FOUNDATION ENGINEERING
Unit - I
PART-B

III yr.
Resmi

1. **Describe the various methods of drilling bore holes for sub surface investigations.** **(April/May 2011) (Nov/Dec 2014) (May/June 2016)**

- When the depth of exploration is large, borings are used for exploration.
- A vertical bore hole is drilled in the ground to get the information about the subsoil strata samples are taken from the bore hole and tested in the laboratory.
- The bore hole may be used for conducting in-situ tests and for locating the water table.

Depending upon the type of soil and the purpose of boring, the following methods are used

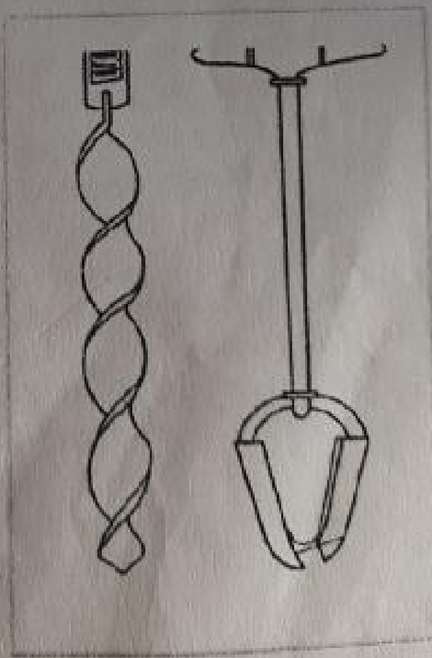
1. Auger Boring **2.Auger and shell boring**

3. Wash Boring **4. Rotary Drilling**

5. Percussion drilling **6. Core Boring**

1. Auger Boring:

1. Augers are used in cohesive and other soft soils above water table.
2. Hands augers are used for depth up to 6m.
3. Mechanically operated augers are used for greater depths a d they can also be used in gravelly soils.
4. Samples recovered from the soil brought up by au ers are badly disturbed nature of soil sample; it becomes difficult to locate the exact changes in the soil strata.
5. It can be operated manually or mechanically. Mechanical augers are driven by power. These are used for making holes in hard strata to a great depth. Even mechanical augers become inconv ni nt for depth greater than 12m and other methods of boring are used.
6. The hand augers used in borin are about 15to 20cm in diameter. It is attached to the lower end of the pipe of about 18mm diameter.
7. When the auger is filled with soil, it is taken out. If the hole is al eady driven, another type of auger known as post hole auger is used for taking soil samples.



Limitation or Disadvantages:

- **Sandy soil below water table, a casing is normally required. For such soils, the method of auger boring becomes slow and expensive.**
- **It cannot be used when there are large cobbles, boulders or other obstructions hich prevent drilling of the hole.**
- **Auger boring is fairly satisfactory for**

highways, railways, airfield exploration at shallow depth. The sub-surface explorations are done quite rapidly and economically by auger boring.

2. Augers and shell Boring:

1. Cylindrical augers and shell with cutting edge on teeth at the lower end can be used for making deep borings.
2. Hand operated rings are used for depth up to 2.m and the mechanical ring up to 50m.
3. This Augers are suitable for soft to stiff clays, shells for very stiff and hard clays and shells or sand pumps for sandy soils.
4. Small boulders, thin soft strata or rock or cemented gravel can be broken by chisel bits attached to drill rods. The hole usually requires a casing.

3. Wash Boring:

1. In wash boring, the hole is drilled by first driving a casing about 2 to 3m long and then inserted into a hollow drill rod with a chisel shaped chopping bit at its lower end. Water is pumped down the hollow drill rod, which is known as wash pipe.

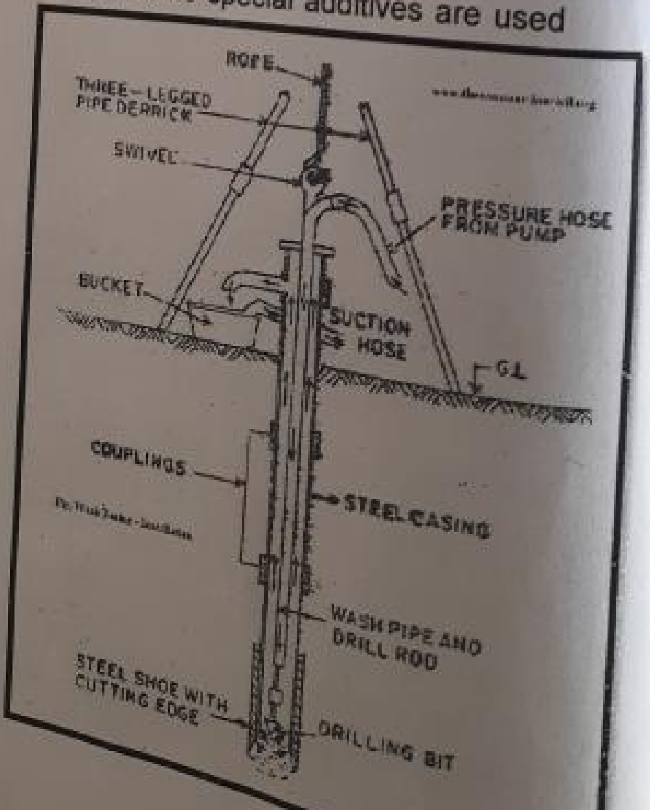
2. Water emerges as a strong jet through a small opening of the chopping bit. The hole is advanced by a combination of chopping action and the jetting action as the drilling bit and the accompanying water jet disint grates the soil.

3. The water and chopped soil particles rise upward through the annular space between the drill rod and the cas ng. The return water also known as wash water which is collected in a tub throu h a T-shaped pipe fixed at the top of the casing.

4. The hole is further advanced by alternately raising and dropping the chopping by a winch. The swivel joint provided at the top of the drill rod facilitates the turning and twisting of the rod. The process is continued even below the costing till the hole begins to cave in. At that stage the bottom of the casing can be extended by providing additional pieces t the top.

5. However in stabl , cohesive soils the casing is required only in the top portion. Sometimes instead of casing, special drilling fluids made of suspension or emulsion of fat clays or bentonite combined with some special additives are used for supporting walls of the

6. The change in strata is provided by the reaction of the chopping bit as the hole is advanced. It is also indicated by a change in color of the wash water. The wash boring is mainly used for advancing a hole in the ground. Once the hole has been drilled, a sampler is inserted to obtain soil samples for testing in the laboratory.



Limitation or Disadvantages:

The equipment used in wash boring is relatively light and inexpensive. The main disadvantage of the method is that it is slow in stiff soils and coarse grained soils. It cannot be used efficiently in hard soils, rocks and the soil containing boulders.

- The method is not suitable for takin ood quality undisturbed samples above ground water table, as the wash water enters the strata below the bottom of the hole and causes an increase i its water content.

4. Percussion Drilling:

1. The percussion drilling method is used for making holes in rocks, boulders and other hard str t .
2. The main advantag of the percussion drilling method is that it can be used for all types of materials. It is particularly useful for drilling holes in is glacial tills containing boulders.
3. In this method a heavy chisel is alternately lifted and dropped in a vertical hole. The material gets pulverized. If the point where chisel strikes is above the water table, water is added to the hole. The water forms slurry with the pulverized material which is removed by a sand pipe.
4. Percussion drilling may require a casing. It is also used for drilling tube wells.

Limitation or Disadvantages:

ξ One of the major disadvantages is that the material at the bottom of the hole is disturbed by heavy blows of the chisel.

x It is not possible to get good quality undisturbed samples. This method is generally more expensive.

5. Rotary Drilling:

1. Rotary boring or drilling is a very fast method of advancing hole in the both rocks and soils.
2. Rotary drilling can be used in clay, sand and rocks.
3. Bore holes of diameter 50mm to 200mm can be easily made by this method.
4. A drill bit, fixed to the lower end of the drill rods, is rotated by a suitable chunk and is always kept in firm contact with the bottom of the hole.
5. A drilling mud, usually a water solution of bentonite with or without other admixtures is continuously forced down the hollow drill rods.
6. The mud entering upwards brings the cuttings to the surface. This method is also known as 'MUD ROTARY DRILLING 'and the hole usually requires no casing.

6. Core Drilling:

1. The core drilling method is used for drilling holes a d for obtaining rock cores.
2. In this method a core barrel fitted with a drilling bit is fixed to a hollow drilling rod. As the drilling rod is rotated, the bit advanc s and cuts an annular hole an intact hole.
3. The core is then removed from its bottom and is retained by a core –lifter and brought to the ground surface.
4. The core drilling may be done us ng e ther a diamond studded bit or cutting edge consists of chilled shot. The diamond driller is superior to the other type of drilling, but it is costlier
5. Water is pumped continuously i to the drilling rod to keep the drilling bit cool and to carry the disintegrated materials to the ground surface.

2.Explain the (i) Seismic refraction method and (ii) Electrical resistivity method of soil exploration. (May/June 2009), (Nov/Dec 2015)

(OR)

Explain in detail the geophysical methods of soil explorations with neat sketch.

(Nov/Dec 2012),(Nov/Dec 2013), (May /June 2013)

(i) SESMIC REFRACTION

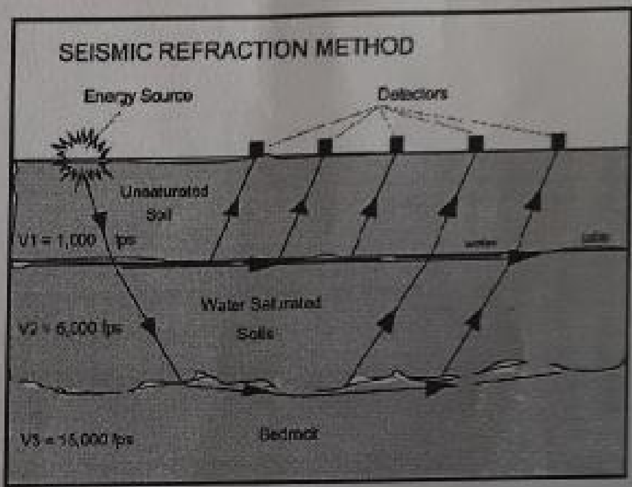
METHOD General

ξ This method is based on the fact that seismic waves have different velocities in different types of soils and besides the wave refract when they cross boundaries between different types of soils.

ξ In this method an artificial impulse are produced either by detonation of explosive or mechanical blow with a heavy hammer at ground surface or at the shadow depth within a hole.

Procedure

- The detectors are generally placed at varying distance from the shot point but along the straight line.
- The arrival time of the first impulse at each geophone is utilized.
- If the successfully deeper strata transmit the waves with increasingly greater velocities the path travelled by the first impulse will be similar to those.
- Those recorded by the nearest recorders pass ent rely through the overburden, whereas those first reaching the after detectors travel downward through the lower velocity material, horizontally within the higher velocity stratum and return to the surface.
- (A T_1 and A T_2) as the function of the distances between the geophones and the shot points (L_1 and L_2).
- A curve obtained which indicates the wav v locity in each stratum and which may be used to determine the depths to the boundaries between the strata.



$$H_1 \quad \frac{l_1 V_1}{2\cos\Delta} \quad \frac{L_1}{2}\sqrt{\frac{V_2\ V_1}{V_2\ V_1}}$$

$$H_2 \quad \frac{l_2 V_2}{2\cos E} \quad 0.85 H_1 \quad \frac{L_2\ L_1}{2}\sqrt{\frac{V_3\ V_2}{V_3\ V_2}}$$

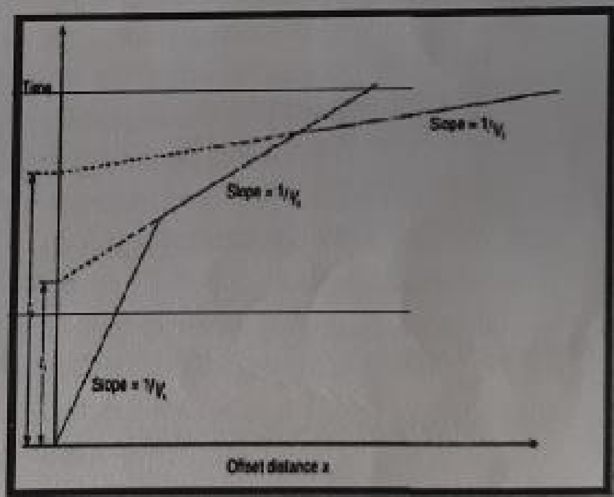
Where H_1 and H_2 are the depths of the strata

$l_1 = AB_1$

$l_2 = AC_1 - AB_1$

$\sin\alpha = (V_1 - V_2)$

$\sin\beta = (V_2/V_3)$



Applications

- **Depth and characterization of the bed rock surfaces.**
- **Buried channel location.**
- Depth of the water table.
- Depth and continuity of stratigraphy interfaces.
- Mapping of faults and other structural features.

Advantages

- Complete picture of stratification of layer up to 10 m depth.
- Refraction observations generally employ fewer source and receiver location and thus relatively cheap to acquire.

These shocks generate three types of waves:

- ξ Longitudinal or compressive wave or primary (p) wave
- ξ Transverse or shear waves or secondary (s) waves
- ξ Surface waves

It is primarily the velocity of longitudinal or the compression waves which is utilized in this method. The equation on the p-waves (V_c) and s-waves (V_s) is given as

$$V_c \quad \sqrt{\frac{E(1-\mu)}{(1+\mu)(1-2\mu)\rho}}$$

$$V_s \quad \sqrt{\frac{E}{2\rho(1+\mu)}}$$

ⱽ

These waves are classified as direct, reflected and efracted waves.

- ξ The direct waves travel in approximately straight line from the source of impulse.
- x The reflected and refracted wave undergoes a change in direction when they encounter a boundary separating med a of different seismic velocities.
- x This method is more suited to the shallow explorations for civil engineering purpose.
- ξ The time required for the impulse to travel from the shot point to various points on the ground surface is determined by means of geophones which transform the vibratio s i to electrical currents and transmit them to a recording unit or oscillog aph, with a timing mechanism.

