frictional resistances consists of pushing the cone alone through the soil strata to be tested, then the cone and the friction jacket, and finally the entire assembly in sequence and noting the respective resistance in the first two cases. The process is repeated at predetermined intervals. After reaching the deepest point of investigation the entire assembly should be extracted out of the soil.

Dynamic cone penetration test :

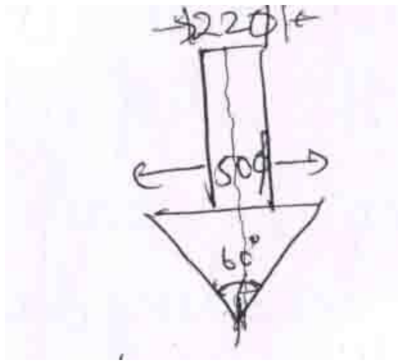


Fig. 15.1 Cone without threads

The cone shall be driven into the soil by allowing the hammer to fall freely through 750 mm each time.

In-situ vane shear test :-

The vane is pushed with a moderately steady force up to a depth of four times the diameter of the borehole or 50 cm, whichever is more, below the bottom. No torque shall be applied during the thrust. The torque applicator is tightened to the frame properly. After about 5 minutes, the gear handle is turned so that the vane is rotated at the rate of 0.1^0/s. The maximum torque reading is noted when the reading drops appreciably from the maximum –

$$\tau = \frac{T}{\pi D^2\left[\left(H/2\right)+\left(D/6\right)\right]}$$

*Under revision

Seismic Refraction :-

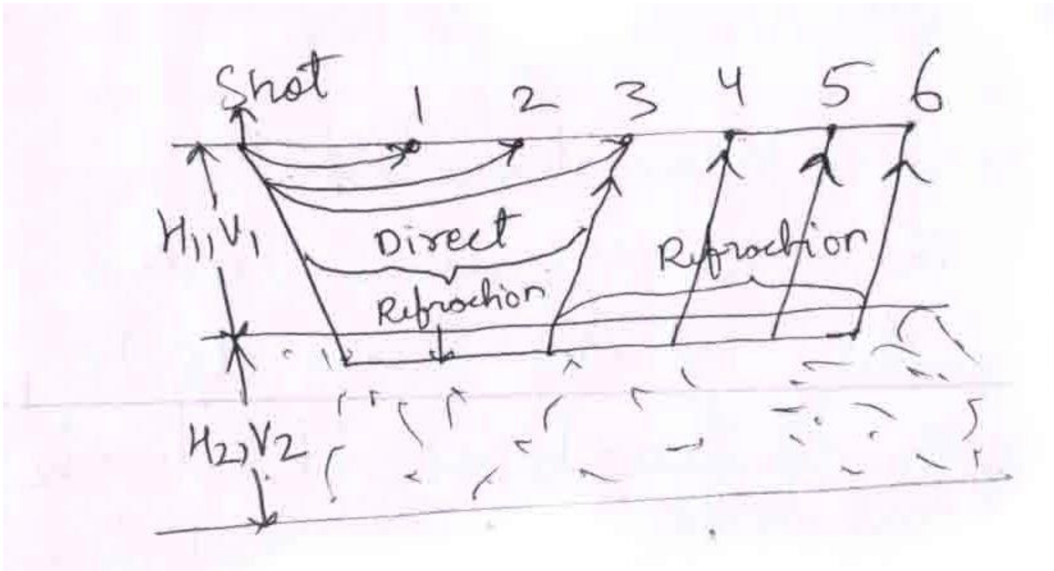


Fig. 15.2 Travel of primary and refracted waves

Here velocity v = $C\sqrt{\frac{Eg}{\gamma}}$

where,

v = velocity of the shock wave,

E = modulus of elasticity of the soil.

g = acceleration due to gravity.

γ= density of the soil

c = a dimensionless constant involving Poisson's ratio.

Electrical Resistivity :-

Resistivity is usually defined as the resistance between opposite faces of a unit cube of the material. Each soil has its own resistivity depending upon the water content, compaction and composition for example, the resistivity is high for loose dry gravel or solid rock and is low for saturated silt.

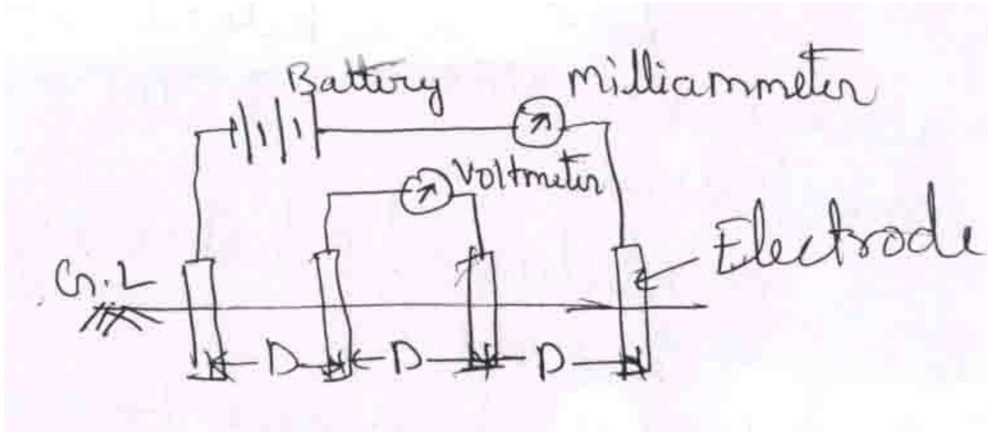


Fig. 15.3 Wenner configuration for electrical resistivity.

$$\rho = 2\pi D \frac{E}{I}$$

Where, D = distance between electrodes (m)

E = potential drop between the inner electrodes (volts)

I = current following between the outer electrodes (Amperes) and

ρ = mean resistivity (ohmm)

*Under revision

LECTURE 16

DETAIL NOTES

SUBSURFACE EXPLORATION

Introduction

Earthwork forms the largest activity of a Civil Engineer. It is well understood that irrespective of the type of civil engineering structure on earth –

• It has to be rested either in soil (e.g., foundations)
• Rested on soil (e.g., pavements) or
• The structure is itself constructed making use of soil (e.g., Earthen dams).

This implies that a better knowledge of the spatial variation of the soils encountered is essential. Therefore, before construction of any civil engineering work a thorough investigation of the site is essential. Site investigations constitute an essential and important engineering program which, while guiding in assessing the general suitability of the site for the proposed works, enables the engineer to prepare an adequate and economic design and to foresee and provide against difficulties that may arise during the construction phase. Site investigations are equally necessary in reporting upon the safety or causes of failures of existing works or in examining the suitability and availability of construction materials. Site investigation refers to the methodology of determining *surface* and *subsurface* features of the proposed area.

Information on *surface conditions* is necessary for planning the accessibility of site ,for deciding the disposal of removed material (particularly in urban areas), for removal of surface water in water logged areas, for movement of construction equipment, and other factors that could affect construction procedures.

Information on sub*surface conditions* is more critical requirement in planning and designing the foundations of structures, dewatering systems, shoring or bracing of excavations, the materials of construction and site improvement methods.

Soil Exploration

The knowledge of subsoil conditions at a site is a prerequisite for safe and economical design of substructure elements. The field and laboratory studies carried out for obtaining the necessary information about the surface and subsurface features of the proposed area including the position of the ground water table, are termed as **soil exploration or site investigation**.

Objectives of soil exploration program

The information from soil investigations will enable a Civil engineer to plan, decide, design, and execute a construction project. Soil investigations are done to obtain the information that is useful for one or more of the following purposes.

1. To know the geological condition of rock and soil formation.
2. To establish the groundwater levels and determine the properties of water.
3. To select the type and depth of foundation for proposed structure
4. To determine the bearing capacity of the site.
5. To estimate the probable maximum and differential settlements.

6. To predict the lateral earth pressure against retaining walls and abutments.
7. To select suitable construction techniques
8. To predict and to solve potential foundation problems
9. To ascertain the suitability of the soil as a construction material.
10. To determine soil properties required for design
11. Establish procedures for soil improvement to suit design purpose
12. To investigate the safety of existing structures and to suggest the remedial measures.
13. To observe the soil performance after construction.
14. To locate suitable transportation routes.

The objectives of soil investigations from various requirements point of view is summarized in Table 1.1

Table 16.1 Objectives of soil investigations

Design requirements	• define stratigraphy/geology. • to determine soil properties required for design. • aid material selection. • to determine the type and depth of foundation
Construction requirements	• to select suitable construction techniques • define equipment and techniques needed • to locate suitable transportation routes
Auditing	• checking a site prior to sale/purchase • to establish procedures for soil improvement to suit design purpose
Monitoring	• to observe the soil performance after construction • determine reasons for poor behaviour • document performance for future reference

Scope of soil investigation

The scope of a soil investigation depends on the type, size, and importance of the structure, the client, the engineer's familiarity with the soil at the site, and local building codes. Structures that are sensitive to settlement such as machine foundations and high-use buildings usually require a thorough soil investigation compared to a foundation for a house. A client may wish to take a greater risk than normal to save money and set limits on the type and extent of the site investigation. If the geotechnical engineer is familiar with a site, he/she may undertake a very simple soil investigation to confirm his/her experience. Some local building codes have provisions that set out the extent of a site investigation. It is mandatory that a visit be made to the proposed site.
In the early stages of a project, the available information is often inadequate to allow a detailed plan to be made. A site investigation must be developed in phases.

Phases of a Soils Investigation

The soil investigation is conducted in phases. Each preceding phase affects the extent of the next phase. The various phases of a soil investigation are given below:

Phase I. Collection of available information such as a site plan, type, size, and importance of the structure, loading conditions, previous geotechnical reports, topographic maps, air photographs, geologic maps, hydrological information and newspaper clippings.

Phase II. Preliminary reconnaissance or a site visit to provide a general picture of the topography and geology of the site. It is necessary that you take with you on the site visit all the information gathered in Phase I to compare with the current conditions of the site. Here visual inspection is done to gather information on topography, soil stratification, vegetation, water marks, ground water level, and type of construction nearby.

Phase II. Detailed soils exploration. Here we make a detailed planning for soil exploration in the form, trial pits or borings, their spacing and depth. Accordingly, the soil exploration is carried out. The details of the soils encountered, the type of field tests adopted and the type of sampling done, presence of water table if met with are recorded in the form of bore log. The soil samples are properly labeled and sent to laboratory for evaluation of their physical and engineering properties.

Phase IV. Write a report. The report must contain a clear description of the soils at the site, methods of exploration, soil profile, test methods and results, and the location of the groundwater. This should include information and/or explanations of any unusual soil, water bearing stratum, and soil and groundwater condition that may be troublesome during construction.

Soil Exploration Methods

1) Trial pits or test pits
2) Boring
3) probes (in situ test) and geophysical methods

Specific recommendations are made by Indian standards regarding the type, extent and details of subsurface explorations and the number, depth and spacing of boreholes for the following civil engineering works. Following is the list of various codes specified for the said purpose:

- Foundations of Multi-storeyed Buildings (IS: 1892, 1979)
- Earth and rockfill Dams (IS: 6955, 1973)
- Power House Sites (IS: 10060, 1981)
- Canals and Cross Drainage Works (IS: 11385, 1985)
- Ports and Harbours (IS: 4651 – Part 1, 1974)

Trial pits or test pits

- Applicable to all types of soils
- Provide for visual examination in their natural condition
- Disturbed and undisturbed soil samples can be conveniently obtained at different depths
- Depth of investigation: limited to 3 to 3.5 m.

Advantages
i) Cost effective
ii) Provide detailed information of stratigraphy
iii) Large quantities of disturbed soils are available for testing
iv) Large blocks of undisturbed samples can be carved out from the pits
v) Field tests can be conducted at the bottom of the pits

Disadvantages
i) Depth limited to about 6m
ii) Deep pits uneconomical
iii) Excavation below groundwater and into rock difficult and costly
iv) Too many pits may scar site and require backfill soils.

Limitations
i) Undisturbed sampling is difficult
ii) Collapse in granular soils or below ground water table

Exploratory borings
Boring is carried out in the relatively soft and uncemented ground (engineering 'soil') which is normally found close to ground surface. The techniques used vary widely across the world.

Location, spacing and depth of borings
It depends on:
i) Type of structure
ii) Size of the structure
iii) Weight coming

General guidelines for location and depth of bore holes
Boreholes are generally located at
- The building corners
- The centre of the site
- Where heavily loaded columns or machinery pads are proposed.
- At least one boring should be taken to a deeper stratum, probably up to the bedrock if practicable
- Other borings may be taken at least to significant stress level.

LECTURE 17

Spacing of Bore Holes – Codal Recommendations

According to IS 1892 (1979) Code of practice for subsurface investigation:

- For a small building one bore hole or test pit at the centre can give necessary data
- For a building covering not more than 4000 sq.m, one bore hole or test pit at each corner and one at centre is adequate.
- For a large project, the number will depend on its geological features and variation of strata. Generally a grid of 50 m spacing should be used with a combination of bore holes and sounding tests.

Borehole Spacing- Guidelines

Table 17.1 gives the general guidelines for the spacing of boreholes

Type of project	**Spacing (m)**	**Spacing (ft)**
Multi-storey building	10-30	30-100
Industrial plant	20-60	60-200
Highway	250-500	800-1600
Residential subdivision	250-500	800-1600
Dams and dikes	40-80	130-260

Depth of Investigation

The depth of investigation depends on

- The size and type of proposed structure
- Sequence of proposed strata.

The depths of boreholes should cover the zone of soil that will be affected by the structural loads. There is no fixed rule to follow. In most cases, the depths of boreholes are governed by experience based on the geological character of the ground, the importance of the structure, the structural loads, and the availability of equipment.

Guidelines for depth of investigation:

1. At least one boring should be taken to deeper stratum, probably up to the bedrock if practicable.

2. Borings should penetrate at least 3 m into rock.
3. Other borings may be taken at least to significant stress level.

4. In compressible soils such as clays, the borings should penetrate at least between 1 and 3 times the width of the proposed foundation or until the stress increment due to the heaviest foundation load is less than 10%, whichever is greater.

*Under revision

5. In very stiff clays, borings should penetrate 5-7 m to prove that the thickness of the stratum is adequate.

6. Borings must penetrate below any fills or very soft deposits below the proposed structure.

7. The minimum depth of boreholes should be 6 m unless bedrock or very dense material is encountered.

Significant depth

The investigation shall be carried out to the point at which the vertical stress due to proposed structure is equal to or less than 10% of original effective stress at the point before the structure is constructed – **significant depth**

Methods of borings

i) Auger boring – preferred for shallow depths , low ground water table
ii) Wash boring: high water table, deeper soil deposit
iii) Rotary drilling: high quality boring, also for rock drilling
iv) Percussion drilling: fast drilling, not taking samples, gravel

Hand Auger

Enables quick assessment of the soils present in the top few metres of the profile. It is limited by depth of water table in sandy soils and the presence of strong layer

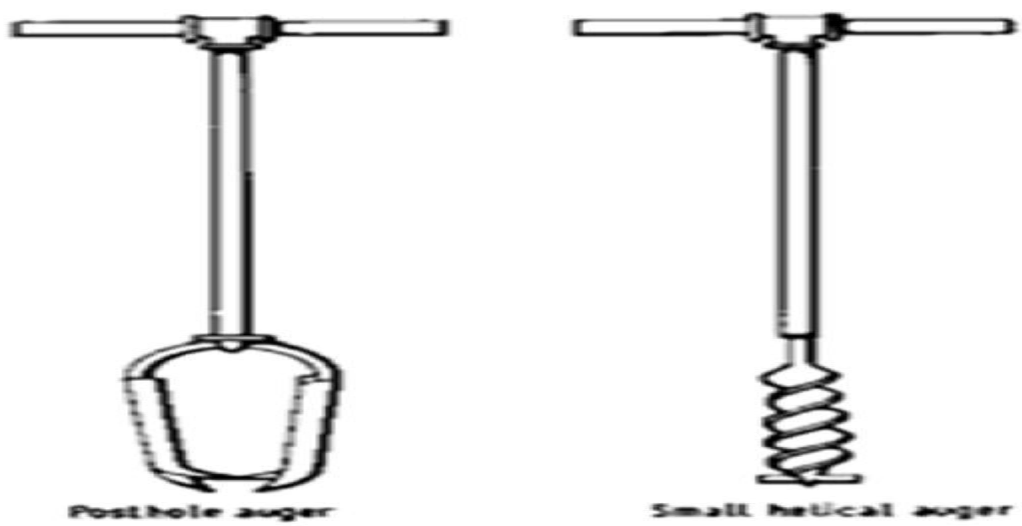


Fig. 17.1 Augers

Manual boring

It is suitable up to depths of 6m to 8 m. The soil samples obtained from auger borings are highly disturbed. In some non-cohesive soils or soils having low cohesion, the wall of the bore holes will not stand unsupported. In such circumstance, a metal pipe is used as a casing to prevent the soil from caving in .

Fig. 17.2 **Manual auger boring**

Mechanical boring

www.afkhami.com/Images/GeoAlbum/SPT.jpg

Fig. 17.3 **Mechanical boring**

*Under revision

LECTURE 18

Wash Boring
Wash boring relies on relatively little drilling action and can form a hole primarily by jetting. This can be undertaken with light equipment without the need for a drilling rig.

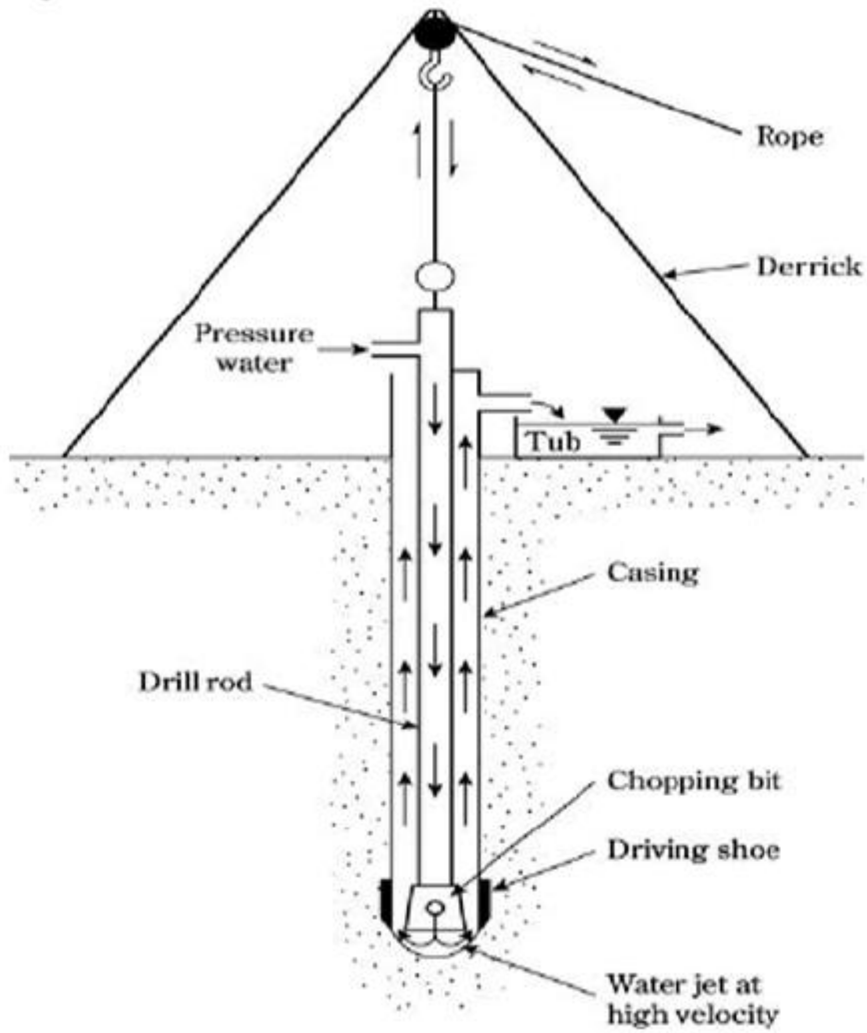


Fig.18. 1 Wash Boring set up

Soil Sampling

Need for sampling
Sampling is carried out in order that soil and rock description, and laboratory testing can be carried out.
Laboratory tests typically consist of:
i). Index tests (for example, specific gravity, water content)

ii). Classification tests (for example, Atterberg limit tests on clays); and
iii) Tests to determine engineering design parameters (for example strength, compressibility, and permeability).