Assumptions

The various assumptions involved are

- ξ All the soil layers are horizontal
- ξ The layers are sufficiently thick to produce a response
- ξ Each layer is homogeneous and isotropic
- ξ Velocity should increase with depth following the Snell's law as given

i_1 is the angle of incidence
i_2 is the angle of refraction

v_1 and v_2 are velocity in two different mediums

r_1, r_2, r3 and r_4 are the distances betwe n th various electrodes

Potential difference between C and D = V_{CD}

$$V_c - V_D \quad \frac{I\rho}{2\pi}\left[\left(\frac{1}{r_r}-\frac{1}{r_2}\right)-\left(\frac{1}{r_3}-\frac{1}{r_4}\right)\right]$$

$$\rho \quad \frac{2\pi V_{CD}}{I}\left[\frac{1}{\left(\frac{1}{r_1}-\frac{1}{r_2}\right)-\left(\frac{1}{r_3}-\frac{1}{r_4}\right)}\right]$$

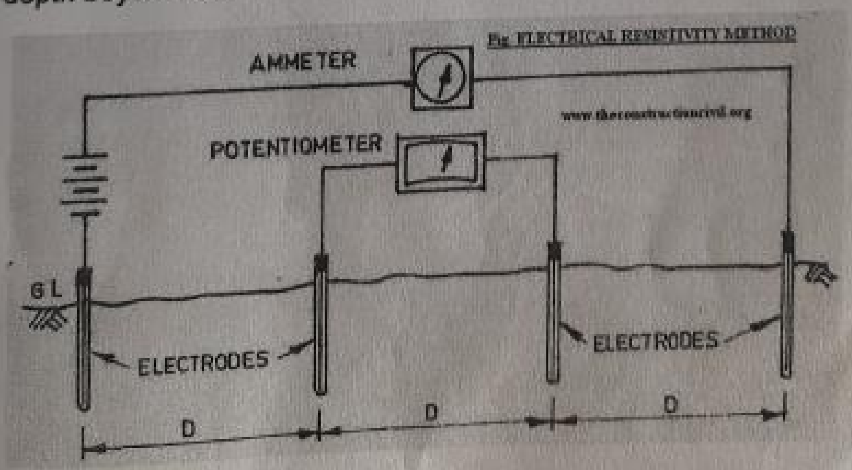
If $r_1=r_4=(r_2/2)=(r_3/2)$Then the resistivity is given as

$$Y \quad \frac{2\Sigma Rr}{I}$$

Where,

Resistances R= V_{CD}/I

- Thus the apparent resistivity of the soil to the depth approximately equal to the spacing r_1 of the electrode can be computed.
- The resistivity unit is often so designed that the apparent resistivity can be read directly on the potentiometer.
- In resistivity mapping or transverse profiling the electrodes are moved from place to place without changing their spacing and the apparent resistivity and any anomalies within a depth a depth equal to the spacing of the electrodes can thereby be determined for a number of points.
- In resistivity sounding or depth profiling the center point of the set up is stationary whereas the spacing of the electrode is varied.
- A detailed evaluation of the results of the resistivity sounding is rather complicated, but preliminary indications of the subsurface conditions may be obtained by plotting the apparent resistivity as a function of electrode spacing.
- When the electrode spacing reaches a value equal to the depth to a deposit with a resistivity materially different from that of overlying strata, the resultant diagram will generally show a more or less pronounced break in the strata depth beyond A_2 .



- Little processing is done on refraction observations with the exception of trace scaling or filtering to help in the process of picking the arrival times of the initial ground motion.
- Because such a small portion of the recorded ground motion is used developing models and interpretations is no more difficult than our previous efforts with other geophysical surveys.
- Provides seismic velocity information for estimat ng material properties.
- Provides greater vertical resolution than elect ical, magnetic or gravity methods.
- Data acquisition requires very limited intrusiv activity is non- destructive.

Disadvantages

- Blind zone effect: If $v_2 < v_1$, then wave refracts more towards normal then the thickness of the strata is neglected.
- Error also introduced due to some dissipation of the velocity as longer the path of travel, geophone receives the erroneous readings.
- Error lies in all assumptio s.

(ii) ELECTRICAL RESISTIVITY METHOD

Electrical resistivity method is based on the difference in the electrical conductivity or electrical resistivity of different soils. Resistivity is defined as the resistance in ohms between opposite phases of a unit cube of a material.

ρ is resistivity in ohm-cm

R is resistance in ohms

A is the cross sectional area (cm^2)

L is the length of the conduction (cm)

Procedure:

- In this method the electrodes and driven approximately 20 cms in to the ground and a dc or a very low frequency ac current of known magnitude is passed between the outer electrodes thereby producing within the soil an electrical field and the boundary conditions.
- The electrical potential at point C is V_c and at the point D is V_d which is measured by means of the inner electrodes respectively.

$$V_c \quad \frac{I\rho}{2\pi}\left(\frac{1}{r_1}-\frac{1}{r_2}\right)$$

$$V_D \quad \frac{I\rho}{2\pi}\left(\frac{1}{r_3}-\frac{1}{r_4}\right)$$

Where ρ is resistivity

I is current

ξ I practice many sever l different arrays are used. For simple sounding a Wenner array is us d. Then the resistivity is given as

$$\Upsilon \quad \frac{2\Sigma Ra}{I}$$

Where a is the spacing between the electrodes.

ξ The Schlumberger array is used for profiling and sounding. In sounding configuration the current electrodes separated by AB are symmetric about the potential electrodes MN.

ξ The current electrodes are then expanded and the resistivity is given as

$$\mathrm{U} \quad \underline{\Sigma s}^{2}\ \frac{a^{2}}{a} \underline{/4}R$$

Applications

ξ Characterize subsurface hydrogeology.
ξ Determine depth to bedrock /over burden thickness.
ξ Determine depth to ground water.
ξ Map stratigraphy.
ξ Map clay aquitards.
ξ Map salt water intrusion.
ξ Map vertical extent of certain types of soil and ground water contamination.

Resistivity profiling

ξ Map faults.
ξ Map lateral extent of conductive contaminant process.
ξ Locate voids.
ξ Map heavy metals soil contamination.
ξ Delineate disposal areas.
ξ Map paleochannels.
ξ Explore for sand and gravels.
ξ Map archaeological sites.

Advantages of this method are

x It is very rapid and economical method.
x It is good up to 30 m depth.
x The instrumentation of this method is very simple.
x It is a non destructive method.

Disadvantages of this method are

ξ It can only detect absolutely different strata like rock and water.
ξ It provides no information about the sample.
ξ Cultural problems cause interference.

ξ Data acquisition can be slow compared to other ... methods, although that difference is disappearing with the very latest techniques.

3. Briefly explain with neat sketch Standard Penetration Test and the correction to be applied to find 'N' value. (May/June 2016), (May/June 2014), (Nov/Dec 2011), (May /June 2012), (Nov/Dec 2013)

Standard Penetration Test.:

1. The standard Penetration Test is the most commonly used in –site test, especially for cohesion less soils which cannot be easily sampled.

2. The test is extremely useful for determining the relative density and the angle to determine the UCC strength of the cohesive soil.

3. The standard penetration test is conducted in a bore hole using a standard split spoon sampler, when the bore hole has been drilled to the desired depth, the drilling tools are removed and the sampler is lowered to the bottom of the hole.

4. The sampler is driven into the soil by a drop hammer of 63.5kg mass falling through a height of 750mm at the rate of 30blows per minutes.

5. The number of hammer blows required to drive 150mm of the sample is counted.

6. The sampler is further driven by 150mm and the number of blows recorded.

7. Likewise the sampler is once again further drive by 150mm and the number of blows recorded. The number of blows recorded for the first 150mm is disregarded.

8. The plumber of blows recorded for the last two 150mm intervals are added to give the standard Penetration Number (N).

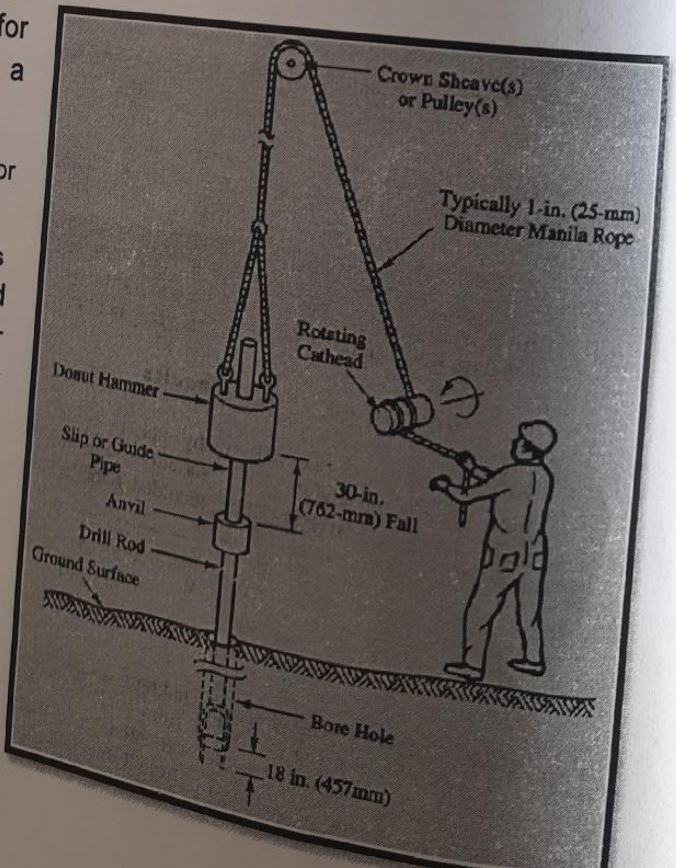
9. In other words, 'N' is equal to the number of blows required for 300mm of penetration beyond a seating drive of 150mm.

10. If the number of blows for 150mm drive exceeds 50, it is taken as refusal and the test is discontinued. The st ndard Penetration number is corrected for decadency correction and our burden correction.

(a) Dilatancy Correction.

ξ Silty fine sands and fine sands below the water table develop pore pressure which is not easily dissipated.

ξ The pore pressure increases the resistance of the soil and hence the Penetration number(**N**).



- Terzaghi and peck recommend the following correction when the observed N value exceeds 15. The corrected Penetration Number,

$$N_c = 15 + ½ [N_R – 15]$$

Where, N_c – corrected value

N_R – Recorded Value

If $N_R \leq 15$, then $N_c = N_R$

(b) Over burden Pressure Correction:

- In granular soils, the overburden pressure affects the penetration resistance.
- Generally, the soil with high confining pressure gives higher penetration number.
- As the confining pressure in cohesion soil increases w th depth, the penetration number for the soils at shallow depths is under estimated and that at greater depths is over estimated for uniformity, the N values obtained from field tests under different effective overburden pressure are corrected to a standard effective overburden pressure.

For dry or moist clean sand, (Gibbs and Holtz)

$$N_c = \frac{N_R \times 350}{\bar{\sigma}+70}$$

N_c - corrected value

N_R - Recorded Value

$\bar{\sigma}$ - effective over burden pressure

It is applicable for $\bar{\sigma} \leq 280 kN/m^2$. Usually the overburden correction is applied first and then dilatancy correction is applied

first and then dilatancy correction is applied.

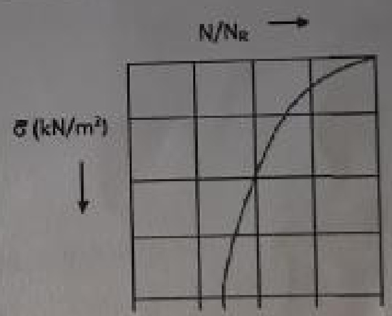


Fig Overburden pressure correction diagram

The correction given by Bazara & peck is

$N = 4N_R$ if $\bar{\sigma} <$ $\frac{71.8 kN/m^2}{1+ 0.0418\bar{\sigma}}$

$N = 4N_R$ if $\bar{\sigma} >$ $\frac{71.8 kN/m^2}{3.25+ 0.0104\bar{\sigma}}$

$N = N_R$ if $\bar{\sigma} = 71.8 kN/m^2$

Correction of N with engineering properties:

- The value of standard Penetration number N depends upon the relative density of the cohesionless soil and the unconfined compressive strength of the cohesive soil.
- If the soil is compact or stiff, the penetration number is high.
- The angle of sheaing resistance (ϕ) of the cohesionless soil depends upon the number N.
- In general, greater the N-value greater the ϕ value.
- The consistency &UCC strength of cohesive soils can be approximately determined from SPT, N-value.

Correction between N, Dr, ϕ

N	Condition	Relative de sity, Dr	Angle of internal friction, ϕ
0-4	Very Loose	0-15%	<28°
4-10	Loose	15-35%	28°-30°
10-30	Medium	35-65%	30°-36°
30-50	Dense	65-85%	36°-42°
>50	Very Dense	>85%	42° & greater

For Clays the following data are give .

Correlation between N and q_u

N	Consistency	q_u(kN/m2)
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

4.Explain in detail the various types of samplers with sketches. (May/June 2016) (May/June 2013) (May/June 2011)

OPEN DRIVER SAMPLER:

- Most commonly used for disturbed samples.

- If the soil encountered in the bore hole is fine sand and it lies below the water table, the sample is recovery becomes difficult.
- For such soil, a spring core catcher device is used to aid recovery.
- As the sampler is lifted springs close and form a dome and retain the sample.
- While taking samples, care should be taken to ensure that the water level in the hole is maintained slightly higher than the piezometric level at the bottom of the hole.
- It is necessary to prevent quick sand conditions.
- The split tube may be provided with a thin metal or plast c tube liner to protect the sample and to hold it together.
- After the sample has been collected, the liner a d the sample it contains are removed from the tube and ends are sealed.

STATIONARY PISTON SAMPLER:

- Stationary piston sampler consists of a sampler with a piston attached to a long piston rod extending up to the ground surface through the drill rods.
- The lower end of the sampler is kept closed with piston while the sampler is lowered through the bore hole.
- When the desired elevation reached, the piston rod is clamped; thereby keeping the piston statio ary and the sampler tube is advanced further into the soil.
- The sampler is then lifted and the piston rod clamped in position.
- The piston prevents the entry of water and soil into the tube, when it is being lowered and also helps to retain the sample during the process of lifting the tube.
- The sampler is therefore very much being suited for sampling in soft soils and saturated sands.

ROTARY SAMPLERS:

- Rotary samplers are core barrel type with an outer tube provided with cutting teeth and a removable thin liner inside. It is used for sampling in stiff cohesive soils.

Driving shoe made up of tool steel about 75mm long. Steel tube of 450mm long.