Factors to be considered while sampling soil
i) Samples should be representative of the ground from which they are taken.
ii) They should be large enough to contain representative particle sizes, fabric, and fissuring and fracturing.

iii) They should be taken in such a way that they have not lost fractions of the *situ soil (for example, coarse or fine particles) and, where strength and compressibility tests are planned ,they should be subject to as little a disturbance as possible.*

Type of soil samples

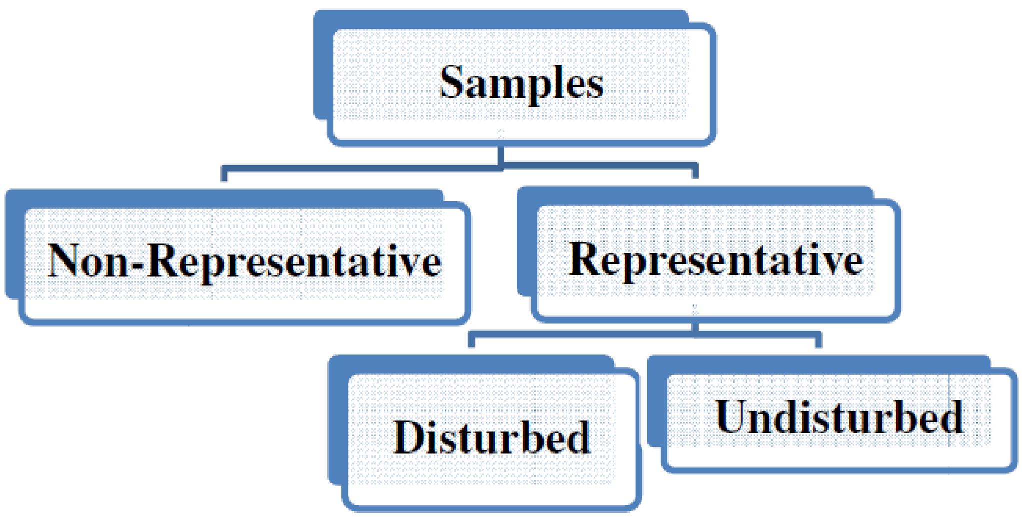


Fig. 18.2 Types of soil samples

Non-Representative Samples

Non-Representative soil samples are those in which neither the in-situ soil structure, moisture content nor the soil particles are preserved.
• They are not representative
• They cannot be used for any tests as the soil particles either gets mixed up or some particles may be lost.
• e.g., Samples that are obtained through wash boring or percussion drilling.

Disturbed soil samples
Disturbed soil samples are those in which the in-situ soil structure and moisture content are lost, but the soil particles are intact.
• They are representative
• They can be used for grain size analysis, liquid and plastic limit, specific gravity, compaction tests, moisture content, organic content determination and soil classification test performed in the lab
• e.g., obtained through cuttings while auguring, split spoon (SPT), etc.

Undisturbed soil samples
Undisturbed soil samples are those in which the in-situ soil structure and moisture content are preserved.
• They are representative and also intact
• These are used for **consolidation, permeability or shear strengths test (Engineering properties)**
• More complex jobs or where **clay** exist
• In **sand** is very difficult to obtain undisturbed sample
• Obtained by using Shelby tube (thin wall), piston sampler, surface (box), vacuum, freezing, etc.,

Causes of Soil disturbances

- Friction between the soil and the sampling tube
- The wall thickness of the sampling tube
- The sharpness of the cutting edge
- Care and handling during transportation of the sample tube
- To minimize friction

The sampling tube should be pushed instead of driven into the ground Sampling tube that are in common use have been designed to minimize sampling disturbances.

Design Features affecting the sample disturbance

- Area ratio
- Inside Clearance
- Outside Clearance
- Recovery Ratio
- Inside wall friction
- Design of non-return value
- Method of applying force
- sizes of sampling tubes

Area ratio

$$\text{Area ratio } A_r = \frac{\mathit{Max.Cross\ sectional\ area\ of\ the\ cutting\ edge}}{\mathit{Area\ of\ the\ soil\ sample}}$$

$$A_r = \frac{D_2{}^2 - D_1{}^2}{D_1{}^2} x\, 100$$

Where, D_1 = inner diameter of the cutting edge.
D_2 = outer diameter of the cutting edge.

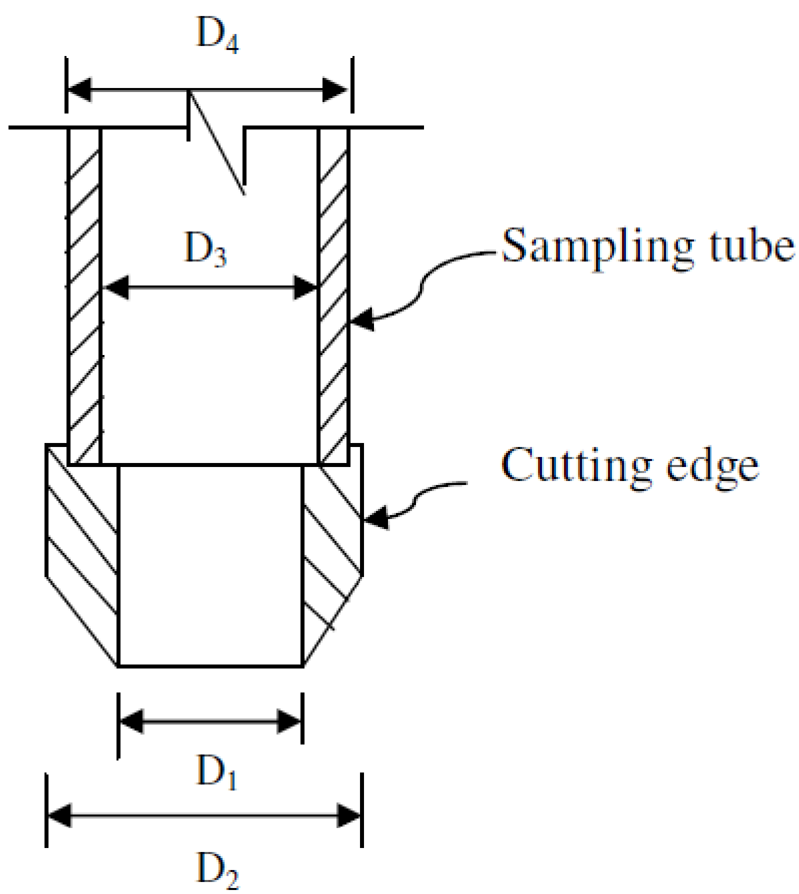


Fig. 18.3 Typical Sampling tube

- For obtaining good quality undisturbed samples, the area ratio should be less than or equal to 10%.

- It may be high as 110% for thick wall sampler like split spoon sampler and may be as low as 6 to 9% for thin wall samples like Shelby tube.

Inside Clearance

$$Ci = \frac{D_3 - D_1}{D_1} x\, 100$$

Where D_3 = inner diameter of the sample tube

- The inside clearance allows **elastic expansion** of the sample when it enters the sampling tube.
- It helps in reducing the **frictional drag** on the sample, and also helps **to retain the core.**
- For an undisturbed sample, the inside clearance should be between **0.5 and 3%.**

LECTURE 19

Outside Clearance

$$Co = \frac{D_2 - D_4}{D_4} \; x\, 100$$

Where D_4 = outer diameter of the sample tube

- Outside clearance facilitates the **withdrawal** of the sample from the ground.
- For **reducing the driving force**, the outside clearance should be as small as possible.
- Normally, it lies between zero and 2%.
- C_o Should not be more than C_i

Recovery Ratio

$$R_r = \frac{L}{H}$$

Where
L = length of the sample within the tube and
H = Depth of penetration of the sampling tube

- R_r = 96 – 98 % for getting a satisfactory undisturbed sample

Inside wall friction

- The friction on the inside wall of the sampling tube causes disturbances of the sample.
- Therefore the inside surface of the sampler should be as smooth as possible.
- It is usually smeared with oil before use to reduce friction.

Design of non-return valve

- The non – return valve provided on the sampler should be of proper design.
- It should have an orifice of large area to allow air, water or slurry to escape quickly when the sampler is driven.
- It should close when the sample is withdrawn.

Method of applying force

- The degree of disturbance depends upon the method of applying force during sampling and depends upon the rate of penetration of the sample.
- For obtaining **undisturbed samples, the sampler should be pushed and not driven**

Type of Soil Samplers

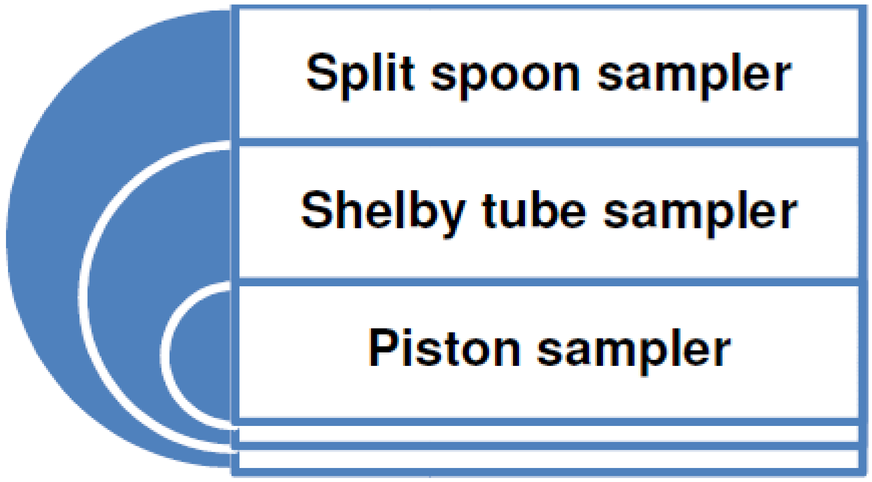
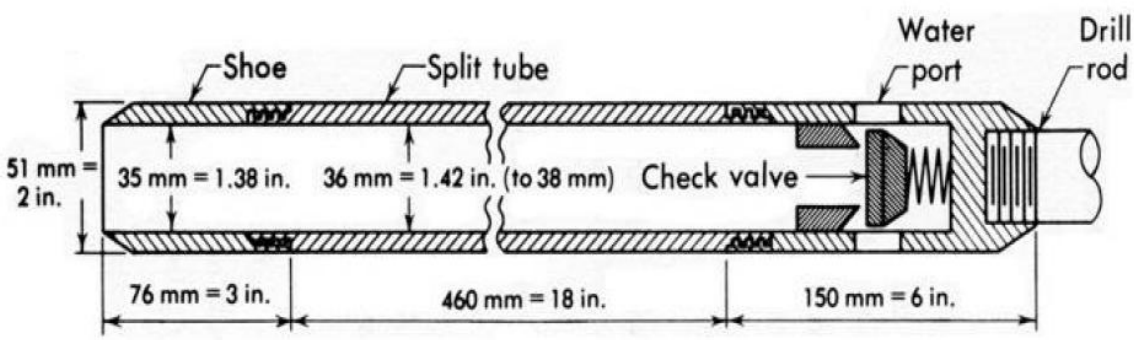


Fig. 19.1 Type of samplers

Split Spoon Sampler

- Has an inside diameter of 35mm and an outside diameter of 50mm.
- Has a split barrel which is held together
  and a cap at the upper end.
- The thicker wall of the standard sampler permits higher driving stresses than the Shelby tube but does so at the expense of higher levels of soil disturbances.
- Split spoon samples are highly disturbed.
- They are used for visual examination and for classification tests.



Source: www.geo.sunysb.edu/.../abstracts06/moss
Fig.19.2 Split spoon sampler

*Under revision

Shelby Tube Sampler

- Thin-walled seamless steel tube of diameter 50mm or 75mm
- The bottom end of the tube is sharpened.
- The tubes can be attached to drilling rods.
- The drilling rod with the sampler attached is lowered to the bottom of the borehole and the sampler is pushed into the soil.
- The soil sample inside the tube is then pulled out.
- The two ends of the sampler are sealed and sent to the lab.
- The samples can be used for consolidation or shear tests.

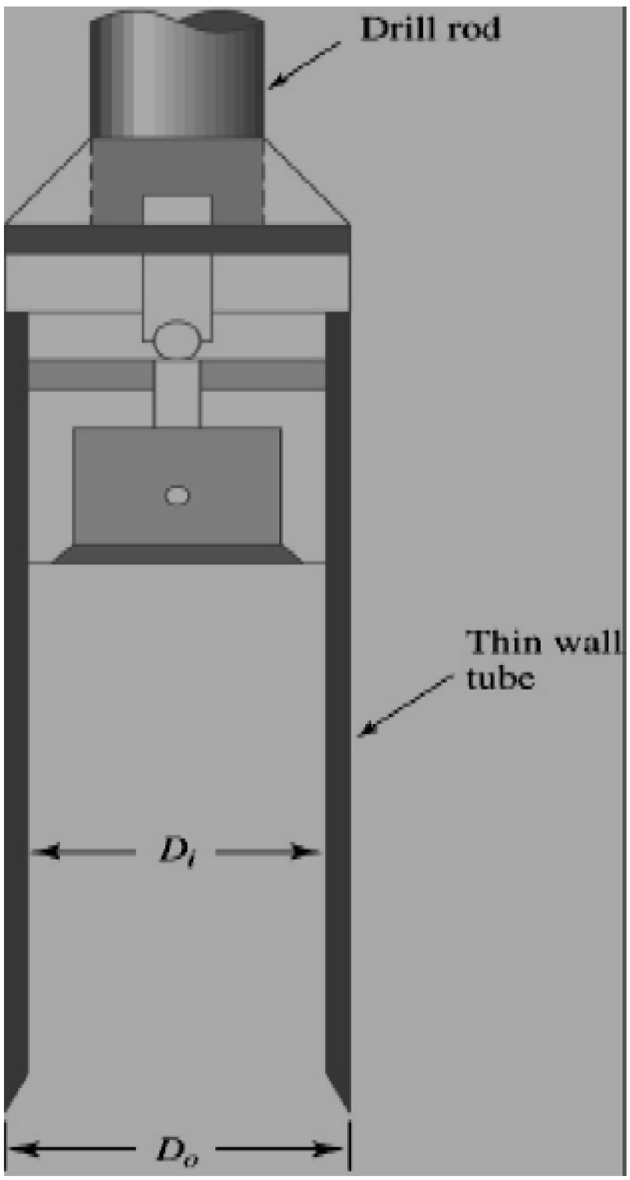


Fig.19.3 Shelby tube sampler

*Under revision

Plate 19.4 Shelby tube sampler

*Under revision

LECTURE 20

Piston Sampler

- For sampling very soft or larger than 76.2mm in diameter to get high quality undisturbed samples, they tend to fall out of the sampler. Then piston samplers are used.
- They consist of a thin wall tube with a piston.
- Initially, the piston closes the end of the thin wall tube. The sampler is lowered to the bottom of the borehole and then the thin wall tube is pushed into the soil hydraulically past the piston.
- Later the pressure is released through a hole in the piston rod. To a large extent, the presence of the piston prevents distortion in the sample by not letting the soil squeeze into the sampling tube very fast and by not admitting excess soil.
- Consequently, samples obtained in this manner are less disturbed than those obtained by Shelby tubes.

*Under revision

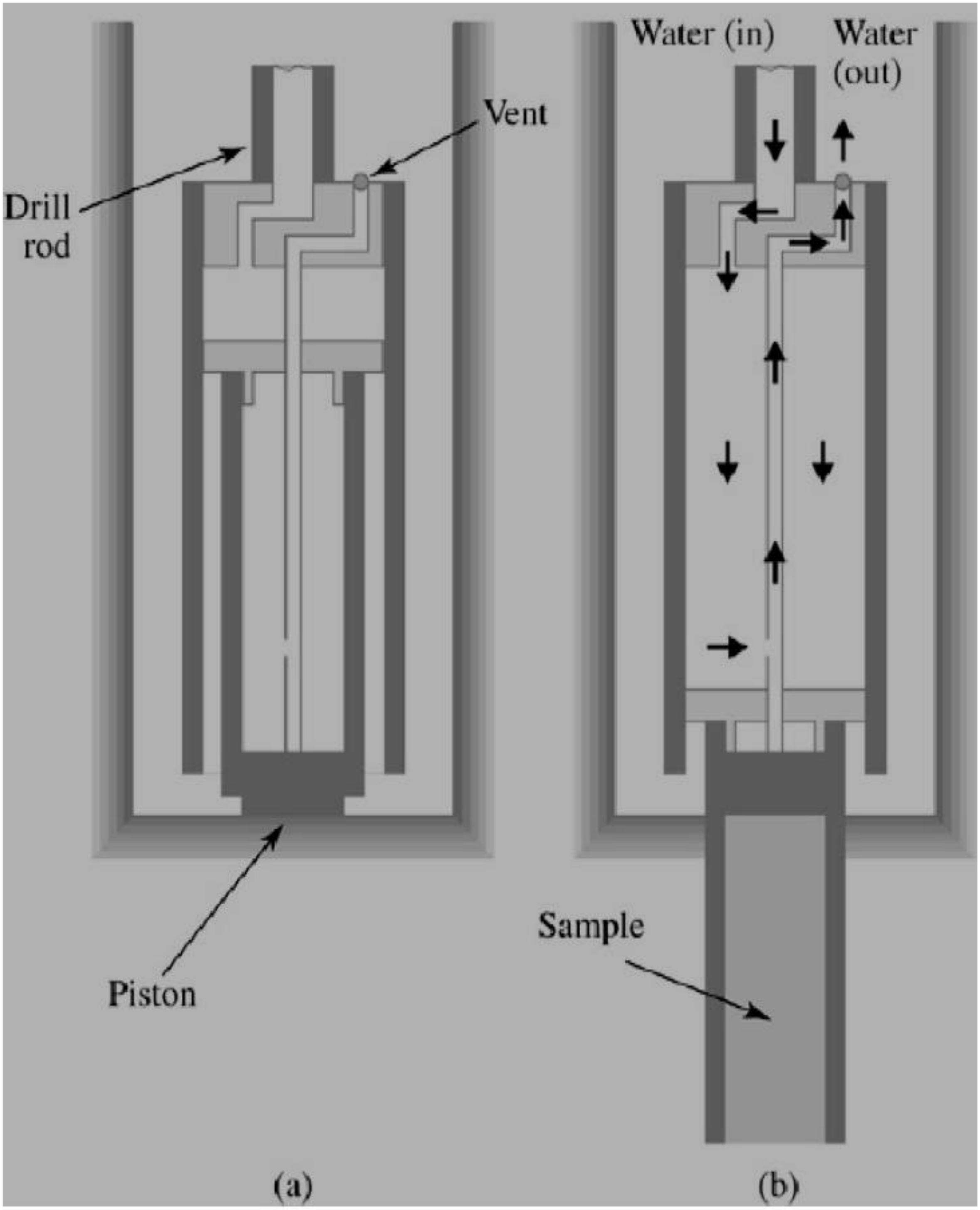


Fig.20.1 Piston sampler

*Under revision

Bore Hole Support or Stabilization of Bore Hole

This necessary to prevent cohesion less soils against caving while drilling bore hole.
Either of the following is used for bore hole support

• **Steel casing –** hydraulically pushed
• **Drilling mud** – Circulation bentonite slurry

Stabilization of bore holes using drilling mud

Bentonite mud or Drilling mud' is a thin mixture of water and bentonite clay, which can be mixed in powder form to the drilling water to create higher density suspension.