- The coupling head provided with check valve and 4 venting port of 10mm diameter.
- After borehole sampler attached to drilling rod and lowered into the hole.
- Drop hammer is used for forcing the sampler.
- When the sample is taken out removing the shoe and coupling transported to lab.
- Spring core catches is used for taking sand below ground water level and spring closes when lifted up and forms a dome.
- Water level slightly above the piezometric level at the bottom of the hole to avoid quick sand condition.

SPLIT SPOON SAMPLER:

The most commonly used sampler for obtai i g disturbed sample of soil is the standard split spoon sampler. It consists ma nly of three parts

i) Driving shoe, made of tool steel, about 75mm long

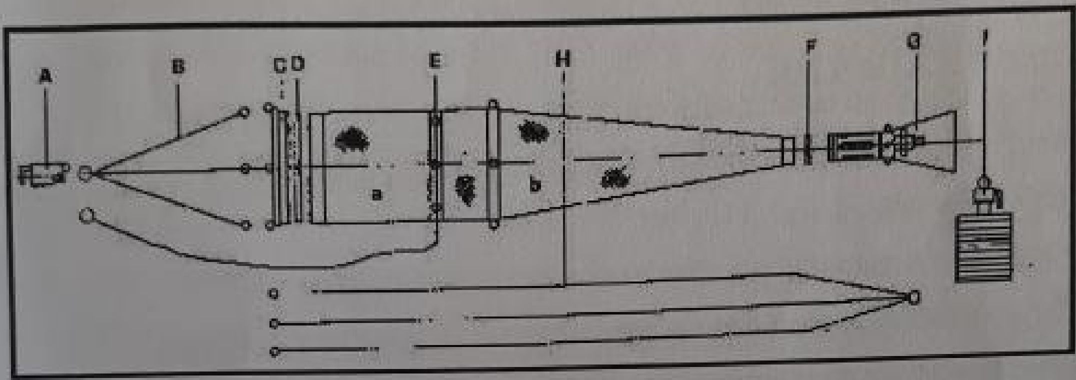
ii) Steel tube abut 450mm long, split longitudinal in two halves and

iii) Coupling at the top of the tube about 150mm long.

- The inside diamete of the split tube is 38mm and the outside diameter is 50mm.
- The coupling h ad may be provided with a check valve and 4 venting ports of 10mm diameter to improve sample recovery.
- This sampler is also used in conducting standard penetration test.
- After the bore hole has been made, the sampler is attached to the drilling rod and lo ered into the hole.
- The sample is collected by jacking or forcing the sampler into the soil by repeated blows of a drop hammer.
- The sampler is then withdrawn.
- The split tube is separated after removing the shoe and the coupling and the sampler is taken out. It is then paced in a container, sealed and transported to the laboratory.

SCRAPER BUCKET SAMPLER:

- If a sandy deposit contains pebbles it is not possible to obtain samples by standard split spoon sampler by standard split spoon sampler or split spoon fitted with a spring core catcher.
- The pebbles come in between the springs and prevent their closure.
- For such deposits, a scraper bucket sampler can be used.
- A scraper bucket sampler can also be used for obtaining the samples of cohesion less soils below the water table.



SHELBY TUBES AND THIN WALLED SAMPLERS:

- Shelby tubes are thin wall tube samplers made of seamless steel.
- The outside diameter of the tube may be between 40 to 125mm.
- The area ratio is less tha 15% and the inside clearance is between 0.5 to 3%.
- The length of th tube is 5 to 10 times the diameter for sandy soils and 10 to 15 times the diameter for clayey soils.
- The diameter generally varies between 40 and 125mm and thickness varies from 1.25 to 3.15mm.
- The sampler tube is attached to the drilling rod and lowered to the bottom of the bore hole.
- It is then pushed into the soil.
- Care should be taken to push the tube into the soil by a continuous rapid motion without impact or twisting.
- The tube should be pushed to the length provided for the sample.
- At least 5 minutes after pushing the tube into its final position, the tube is turned revolutions to shear the sample off at the bottom before it is withdrawn.
- The tube is taken out and its ends are sealed before transportation. Shelby tubes are used for obtaining undisturbed samples of clay.

DENISON SAMPLER:

- The Denison sampler is a double walled sampler.
- The outer barrel rotates and cuts into the soil.
- The sample is obtained in the inner barrel.
- The inner barrel is provided with a liner. It may also be provided with a basket type core retainer.
- The Denison sampler is mainly used for obtaining samples of stiff to hard cohesive soils and slightly cohesive sands.
- However, it cannot be used for gravelly soils, loose cohesion less sands and silts below ground water table and very soft cohesive soils.

HAND-CURVED SAMPLES:

- Hand curved samples can be obtained if the soil is exposed, as in a test pit, shaft or tunnel.
- Hand curved samples a e also known as chunk samples.
- The soil should h ve at least a trace of cohesion so that it can stand unsupported for som times.
- To obtain a sample, a column of soil is isolated in the pit.
- The soil is carefully removed from around the soil column and it is properly trimmed.
- An open ended box is then placed over the soil column.
- The space between the box and the soil column is fitted with paraffin.
- A spade or a plate with sharp edges is inserted below the box and the sample is cut at its base.
- The box filled with the soil sample is removed.
- It is turned over and the soil surface in the box is trimmed and any depression is filled with paraffin.
- A chunk sample may be obtained without using the box if the soil is cohesive.
- A column of soil is isolated.
- The block of soil is carefully removed from the soil column with sharp knife.
- The chunk sample is then coated with paraffin wax to prevent loss of moisture.
- Samples from open pits can also be obtained by pressing a sampling tube provided with a cutting edge.
- The soil surrounding the outside of the tube is carefully removed while the tube is being pushed into the soil. Hand – curved samples are undisturbed.

5.Explain in detail the cone penetration test with sketches. (Nov/Dec 2014) (Nov/Dec 2013) (May/June 2011)

Cone Penetration Test

- The cone test was developed by the dutch government, soil mechanics laboratory at Defit and is therefore also known as Dutch cone test.
- The test is conducted either by the Static method or by dynamic method.

Static Cone Penetration Test.:

- The Dutch cone has a apex angle of 60 and an overall diagram of 35.7mm giving an overall diag am of 35.7mm giving an end area of $10cm^2$.
- For obtaining the cone resistance, the cone is pushed downward at a steady rate of 10mm/s c through a depth of 35mm each time.
- The cone is pushed by applying thrust and not by driving.
- After the cone resistance has been determined the cone is withdrawn.
- The sleeve is pushed onto the cone both are driven together into the soil and the combined resistance is also determined.
- The resistance of the sleeve alone is obtained by subtracting the cone resistance from the combined resistance.
- A modification of the dutch cone pentrometers is the refined dutch cone.
- It has got a friction sleeve of limited length above the cone point.
- It is used for obtaining the point resistance of the cone and the frictional resistance of the soil above cone point.
- For effective use of the cone penetration test, some reliable calibration is required.
- This consists of comparing the results with those dutch cone obtained from conventional tests conducted on undisturbed sample in a laboratory.
- It is also convenient to compare the cone test results with SPT results, are related to the SPT number N, indirect correlations are obtained between the cone tests and the engineering properties of the soil.

The following relation holds approximately good between the poi t resistance of the cone (9c) and the standard penetration Number (N)

i)	Gravels 9c	=	800N to 1000N
ii)	Sands 9c	=	500N to 600N
iii)	Silly sands 9c	=	300N to 400N
iv)	Sills & clayey 9c	=	200N wh r 9c is in KN/m2 silts

b. **Dynamic Cone Penetration Test.**

- x The test is conducted by driving the cone by blows of a hammer.
- x The number of blows for driving the cone through a specified distance is a measure of dynamic cone resista ce.
- x It is performed either by using a 50mm cone without bentonite slurry or by using a 65mm cone with bentonite slurry (IS 4968 – part I &II 1976) The driving energy is given by 65kg hammer falling through a height of 75cm.
- ξ The number of blows required for 30cm of penetration is taken as the dynamic cone resistance (N_{cbr}).
- ξ If the skin friction is to be eliminated, the test is conducted in a cased bore hole.
- ξ When a 65mm cone with bentonite slurry is used, the set up should have arrangement for circulating slurry so that the friction on the driving rod is eliminated.
- ξ The dynamic cone resistance (N_{cbr}) is correlated with the SPT number N.
- ξ The following approximate relations may be used when a 50mm diameter cone is used.

N_{cbr}	Depth
1.5N	<3m
1.75N	3-6m
2.0N	>6m

- ξ The central building research Institute, Roorkee has developed the following correlation between the dynamic cone resistance (N_{bcr}) of SPT Number N.
- ξ It is applicable for medium to fine sand.

Ncbr	Depth
1.5N	<4m
1.75N	4-9m
2.0N	>9m

6.The field N value in a deposit of fully subm rg d fine was 40at a depth of 6m. The average saturated unit weight of the soil is 19 kN/m^3. Calculate the corrected N value as per IS: 2131- 1981.

(April/May 2015)

Solution:

x Since the soil is subme ged fine sand, dilatancy correction is also to be applied in addition to the correction for overburden pressure.

$$\gamma' = \gamma_{sat} - \gamma_w$$

$$= 19 - 9.8$$

$$= 9.2 \text{ kN/m}^3$$

Effective overburden pressure $\bar{p}$ at 6 m depth = 9.2 X 6

$$= 55.2 \text{ kN/m}^2$$

(a) Correction for overburden pressure:

According to IS : 2131- 1981

$$C_N = 0.77 \log 2000/\bar{p}$$

$$= 0.77 \log 2000/55.2$$

$$= 1.2$$

$$N' = C_N N$$

$$= 1.2 \times 40$$

$$= 48$$

(b) Correction for dilatancy effect:

$$N'' = 15 + 0.5 (N' - 15)$$

$$= 15 + 0.5 (48 - 15)$$

$$= 31.5$$

UNIT II

SHALLOW FOUNDATIONS

2 marks

1. **What is ultimate bearing capacity? (April/May 2004), (May/June 2013)**
   The ultimate bearing capacity is defined as the minimum gross pressure intensity at the base of the foundation at which the soil fails in shear.

2. **What is consolidation settlement? (April/May 2004)**
   The consolidation settlement is the long term settlement taking place over a long period of time due to the gradual expulsion of water without replacing it by air from the soil pores.

3. **List the various components of settlement. (Nov/Dec 2005), (April/May 2010)**
   The settlement of foundation base is due to (a) Elastic / Immediate settlement (b) Consolidation Settlement (c) Secondary Consolidation Settlement

4. **Give the Terzaghi‟s bearing capacity equation of strip footing for local shear failure. (Nov/Dec 2005)**
   $Q_f = \frac{2}{3} C N'_c + \gamma D_f N'_q + 0.5 \gamma B N'_\gamma$

5. **Compare general and local shear failure. (May/June 2009)**

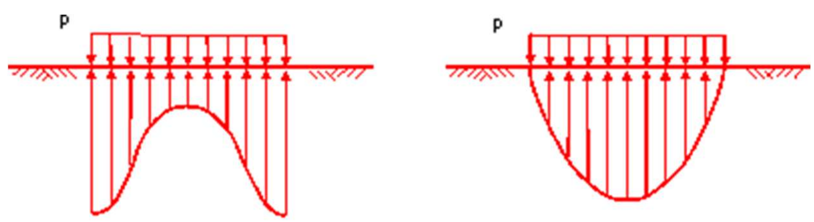
S.NO	General Shear Failure	Local Shear Failure
1	Well defined failure pattern	Well defined wedge and slip surfaces only beneath the foundation
2	A sudden – catastrophic failure accompanies by tilting of foundation.	There is no tilting of foundation. Slip surface not visible beyond the edges of the foundation.
3	Bulging of ground surface adjacent to the foundation.	Slight bulging of ground surface adjacent to the foundation.
4	The load – Settlement curve indicates the ultimate load clearly.	The load – Settlement curve does not indicate the ultimate load clearly.

6. **What is meant by allowable settlement? (May/June 2009)**
   Allowable Settlement:
   - It is the maximum settlement beyond which the foundation fails due to excessive settlement
   - Permits a maximum allowable settlement of 40 mm for isolated foundation on sand and 65 mm for those on clay.
   - For raft foundations on sand 40 mm to 65 mm and that on clay 65 mm to 100 mm.

7. **List the factors affecting bearing capacity of soil. (April/May 2010)**
   - Nature of the soil and its Physical and engineering properties.
   - Nature of the foundation.
   - Total and differential settlements that the structure can withstand without functional failure.
   - Location of ground water table.
   - Initial Stresses, if any.

8. **What is spread footing? (Nov/Dec 2009)**
   Spread footing is a foundation which transmits the load to the ground through one or more stepped footings. One spread foot is PAD. Two or more spread step is stepped footing.

9. **Sketch the pressure distribution beneath a rigid footing on cohesive and cohesionless soil. (Nov/Dec 2009), (May/June 2012)**



10. **Define safe bearing capacity. (Nov/Dec 2010)**
    Safe bearing capacity is the maximum intensity of loading that the soil will safely carry with a factor of safety without risk of shear failure of soil irrespective of any settlement that may occur.

11. **What is the equation used to determine the immediate settlement? (Nov/Dec 2010)**

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

Where, I = influence factor which depends on the shape of footing and rigidity

q = intensity of contact pressure

B = least lateral dimension of footing

E and μ = Young"s modulus and Poisson"s ratio of the soil

12. **What are the criteria used for the determination of bearing capacity? (Nov/Dec 2010) The following criteria must always be used in evaluating the bearing**

capacity.

1. Adequate factor of safety against failure (collapse)
2. Adequate margin against excessive settlement. Although failure or collapses of foundation have been reported from time to time, by far the most common difficulty of foundations arises from excessive settlement. Therefore, this criterion warrants skillful and careful attention of the practicing engineers.

(Or)

Determination of bearing capacity:

1. Skin friction
2. Bearing load
3. Base area etc.

13. **A footing was designed based on ultimate bearing capacity arrived for the condition of water table at the ground surface. If there is a chance for raise in water level much above the ground level do you expect any change in the bearing capacity, why?** (Nov/Dec 2010)

The raise in water level above the ground level would not change the bearing capacity of the soil. Because the soil under submerged condition when the water reaches the ground surface. Therefore the water above the ground level does not affect the unit weight of soil.