Advantages

• It is advantages over water.
• Firstly, it is more viscous and can therefore lift cuttings adequately at a lower velocity.
• Secondly it will cake the edges of the borehole, and the outside of the core, and will largely eliminate the seepage of water out of the borehole, thus reducing problems of loss of return.
• Hence, smaller volumes of flush fluid will be required and the fluid may he recirculated via a settling tank (where the cuttings are allowed to drop out of suspension).
• The cake formed on the outside of the borehole has the effect of considerably improving Borehole stability and the prevention of softening of weak rock cores.

Disadvantages

• The bentonite mud-soil cakes are difficult to dispose of, at the end of drilling a borehole. The mud cannot simply be tipped on the site, and it cannot be discharged into nearby sewers.
• Bentonite mud must be properly mixed, using appropriate equipment, in order to ensure that it is of the correct consistency and does not contain unmixed dry bentonite lumps, capable of clogging flush ports in the core barrel.

In Situ Testing

- There is a wide variety of different tests that can be used for evaluating the properties of the ground.
- It is often preferable to do an *in situ test in an attempt to* measure a particular parameter, rather than obtain a sample and do a laboratory test.
    a. sampling results in disturbance (reduces strength and stiffness)
    b. sometimes only best (strongest) material is recovered, and is not representative of overall *in situ material*

Parameters obtained from In Situ Testing

- Typical parameters that may be obtained either directly, or indirectly from *in situ tests:*

1. strength
2. stiffness
3. permeability
4. relative density

In-situ Tests

In situ testing is a division of field testing corresponding to the cases where the ground is tested in-place by instruments that are inserted in or penetrate the ground. In-situ tests are normally associated with tests for which a borehole either is unnecessary or is only an incidental part of the overall test procedure, required only to permit insertion of the testing tool or equipment. The role of specialized in-situ testing for site characterization and the research and development of in-situ techniques have received considerable attention over the last 15 years or so. The use of specialized in-situ testing in geotechnical engineering practice is rapidly gaining increased popularity. In Europe, specialized in-situ testing has been commonly used for more than 25 years. Improvements in apparatus, instrumentation, and technique of deployment, data acquisition and analysis procedure have been significant. The rapid increase in the number, diversity and capability of in-situ tests has made it difficult for practicing engineers to keep abreast of specialized in-situ testing and to fully understand their benefits and limitations. Table below summarizes the primary advantages and disadvantages of in-situ testing

Table 20.1 Summary of advantages and disadvantages of in-situ testing

Advantages	Disadvantages
• Tests are carried out in place in the natural environment without sampling disturbance , which can cause detrimental effects and modifications to stresses, strains, drainage, fabric and particle arrangement. • Continuous profiles of stratigraphy and engineering properties/characteristics can be obtained. • Detection of planes of weakness and defects are more likely and practical. • Methods are usually fast, repeatable, produce large amounts of information and are cost effective Tests can be carried out in soils that are either impossible or difficult to sample without the use of expensive specialized methods. • A large volume of soil may be tested than is normally practicable for laboratory testing. This may be more representative of the soil mass.	• Samples are not obtained; the soil tested cannot be positively identified. The exception to this is the SPT in which a sample, although disturbed, is obtained. • The fundamental behaviour of soils during testing is not well understood. • Drainage conditions during testing are not known. • Consistent, rational interpretation is often difficult and uncertain. • The stress path imposed during testing may bear no resemblance to the stress path induced by full-scale engineering structure. • Most push-in devices are not suitable for a wide range of ground conditions. • Some disturbance is imparted to the ground by the insertion or installation of the instrument. • There is usually no direct measurement of engineering properties.

*Under revision

LECTURE 21

The in-situ tests that are most commonly used in practice are:

(i) Standard penetration test (SPT)
(ii) Cone – penetration test (CPT)
(iii) Piezo-cone penetration test (CPTU)
(iv) Field vane shear test (FVT)
(v) Pressure meter test (PMT)
(vi) Dilatometer test (DMT)
(vii) Becker Penetration Test (BPT) and
(viii) Iowa Bore hole shear test (BHST)
(ix) Plate load test

Standard Penetration Test (SPT):
One of the most common in-situ tests is the standard penetration test or SPT. This test which was originally developed in the late 1920s, is currently the most popular and economical means to obtain subsurface information (both inland and offshore). It offers the advantage of low cost, applicability to many soil types, samples are obtained (although disturbed) and a large database from which many useful correlations have been developed.

Procedure:
The standard penetration test is conducted in a borehole using a standard split-spoon sampler.

(i) When the borehole (55 to 150 mm in dia) has been drilled to the desired depth, the drilling tools are removed and the split-spoon sampler, attached to standard drill rods of required length is lowered to the bottom of the borehole and rested at the bottom .

(ii) The split-spoon sampler is then driven into the soil for a distance of 450 mm in three stages of 150 mm each by blows of a drop hammer of 63.5 kg mass falling vertically and freely through a height of 750 mm at the rate of 30 blows per minute (IS 2131 – 1981). The number of blows required to penetrate every 150-mm is recorded while driving the sampler. If full penetration is obtained, the blows for the first 150 mm is retained for reference purposes, but not used to compute the SPT value because the bottom of the boring is likely to be disturbed by the drilling process and may be covered with loose soil that may fall from the sides of the boring. The number of blows required for the next 300 mm of penetration is recorded as the SPT value. The number of blows is designated as the "Standard Penetration Value" or "Number" N.

(iii) The slit-spoon sampler is then withdrawn and is detached from the drill rods. The split barrel is disconnected from the cutting shoe and the coupling. The soil sample collected inside the split barrel is carefully collected so as to preserve the natural moisture content and transported to the laboratory for tests. Sometimes, a thin liner is inserted within the split-barrel so that at the end of the SPT, the liner containing the soil sample is sealed with molten wax at both its ends before it is taken away to the laboratory. Usually SPT is carried out at every 0.75-m vertical interval or at the change of stratum in a borehole. This can be increased to 1.5 m if the depth of borehole is large. Due to the presence of boulders or rocks, it may not be possible to drive the sampler to a distance of 450 mm. In such a case, the N value can be recorded for the first 300-mm penetration.

The boring log shows refusal and the test is halted if:
(i) 50 blows are required for any 150 mm penetration
(ii) 100 blows are required for 300 mm penetration
(iii) 10 successive blows produce no advance

Plate 21.1 Standard Penetration testing

*Under revision

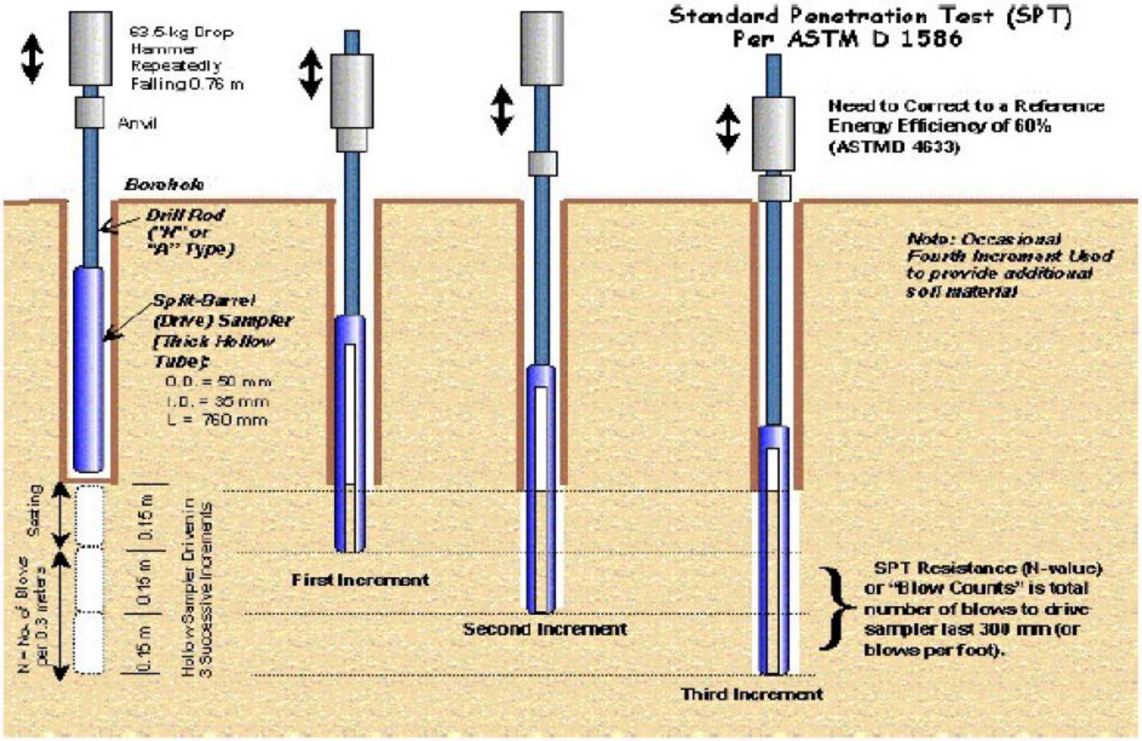
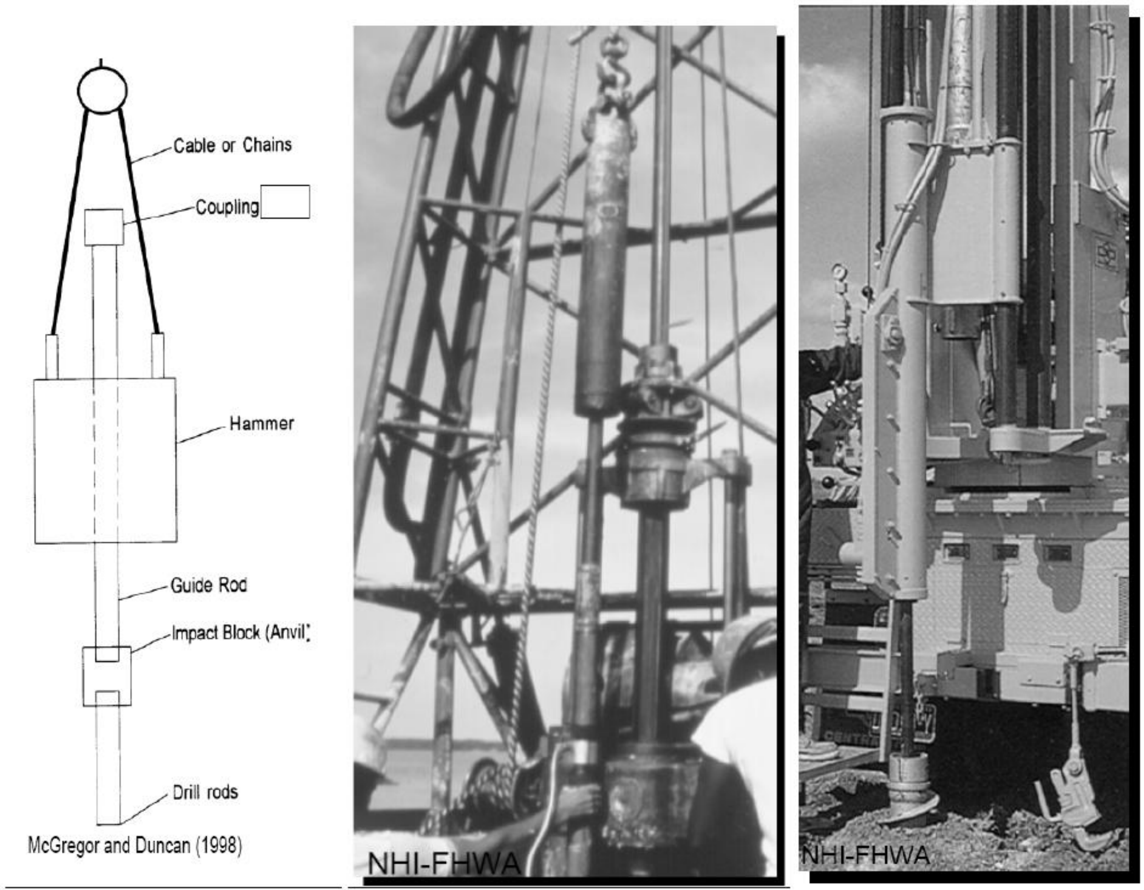


Fig.21.2 Stages of Standard Penetration testing



a) Donut hammer **b)Safety Hammer** **c) Automatic hammer**

Fig. 21.3 Types of SPT hammers

Precautions:
Some of the precautions to be observed to avoid some of the pitfalls of the test are as follows:
(i) The drill rods should be of standard specification and should not be in bent condition.
(ii) The split spoon sampler must be in good condition and the cutting shoe must be free from wear and tear.
(iii) The drop hammer must be of right weight and the fall should be free, frictionless and vertical.
(iv) The height of fall must be exactly 750 mm. Any change in this will seriously affect the N value.
(v) The bottom of the borehole must be properly cleaned before the test is carried out. If this is not done, the test gets carried out in the loose, disturbed soil and not in the undisturbed soil.
(vi) When a casing is used in borehole, it should be ensured that the casing is driven just short of the level at which the SPT is to be carried out. Otherwise, the test gets carried out in a soil plug enclosed at the bottom of the casing.

(vii) If the water level in the borehole is lower than the ground water level, ‘quick’ condition may develop in the soil and very low N values may be recorded.
In spite of all these imperfections, SPT is still extensively used because the test is simple and relatively economical. It is the only test that provides representative soil samples both for visual inspection in the field and for natural moisture content and classification tests in the laboratory. Because of its wide usage, a number of time-tested correlations between N value and soil parameters are available, mainly for cohesionless soils. Even design charts for shallow foundations resting on cohesionless soils have been developed on the basis of N values. The use of N values for cohesive soils is limited, since the compressibility of such soils is not reflected by N values.

SPT values obtained in the field for sand have to be corrected before they are used in empirical correlations and design charts. IS: 2131 – 1981 recommends that the field value of N corrected for two effects, namely,

(a) effect of overburden pressure, and (b) effect of dilatancy

(a) Correction for overburden pressure:
Several investigators have found that the overburden pressure influences the penetration resistance or the N value in a granular soil. If two granular soils possessing the same relative density but having different confining pressures are tested, the one with a higher confining pressure gives a higher N value. Since the confining pressure (which is directly proportional to the overburden pressure) increases with depth, the N values at shallow depths are underestimated and the N values at larger depths are overestimated. Hence, if no correction is applied to recorded N values, the relative densities at shallow depths will be underestimated and at higher depths, they will be overestimated. To account for this, N values recorded (N_R) from field tests at different effective overburden pressures are corrected to a standard effective overburden pressure.
The corrected N value is given by

$$N_C = C_N N_R$$

Where, N_C = corrected value of observed N value
C_N = correction factor for overburden pressure
N_R = Recorded or observed N value in the field
The correction proposed by Peck, Hanson and Thornburn (1974) is given by the equation:

$$C_N = 0.77 \log_{10} \frac{2000}{\sigma'}$$

Where, σ' = Effective overburden pressure at the depth at which N value is recorded, in kPa .

*Under revision

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(b) Correction for dilatancy:

Dilatancy correction is to be applied when N_c obtained after overburden correction, exceeds 15 in saturated fine sands and silts. IS: 2131 – 1981 incorporates the Terzaghi and Peck recommended dilatancy correction (when $N_c > 15$) using the equation .

$$\mathbf{N'c = 15 + 0.5\,(N' - 15)}$$

Where N'_c = final corrected value to be used in design charts.

$N_c > 15$ is an indication of a dense sand, based on the assumption that critical void ratio occurs at approximately $N_c = 15$. The fast rate of application of shear through the blows of a drop hammer is likely to induce negative pore water pressure in saturated fine sand under undrained condition of loading. Consequently, a transient increase in shear resistance will occur, leading to a SPT value higher than the actual one.

Note: The overburden correction is applied first. This value is used as observed N value and then the dilatancy correction is applied.

Correlation of 'N' with engineering properties:

The value of standard Penetration number depends upon the relative density of the cohesionless soil and the UCC strength of the cohesive soil.
The angle of shearing resistance (φ) of the cohesionless soil depends upon the number N. In general, greater the N-value, greater is the angle of shearing resistance. Table below gives the average values of φ for different ranges of N.

Table 22.1 Correlation between N value and angle of shearing resistance

N	Denseness	φ
0-14	Very loose	$25^o – 32^o$
4-10	Loose	$27^o – 35^o$
10-30	Medium	$30^o – 40^o$
30-50	Dense	$35^o – 45^o$
>50	Very dense	$> 45^o$

The consistency and the UCC strength of the cohesive soils can be approximately determined from the SPT number N. Table **22.2** gives the approximate values of UCC strength for different ranges of N.

Table 22.2 correlation between N value and UCC strength

N	Consistency	q_u (kN/m^2)
0-2	Very soft	< 25
2-4	Soft	25 – 50
4-8	Medium	50 – 100
8-15	Stiff	100 – 200
15-30	Very stiff	200 – 400
>30	Hard	> 400

It can also be determined from the following relation

$q_u = 12.5\ N$

Where, q_u = UCC strength (kN/m^2)

Cone Penetration Test (CPT)

(a) Dynamic Cone Penetration Test (DCPT)

In this test, a cone, which has an apex angle of 60^o and attached to drill rods is driven into the soil by blows of a hammer of 63.5 kg, falling freely from a height of 750 mm. The blow count for every 100-mm penetration of the cone is continuously recorded. The cone is driven till refusal or upto the required depth and the drill rods are withdrawn, leaving the cone behind in the ground. The number of blows required for 300-mm penetration is noted as the dynamic cone resistance, N_{cd}. The test gives a continuous record of N_{cd} with depth. No sample, however, can be obtained in this test.

Dynamic cone penetration tests are performed either by using a 50 mm diameter cone without bentonite slurry (IS: 4968 – Part I – 1976) or a 65 mm diameter cone with bentonite slurry (IS: 4968 – Part II – 1976). When bentonite slurry is used, the set-up has an arrangement for the circulation of slurry so that friction on the drill rod is eliminated. The dynamic cone test is a quick test and helps to cover a large area under investigation rather economically. It helps in identifying the uniformity or the variability of the subsoil profile at the site and reveals local soft pockets, if any. It can also establish the position of rock stratum, when required. The test is much less expensive and much quicker than the SPT. If the tests are carried out close to a few boreholes, the data from DCPT can be compared with the SPT data and correlation between the two established for the particular site conditions. The correlation can then be used to obtain N values from N_{cd} values.

Some approximate correlations between N_{cd} and N, applicable for medium to fine sands are given below:

When a 50 mm diameter cone is used,

N_{cd} = 1.5 N for depths upto 3 m

N_{cd} = 1.75 N for depths from 3 m to 6 m

N_{cd} = 2.0 N for depths greater than 6 m

(b) *Static cone penetration test (CPT)*

The static cone penetration test, simply called the cone penetration test (CPT), is a simple test that is presently widely used in place of SPT, particularly for soft clays and fine to medium sand deposits. The test was developed in Holland and is, therefore, also known as the Dutch cone test. The test assembly is shown in Fig **22.2** The penetrometer that is commonly used is a cone with an apex angle of 60^o and a base area of 10 cm^2.