14. **Discuss the methods for determining immediate settlement of foundation on clay. (April/May 2011)**

The immediate settlement is mainly due to the expulsion of air and to the elastic deformation and reorientation of the soil particle on loading. Based on the theory of elasticity principles, immediate settlement is computed from the following equations.

$$Si = \frac{I\, q\, B\, (1-\mu^2)}{E}$$

Where, I = influence factor which depends on the shape of footing and rigidity

q = intensity of contact pressure

B = least lateral dimension of footing

E and μ = Young"s modulus and Poisson"s ratio of the soil

15. **A footing 2m square is laid at a depth of 1.3 m below the ground surface. Determine the net ultimate bearing capacity using BIS formula. Take γ =20 kN/m^3, ϕ =30° and c= 0. For ϕ =30°, take N_c =30.1, N_q =18.4 and N_γ = 22.4. (April/May 2011)**

$$q_{nu} = q(N_q-1)\, S_q\, d_q + 0.5\, \gamma\, B\, N_\gamma\, S_\gamma\, d_\gamma$$

$$q = 1.3 \times 20 = 26 \text{ kN/m}^2$$

$S_q = 1.2$ for square footing

$S_\gamma = 0.8$ for square footing

$d_q = d_\gamma = 1 + 0.1\ D_f \tan (45^\circ + \phi)$

$= 1 + 0.1 \times 1.3/2 \times \tan(45^\circ + 30^\circ/2)$

$= 0.112 + 1$

$= 1.112$

$q_{nu} = 26\ (18.4 - 1)1.2 \times 1.112 + 0.5 \times 20 \times 2 \times 0.8 \times 1.112$

$= 603.68 + 17.792$

$= 621.472\ kN/m^2$

16. Define punching shear failure. (Nov/Dec 2012)

Punching Shear failure occurs when there is relatively high compression of soil under the footing, accompanied by shearing in the vertical direction around the edges of the footing. Punching shear may occur in relatively loose sand with relative density.

17. What is mean by swelling potential? (Nov/Dec 2012)

The swelling potential of expansive soils is defined as the percentage swell of a laterally confined soil sample, when tested in a consolidometer test, when soaked under a surcharge load of 7 kN/m^2 after being compacted to maximum dry density at O.M.C (Optimum moisture content) according to AASHO compaction test.

18. What is net pressure intensity? (May/June 2013)

The difference in intensity of gross pressure after the construction of a structure and the original overburden pressure is called Net pressure

$$\text{Net pressure intensity} = \frac{\text{Net load on the base of the foundation}}{\text{Area of footing}}$$

19. What is safe bearing pressure? (May/June 2013)

In conventional design, the allowable bearing capacity should be taken as the smaller of the following two values.

- The safe bearing capacity based on ultimate capacity
- The allowable bearing pressure on tolerable settlement.

20. What is the total settlement of a footing? (May/June 2013)

Total settlement is defined as the settlement due to elastic settlement, consolidation settlement and secondary settlement.

$$S = S_i + S_c + S_s$$

21. **What are the major criteria to be satisfied in the design of a foundation? (Nov/Dec 2013)**

    Foundation design is based on providing a means of transmitting the loads from a structure to the underlying soil without

    - A soil shear failure, shear failure means that, it is a plastic flow and/ or a lateral expulsion of soil from beneath the foundation.
    - Causing excessive settlements of the soil under the imposed loads.

22. **What is the effect of rise of water table on the bearing capacity and the settlement of a footing on sand? (Nov/Dec 2013)**
    The pressure of water affects the unit weight of soil. Hence bearing capacity is affected due to the effect of water table. For practical purpose it is more sensitive when the water table rises above depth 13 m from footing.

23. **Define 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.

24. **Define settlement and its types.**
    The term settlement indicates the sinking of the structure due to compression and deformation of the underlying soil.
    Types:
    1. Uniform settlement 2. Non uniform or differential settlement

<u>13 MARKS</u>

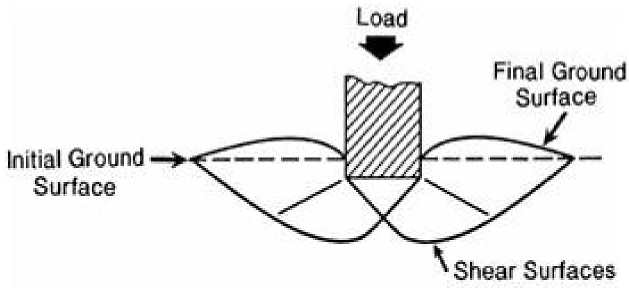
1. **Explain in detail the various types of shear failure.**

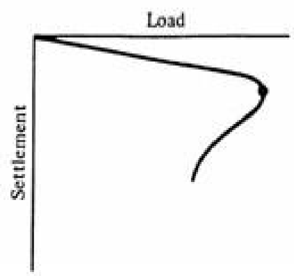
Types of shear failure:

- General shear failure
- Local shear failure
- Punching shear failure

Characteristics of General Shear Failure:

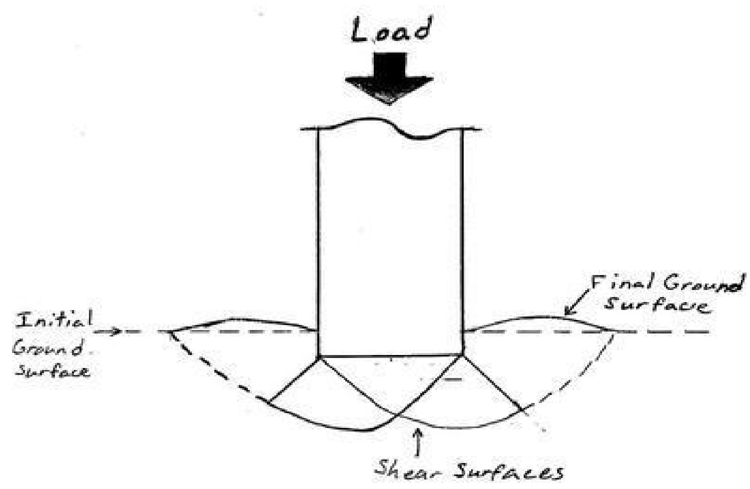
- It has well defined failure surface reaching up to ground surface.
- There is considerable bulging of sheared mass or soil adjacent to the footing.
- Failure is accompanied by tilting of foundation.
- Failure is sudden with pronounced peek resistance.
- The ultimate bearing capacity is well defined.





Characteristics of Local Shear Failure:

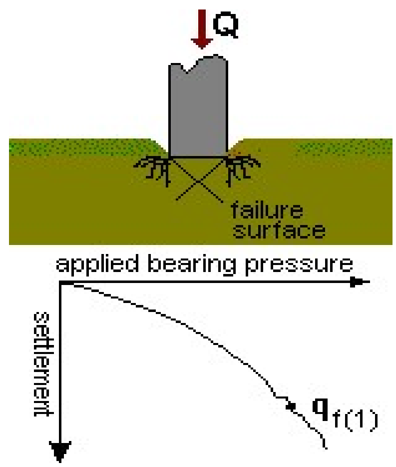
- Failure pattern is clearly defined.
- Failure surface do not reach ground surface.
- There is only slight building of soil around the footing.
- Failure is not sudden and there is no tilting of footing.
- Failure is defined by large settlement.
- Ultimate bearing capacity is not well defined.

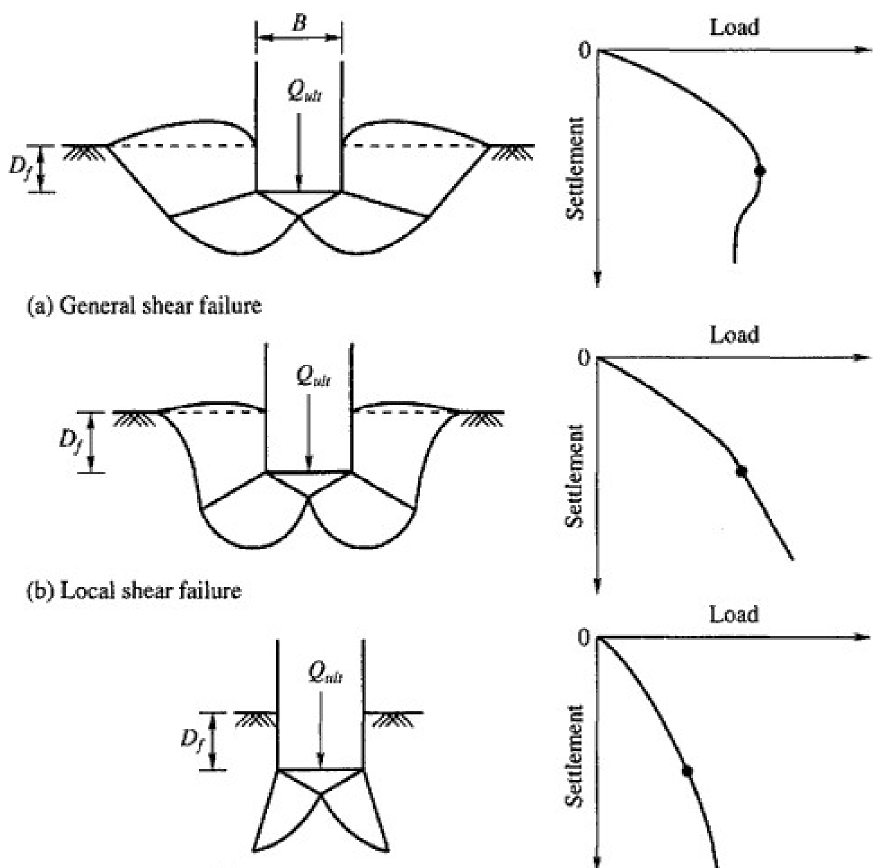


Local Shear Failure

Characteristics of Punching Shear Failure:

- No failure pattern is observed.
- The failure surface which is vertical or slightly inclined follows the perimeter of base.
- There is not building of soil amount the footing.
- There is no tilting of foundation.
- Failure is characteristics in terms of very large settlement.





(a) General shear failure

(b) Local shear failure

(c) Punching shear failure

2. **List the various factors that affect the depth of foundation.**

Depth of Footing

To perform its function properly a footing must be laid at a suitable bellow the ground surface. The vertical distance between ground surface and the base of the footing is known as the depth of the footing(Df).The depth of the footing contains the ultimate bearing capacity and the settlement. While fixing the depth of footing, the following points should be considered.

1. Depth of top soil:
   The footing should be located bellow the top soil consisting the organic materials which eventually decompose. The top soil should be removed over an area slightly larger than the footing.

2. Frost depth:
   The footing should be carried bellow the depth of frost penetration. If the footing is located at insufficient depth, it would be subjected to the frost damage due to formation of ice lenses and consequent frost heave. During summer, thawing occurs from the top downwards and the melted water is entrapped

3. Zone of soil volume change:
   Some clay, especially clays having high plasticity, such as black cotton soil, undergoes excessive volume changes. Such soil shrinks upon drying and swells upon wetting. The volume changes are generally greater near the ground surface and decreases with increase in depth. Large volume change beneath a footing may cause lifting and dropping. The footing should be placed bellow as strata that are subjected to large volume change.

4. Adjacent footing and property lines:
   - The footing should be so located that no damage is done to the existing structure. The adjacent structure may be damaged by construction of a new footing due to vibrations, undermining or lowering of the water table. The new footing may also impose additional load on the existing footing which may cause settlement.
   - In general deeper the new footing and closer to the existing structure the greater is the potential damage to the existing structure. This is this is particularly more severe if the new footing is lower than the existing footing.
   - As far as possible, the new footing should be placed at a small depth as the old ones and the sites of excavation adjacent to the existing structure should be suitably supported. If the footings are placed at the different levels, the slope of the line joining the two footings should not be steeper than two horizontal to one vertical as per IS: 1904-1978.

5. Sloping ground:
   If a footing is located adjacent to a sloping ground, the sloping ground surface should not encroach upon a frustum of bearing material under the footing having sides making an angle of 30" with the horizontal. Moreover, the minimum distance from lower edge of the footing to be sloping ground surface should be 90cm.

6. Water table :
   The footing should be placed above ground water table as far as possible. The presence of ground water within the soil immediately around a footing in undesirable as it reduces the bearing capacity of the soil and there are difficulties during construction. The water proofing problem also arises due to dampness.

7. Scour depth:
   The footings located in streams, on water fronts or other locations where there is a possibility of scouring should be placed below the potential scour depth.

8. Underground defects:
   The depth of footing is also affected by the presence of underground defects such as faults, causes and mines. If there are manmade discontinuities, such as sewer lines, water mains, underground cables, these should be shifted or footing should be relocated.

9. Root holes:
   If there are root holes or cavities caused by burrowing animals or worms, the footing should be placed bellow such a zone of weakened soil.

10. Minimum depth:
    IS 1904 – 1978 specifies that all foundations should extend to a depth of a least 50cm below the natural ground surface. However in case of rocks, only its top soil should be removed and the surface should be cleaned and if necessary stepped.

    The minimum depth of foundation (D_f) according to Rankine"s formula

$$(D_f)_{min} = \frac{q(1-\sin\phi")^2}{\gamma(1+\sin\phi")}$$

3. **Explain Terzaghi"s bearing capacity theory.** (16)

Terzaghi"s bearing Capacity Theory

Terzaghi (1943) was the first to propose a comprehensive theory for evaluating the safe bearing capacity of shallow foundation with rough base.

Assumptions

1. Soil is homogeneous and Isotropic.

2. The shear strength of soil is represented by Mohr Coulombs Criteria.

3. The footing is of strip footing type with rough base. It is essentially a two dimensional plane strain problem.

4. Elastic zone has straight boundaries inclined at an angle equal to to the horizontal.

5. Failure zone is not extended above, beyond the base of the footing. Shear resistance of soil above the base of footing is neglected.

6. Method of superposition is valid.

7. Passive pressure force has three components (PP_C produced by cohesion, PP_q produced by surcharge and PP_γ produced by weight of shear zone).

8. Effect of water table is neglected.

9. Footing carries concentric and vertical loads.

10. Footing and ground are horizontal.

11. Limit equilibrium is reached simultaneously at all points. Complete shear failure is mobilized at all points at the same time.