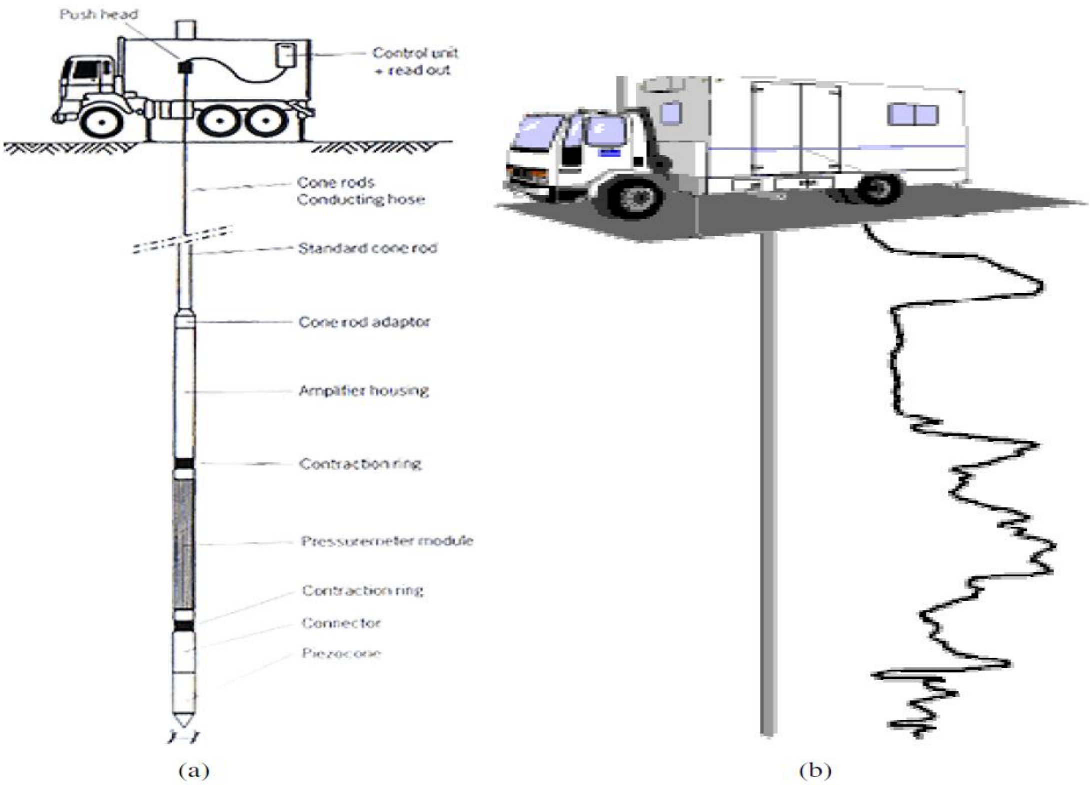

Fig.22.1 Cone penetration set up

The sequence of operations of the penetrometer is as follows:
1. *Position 1:* The cone and the friction jacket is in a stationary position.
2. *Position 2*: The cone is pushed into the soil by the inner drill rod/sounding rod to a depth '**a**', at a steady rate of 20 mm/s, till a collar engages the cone. The tip resistance q_c called the cone or point resistance, can be calculated by the force Q_c read on a pressure gauge.
The tip resistance, $q_c = Q_c / A_c$
Where A_c is the base area
Normally the value of **a = 40 mm**

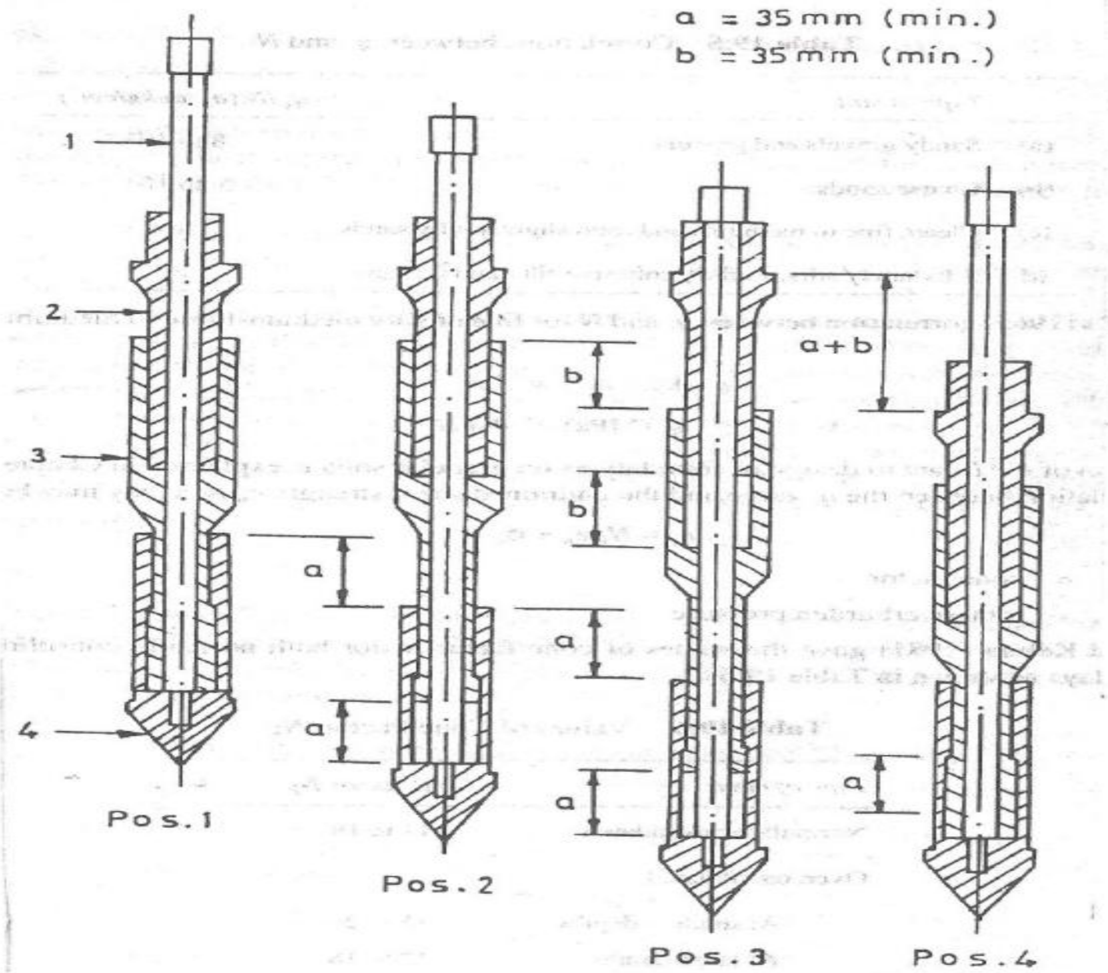

Fig. 22.2 Sequence of operations of CPT

LECTURE 23

3. *Position 3*: The sounding rod is pushed further to a depth '**b**'. This has the effect of pushing the friction jacket and the cone assembly together. The total force Q_t required for this is again read on the pressure gauge. The force required to push the friction jacket along,

Q_f is then obtained as $Q_t – Q_c$
The side or the skin friction, $f_s = Q_f / A_f$
Where, A_f is the surface area of the friction jacket.
Normally the value of **b = 40 mm**
4. *Position 4*: The outside mantle tube is pushed down to a distance (a+b), bringing the cone and the friction jacket to position 1.

The procedure illustrated above is continued till the proposed depth of sounding is reached. CPT gives a continuous record of variation of both cone resistance and friction resistance with depth. Unlike the SPT and the DCPT, this test measures the static resistance of the soil. From CPT soil sample is not obtained. The test is not suitable in gravels and very dense sands owing to the difficulty experienced in pushing the cone and the anchorage system. Data from CPT is often used to estimate the point bearing resistance of a pile foundation. In granular soils, correlations have been established between q_c and N. table below shows the correlations

Table 23.1 Correlations between q_c and N

Type of soil	q_c / N (kPa)
a) Sandy gravels and gravels	800 to 1000
b) Coarse sands	500 to 600
c) Clean, fine to medium sands and slightly silty sands	300 to 400
d) Silts, sandy silts, slightly cohesive silt-sand mixtures	200

c) Piezo-cone Penetration Test
The piezo-cone penetration test (CPTU) is an example of improved in-situ techniques that have been developed for site investigations. It is an extension to the standard cone penetration test (CPT). The standard cone penetrometer consists of a 35 – mm diameter rod with a 60^o conical-shaped tip equipped with electronic sensing elements to measure tip resistance and the local side friction on a sleeve. The piezo-cone penetrometer in addition incorporates a transducer to measure pore water pressure. The penetrometer is pushed at a constant rate of 2 cm/s into the ground. In general, no pre-boring is required. In addition to the standard three-channel piezo-cone penetrometer, new penetrometers have been developed that include additional sensing elements for measurement of temperature, inclination, lateral stress and seismic waves. In particular, the seismic cone penetrometer (CPTS), which houses an accelerometer, has proven itself most beneficial in the assessment of liquefaction and determining dynamic soil properties. The CPTU is one of the most commonly used specialized in-situ tests and is ideal for use in sands, silts, clays and mine-tailing materials. A reliable, continuous stratigraphic profile, together with important engineering properties, can be interpreted from its results. Further, because the data are collected on a continuous basis, detection of thin, weak layers and /or more pervious layers is possible. This is important

because these weaker zones ultimately control the behaviour and performance of the soil mass and, in particular, are significant to slope stability and seepage considerations, liquefaction and dynamic stability analysis, and foundation design. These weaker layers often go undetected by conventional borehole drilling and sampling operations. The CPTU also avoids the disturbance effects commonly encountered when drilling boreholes and sampling below the water table. Thus, CPTU greatly enhances the quality of data gathered in a sub-surface investigation. In addition, because of its relative simplicity, ease of operation and high rate of production in terms of depth of soil investigated, the CPTU has proven itself to be very cost-effective. The interpreted data provide information on: stratigraphy in terms of thickness, gradation and soil type; density (void ratio) and /or state (state parameter) of the more sandy or coarser materials; friction angle (strength) of the more sandy or coarser materials; undrained strength and stress history (OCR) of the more clayey or fine-grained materials; and liquefaction potential evaluation and cyclic resistance. Further, as the cone penetrometer is pushed through the ground, excess porewater pressure is induced, the magnitude of which is controlled by the hydraulic conductivity of the material. If the pushing of the cone is interrupted and the cone is held stationary for a sufficient period of time, the porewater pressure measured by the cone will stabilize to the in-situ piezometric pressure at the cone tip. Monitoring the rate of porewater pressure change to this stabilized pressure provides data from which the in-situ hydraulic conductivity can be calculated. From the above operations, the piezometric pressure, gradients and hydraulic conductivity within the ground can also be obtained.

Geophysical Methods

It is a non-intrusive method of “seeing” into the ground. Unlike direct sampling and analysis, such as obtaining a soil or water sample and sending it to a laboratory, the geophysical methods provide non-destructive, in situ measurements of physical, electrical or geochemical properties of the natural or contaminated soil and rock. Geophysical methods encompass a wide range of surface and down-hole measurement techniques which provide a means of investigating subsurface hydrogeologic and geologic conditions. These methods have also been applied to detecting contaminant plumes and locating buried waste materials. All of the geophysical methods, like any other means of measurements, have advantages and limitations. There is no single, universally applicable geophysical method, and some methods are quite site specific in their performance. Thus, the user must select the method or methods carefully and understand how they are applied to specific site conditions and project requirements.

Usefulness of geophysical methods

i) To provide a greater volume of measurement

Data obtained from borings or monitoring wells come from a very localized area and are representative of material conditions at the bore-hole. Geophysical methods, on the other hand, usually measure a much larger volume of the subsurface material

ii) As anomaly detectors

As a result of geophysical measurements being relatively rapid, a larger number of measurements can be taken, for a given budget. With a greater number of measurements plus the fact that the measurement encompasses a larger volume of subsurface material, the geophysical methods can detect anomalous areas which may pose potential problems, and thus are essentially anomaly detectors.

Once an overall characterization of a site has been made using geophysical methods and anomalous zones identified, drilling and sampling programmes are made more effective by:

- **locating boreholes** and **monitoring wells** to provide samples that are **representative of both anomalous** and **background conditions**;
- **minimizing the number of borings**, samples, piezometers and monitoring wells required to characterize accurately a site;
- reducing field investigation time and cost; and
- significantly **improving the accuracy** of the overall investigation.
- This approach yields a much **greater confidence** in the final results, with **fewer borings or wells**, and an overall **cost savings**.
- It makes good sense to minimize the number of monitoring wells at a site while optimizing the location of those installed.
- If the wells are located in the wrong position, they do not provide representative data and a large amount of relatively useless data would accrue.
- Using the geophysical method in a systematic approach, drilling is no longer being used for hit-or-miss reconnaissance, but is being used to **provide the specific quantitative assessment of subsurface conditions**.
- Boreholes or wells located with this approach may be thought of as **smart holes** because they are scientifically placed, for a specific purpose, in a specific location, based on knowledge of site conditions from geophysical data.

Geophysical measurements

a) Mechanical Wave Measurements
b) Electromagnetic Wave Techniques
Mechanical Wave Measurements
– Crosshole Tests (CHT)
– Downhole Tests (DHT)
– Spectral Analysis of Surface Waves
– **Seismic Methods(Reflection and Refraction)**
– Suspension Logging
Electromagnetic Wave Techniques
– Ground Penetrating Radar (GPR)
– Electromagnetic Conductivity (EM)
– **Surface Resistivity (SR)**
– Magnetometer Surveys (MT)

Seismic Methods – Principle Involved
It is based on fact that **compression waves** or **shear waves** travel at **different speeds** in the ground, and that waves reflect off at interfaces between materials of different density or stiffness.

There are two methods based on seismic waves. They are:
Seismic Reflection Method
Seismic Refraction Method

Seismic Reflection

It is well know that reflections of sound waves (compression waves) from the subsurface arrive at the geophones at some measurable time after the source pulse. If we know the speed of sound in the earth and the geometry of the wave path, we can convert that seismic travel time to depth. By measuring the arrival time at successive surface locations we can produce a profile, or cross-section, of seismic travel times. In practice, the speed of sound in the earth varies enormously. Dry sand might carry sound waves at 250 m/s or less. At the other extreme, unfractured granite might have a velocity in excess of 6,000 m/s. More is the number of layers between the surface and the layer of interest, more complicated is the velocity picture. Various methods are used to estimate subsurface velocities including refraction analysis, borehole geophysical measurements, estimates from known lithologic properties, and analysis of reflection times at increasing offsets. Generally, a combination of velocity estimation methods will give the best results.

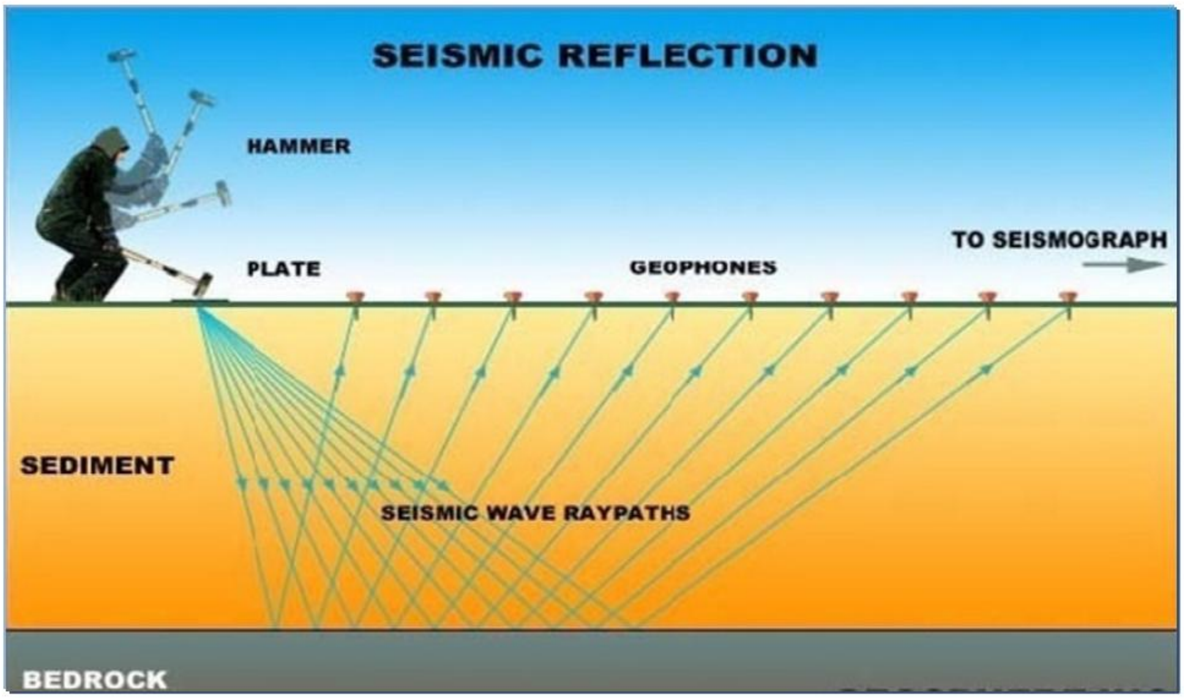

Fig. 23.1 Seismic Reflection method

Seismic Refraction

When a sound wave crosses an interface between layers of two different velocities, the wave is refracted. That is, the angle of the wave leaving the interface will be altered from the incident angle, depending on the relative velocities. Going from a low-velocity layer to a high-velocity layer, a wave at a particular incident angle (the "critical angle") will be refracted along the upper surface of the lower layer. As it travels, the refracted wave spawns upgoing waves in the upper layer, which impinge on the surface geophones. Sound moves faster in the lower layer than the upper, so at some point, the wave refracted along that surface will overtake the direct wave. This refracted wave is then the first arrival at all subsequent geophones, at least until it is in turn overtaken by a deeper, faster refraction. The difference in travel time of this wave arrival between geophones depends on the velocity of the lower layer. If that layer is plane and level, the refraction arrivals form a straight line whose slope corresponds directly to that velocity. The point at which the refraction overtakes the direct arrival is known as the "critical distance", and can be used to estimate the depth to the refracting surface.

a) Seismograph

b) Spectrum Analyzer

c) Portable Analyzer

d) Velocity Recorder

Plate 23.2 Geophysical Equipment

LECTURE 24

Plate 24.1 Geophones

*Under revision

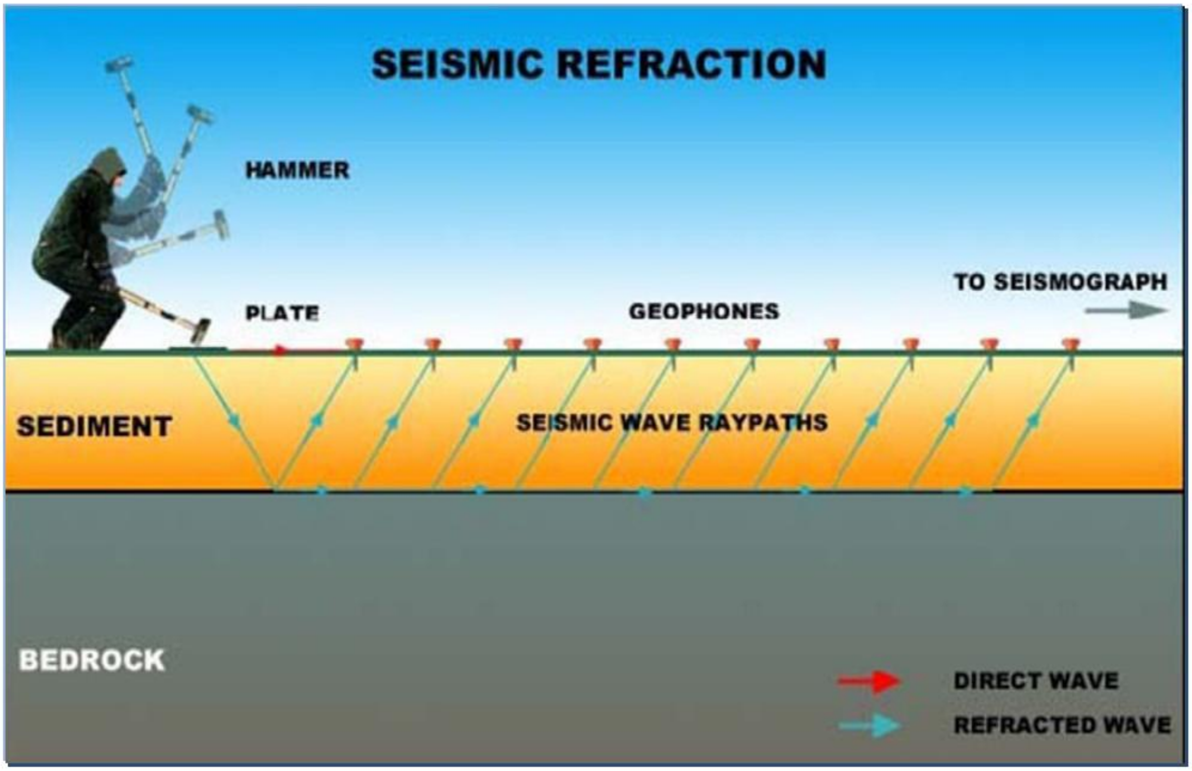

Fig.24.2 Seismic refraction method

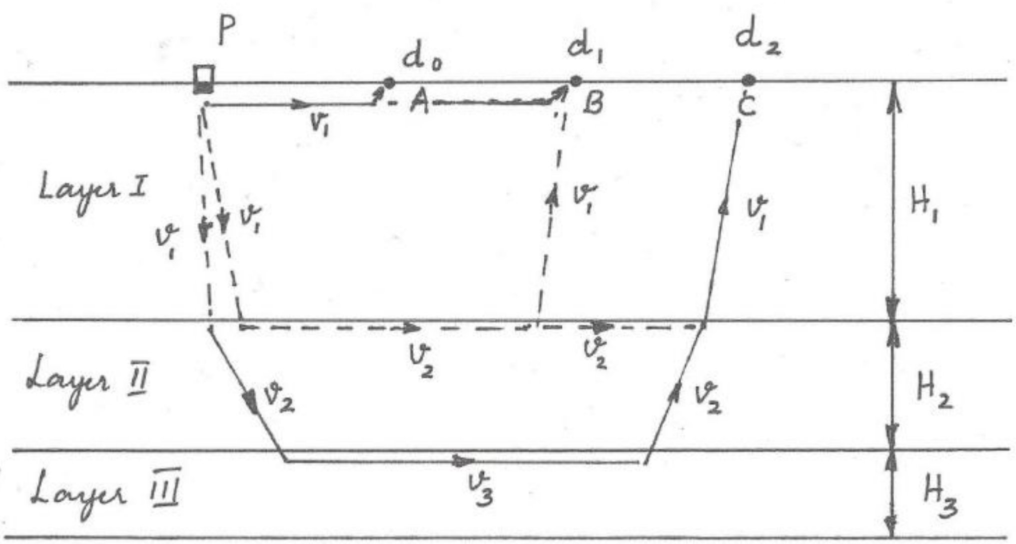

Fig. 24.3 Seismic refraction method

*Under revision

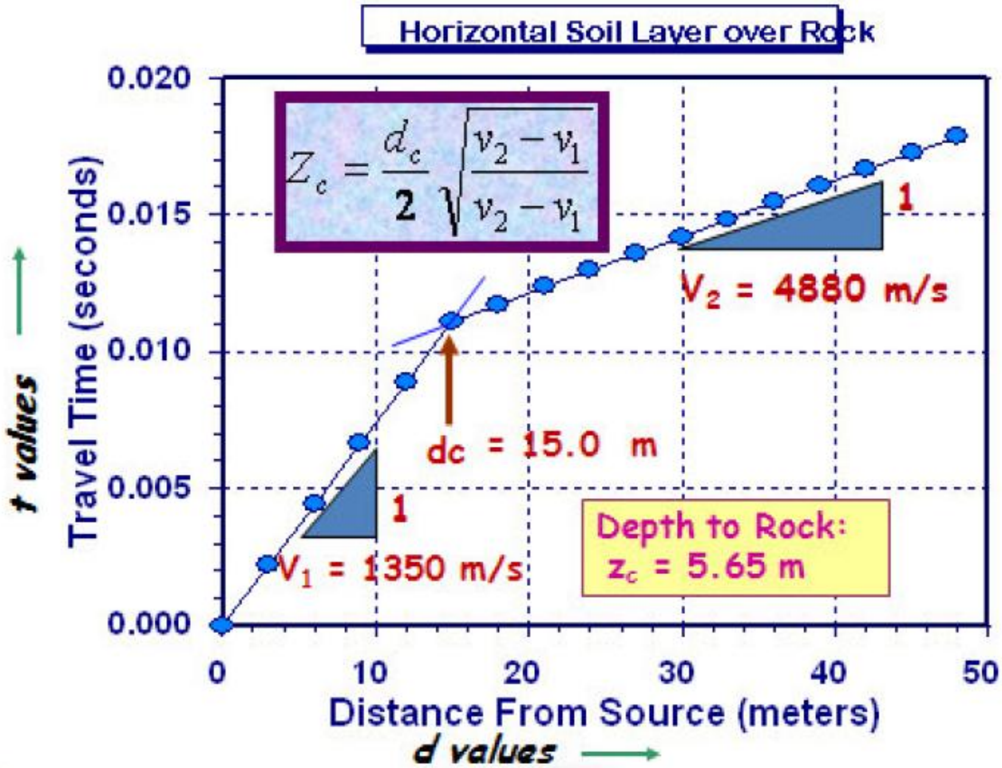

Fig.24.4 Plot of Travel time versus distance obtained from seismic refraction method

Now at critical distance d_1(or d_c), both the primary wave and refracted wave arrive simultaneously. Therefore, Time taken by primary wave to travel distance d_1 = time taken by refracted wave to travel distance ($2H_1$+ d_1).

$$\frac{d_1}{v_1} = \frac{2H_1}{v_1} + \frac{d_1}{v_2}$$

$$or \quad \frac{2H_1}{v_1} = \frac{d_1}{v_1} - \frac{d_1}{v_2}$$

$$or \quad \frac{2H_1}{v_1} = d_1\left[\frac{v_2 - v_1}{v_1 v_2}\right]$$

$$H_1 = \frac{d_1}{2}\left[\frac{v_2 - v_1}{v_2}\right]$$

---------- (1.1)

This equation gives reliable results when the waves are produced by sinusoidal force and not by impact. When impact loads are used the following empirical relation gives more reliable results:

$$H_1 = \frac{d_1}{2}\sqrt{\frac{v_2 - v_1}{v_2 - v_1}}$$

------------ (1.2)

Applicability of Seismic refraction method

- The seismic refraction method is commonly applied to shallow investigations up to about 100m.
- However, with sufficient energy, surveys to several hundred meters are possible.

LECTURE 25

Limitations of Seismic Methods

The Method cannot be used where a hard layer overlies a soft layer, because there will be no measurable refraction from a deeper soft layer. Test data from such an area would tend to give a single-slope line on the travel-time graph, indicating a deep layer of uniform material.

The method cannot be used in an area covered by concrete or asphalt pavement, since these materials represent a condition of hard surface over a soft stratum.

- A frozen surface layer also may give results similar to the situation of a hard layer over a soft layer.
- If the area contains some underground features, such as buried conduits, irregularly dipping strata, discontinuities such as rock faults or earth cuts, irregular water table, and the existence of thin layers varying materials, the interpretation of the results becomes very difficult.
- The method require sophisticated and costly equipment.
- For proper interpretations of the seismic survey results, the services of an expert are required.

Surface Resistivity Methods

These methods make use of Electromagnetic Wave Geophysics. These are nondestructive methods, non-invasive and are conducted across surface. Here measurements of electrical & magnetic properties of the ground namely, resistivity (conductivity), permittivity, dielectric, and magnetic fields are measured. They cover wide spectrum in frequencies (10 Hz $< f < 10^{22}$ Hz).

The popular and most widely used surface resistivity methods are:

- Electrical Profiling Method
- Electrical Sounding Method

Electrical Resistivity

"Electrical Resistivity is the physical property of a material, which is defined as the resistance of the material to the passage of electrical current".

It is expressed as:

$$\rho = \frac{RA}{L}$$

Where R = Electrical Resistance (Ohms)

A = area of cross section (m^2)

L = Length of Conductor (m)

ρ = Electrical Resistivity (Ohm-m)

Electrical Profiling Method
Test Procedure
Four electrodes are placed in a straight line at equal distances as shown in Fig 25.1. The two outer electrodes are known as current electrodes and the inner electrodes are known as potential electrodes. The mean resistivity of the strata is determined by applying a direct current of 50 to 100 milli amperes between the two inner electrodes (here a null-point circuit is used that requires no flow of current at the instant of measurement).

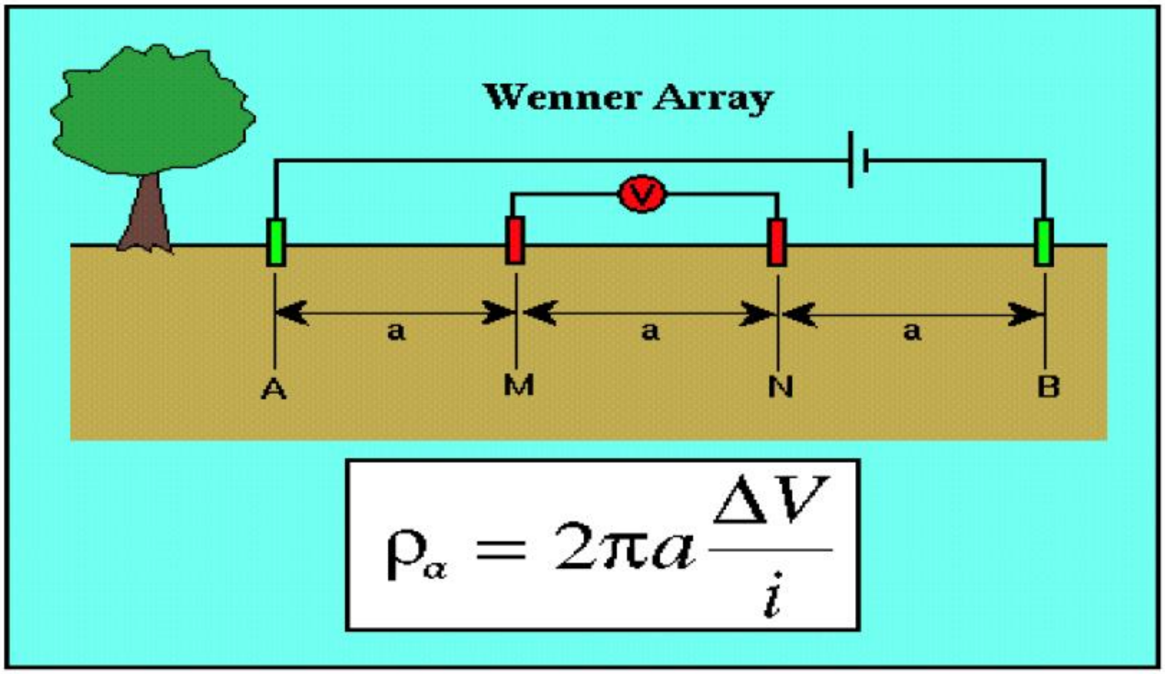

Fig. 25.1. Wenner arrangement
In a semi-infinite homogeneous isotropic material, the electrical resistivity, is given by the formula:

$$\rho = 2\pi a \frac{\Delta V}{i}$$

Where,
a = distance between electrodes (m).
ΔV = potential drop between the inner electrodes.
i = current flowing between the outer electrodes (Amps).
ρ = Mean resistivity (Ohm-m).

*Under revision

The calculated value is the apparent resistivity, which is the weighted average of all materials within the zone created by the electrical field of the electrodes. The depth of material included in the measurement is approximately the same as the spacing between the electrodes. The electrodes are moved as a group, and different profile lines are run across the area. The test is repeated after changing the spacing ('a') and again determining the mean resistivity with the new spacing. It is necessary to make a preliminary trial on known formations, in order to be in a position to interpret the resistivity data for knowing the nature and distribution of soil formations.

Applications of Electrical Profiling method

- This method is useful for establishing boundaries between different strata.
- The method is generally used for locating sand and gravel deposits (or ore deposits) within a fine-grained soil deposit.

Electrical Sounding Method

Test procedure

In this method a centre location for the electrodes is selected. A series of resistivity readings is obtained by gradually increasing the electrode spacing. As the depth of the current penetration is equal to the electrode spacing, the changes in the mean resistivity is correlated to the changes in strata at that location.

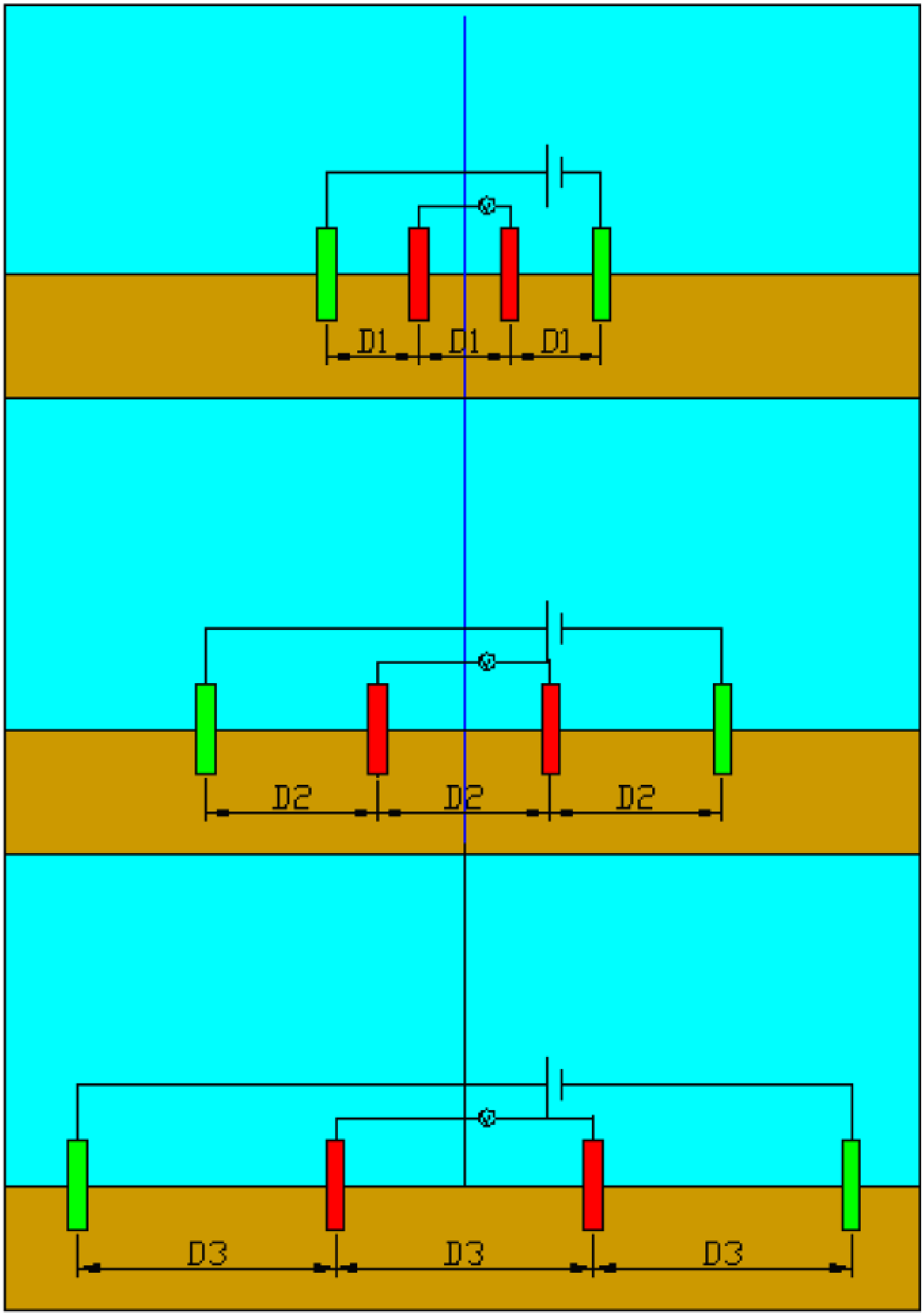

Fig. 25.2. Electrode arrangement for electrical sounding method

*Under revision

LECTURE 26

Table 26.1 Representative Resistivity values (after Peck, et. Al., 1974)

SL.No	Material	Resistivity, ρ (Ohm-cm x 10^3)
1	Clay and saturated silt	0-10
2	Sandy clay and wet silty sand	10-25
3	Clayey sand and saturated sand	25-50
4	Sand	50-150
5	Gravel	150-500
6	Weathered rock	100-200
7	Sound rock	150-4000

Applications of Electrical Sounding method

- This method is useful in studying the layering of materials.
- The method is capable of indicating sub-surface conditions where a hard layer underlies a soft layer and also the situation of a soft layer underlying a hard layer.

Limitations of Electrical Resistivity Methods

- The methods are capable of detecting only the strata having different electrical resistivity.
- The results are considerably influenced by surface irregularities, wetness of the Strata and electrolyte concentration of the ground water.
- As the resistivity of different strata at the interface changes gradually and not abruptly as assumed, then the interpolation becomes difficult.
- The services of an expert in the field are needed.

Boring Log

During soil exploration all suitable details are recorded and presented in a boring log.

Additional information consisting mainly of lab and field test result is added to complete the boring log.

Details of Boring Log

The ground conditions discovered in each borehole are summarised in the form of a bore log. The method of investigation and details of the equipment used should be stated on each log. The location, ground level and diameter of the hole should be specified.
The names of the client and the project should be mentioned.

Other Details of Boring Log

- The soil profile with elevations of different strata.
- Ground water level.
- Termination level of the bore hole.
- The depth at which samples were taken or at which in-situ tests were performed.
- The type of soil samples.
- N-values at the measured elevation.
- The results of important laboratory tests.

Table 26.2 Typical Bore Log

BORE LOG

Project : Proposed Commercial complex
Client: Sri Sreenivas
Location: J.P.Nagar, Bangalore

Type of Boring: Manual
Bore Hole Diameter: 150 mm
GWT from GL : Nil
Date of Commencement : 20-11-08
Date of Completion : 20-11-08

BH. No.: 1

Depth (m)	Type of sampling	S.P.T value (N)	Layer (m)	Legend	Description	Remarks
0.5					Brownish Sandy.soil	
1.0	DS					
1.5	SPT	4+6+8 (N=14)	1.5			
2.0	DS					
2.5					Yellowish Sandy.soil	
3.0	SPT	7+11+13 (N=24)				
3.5						
4.0			4.0		Weathered Rock	
4.5	SPT	18+23+31 (N=54)				
5.0			5.0			
5.5					Soft Disintegrated Rock	
6.0	SPT	Refusal Stratum	6.0			
					Borehole Terminated @ 6.0 m	

SPT – Standard Penetration Test
DS – Disturbed Sample
R – Refusal

Fig.1.19 Typical Boring Log

*Under revision

Soil Exploration Report

At the end of the soil exploration program, the soil and rock samples, collected from the Field are subjected to visual observation and laboratory tests. Then, a soil exploration report is prepared for use by the planning and design office. Any soil exploration report should contain the following information:

1. Scope of investigation.
2. General description of the proposed structure for which the exploration has been conducted.
3. Geological conditions of the site.
4. Drainage facilities at the site.
5. Details of boring.
6. Description of subsoil conditions as determined from the soil and rock samples collected.
7. Ground water table as observed from the boreholes.
8. Details of foundation recommendations and alternatives.
9. Any anticipated construction problems.
10. Limitations of the investigation.