12. The properties of foundation soil do not change during the shear failure

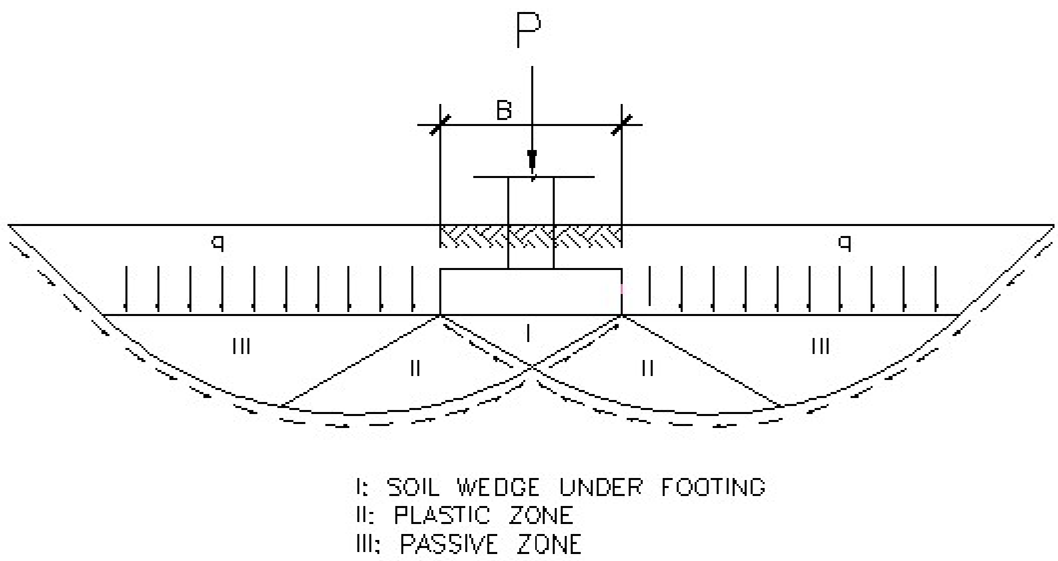
Limitations

1. The theory is applicable to shallow foundations

2. As the soil compresses, increases which is not considered. Hence fully plastic zone may not develop at the assumed.

3. All points need not experience limit equilibrium condition at different loads.

4. Method of superstition is not acceptable in plastic conditions as the ground is near failure zone.



Terzaghi‟s concept of Footing with five distinct failure zones in foundation soil

Concept

A strip footing of width B gradually compresses the foundation soil underneath due to the vertical load from superstructure. Let qf be the final load at which the foundation soil experiences failure due to the mobilization of plastic equilibrium. The foundation soil fails along the composite failure surface and the region is divided in to five zones, Zone 1 which is elastic, two numbers of Zone 2 which are the zones of radial shear and two zones of Zone 3 which are the zones of linear shear. Considering horizontal force equilibrium and incorporating empirical relation, the equation for ultimate bearing capacity is obtained as follows.

$$\mathbf{q_{ult} = (P_p)_\gamma + (P_p)_c + (P_p)_q}$$

$(P_p)_\gamma$ = Component produced by cohesive stress.

$(P_p)_c$ = Component produced by surcharge $q = \gamma D_F$

$(P_p)_q$ = Component produced by the weight of soil in zone II, III.

$$\mathbf{q_{ult} = C\, N_c + q\, N_q + 0.50\, \gamma\, B\, N_\gamma}$$

N_c, N_q, N_γ = Bearing Capacity factor which are dimensionless depend on angle of shear resistance υ.

$$N_q = \left[\frac{a^2}{2\cos^2\left(45+\frac{\varphi}{2}\right)}\right]$$

$$a = e^{\left(\frac{3\pi}{4}-\frac{\varphi}{2}\right)\tan\varphi}$$

$$N_c = (N_q - 1)\cot\varphi$$

$$N_\gamma = \frac{1}{2}\left[\frac{K_\gamma}{\cos^2\varphi} - 1\right]\tan\varphi$$

Ultimate bearing capacity,

γ q f = cNc + γDNq + 0.5γBNγ

If the ground is subjected to additional surcharge load q, then

γ q f = cNc + (γD + q)Nq + 0.5γBNγ

Net ultimate bearing capacity, qn = cNc + γD(Nq – 1) + 0.5γBNγ − γD

γ qn = cNc + γD(Nq −1) + 0.5γBN

Safe bearing capacity, qs = cNc + γD (Nq – 1) + 0.5γBNγ1/F + γD

Here, F = Factor of safety (usually 3)

c = cohesion

γ = unit weight of soil

D = Depth of foundation

q = Surcharge at the ground level

B = Width of foundation

Nc, Nq, Nγ = Bearing Capacity factors

Nc = cot υ (Nq –1)

$$N_q = e^{2(3\pi/4-\upsilon/2)\tan\upsilon} / [2 \cos^2(45+\upsilon/2)]$$

$$N_\gamma = (1/2) \tan\upsilon \left(K_{pr} / \cos^2\upsilon - 1 \right)$$

Kp=passive pressure coefficient.

$$K_p = \frac{1+\sin\varphi}{1-\sin\varphi} = \text{coefficient of passive earth pressure.}$$

Strip footings: $Q_u = c\,N_c + \gamma\,D\,N_q + 0.5\,\gamma\,B\,N_\gamma$

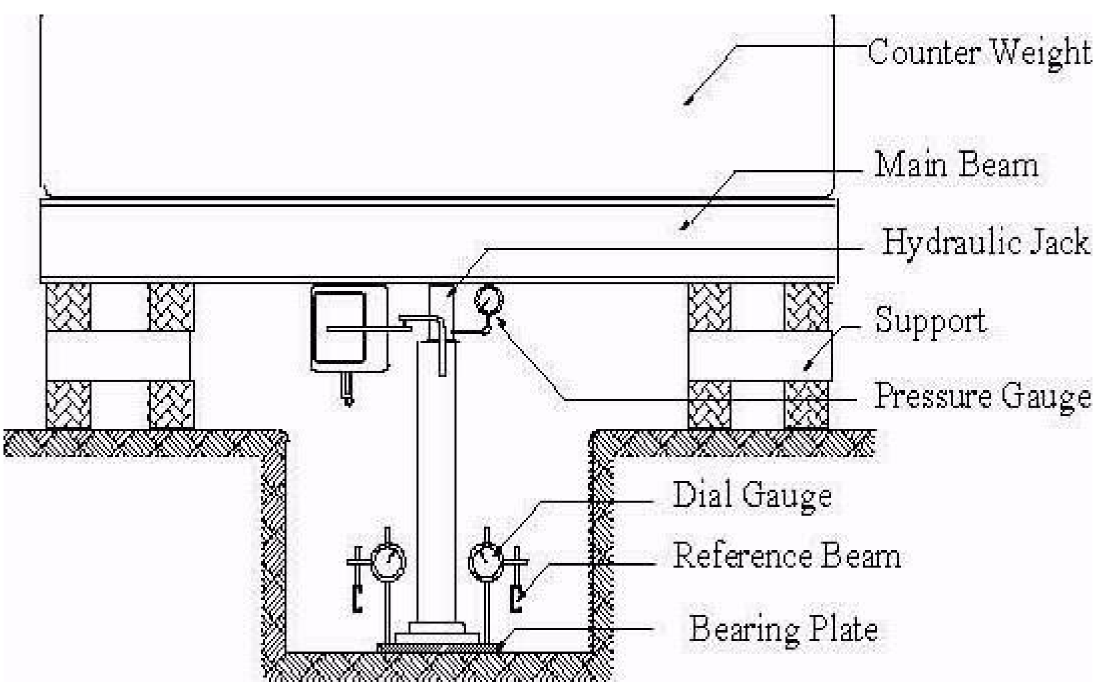
Square footings: $Q_u = 1.3\,c\,N_c + \gamma\,D\,N_q + 0.4\,\gamma\,B\,N_\gamma$

Circular footings: $Q_u = 1.3\,c\,N_c + \gamma\,D\,N_q + 0.3\,\gamma\,B\,N_\gamma$

4. Explain plate load test with sketch.

PLATE LOAD TEST

The allowable bearing pressure can be determined by conducting a plate load test at the site. The conduct a plate load test, a pit of the size 5Bp X 5Bp,where Bp is the size of the plate, is excavated to the depth equal to the depth of foundation(D_f).The size of the plate is usually 0.3m square. It is made of steel and is 25mm thick. Occasionally circular plates are also used. Sometimes large size plates of 0.6m square are used.



A central hole of size Bp X Bp is excavated in the pit the depth of the central hole(D_p)is obtained from the following relation

$$\frac{D_p}{B_p} = \frac{D_f}{B_f}$$

$$D_p=(D_f/B_f)B_p$$

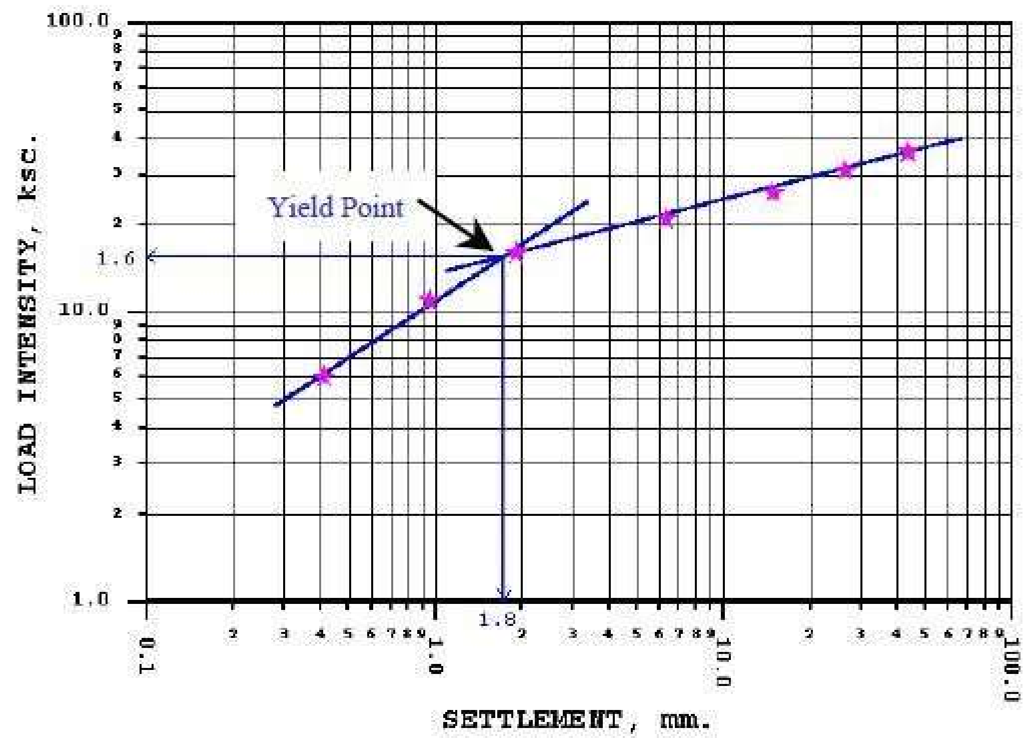
$$=(B_p/B_f)D_f$$

Where,

B_f-width of the pit B_p-size of plate

The conducting the plate load test, the plate is placed in the central hole and the load is applied by means of a hydraulic jack. the reaction to the jack is provided by means of a reaction beam. Sometimes truss are used instead of a reaction beam to take up the reaction. Alternatively, a loaded platform can be used to provide reaction.

A seating load of KN/m^2 is first applied, which is released after the sometimes. The is then applied in increments of about 20% of the estimated safe load or 1/10th of the ultimate load. The settlement is recorded after 1,5,10,20,40,60 minutes and further after an internal of one hour. These hourly observations are continued for clayey soils, until the rate of settlement is less than 0.2mm per hour. The test is conducted until failure or at least until the settlement of 25mm has occurred



The ultimate load for the plate is indicated by a break on the log-log between the load intensity q and the settlements. If the break is not will defined the ultimate load is taken as the corresponding to the settlement of 1/5th of the plate width (Bp) on the natural plot.the ultimate load is obtained from the intersection of the tangents drawn.

Advantages:

1. The ultimate bearing capacity of the proposed foundation $q_u(f)$ can be obtained from the following relations

a) For clayey soils $q_u(f)=(q_u)_p$ ———————————— 1

b) For sandy soils $q_u(f)=(q_u)_p \times B_f/B_p$ ———————————2

where,

B_f –foundation width

2. The plate load test can also be used to determine the settlement for a given intensity of loading (q_0).The relations between the settlement of the plate (s_p)and that of the foundation (s_f) for the same load intensity

a) For clayey soils, $s_f = s_p(B_f/B_p)$ ———————— 3

where s_p is obtained from the load intensity settlement curve for q_0

b) For sandy soils

$$s_f = s_p\left[\frac{B_f(B_p+0.3)}{B_p(B_f+0.3)}\right]^2 \qquad 4$$

Where B_f – width of foundation in meters

B_p – width of the plate in meters

3. For designing a shallow foundation for an allowable settlement of s_f,a trial and error procedure is adopted.First of all a value of B_f is assumed and value of q_0 is obtained as

$q_0 = Q/A_f$ ———————— 5

where A_f - area of footing

Q – Load

For the computed value of q_0 the plate settlement(s_p) is determined from the load – settlement curve obtained from the plate load test the values of s_f is computed equation 3if the soil is clay and using 4 if sand.The computed with the allowabl settlement.The procedure is repeated till the computed value is equal to the allowable settlement

4.The plate load test is can be also be used for the determination of the influence factor,I

$S = \frac{l-u^2}{E_s}$ x I x qB

The below graph shows a plot between settlements and the load qB,The slope of the line is equal to $\frac{1-u^2 I}{E_s}$

LIMITATIONS OF PLATE LOAD TEST:

1. SIZE EFFECT:

The results of the plate load test reflect the strength and the settlement characteristics of the soil within the pressure bubls.As the pressure bulb depands upon the size of the loaded area it is much deepers for the actual foundation as compared to that of plate.The plate load test doesnot truely represent the actual conditions to a large depth.

2. SCALE EFFECT:

The ultimate bearing capacity of saturated clays is independent of the size of the plate but for cohesionless soils.It increases with the size of the plate to rduce scale effect,it is desirable to repeat the plate load test with plates of two or three different sizes and the average of the bearing capacity values obtained.

3. TIME EFFECT:

A plate load test is essentially a test of short duration for clayey soils it does not give the ultimate settlement.the load settlement curve is not truely representative.

4. INTERPRETATION OF FAILURE:

The failure load is not well defined except in the case of a general shear failure an error of personal interpretation may be involved in other type of failures

5. REACTION LOAD:

It is not practicable to provide a reaction of more than 250KN.Hence the test on a plate of size larger than 0.6m width is difficult.