The following graphic presentations also need to be attached to the soil exploration report:

1. Site location map.
2. Location of borings with respect to the proposed structure.
3. Boring logs.
4. Laboratory test results.
5. Other special presentations.

The boring log is the graphic representation of the details gathered from each borehole.

LECTURE 27

BEARING CAPACITY

Foundation :-

The lowest part of the structure which is in contact with soil and transmits loads to it.

Footing :-

The portion of the foundation of the structure, which transmits loads directly to the foundation soil.

Bearing Capacity :-

The load carrying capacity of foundation soil or rock which enables it to bear and transmit loads from a structure.

Ultimate bearing capacity :-

Maximum pressure which a foundation can withstand without the occurrence of shear failure of the foundation.

Gross bearing capacity :-

The bearing capacity inclusive of the pressure exerted by the weight of the soil standing on the foundation or the surcharge pressure as it is sometimes called.

Net bearing capacity :

Gross bearing capacity minus the original overburden pressure or surcharge pressure at the foundation level.

Safe bearing capacity :

Ultimate bearing capacity divided by the factor of safety. The factor of safety in foundation may range from 2 to 5, depending upon the importance of the structure and the soil profile at the site.

*Under revision

Allowable bearing pressure :-

The maximum allowable net loading intensity on the soil at which the soil neither fails in shear nor undergoes excessive or intolerable settlement, detrimental to the structure.

Methods of determining Bearing Capacity :-

1. Bearing capacity tables in various building codes.
2. Analytical methods.
3. Plate bearing tests.
4. Penetration tests.
5. Model tests and prototype tests.
6. Laboratory tests.

Analytical Methods :-

The following analytical approaches are available :

1. The theory of elasticity (Schleicher's method).
2. Earth pressure theory (Rankine's method).
3. The theory of plasticity (Fellenius method).

Schleicher's Method :-

$$S = \frac{Kq(1-\nu^2)\sqrt{A}}{E}$$

where, K = shape coefficient .

q = net pressure applied from the slab on to the soil.

A = area of the bearing slab.

E = Modulus of elasticity of soil.

ν = Poisson's ratio for the soil.

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Rankine's Method

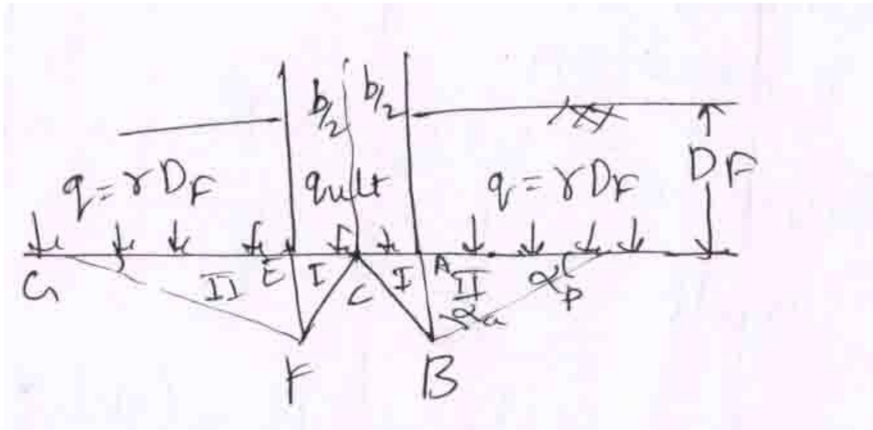


Fig. 27.1

$$q_{ult} = \frac{1}{2}\gamma b N_r + \gamma D_F Nq^2$$

where, $N_\gamma = \frac{1}{2}\sqrt{N_\phi}\,(N_\phi{}^2 - 1)$

$N_q = N_\phi{}^2$

Both are known as bearing capacity factors.

Fellenius Method

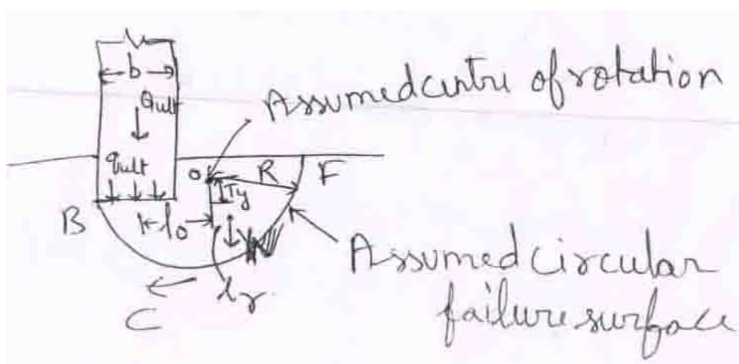


Fig. 27.2

Moments about the centre of rotation

$Q_{ult}\, l_0 = wl_r + CR$

$Q_{ult} = W\frac{l_r}{l_0} + C\frac{R}{l_0}$

$$q_{ult} = \frac{Wl_r}{bl_0} + \frac{CR}{bl_0} = \frac{(wl_r + CR)}{bl_0}$$

*Under revision

LECTURE 28

Terzaghi's Method :-

$$q_{ult} = cN_C + \gamma D_F N_q + \frac{1}{2}\gamma b N_\gamma$$

N_c, N_q and N_γ are called bearing capacity factors.

For square footings

$$q_{ult\,c} = 1.3cN_C + \gamma D_F N_q + 0.4\gamma b N_r$$

Safe Bearing Capacity :-

Procedure

1. The surcharge pressure γD_F is deducted from the gross ultimate bearing capacity q_{ult}, to give the net ultimate bearing capacity q_{ne+ult}.
2. The net ultimate bearing capacity is divided by the chosen factor of safety η , to give the net safe bearing capacity q_{ns}.
3. Finally the surcharge pressure is added to the net safe bearing capacity, to give the safe bearing capacity, q_s, against shear failure.

$$q_{netult} = q_{ult} - \gamma D_F$$

$$q_{ns} = \frac{q_{netult}}{\eta}$$

$$q_s = q_{ns} + \gamma D_F = \frac{q_{netult}}{\eta} + \gamma D_F$$

$$q_s = \frac{(q_{ult} - \gamma D_F)}{\eta} + \gamma D_F$$

$$q_s = \frac{q_{ult}}{\eta} + \frac{\gamma D_F(\eta - 1)}{\eta}$$

$$q_{netult} = q_{ult} - \gamma D_F = cN_C + \gamma D_F(N_q - 1) + \frac{1}{2}\gamma b N_\gamma$$

*Under revision

Effect of Water Table fluctuation

The basic theory of bearing capacity is derived by assuming the water table to be at great depth below and not interfering with the foundation. However, the presence of water table at foundation depth affects the strength of soil. Further, the unit weight of soil to be considered in the presence of water table is submerged density and not dry density. Hence, the reduction coefficients R_{W1} and R_{W2} are used in second and third terms of bearing capacity equation to consider the effects of water table.

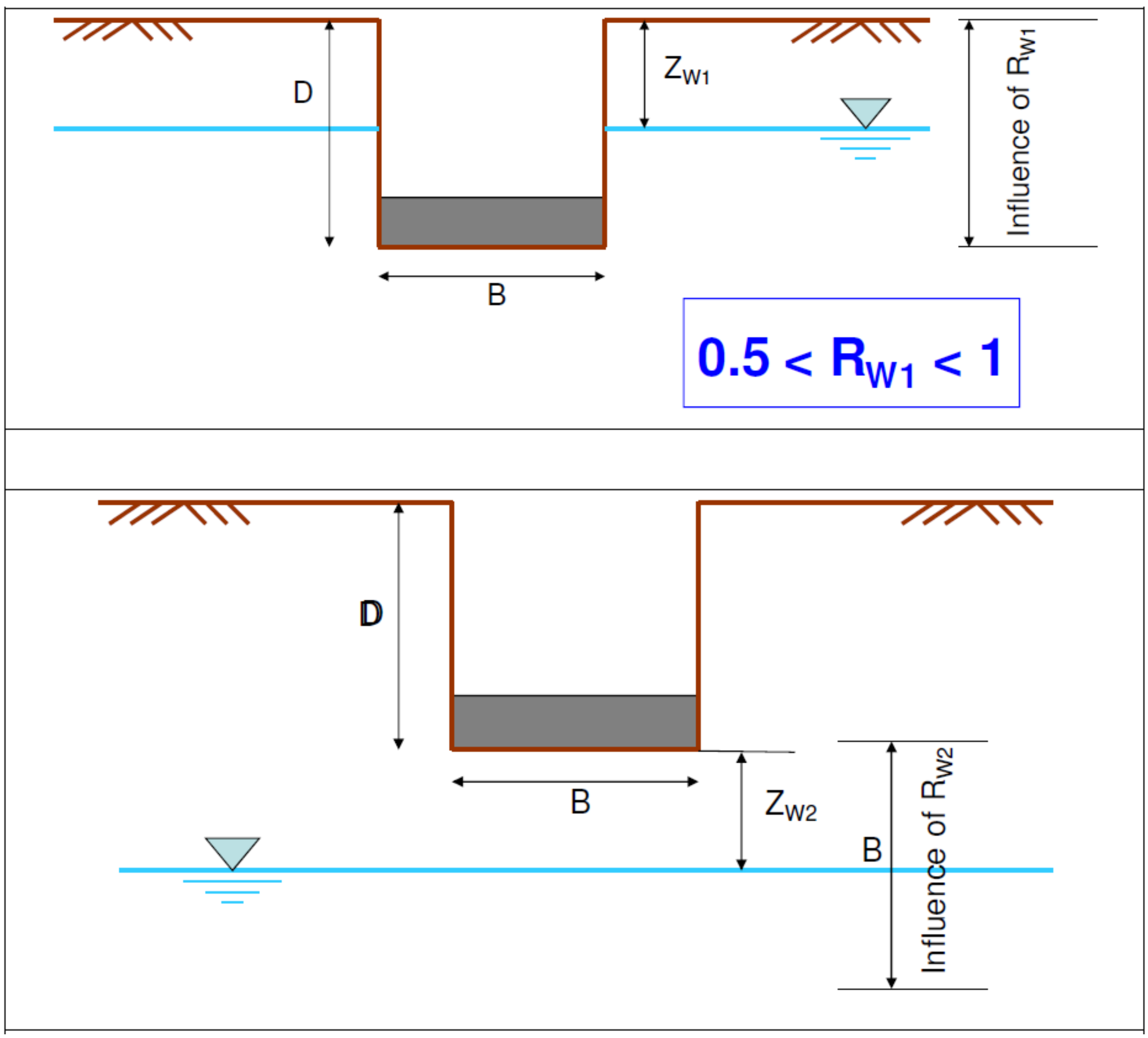


Fig. 28.1 Effect of water table on bearing capacity.

*Under revision

Ultimate bearing capacity with the effect of water table is given by,

$$q_f = cN_c + \gamma DN_q R_{w1} + 0.5 \gamma BN_\gamma R_{w2}$$

$$R_{w1} = \frac{1}{2}\left[1 + \frac{Z_{w1}}{D}\right]$$

where Z_{W1} is the depth of water table from ground level.

1. $0.5 < R_{w1} < 1$.
2. When water table is at the ground level ($Z_{w1} = 0$), $R_{w1} = 0.5$.
3. When water table is at the base of foundation ($Z_{w1} = D$), $R_{w1} = 1$.
4. At any other intermediate level, R_{w1} lies between 0.5 and 1.

*Under revision

Here ,

$$R_{w2} = \frac{1}{2}\left[1 + \frac{Z_{w2}}{B}\right]$$

where Z_{W2} is the depth of water table from foundation level.

1. $0.5 < R_{w2} < 1$
2. When water table is at the base of foundation ($Z_{w2} = 0$), $R_{w2} = 0.5$.
3. When water table is at a depth B and beyond from the base of foundation ($Z_{w2} >= B$), $R_{w2} = 1$.
4. At any other intermediate level, R_{w2} lies between 0.5 and 1.

LECTURE 29

Settlement :

Settlement is the vertically downward movement of structure due to the compression of underlying soil because of increased load.

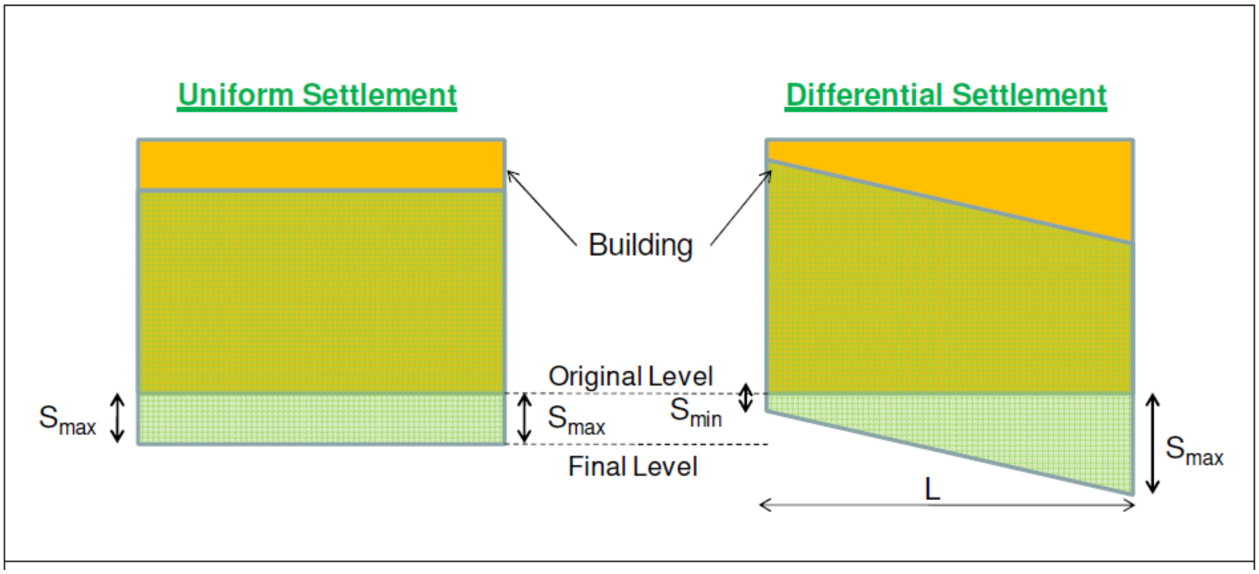


Fig. 29.1 : Concepts of uniform and differential settlement

Maximum Settlement : It is the absolute maximum downward movement of any part of building element.
Maximum Settlement = S_{max}

Differential Settlement : It is the maximum difference between two points in a building element.
Differential Settlement = S_{max} - S_{min}

Angular Distortion : It is another method of expressing differential settlement.
Angular Distortion = Differential Settlement/Length of element
= $(S_{max} - S_{min})/L$

Fig. 29.2 represents soil movement under different circumstances at the ground level. The fluctuation in the elevation of ground level depends on seasonal changes in expansive (Indian Black Cotton) soils and,

*Under revision

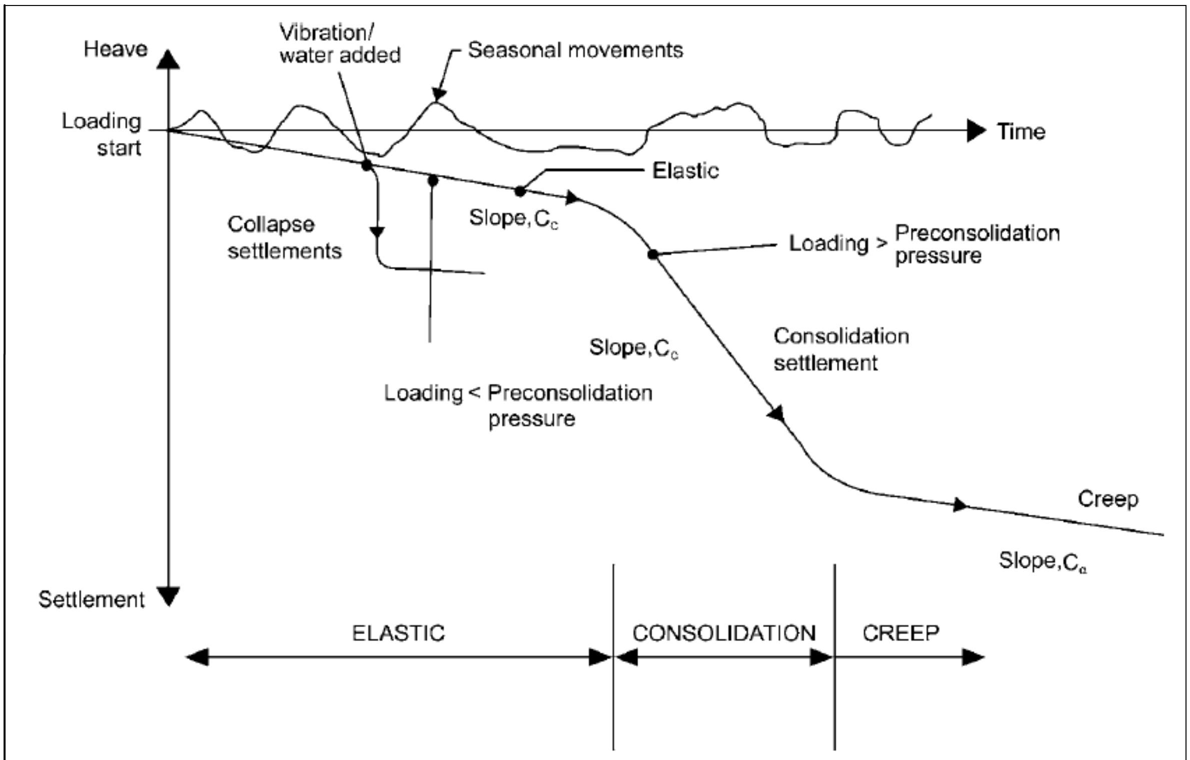


Fig.29.2 : Soil movement in different situations

Table 29.1 presents the different types of movements experienced by various soils. It can be noticed that only few soils such as clay can experience swell. Further, consolidation settlement and creep settlement are more pronounced in clay. Immediate or elastic settlement is observed in each and every soil. First three, namely Immediate, Consolidation and Creep settlement cause downward movement of ground while swell causes upward movement of ground.

Table 29.1: Types of Movement in different soils

Principal Soil Type	Type of Movement			
	Immediate	Consolidation	Creep	Swell
Rock	Yes	No	No	Some
Gravel	Yes	No	No	No
Sand	Yes	No	No	No
Silt	Yes	Minor	No	Yes
Clay	Yes	Yes	Yes	Yes
Organic	Yes	Minor	Yes	Yes

Extract from IS 1904 -1986 : General Requirements for Design & Construction of Foundation

IS 1904-1986 presents Table 1 which gives details about the permissible settlement in steel structures, reinforced concrete structures, multi-storeyed buildings and water towers and silos in two different types of soils, namely (1) Sand and hard clay and (2) Plastic clay. The settlements considered are maximum settlement, differential settlement and angular distortion or tilt. The details in this table can be followed in the absence of more precise settlement suggested by the user. In case of multi storeyed buildings both RC frames and load bearing wall structures are considered. Load bearing structures with L/H 2 and 7 are dealt with. Two types of foundations considered are isolated footing and raft foundation. Table 29.2 gives the extract of IS code and Table 29.3 presents the same table in different form for steel and RC structures. A maximum settlement of 75 mm, differential settlement of 0.0015L and angular distortion of 1 in 666 is permitted for isolated footings.

*Under revision

Table 29.2 : Permissible uniform and differential settlement and tilt for shallow foundations.