6. WATER TABLE:

The level of water table affects the bearing capacity of the sandy soils.If the water table is above the level of the footing it has to be lowered by pumping before placing at the water table level if it is within about 1m below the footing.

5. A footing 2m square rests on a soft cay soil with its base at depth of 1.5m from ground surface. The clay stratum is 3.5m thick and is underlain by a firm sand stratum. The void ratio of clay is 1.08 and compression index is 0.18, cohesion is 50 kN/m^2 . Compute the settlement that would result if the load intensity equal to the safe bearing pressure of soil were allowed to act on the footing . Natural water table is quite close to the ground surface. For given conditions, bearing capacity factor(N_c) is obtained as 6.9. Take factor of safety as 3. Assume load spread of 2(vertical) to (horizontal).

Solution :

c=0.5 kg/cm^2 =5 t/m^2

(a) determination of submerged unit weight γ''

$$\gamma''=\frac{G-1}{1+e}\ \gamma_w$$

Where e=ω_{sat} G= 0.4 × 2.7 = 1.08 and γ_w= 1 t/m^3

$$\gamma''=\frac{2.7-1}{1+1.08}\times 1 = 0.817\ t/m^3$$

(b) Determination of footing load

q_{nf}= c N_C = 5× 6.9 = 34.5 t/m^2

$$q_s=\frac{q_{nf}}{F}+\gamma' D=\frac{34.5}{3}+0.817\times 1.5$$

=12.726 KN/m^3

Q_s =q_s × area =12.726 × 2 × 2 = 50.9 t

(c) Determination of settlement: consider level A A at mid depth of clay.

$\varsigma_o''=\gamma'\ 2.5 = 0.817\times 2.5 = 2.043\ t/m^3$

Area if spread at level AA= 3× 3 = 9m^2.

$$\therefore \Delta\varsigma'=\frac{Q_s}{A}=\frac{50.9}{9}=5.656\ t/m^2$$

C_c=0.009(ω_L-10)= 0.009(30-10)=0.18

e_o = e = 1.08

$$\Delta H=\frac{C_c H}{1+e_o}\log_{10}\frac{\varsigma_o''+\Delta\varsigma'}{\varsigma_o''}$$

$$=\frac{0.18\times 2}{1+1.08}\log_{10}\frac{2.043+5.656}{2.043}=0.1\text{m}=10\text{cm}$$

6. **A square foundation of size 1.8m × 1.8m is to be built at a depth of 1.6m on a uniform clay strata having the following properties : $\emptyset = 0°, c = 30KN/m^3$ and γ = 182 $KN/$**

m^3. Find the safe load that the foundation can carry with a factor of safety of 3. Use Terzaghi's bearing capacity theory. If the ground water table subsequently rises from depth of 6m to the ground surface, find the load carrying capacity of the foundation. The submerged density of the soil is 10.5 KN/m^3.

Given data:

Square foundation size= 18m× 18m

Depth of foundation =1.6m

$\emptyset = 0$

C=30 KN/m^3

$\gamma = 182 KN/m^3$

Fos =3

γ_{sub}=10.5 KN/m^3.

To find:

i) case – i : water table at 6 m from G.L. safe load

ii) case –ii: water table at the ground surface safe load

Solution:

Case (1) : water table at 6m from ground surface.

Safe load $q_s = \frac{1}{F}[1.3CN_c + \gamma D\ Nq - 1\ RW_1 + 0.4\gamma BN_r RW_2] + \gamma D$

ZW_2=6-1.6=4.4m

RW_1=1

Since $ZW_2 > B, RW_2 = 1$

For $\emptyset = 0$ Terzaghi's Bearing capacity factor

N_C=5.7, N_q=1.0, N_r=0

$q_s = \frac{1}{3}[1.3 \times 30 \times 5.7 + 18.2 \times 1.6(1-1) + 0] + 18.2 \times 1.6$

$=(0.33 \times 222.3) + 29.12$

$=(74.1)+29.12$

$=103.22 KN/m^2$

Case (2) if water table at the ground surface

$q_s=\frac{1}{F}[1.3CN_c+\gamma_{sub} D(N_q-1)RW_1+0.4\gamma BN r\ RW_2]+\gamma D$

$RW_1=RW_2=0.5$

$q_s=\frac{1}{3}[1.3\times 30\times 5.7+\ 10.5\times 1.6\ 1-1\ \times 0.5\ +0]+10.5\times 6$

$=(0.33\times 222.3)+16.96$

$q_s=90.319\ KN/m^2$

7. **The result of two pate load tests for a settlement of 25.4 mm are given**

Plate diameter	load
0.3m	31 KN
0.6m	65KN

A square column foundation is to be designed to carry a load of 800KN with an allowable settlement of 25.4mm. Determine the size of the foundation using housel's method.

Given data:

$Q_1=31KN$, $d_1=0.3m$

$Q_2=65KN$, $d_2=0.6m$

Q=800KN, 25.4mm

To find:

Size of the foundation using housel"s method.

Solution:

$Q_1 =A_1m +P_1n$

$Q_2=A_2m+P_2n$

$Q = Am + Pn$

$A_1 = \frac{\pi}{4} d_1^2 = \frac{\pi}{4} \times 0.3^2 = 70.685 \times 10^{-3} m^2$

$A_2 = \frac{\pi}{4} d_2^2 = \frac{\pi}{4} \times 0.6^2 = 282.74 \times 10^{-3} m^2$

$P_1 = \pi \times d_1 = \pi \times 0.3 = 0.942m$

$P_2 = \pi \times d_2 = \pi \times 0.6 = 1.884m$

$31 = 70.685 \times 10^{-3} m + 0.942n$ --------------------------A

$65 = 282.74 \times 10^{-3} m + 1.884m$ ----------------------------B

Solving equation A and B

M=21.22, n=31.31

$Q = Am + Pn$

$800 = B^2 \times 21.22 + \ldots\ldots 31.31 \times 4B$

B=3.86m say $4m \times 4m$

Result: Size of the foundation : $4m \times 4m$

8. **A square footing for a column is 2.5m x 2.5m and carries a load of 2000kN. Find the factor of safety against bearing capacity failure, if the soil has the following properties.**

Given:

$C = 50 Kn/m^2$

$\emptyset = 15°$

$\gamma = 17.6 kN/m^3$

$N_c'' = 12.5$,

$N_q'' = 4.5$ and

$N_y'' = 2.5$. the foundation is taken to a depth of 1.5m.

$N_c'' = 12.5$ $N_q'' = 4.5$ $N_y'' = 2.5$

D=1.5m B=2.5m $\emptyset = 15°$

$\gamma = 17.6 kN/m^3$

C=0

$q_f = \gamma D\ N_q'' + 0.4\ \gamma B\ N_y''\ q_{nf} =$

$\gamma D\ (N_q'' - 1) + 0.4\ \gamma B\ N_y''$

$q_s = \frac{q_{nf}}{F} + \gamma D$ and F=3

Maximum safe load $= B^2 \times q_s$

9. **Compute the ultimate load that an eccentrically loaded square footing of width 2m width, an eccentricity of 0.315m can take at a depth of 0.45m in soil with $\gamma = 17.75 kN/m^3$, C=9kN/m² and $\emptyset = 35°$,N_c=52, N_q=35 and N_y=42.**

$q_f = \gamma D\ N_q'' + 0.4\ \gamma B\ N_y''\ q_{nf} =$

$\gamma D\ (N_q'' - 1) + 0.4\ \gamma B\ N_y''$

$q_s = \frac{q_{nf}}{F} + \gamma D$ and F=3

Maximum safe load $= B^2 \times q_s$

10. **The following data was obtained from a plate load test carried out on a 60cm square test plate at a depth of 2m below ground surface on a sandy soil which extends up to a large depth. Determine the settlement of a foundation 3.0m x 3.0m carrying a load of 1100KN and located at a depth of 3m below ground surface.**

Load intensity, KN/m2:	**50**	**100**	**150**	**200**	**250**	**300**	**350**	**400**
Settlement, mm :	**2.0**	**4.0**	**7.5**	**11.0**	**16.3**	**23.5**	**34.0**	**45.0**

Solution :

The load- settlement curve is shown in fig

Load intensity on the foundation $= \frac{1100}{3 \times 3} = 122$ kN /m²

From fig. Settlement of the test pate, S_p corresponding to a load intensity of is 122 kN/m² from equation.

$S_f = S_p\ [B_f\ (B_p + 30) / B_p\ (B_f + 30)]^2$

$= 5[300(60+30)/60(300+30)]^2$

$=9.3mm$

The effect of embedment must now be taken into account. Depth of embedment D is equal to or foundation measured from the level at which the test plate is placed.

Thus D=3.0-2.0 = 1m.

Using fox"s depth factor : for $D/\sqrt{LB} = 1/\sqrt{3 \times 3}$ =0.33 and L/B =1

Depth correction factor =0.91

Actual settlement of foundation = 0.91 x 9.3 =8.5 mm

11. **A foundation, 2.0m square is installed 1.2 Above the water table and a submerged density of $10kN/m^3$. The strength parameters with respect to effective stress c'=0 and $\varphi = 30^0$. Find the gross ultimate bearing capacity for the following conditions.**

1. Water table is well below the base of the foundation.

2. Water table raise to the level of the base of the foundation and

3. The water table rise to ground level. (For $\upsilon = 30^0$, Assume $N_q = 22$ and $N_r = 20$).

Solution:

Square footing (2m × 2m)

$C = 0, \upsilon = 30^0$

$N_q = 22, N_r = 20$

$\gamma = 19kN/m^3$

$\gamma_{sub} = 10kN/m^3$

i) Water table is well below the base of the foundation:

$q_f = 1.3\ cN_c + \gamma D\ N_q\ Rw_1 + 0.4\gamma\ B\ N_r\ Rw_2$

$Rw_1 = 1, Zw_2 > B,$

$Rw_2 = 1$

$q_f = 0 + 19 \times 1.2 \times 22 \times 1 + 0.4 \times 19 \times 2 \times 20 \times 1$

$q_f = 805.6\ kN/m^2$

Net ultimate bearing capacity (q_{nf})

$Q_{nf} = q_f - \gamma D = 805.6 - 19 \times 1.2 = 782.8 kN/m^2$

ii) water table at base of the foundation:

$Rw_1 = 0.5\ 1 + \frac{2w1}{D}$

$0.5(1+1) = 1$

$Rw_2 = 0.5\ 1 + \frac{2w2}{D}$

$0.5(1+0) = 0.5$

$q_f = 1.3\ cN_c + \gamma D\ N_q\ Rw_1 + 0.4\ \gamma\ B\ N_r\ R\ w_2$

$q_f = 0 + 19 \times 1.2 \times 22 \times 1 + 0.4 \times 10 \times 2 \times 20 \times 0.5$

$q_f = 501.6 + 80 = 581.6\ kN/m^2$

$q_{nf} = q_f - \gamma\ D = 581.6 - 19 \times 1.2 = 558.8\ kN/m^2$

ii) water table rises the ground level

$Rw_1 = 0.5$

$Rw_2 = 0.5$

$\gamma = \gamma_{sub}$

$q_f = 1.3\ cN_c + \gamma_{sub}\ D\ N_q\ Rw_1 + 0.4\ \gamma_{sub}\ B\ N_r\ R\ w_2$

□ □ □ □ □ □ □ □ × 1.2 × 22 × 0.5 + 0.4 × 10 × 2 × 20 × 0.5

$q_f = 132 + 80 = 212\ kN/m^2$

$q_{nf} = q_f - \gamma_{sub}\ D = 212 - 10 \times 1.2 = 200\ kN/m^2$

12. **A footing 2.m square carries a gross pressure of 350 kN/m² at a depth of 1.2m in sand. A saturated unit weight of sand is 20 kN/m² and the unit weight of sand above water table is 16 kN/m³. The shear strength parameters are C' =0 , ∅ = 30° (for ∅ = 30°, N_q=22, N_r=20). Determine the factor of safety with respect to shear failure for the following cases**

i) W.T is 5m below the ground level

ii) W.T is 1.2m below the ground level

solution:

We will follow IS code method

For square footing in soil having c=0

$q_f = \varsigma\ N_q + 0.4\ \gamma_b\ N_r\ W''$

case i): W.T at 5 m below G.L

ς=16x1.2

= 19.2 kN/m^2

D_W =5m

D + B =3+1.2 = 4.2m

Since $D_W >$ (D + B), W"=1

Also γ = 16 kN/m^2

$q_f = \varsigma\ N_q + 0.4\ \gamma_b\ N_r\ W''$

= 19.2 x 22 + 0.4 x 16 x 3 x 20 x 1

=806.4 KN/m^2

$q_{nf} = q_f - \gamma D$

=806.4 – 16 x 1.2

= 787.2 kN/m^2

Safe bearing capacity , $q_s = \frac{q_{nf}}{F} + \gamma D$

=787.2/F + 16 x 1.2

350=787.2/F +19.2

F=2.38

Case ii): water table at 1.2m below the G.L

$D_w = D \Rightarrow W'' = 0.5$

$\gamma = \gamma_{sat}$ =20 kN/m^3

ς = 16 × 1.2 = 19.2 kN/m^2

$q_f = \varsigma\ N_q + 0.4\ \gamma_b\ N_r\ W''$

$= 19.2 \text{ x}22+0.4 \text{ x}(20-9.81) \text{ x } 3\text{x } 20$

$q_f = 666.96 \text{ kN/m}^2$

$q_{nf} = q_f - \gamma D$

$=666.96 - 16\text{x}1.2$

$=647.76 \text{ kN/m}^2$

Safe bearing capacity $q_s = \frac{q_{nf}}{F} + \gamma D$

$350 = 67.76/F\ \ +19.2$

$F = 1.96$

13. **A circular footing is resting on a stiff saturated clay with unconfined compression strength of 250 kN/m^2. The depth of foundation is 2m. Determine the diameter of the footing if the column load is 700 KN.**

Assume a factor of safety as 2.5. the bulk unit weight of soil is 20KN /m^3.

For stiff saturated clay, $\emptyset = 0$

N_c=5.7, N_q =1 and N_r=0

$q_f = 250 \text{ KN/m}^2 \because c = 250/2 = 125 \text{ KN/m}^2$

$q_f = 1.3\ c\ N_r + \gamma D\ N_r + 0.4\ \gamma\ B\ N_r$

$=966 \text{ KN/m}^2$

$q_{nf} = q_f - \gamma D$

$= 966 - 20 \text{ x}2 = 926 \text{ KN/m}^2$

$q_s = \frac{q_{nf}}{F} + \gamma D = 926/2.5 + 40$

$=410.4 \text{ KN/m}^2$

$P = q_s \text{ x } A$

$= 700 - 410.4 \text{ x ml}^2/4$

$d=\sqrt{\frac{4\times 700}{\pi\times 410.4}}=1.47m$

what will be the change in ultimate , net ultimate and safe bearing capacity if the water table is at ground level ?

$q_{nf} = 1.3\ cN_c + \gamma'' D\ N_q$

$=1.3 \times 125 \times 5.7 + 10 \times 2 \times 1$

$=946.25\ KN/m^2$

$q_n = 946.25 - 20$

$= 926.25\ KN/m^2$

$q_s = 526.25/2.5 = 390.5\ KN/m^2$

UNIT III

FOOTINGS AND RAFTS

PART A

1.What are types of foundation? (Nov/Dec 2015)

- Shallow foundation
- Deep foundation

2.What are the footings comes under shallow foundation? (Nov/Dec 2015)

- Spread footing or pad footing,
- Strap footings,
- Combined footings,
- Raft or mat foundation

3.Under what circumstances, a strap footing is adopted? (May/June 2016)

When the distance between the two columns is so great, so that trapezoidal footing is very narrow and so it is uneconomical. It transfers the heavy load of one column to other column.

4.What is safe bearing pressure? (May/June 2013)

In conventional design, the allowable bearing capacity should be taken as the smaller of the following two values.

- The safe bearing capacity based on ultimate capacity
- The allowable bearing pressure on tolerable settlement.

5.What is a mat foundation? Where mat foundation is used? (April/May 2015) (Nov/Dec 2013) (Nov/Dec 2012) (May/June 2011)

It is a combined footing that covers the entire area beneath a structure and supports all the walls and columns.

It is used when the area of isolated footing is more than fifty percentage of whole area or the soil bearing capacity is very poor.

6.Define floating foundation? Give the advantages of floating foundation.

(May/June 2011)

It is defined as a foundation in which the weight of the building is approximately equal to the full weight of the soil including water excavated from the site of the building.

The structural load on a floating foundation is reduced,

$Q' = Q - W_s$, where Q – gross load and W_s – excavated soil weight.

7.Define spread footing? **(May/June 2014)**

It is a type of shallow foundation used to transmit the load of isolated column, or that of wall to sub soil. The base of footing is enlarged and spread to provide individual support for load.

8.List the different types of raft foundation. Under what circumstances, a raft footing is adopted? **(Nov/Dec 2013) (Nov/Dec 2011)**

- Flat plate
- Flat plate thickened under column
- Beam and slab construction
- Box structures
- Mat on piles

Raft foundation is used where settlement above highly compressible soils, by making the weight of the structure and raft approximately equal to the weight of the soil excavated. Flat type is commonly used since uniform thickened bottom slab is provided over the entire area.

9.What is mean by proportioning of footing? **(May/June 2012)**

Portioning of footing is defined as the arrangement of footing in the combined footing system, in which it is arranged in such a way that, the centroid of the area in contact with the soil lies on the line of action of the resultant of the loads.

10.What are the assumptions made in combined footing? (Nov/Dec 2010)

- The footing is rigid and rests on a homogenous soil to give rise to linear stress distribution on the bottom of the footing.
- The resultant of the soil pressure coincides with the resultant of the loads, and then it is assumed to be uniformly distributed.

11.What is the function of strap beam in a strap footing? (Nov/Dec 2010)

- The strap connects the two isolated footing such that they behave as one unit.
- The strap simply acts as a connecting beam.

PART-B

1.What is combined Footing? Elaborate the proportioning of rectangular combined footing. (Nov/Dec 2015) (May/June 2014) (May/June 2013) (Nov/Dec 2012)

- A combined footing supports two columns.
- When a foundation is built close to an existing building or the property line, there may not be sufficient space for equal projections on the sides of the exterior column.
- This results in a eccentric loading on the footing. It may lead to yilting of the foundation.
- To counteract the tilting tendency a combined footing is provided which joins the exterior column with interior column.
- A combined footing is also required when the two individual footings overlap.
- The footing is proportioned such that the centre of gravity of the footing lies on the line of action of the resultant of the column loads.
- The pressure distribution thus becomes uniform.
- A combined footing is generally rectangular in plan if sufficient space is available beyond each column, If one of the columns is near the property line, the rectangular footing can still be provided if the interior column is relatively heavier.

- However, if the interior column is lighter, a trapezoidal footing is required to keep the resultant of the column loads through the centroid of the footing.
- Thus the resultant of the soil reaction is made to coincide with the resultant of the column loads.

Rectangular Combined Footing:

- The design of a combined footing consists of selecting length and width of the footing such that the centroid of the footing and the resultant of the column loads coincide.
- With the dimensions of the footing established, the shear force and bending moment diagram are drawn.
- The thickness of the footing is selected from the bending moment and shear force considerations.
- The footing is designed as a continuous beam supported by two columns in the longitudinal direction.
- The reinforcement is provided as in a continuous beam.

The procedure consists of following steps:

1. Determine the total column loads.

   $Q = Q_1 + Q_2$

   Where Q_1 – exterior column load

   Q_2 – interior column load
2. Find the base area of the footings.

   $A = Q / q_{na}$

Where q_{na}– allowable soil pressure.

3. Locate the line of action of the resultant of the column loads measured from one of the column, say exterior column.

   $\bar{x} = Q_2 \text{ X } x_2/Q$

   Where x_2 - distance between columns.
4. Determine the total length of footing.

   $L = 2(\bar{x} + b_1/2)$

   Where b_1 – width of exterior column.

5. Find the width of the footing.

   B = A/L

6. As the actual width and length that are provided may be slightly more due to rounding off, the actual pressure is given by

   $q_0 = Q / A_0$

   Where A_0 – actual area

7. Draw the shear force and bending moment diagrams along the length of the footing, considering the pressure q_0. For convenience the column loads are taken as concentric column loads acting at the centres.

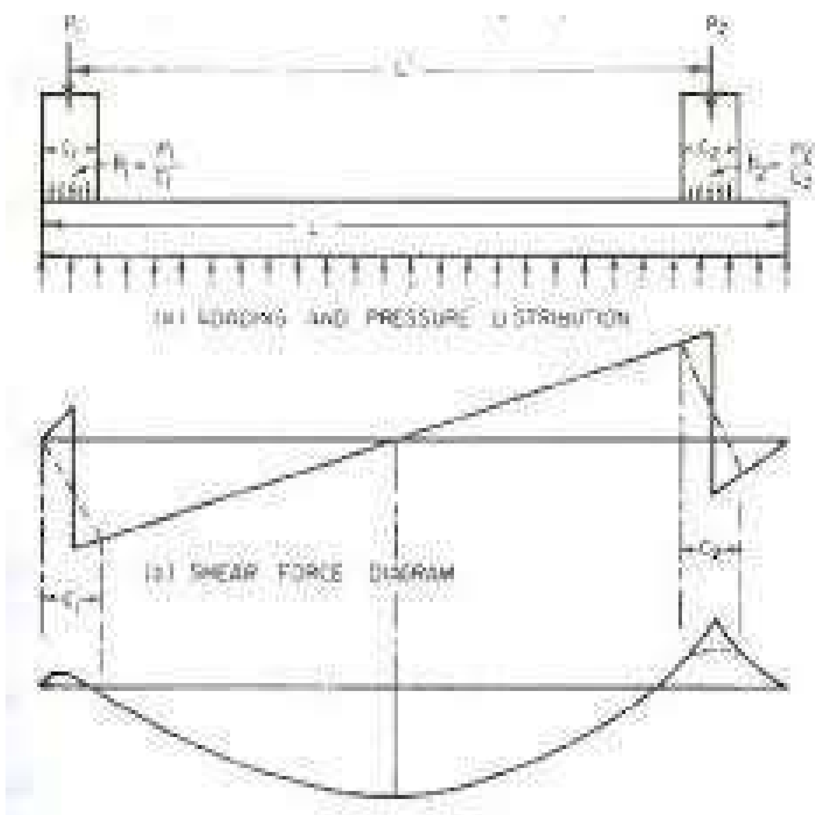
8. Determine the bending moment at the face of the columns and the maximum bending moment at the point of zero shear.

9. Find the thickness of the footing for the maximum bending moment.

10. Check the diagonal shear and punching shear as in the case of isolated footings. Check for bond at the point of contra flexure.

11. Determine the longitudinal reinforcement for the maximum bending moment.

    For transverse reinforcement, assume a width of (b + d) to take all the bending moment in the short direction, where b is the column side and d is the effective depth.



2.Elaborate the proportioning of Trapezoidal Combined Footing: (Nov/Dec 2015) (May/June 2014) (Nov/Dec 2013) (Nov/Dec 2012)

Trapezoidal combined footings are provided to avoid eccentricity of loading with respect to the base. Trapezoidal footings are required when the space outside the exterior column is limited and the exterior column carries the heavier load.

1. Determine the total column loads.

$$Q = Q_1 + Q_2$$

Where Q_1 – exterior column load

Q_2 – interior column load

2. Find the base area of the footings.

$$A = Q / q_{na}$$

Where q_{na}– allowable soil pressure.

3. Locate the line of action of the resultant of the column loads measured from one of the column, say exterior column.

$$\bar{x} = Q_2 \text{ X } x_2/Q$$

Where x_2 - distance between columns.

4. Determine the distance 'x' of the resultant from the outer face of the exterior column.

$$x' = \bar{x} + b_1 / 2$$

where b_1 – width of exterior column.

A trapezoidal footing is required if $L/3 < x' < L/2$

Where L – length of the trapezoidal footing determined from $L = 2(\bar{x} + b_1/2)$

If $x' = L/2$, a rectangular footing is provided. However if $x' < L/3$, a combined footing cannot be provided. In such a case, a strap footing is suitable.

5. Determine the width B1 and B2 from the following relations.

$$\frac{B1 + B2}{2} \; X\, L = A$$

$$\frac{(B1 + 2B2)L}{(B1 + B2)3} = x$$

Solving the above two equations, we get

$$B2 = \frac{2A}{L}\left(\frac{3x'}{L} - 1\right)$$

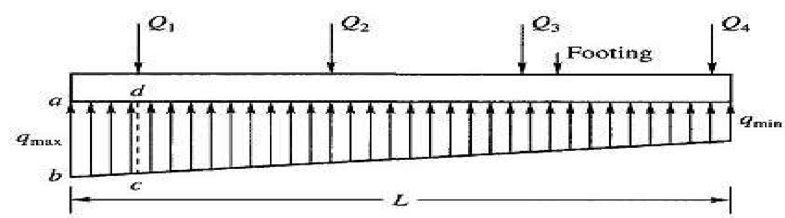
$$B1 = \frac{2A}{L} - B2$$

Once the dimension B1 and B2 has been found, the rest of the design can be done as in the case of rectangular combined footing.

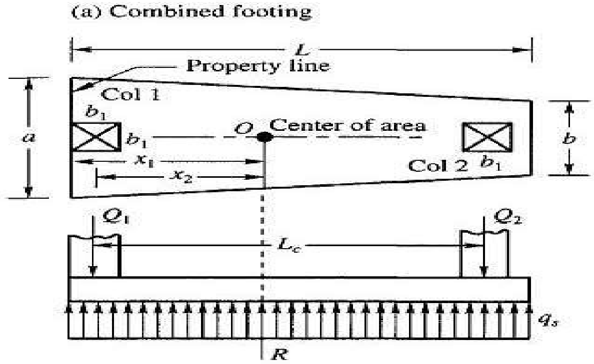
6. As the actual width and length that are provided may be slightly more due to rounding off, the actual pressure is given by

   $q_0 = Q / A_0$

   Where A_0 – actual area
7. Draw the shear force and bending moment diagrams along the length of the footing, considering the pressure q_0. For convenience the column loads are taken as concentric column loads acting at the centers.
8. Determine the bending moment at the face of the columns and the maximum bending moment at the point of zero shear.
9. Find the thickness of the footing for the maximum bending moment.
10. Check the diagonal shear and punching shear as in the case of isolated footings. Check for bond at the point of contra flexure.
11. Determine the longitudinal reinforcement for the maximum bending moment.



(a) Combined footing



(b) Trapezoidal combined footing

3.Define mat foundation. What are the various types of raft foundations? (Nov/Dec 2015), (May/June 2016) (Nov/Dec 2012)

MAT FOUNDATION:

- A raft or mat is a combined footing that covers the entire area beneath the structure and supports all the walls and columns; when the allowable soil pressure is low, or the building loads are heavy, the use of spread footings would cover more than one-half of the area and it may prove more economical to use mat or raft foundation.
- They are also used where the soil mass contains compressible less or the soil is sufficiently erratic so that the differential settlement would be difficult to control.
- The mat or raft trends to bridge over the erratic deposits and eliminates the differential settlement.
- Raft foundation is also used to reduce settlement above highly compressive soils, by making the weight of structure and raft approximately equal to the weight of the soil excavated.

TYPES OF RAFT FOUNDATION:

(a) FLAT TAPE TYPE:

- In this type of mat foundation a mat of uniform thickness is provided.
- This type is most suitable when the column loads are relatively light and the spacing of columns is relatively small and uniform.

(b) FLAT PLATE THICKENED UNDER COLUMN:

- When the column loads are heavy this column is thickened to provide enough thickness for negative bending moment and diagonal shear.
- Sometimes instead of thickening a slab, a pedestal is provided under each column above the slab to increase the thickness.