TABLE 1 PERMISSIBLE DIFFERENTIAL SETTLEMENTS AND TILT (ANGULAR DISTORTION) FOR SHALLOW FOUNDATION IN SOILS

(*Clause* 16.3.4)

Sl No.	Type of Structure	Isolated Foundations						Raft Foundations					
		Sand and Hard Clay			Plastic Clay			Sand and Hard Clay			Plastic Clay		
		Maximum settlement	Differential settlement	Angular distortion	Maximum settlement	Differential settlement	Angular distortion	Maximum settlement	Differential settlement	Angular distortion	Maximum settlement	Differential settlement	Angular distortion
		mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm	mm
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
i)	For steel structure	50	·003 3L	1/300	50	·003 3L	1/300	75	·003 3L	1/300	100	·003 3L	1/300
ii)	For reinforced concrete structures	50	·001 5L	1/666	75	·001 5L	1/666	75	·002 1L	1/500	100	·002L	1/500
iii)	For multistoreyed buildings												
	a) RC or steel framed buildings with panel walls	60	·002L	1/500	75	·002L	1/500	75	0·002 5L	1/400	125	0·003 3L	1/300
	b) For load bearing walls												
	1) L/H = 2*	60	·000 2L	1/5 000	60	·000 2L	1/5 000	Not likely to be encountered					
	2) L/H = 7*	60	·000 4L	1/2 500	60	·000 4L	1/2 500	Not likely to be encountered					
iv)	For water towers and silos	50	·001 5L	1/666	75	·001 5L	1/666	100	·002 5L	1/400	125	·002 5L	1/400

NOTE — The values given in the table may be taken only as a guide and the permissible total settlement/different settlement and tilt (angular distortion) in each case should be decided as per requirements of the designer.

L denotes the length of deflected part of wall/raft or centre-to-centre distance between columns.

H denotes the height of wall from foundation footing.

*For intermediate ratios of L/H, the values can be interpolated.

Table 29.3 : Permissible uniform and differential settlement and tilt, for shallow foundations.

	Sand & Hard Clay			**Plastic Clay**		
	Max. Settlement	**Diff. Settlement**	**Angular distortion**	**Max. Settlement**	**Diff. Settlement**	**Angular distortion**
Isolated foundation i) **Steel** **ii) RCC**	50mm 50mm	0.0033L 0.0015L	1/300 1/666	50mm 75mm	0.0033L 0.0015L	1/300 1/666
Raft foundation **i) Steel** **ii) RCC**	75mm 75mm	0.0033L 0.002L	1/300 1/500	100mm 100mm	0.0033L 0.002L	1/300 1/500

*Under revision

LECTURE 30

Table 30.1 : Limiting Values of movement for Geotechnical Structures

Design Application	Parameter	Typical Movement
Shallow Foundation	Allowable Bearing Pressure	25 mm for buildings
Deep Foundation	Skin Friction	10 mm for skin friction to mobilize
Retaining Wall	Active & Passive earth pressure	0.1% H for Ka & 1% H for Kp to mobilize in dense sand
Pavement	Rut depth based on strain due to no. of repetitions	20 mm rut depth in major roads & 100 mm rut depth in minor roads
Embankment	Self weight settlement	0.1% height of embankment
Drainage	Total settlement	100 to 500 mm

Total Settlement

Total foundation settlement can be divided into three different components, namely Immediate or elastic settlement, consolidation settlement and secondary or creep settlement as given below.

$$\mathbf{S = S_I + S_C + S_S}$$

Here, S = Total Settlement
S_I = Immediate / Elastic Settlement
S_C = Consolidation Settlement
S_S = Secondary Settlement

Immediate Settlement

- Immediate settlement is also called elastic settlement.
- It is determined from elastic theory.
- It occurs in all types of soil due to elastic compression.
- It occurs immediately after the application of load.
- It depends on the elastic properties of foundation soil, rigidity, size and shape of foundation.

Immediate settlement is calculated by the equation mentioned below.

$$S_I = \left(\frac{1-\mu^2}{E} \right) qBI_\rho$$

Here,

S_I = Immediate settlement.
μ= Poisson's Ratio of foundation soil.
E = Young's modulus of Foundation Soil.
q = Contact pressure at the base of foundation.
B = Width of foundation.
I_ρ = Influence Factor.

Table 30.2 presents the typical values of Poisson's ratio in different soils. Table 30.3 represents the ranges of soil modulus in clayey soil of different consistencies in undrained state. In the absence of more accurate data, the values in tables can be used. The influence factor $I_\rho$depends on the shape and flexibility of footing. Further, in flexible footing I_ρ is not constant. Table 30.4 presents the different values of I_ρ .

*Under revision

Table 30.2 : Typical Range of Poisson's Ratio for different soils

Type of Soil	**Poisson's Ratio**
Saturated Clay	0.5
Sandy Clay	0.3 – 0.4
Unsaturated Clay	0.35 – 0.4
Loess	0.44
Silt	0.3 – 0.35
Sand	0.15 – 0.3

Table 30.3 : Typical Range of Soil Modulus in undrained state

Soil Type	**Soil Modulus (kPa)**
Very Soft Clay	400 – 3000
Soft Clay	1500 – 4000
Medium Clay	3000 – 8500
Hard Clay	7000 – 17000
Sandy Clay	28000 – 42000

Table 30.4 : Typical Values of Influence Factors I_ρ

Shape of Footing	**Flexible**			**Rigid**
	Center	**Corner**	**Mean**	
Circle	1.00	0.64	0.85	0.80
Rectangle L/B = 1	1.12	0.56	0.95	0.90
Rectangle L/B = 1.5	1.36	0.68	1.20	1.09
Rectangle L/B = 2	1.52	0.77	1.31	1.22
Rectangle L/B = 5	2.10	1.05	1.83	1.68
Rectangle L/B = 10	2.52	1.26	2.25	2.02
Rectangle L/B = 100	3.38	1.69	2.96	2.70

LECTURE 31

Foundation settlements :-

Source of settlement :-

Foundation settlements may be caused due to some or a combination of the following factors.

1. Elastic compression of the foundation and the underlying soil, giving rise to what is known as immediate, contact settlement.
2. Plastic compression of the underlying soil, giving rise to consolidation, settlement of fine grained soils both primary and secondary.
3. Vibration due to pile driving blasting and oscillating machinery in granular soils.
4. Seasonal swelling and shrinkage of expansive clays.

Bearing capacity of sands :-

$$q_{netult} = \alpha \gamma b N_{\gamma} + \gamma D_F (N_q - 1)$$

where, α = shape factor which is given as

0.5 for continuous footing of width ‘b’

0.4 for square footing of side (b) and

0.3 for circular footing of diameter ‘d’

Bearing capacity of Clays :-

$\phi = 0$

$$q_{ult} = cN_c + \gamma D_F = 5.7C + \gamma D_F$$

q_{netult} = 5.7 c, for continuous footings.

q_{netult} = 1.3 x 5.7 c = 7.4 c

For square or circular footings (c being the cohesion)

Q. What is the minimum depth required for a foundation to transmit a pressure 60 kN/m^2 in a cohesionless soil with γ = 18 kN/m^3 and $\phi = 18^0$? What will be the bearing capacity if a depth of 1.5 m is adopted according to Rankine's approach ?

Sol^n :

γ = 18 kN/m^3, $\phi = 18^0$, q = 60 kN/m^2

minimum depth of foundation, according to Rankine,

$$D_F = \frac{q}{\gamma}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2 = \frac{60}{18}\left(\frac{1-\sin 18}{1+\sin 18}\right)^2 = 0.93m \simeq 1\text{m}$$

If D_F = 1.5 m

$$q_{ult} = \gamma D_F\left(\frac{1+\sin\phi}{1-\sin\phi}\right)^2 = 18\times 1.5\left(\frac{1+\sin 18}{1-\sin 18}\right)^2 = 96.8 kN/m^2$$

LECTURE 32

Pile Foundations :

Two general forms of deep foundation are recognized :

1. Pile Foundation, 2. Pier, Caisson or well foundation.

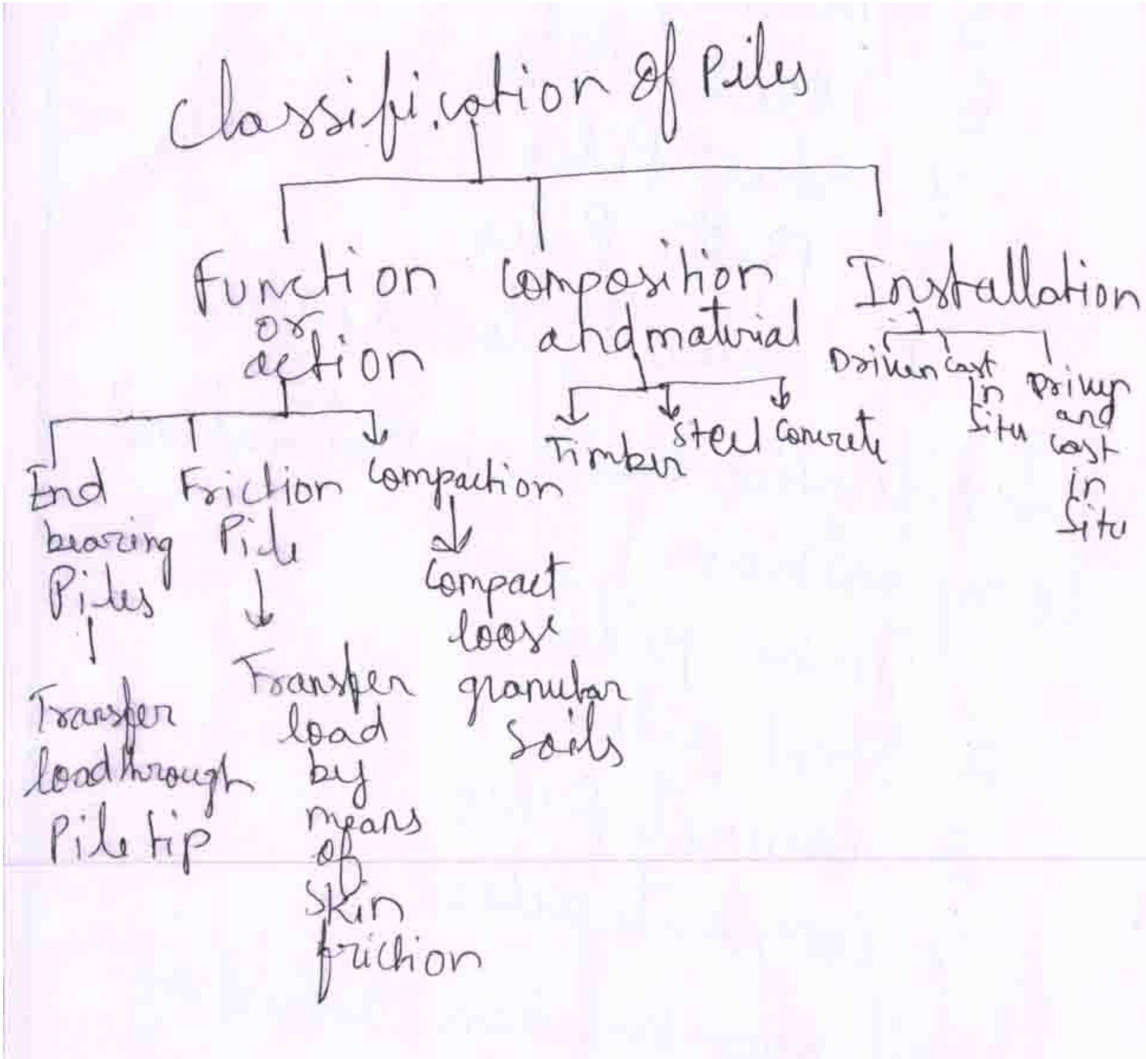


Fig. 32.1

*Under revision

Classification based on Function

1. End bearing piles
2. Friction piles
3. Tension or uplift piles
4. Compaction piles
5. Anchor piles
6. Fender piles
7. Sheet piles
8. Batter piles
9. Laterally loaded piles.

Classification based on Material and Composition

1 Timber piles
2. Steel piles
3. Concrete piles
4. Composite piles

Classification based on method of installation :-

1. Driven piles
2. Cast in-situ piles
3. Driven and cast in-situ piles

Pile driving :-

The operation of forcing a pile into the ground is known as pile driving.

Pile hammers are of the following types :-

1. Drop hammer
2. Single acting hammer
3. Double acting hammer
4. Diesel hammer
5. Vibratory hammer

Pile capacity :-

The following is the classification of the methods of determining pile capacity:-

1. Static analysis.
2. Dynamic analysis.
3. Load tests on pile.
4. Penetration tests.

Static Analysis :-

$$Q_{up} = Q_{eb} + Q_{SF}$$

where, Q_{up} = ultimate bearing load of the pile

Q_{eb} = end bearing resistance of the pile and

Q_{SF} = skin friction resistance of the pile

$Q_{eb} = q_b A_b$

$Q_{SF} = F_s A_s$

Here

q_b = bearing capacity in point bearing for the pile

F_s = unit skin friction for the pile soil system

A_b = bearing area of the base of the pile and

A_s = Surface area of the pile in contact with the soil.

For piles in sand

$$q_b = qN_q$$

The surcharge 'q' is given by

$q = \gamma z$ if $z < z_c$ and

$q = \gamma z_c$ if $z > z_c$

$I_D < 30\%$ $z_c = 10D$

$I_D > 70\%$ $z_c = 30D$

For piles in clay $q_b = q_c$

LECTURE 33

For unit skin friction reistance.

$$F_s = C_a + \sigma_h \tan\delta$$

C_a = adhesion

σ_h = average lateral pressure of soil against pile surface

δ = angle of wall friction

For piles in sands

$Fs = \sigma_h \tan\delta$,

For piles in clays

$F_s = C_a$

$C_a = \alpha c$

where α is called the adhesion factor

Dynamic Analysis

Engineering news formula

$$Q_{ap} = \frac{(Wn + ap)H}{6(S + 2.5)}$$ ---- Double acting steam hammers.

where W_n = weight of hammer (newtons)

a = effective area of piston mm^2

p = mean effective steam pressure (N/mm^2)

H = height of fall of hammer (metres)

S = Final penetration of pile per blow (mm) and

Q_{ap} = allowable load on the pile (kN)

*Under revision

Load test on pile :-

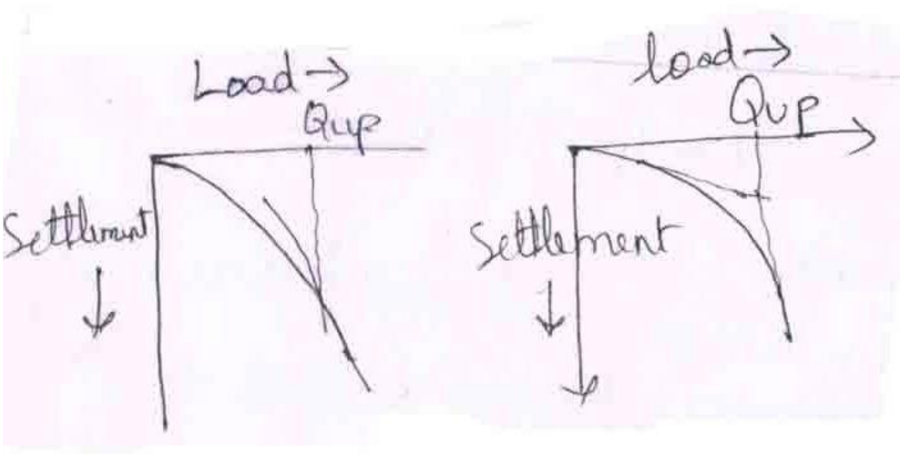


Fig. 33.1

Determination of ultimate load from load settlement curve for a pile.

The allowable load on a single pile may be obtained as one of the following [IS : 2911 (Part I) – 1974]

1. 50% of the ultimate load at which the total settlement is equal to one tenth the diameter of the pile.
2. Two third of the load which causes a total settlement of 12 mm.
3. Two third of the load which causes a net (plastic) settlement of 6 mm (total settlement minus elastic settlement).

Design of Pile Groups :-

The ultimate load will be equal to

$$Q_{ug} = PL\gamma_F + A\gamma_p$$

where, A = cross sectional area of pile group, at base = B x B = B^2

P = perimeter of pile group

P = 4B

τ_F = shear strength of soil = $\bar{c} = \tau = qu/2$ (for clayey soils)

$$\therefore Q_{ug} = 4BL\gamma_F + B^2\gamma p$$

$$= 4BL\overline{C} + B^2(9c_p)$$

In absence of any other specific (data) both $\bar{c}$ and c_p may be taken equal to $qu/2$. If, however, piles of the group are so spaced that they act individually, rather than acting in the group, the total load capacity of 'n' piles is given by :-

$Q_{un} = nQ_{up}$, where Q_{up} = load of individual pile.

The ultimate load (Q_U) of the pile groups will be then equal to lesser of Q_{ug} and Q_{un}, determined above and the permissible load will be equal to Q_U/F.

Q. In a 16 pile group, the pile diameter is 45 cm and centre to centre spacing of the square group is 1.5 m. If c = 50 kN/m^2, determine whether the failure would occur with the pile acting individually, or as a group ? Neglect bearing at the tip of the pile. All piles are 10 m. long. Take m = 0.7 for shear mobilization around each pile.

Solution :

n = 16, d = 45 cm, L = 10 m

width of group = B = (150 x 3)+45=495 cm = 4.95 m

(i) For the group

Q_{ug} = c x perimeter x length = c x 4B x L

= 50 x 4 x 4.95 x 10 = 9900 kN

(ii) For the piles acting individually

$Q_{ug} = nQ_{up} = n\{m_cA_p\}$

$A_p = \pi dL = \pi \times 0.45 \times 10$

$\therefore$ $Q_{ug} = 16 \times 0.7 \times 50 \times \pi \times 0.45 \times 10 = 7917$ kN which is less than the load carried by the group action. Hence the foundation will fail by the piles acting individually , and the load at failure would be 7917 kN.

LECTURE 34

DETAIL NOTES

Pile Foundation

It is a foundation system that transfers loads to a deeper and competent soil layer.

When To Use Pile Foundations

Inadequate Bearing Capacity of Shallow Foundations
To Prevent Uplift Forces
To Reduce Excessive Settlement

PILE CLASSIFICATION

Friction Pile

- Load Bearing Resistance derived mainly from skin friction

End Bearing Pile

- Load Bearing Resistance derived mainly from base

Friction Pile

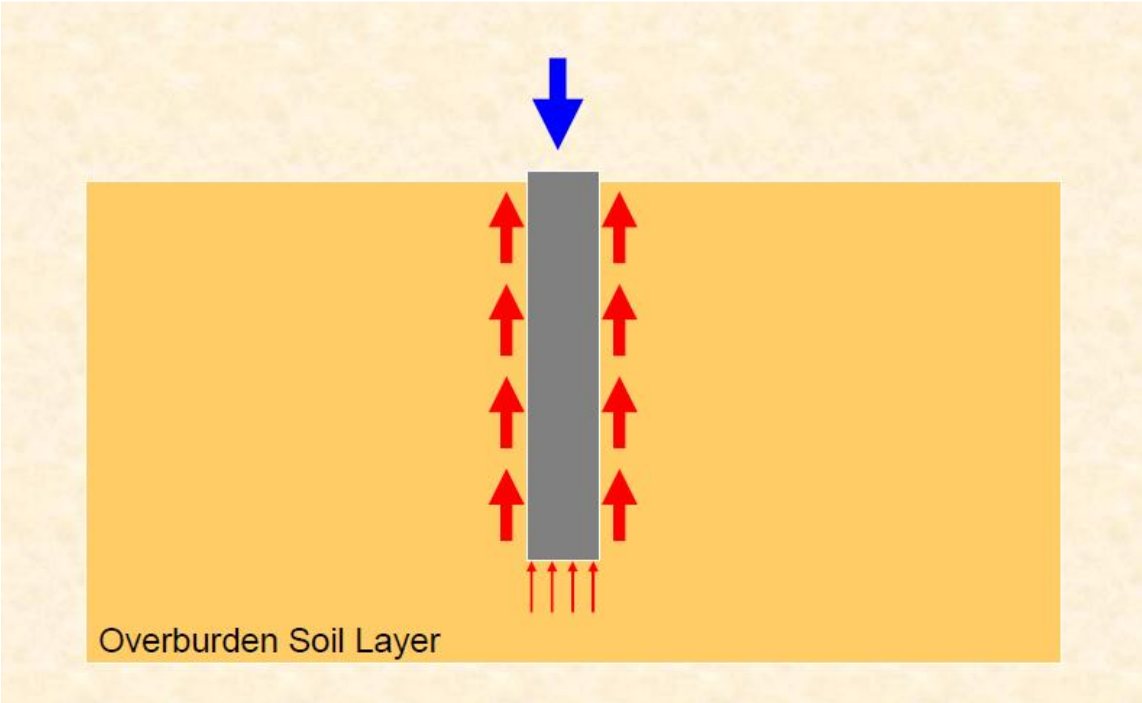


Fig. 34.1

End Bearing Pile

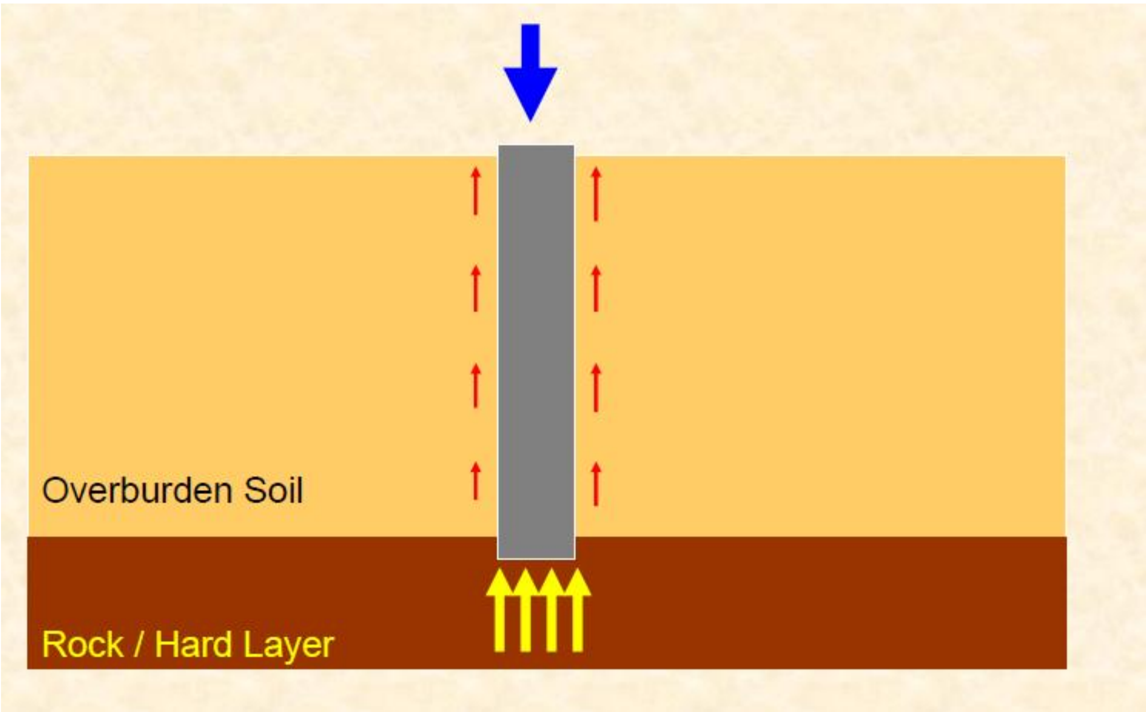


Fig. 34.2

SELECTION OF PILES

Factors Influencing Pile Selection

- Types of Piles Available in Market
- Installation Method
- Ground Conditions
- Site Conditions & Constraints (eg Accessibility)
- Type and Magnitude of Loading
- Development Program & Cost

Pile Capacity Design

Piles installed in a group may fail:

- Individually
- As a block

Piles fail individually
When installed at large spacing

Piles fail as a block
When installed at close spacing

*Under revision

Pile Capacity Design Single Pile Capacity

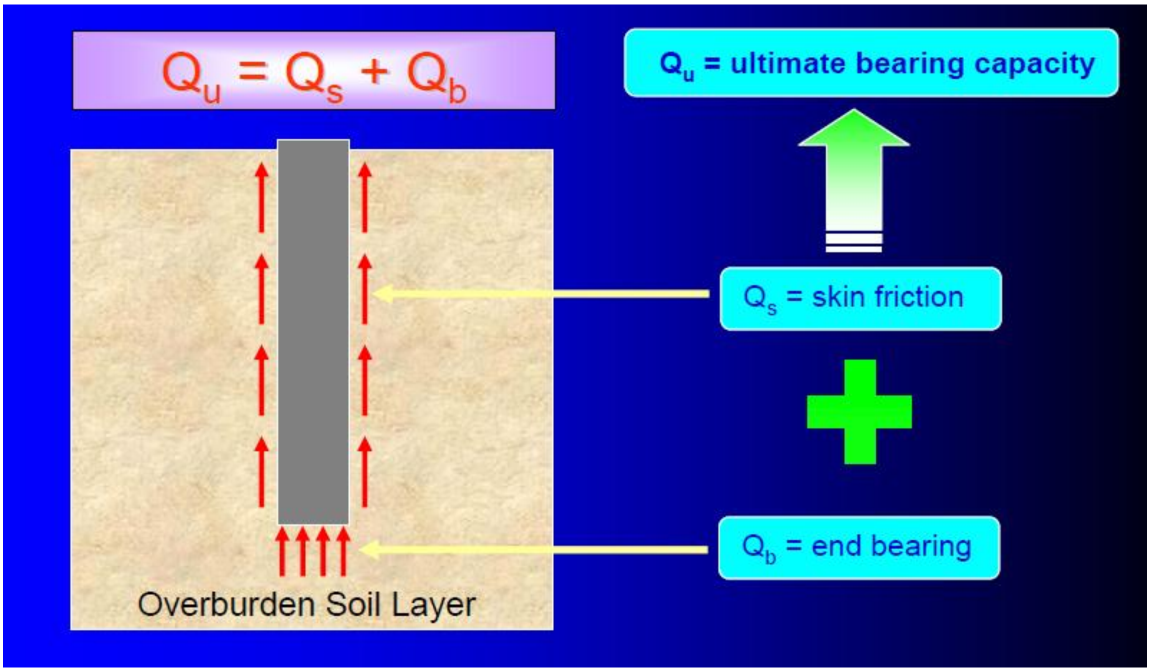
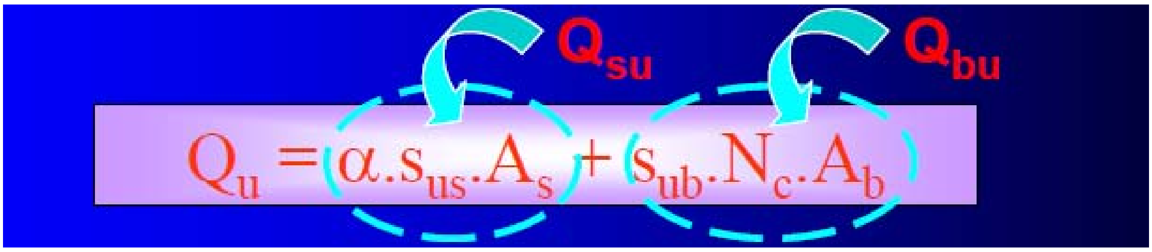


Fig. 34.3

Single Pile Capacity : In Cohesive Soil



Q_u = Ultimate bearing capacity of the pile
α = adhesion factor
S_{us} = average undrained shear strength for shaft
S_{ub} = undrained shear strength at pile base
N_c = bearing capacity factor (taken as 9.0)
A_b = cross sectional area of pile base
A_s = surface area of shaft

*Under revision

LECTURE 35

Values of undrained shear strength, S_u can be obtained from the following:

- Unconfined compressive test
- Field vane shear test

PILE INSTALLATION METHODS

- Diesel / Hydraulic / Drop Hammer Driving
- Jacked-In
- Prebore Then Drive
- Prebore Then Jacked In
- Cast-In-Situ Pile

Fig. 35.2

Diesel Drop Hammer Driving

Fig. 35.3

Jacked - In Piling

LECTURE 36

Fig. 36.1

Cast-In-Situ Piles (Micropiles)

RC Square Piles

Fig. 36.2

Pile Marking

Fig. 36.3

Pile Lifting

Fig. 36.4

Pile Fitting to Piling Machine

LECTURE 37

Fig. 37.1

Pile Positioning

*Under revision

Fig. 37.2

Pile Joining

Fig. 37.3

Steel H Piles

LECTURE 38

Fig. 38.1

Over Driving of Steel Piles

Fig. 38.2

BORED PILING MACHINE

Fig. 38.3

Completed Bored pile

LECTURE 39

Fig. 39.1

Damage to Timber Pile

*Under revision

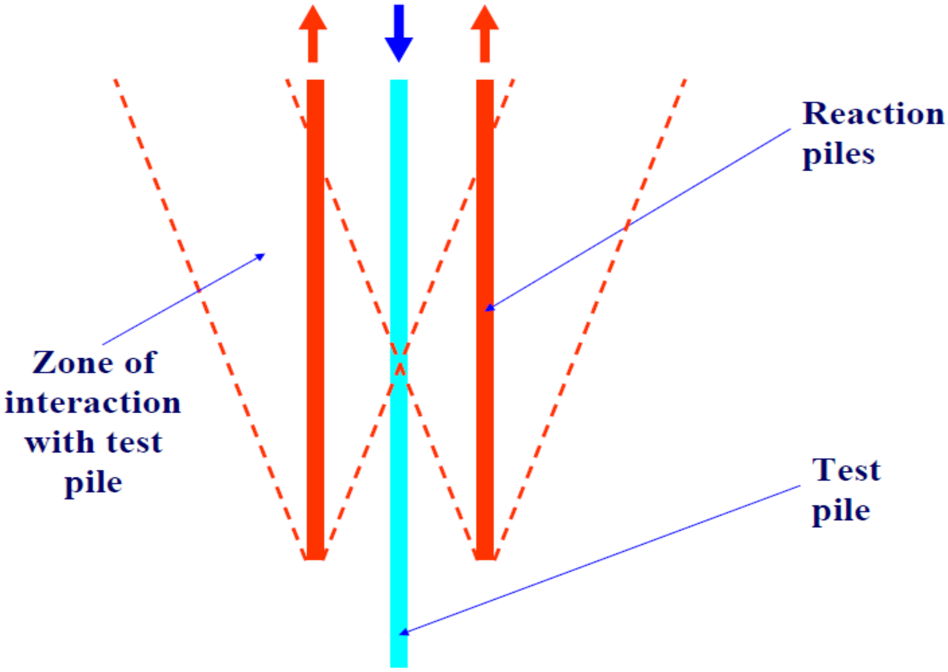


Fig. 39.2

Testing Set- up

- Latest version of ASTM D1143
- Published April 2007

Designation: D 1143/D 1143M – 07

Standard Test Methods for
Deep Foundations Under Static Axial Compressive Load[1]

This standard is issued under the fixed designation D 1143/D 1143M; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ϵ) indicates an editorial change since the last revision or reapproval.

This standard has been approved for use by agencies of the Department of Defense.

LECTURE 40

ASTM D1143

- **Clear distance of at least 5 times the maximum diameter**
    - **Caution on factors influencing results:**
    - **"Possible interaction ……….from anchor piles……."**

- **Probable causes of erratic and unpredictable pile capacities:**

    - Testing set-up using reaction piles
    - Drilling to the casing tip to form "bored pile"

Dynamic load test

Dynamic pile load test procedure is standardized by ASTM D4945-00 Standard Test Method for high strain dynamic testing of piles. It consists of estimating soil resistance and its distribution from force and velocity measurements obtained near the top of a foundation impacted by a hammer or drop weight. The impact produces a compressive wave that travels down the shaft of the foundation. A pair of strain transducers receives the signals necessary to compute force, while measurements from a pair of accelerometers are integrated to yield velocity. These sensors are connected to an instrument (such as a pile driving analyzer), that records, processes and displays data and results The capacity of the hammer should be large enough to achieve sufficient pile settlement so that the resistance of the tested pile can be fully mobilized. The load should be applied axially on the pile. The pairs of accelerometers and strain transducers are fixed to opposite sides of the tested pile, about at least one shaft diameter below the head, either by drilling and bolting directly to the pile or by welding mounting blocks to ensure a reasonably uniform stress field at the measuring elevation The modern change, which is considered as one of the significant beneficial changes that have been made to the system since the last Irish survey in 2000 is the introduction of theodolite to measure the pile displacement during the impact This dynamic pile load testing technique that has been used most often in Ireland and UK is called SIMBAT There are two known methods, based on wave propagation theory, for the analysis and interpretation of the dynamic pile load test. The CASE method, IMPEDANCE method and TNO method are considered as direct methods. CAPWAP, TNO wave and SIMBAT are considered as indirect methods.

SIMBAT Dynamic load test method

The SIMBAT is used mainly for bored piles . It is well accepted in France, Eire and the UK and to a lesser extent in Italy and Spain . Essential preparations should be made before the test is carried out in the field. A pile cap, as an extension to the shaft head (with the same diameter as the shaft), should be constructed. The length of the cap should be at least 1.5 to 2.5 shaft diameter. The cap must be cylindrical, smooth, well-reinforced and of good quality concrete. The side of the cap is instrumented with two strain gauges, two accelerometers and electronic theodolite target. The electronic theodolite is placed 3 to 5m from the pile head. A schematic sketch of SIMBAT equipment and instrumentation is presented in Figure (40.1), whereas a photograph of a complete set of the equipment is shown in Figure (40.2). A series of hammer blows are made with the hammer drop height progressively increased and decreased . The main difference between this system and other dynamic load test systems is the using of an electronic scanning theodolite that records penetration for each blow and records real time elastic displacement . The interpretation of the dynamic pile load test data according to the SIMBAT procedure includes measuring of velocity from the integral of the acceleration and then correcting velocity using the theodolite as an adjustment signal. The measured force at the pile top is separated into components upward and downward. The dynamic (or total) reaction is calculated for each hammer blow and plotted versus cumulative penetration for the whole set of blows. The dynamic load is converted to static load and the predicted static load-settlement curve can be plotted. The static plot is verified by modeling.

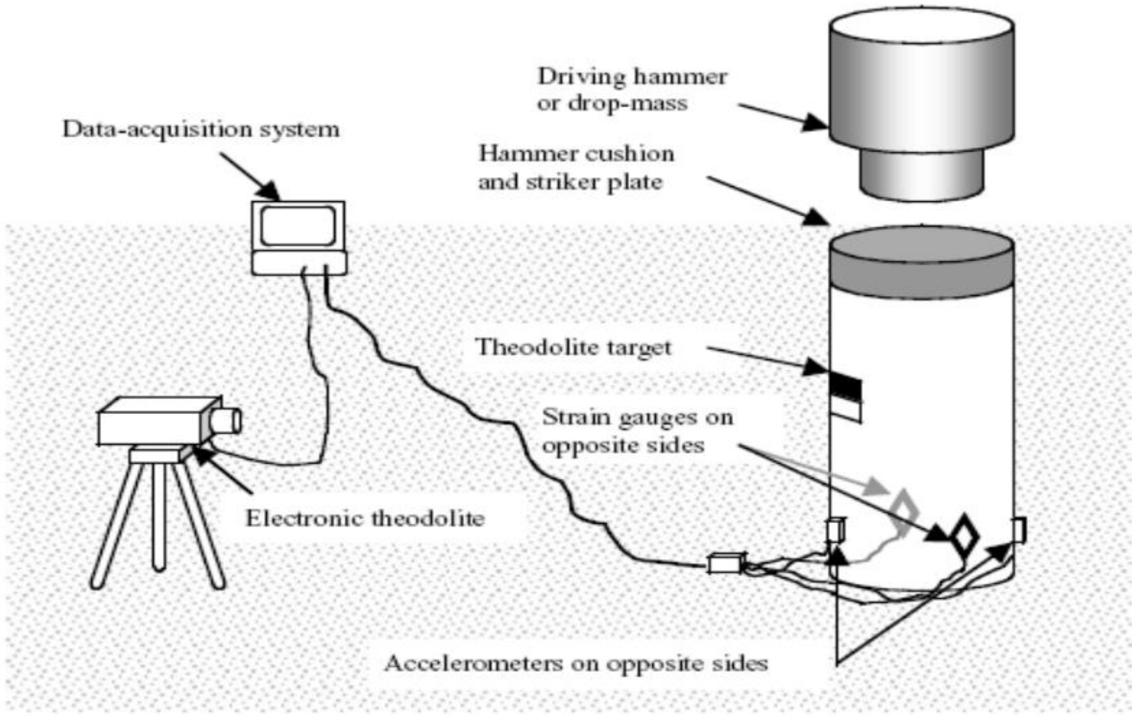


Figure 40.1: Schematic of SIMBAT Instrumentation

Figure 40.2: Complete SIMBAT kit including data collection unit, accelerometers, digital theodolite cable reels and waterproof carry case

Figure 40. 3: SIMBAT Dynamic Pile Load Test

Figure 40.4 : Static Pile Load Test

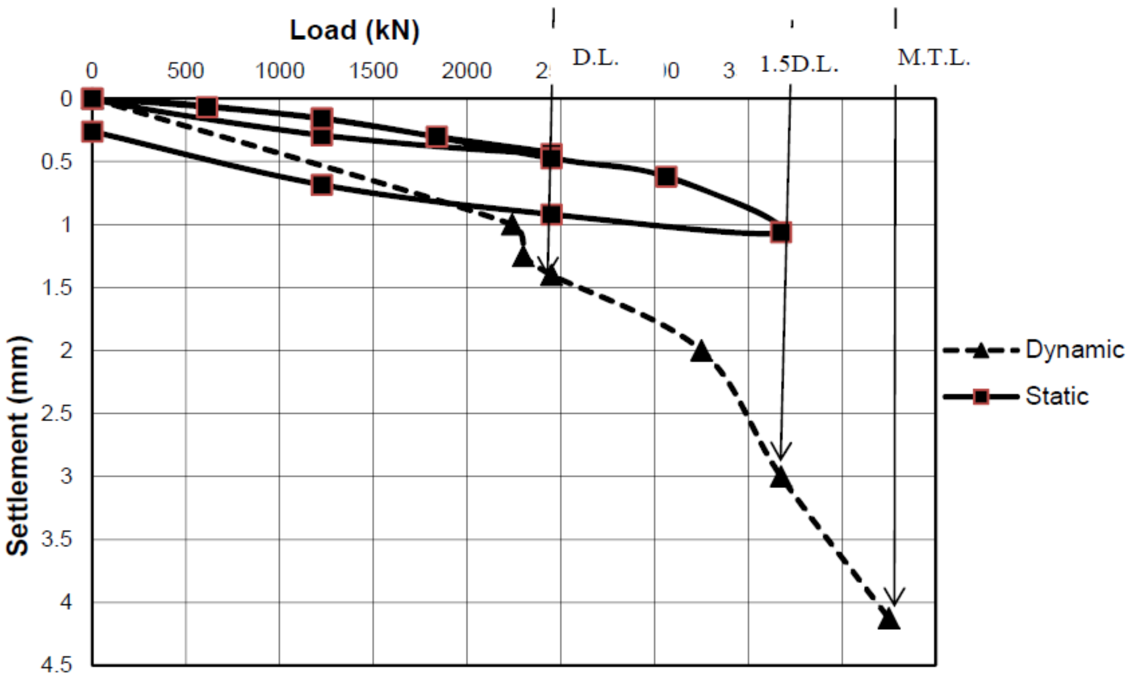


Figure 40.5: Load-Settlement Curves

*Under revision

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