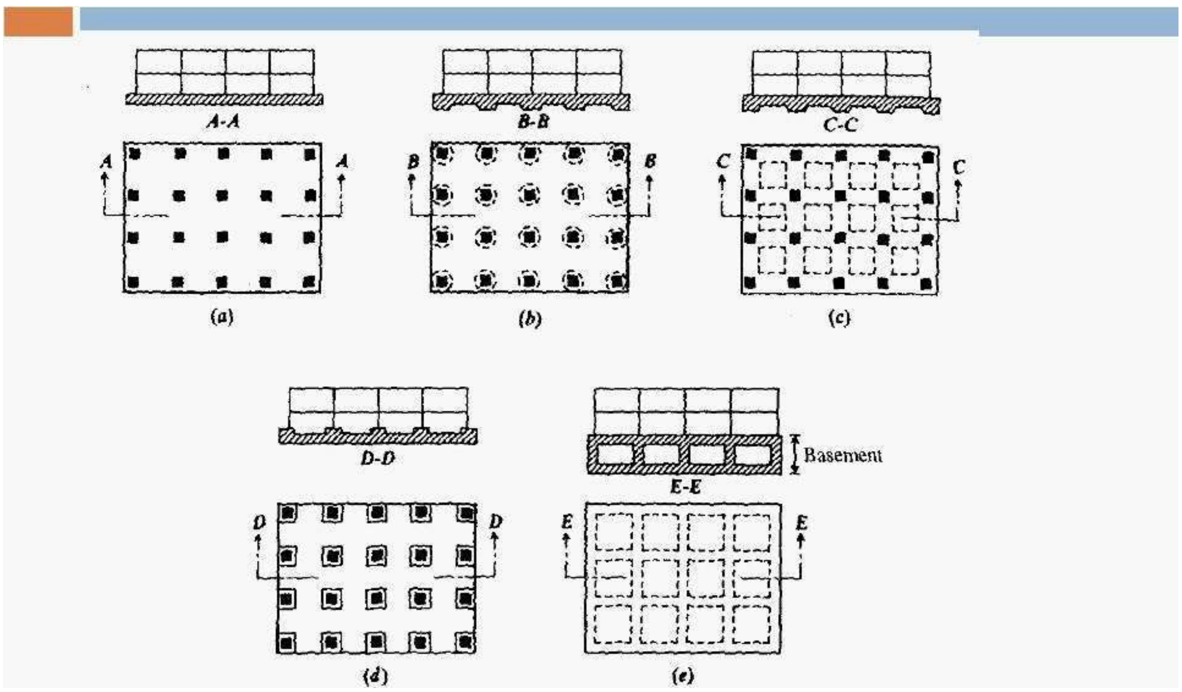
(c) BEAM AND SLAB CONSTRUCTION:

- In this type of construction, the beams run in two perpendicular directions and a slab is provided between the beams.

- The columns are located at the intersection of beams.
- This type is suitable when the bending stresses are high because of large column spacing and unequal column loads.

(d) BOX STRUCTURES:

- In this type of mat foundation, a box structure is provided in which the basement walls acts as a stiffeners for the mat.
- Boxes may be made of cellular construction or rigid frame consisting of slabs and basement walls.
- This type of mat foundation can resist very high bending stresses.



(e) MATS PLACED ON PILES:

- The mat is supported on the piles in this type of construction.
- This type of mat is used where the soil is highly compressible and the water table is high.
- This method of construction reduces the settlement and also controls buoyancy.

4.Explain various types of foundation with neat sketches. (May/June 2016)

SHALLOW FOUNDATION:

- According to Terzaghi, if the depth of a footing is less than or equal to the width, it may be considered a shallow foundation.

STRIP FOOTING:

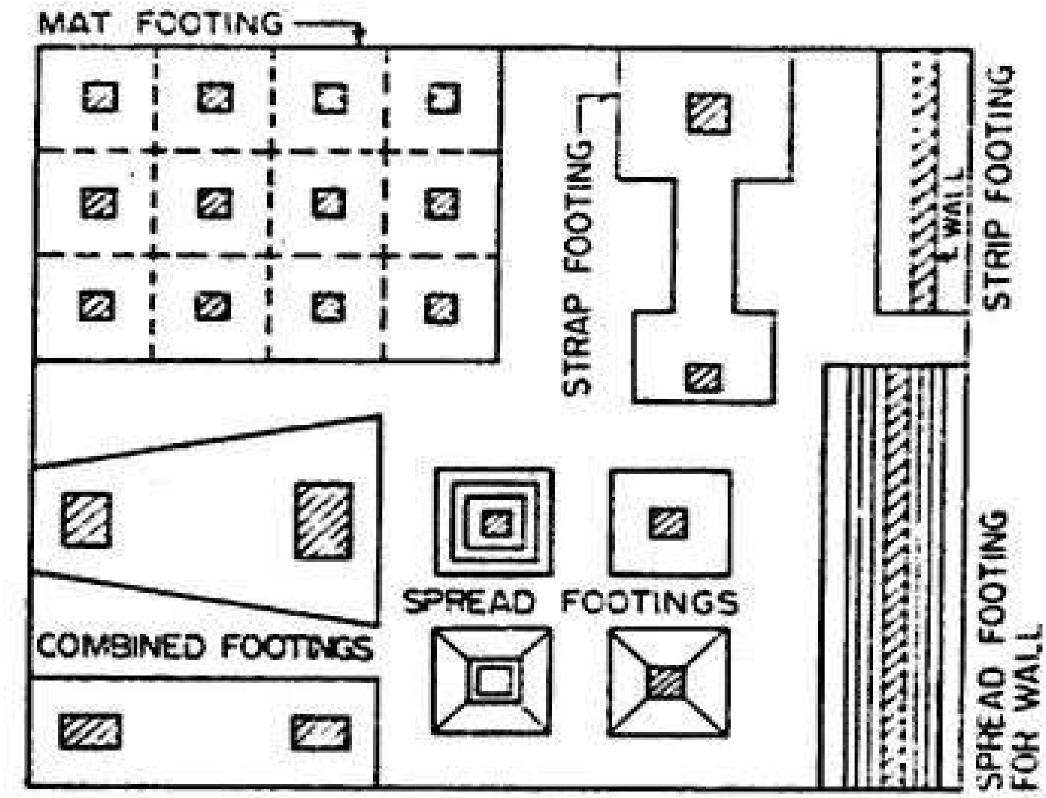
- A strip footing is provided for a load bearing wall.
- A strip footing is also provided for a row of columns which are so closely placed that their spread footings overlap or nearly touch each other. In such a case that is more economical to provide a strip footing than to provide a number of spread footings in one line.
- A strip footing is also known as continuous footing. Generally, footing required to support a wall is known as a continuous, wall footing.

SPREAD FOOTING:

- A spread or isolated or pad footing is provided to support an individual column.
- An isolated footing may be square, circular or rectangular in shape of uniform thickness. Sometimes it is stepped or hunched to spread the load over a large area.

STRAP FOOTING:

- A strap footing consists of two isolated footing connected with a structural strap beam or lever.
- The strap connects the two columns such that they behave as one unit.
- The strap simply acts as a connecting beam and does not take any soil reaction.
- The strap is designed as a rigid beam.
- The individual footings are so designed that their combined line of action passes through the resultant of the total load.
- A strap footing is more economical than a combined footing when the allowable soil pressure is relatively high and the distance between columns is large.



MAT OR RAFT FOUNDATION:

- A mat or raft foundation is large slab supporting a number of columns and walls under the entire structure or a large part of the structure.
- A mat is required when the allowable soil pressure is low or where the columns and walls are so close that individual footings would overlap or nearly touch each other.
- Mat foundations are useful in reducing the differential settlements on non-homogeneous soils or where there is a large variation in the loads on individual columns.

COMBINED FOOTING:

- A combined footing supports two columns.
- It is used when the two columns are close to one column that a spread footing would be eccentrically loaded when kept entirely within the property line.

- By combining it with that of an interior column, the load is evenly distributed.
- A combined footing may be rectangular or trapezoidal in plan.

DEEP FOUNDATIONS:

- If the depth is more, the footings are considered as deep footings. Meyerhof developed the theory of bearing capacity for such footings.

1. **PILE FOUNDATIONS:**

    - The foundations are intended to transmit structural loads through zones of poor soil to the depth where the soil has the desired capacity to transmit the loads.
    - They are somewhat similar to columns in that loads developed at one level are transmitted to a lower level; but piles obtain lateral support from the soil in which they are embedded so that there is no concern with regard to buckling and it is in this respect of that they differ from columns.
    - Piles are slender foundation units which are usually driven into a place. They may also be cast-in-place.
    - A pile foundation usually consists of a number of piles, which together support a structure. The piles may be driven or placed vertically or with a batter.

2. **PIER FOUNDATION:**

    - Pier foundations are somewhat similar to pile foundation but are typically larger I area than piles.
    - An opening is drilled to the desired depth and concrete is poured to make a pier foundation.
    - Much distinction is now being lost between the pile and the pier foundation, adjectives such as driven, bored or drilled and, cast in-situ and pre cast being used to indicate the method of installation and construction. Usually pier foundations are used for bridges.

3. CAISSON FOUNDATION:

- A caisson is a structural box or chamber that is sunk into place or built in place by systematic excavation below the bottom.
- Caissons are classified as 'Open Caisson', 'Pneumatic Caisson and box or floating caisson.

- **Open caisson** may be box type of pile type. The top and bottom are open during installation for open caissons. The bottom may be finally sealed with concrete or may be anchored into rock.
- **Pneumatic caisson** is one in which compressed air is used to keep water from entering the working chamber, the top of the caisson is closed. Excavation and concreting is facilitated to be carried out in the dry. The caisson is sunk deeper as the excavation proceeds and on reaching the final position, the working chamber is filled with concrete.

- **Box or floating caisson** is one in which the bottom is closed.
- It is cast on land and towed to the site and launched in water after the concrete has got cured.
- It is sunk into position by filling the inside with sand, gravel, concrete.

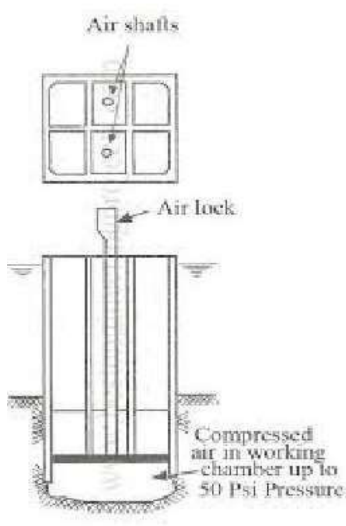


Fig 2 Pneumatic Caissons

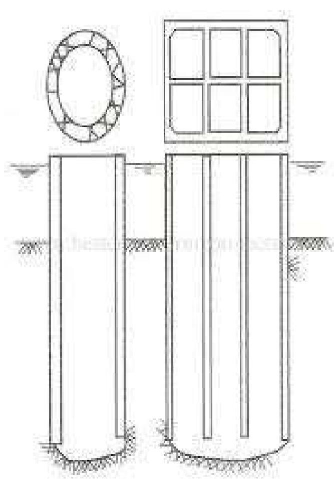


Fig 1 Open Caissons

- False bottoms are temporary base of timber are sometimes used for floating the caisson to the site

5.(i).State the design requirement of a foundation? Explain the conventional method of design of raft foundation. (May/June 2014)

1. The pressure coming on the soil from the superstructure should be below the safe bearing capacity of soil

2. The foundation must not settle more to damage the structure

3. The foundation is located such that loose fill, etc., are avoided.

4. The foundation is located such that any future influence does not adversely affect its performance

5. The foundation should be located below the depth of frost penetration

6. The foundation should be located below the constant moisture zone in highly expansive and swelling soils

7. The foundation should be located below the depth of scour.

Conventional method of design of raft foundation

Assumption:

1. Raft is rigid
2. Contact pressure is uniform or linear or planar as per super structure loading.
3. So the centroid of the soil pressure coincides with the line of action of the resultant force of all the loads action on the mat foundation.

Design procedure

1. Compute the column loads (dead load, live load, wind load, earthquake load, snow load etc. From super structure)
2. Determine the line of action of all the loads
3. Calculate the contact pressure as per the assumption and the conventional– empirical analysis design formula

   $q = (Q_t / A) \pm (Q_t e_x / I_y) x \pm (Q_t e_y / I_x) y$

Where Q_t = total load on mat

A = total area of the mat

X, Y = coordinates of any given points on the with respect to the x and y axes

passing through the centroid of the area of the mat.

e_x, e_y = eccentricities of the resultant forces.

I_x , I_y = moment of inertia of the mat with respect to the x and y axes

respectively.

4. The mat is treated as strip in X and Y direction for the analysis for shear force and bending moment

5. The design dimensions and reinforcement are arrived in both the direction.

5(ii) DESIGN PROCEDURE FOR STRAP FOOTING :: [April / May 2015]

1) Assume a reasonable value of eccentricity e between the load Q_1 and reaction R_1 on the exterior column.

2) Determine the length of the footing of the exterior column. $L_1 = 2(e + 0.5b_1)$

where, b_1 = width of the exterior column.

3) Compute the reaction R_1 by taking moments about the line of action of R_2.

$$R_1 = Q_1 x_2 / S.$$

where,

$x_2 \rightarrow$ Distance between Q_1 & Q_2.

$S \rightarrow$ Distance between R_1 & R_2.

4) Compute the areas A_1 & A_2.

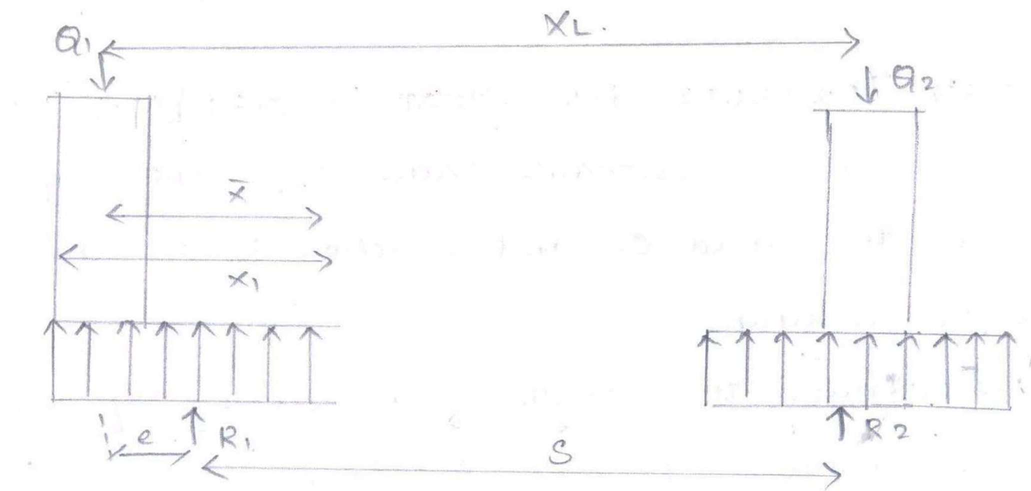
$$A_1 = R_1 / q_{na} \quad ; \quad A_2 = R_2 / q_{na}.$$

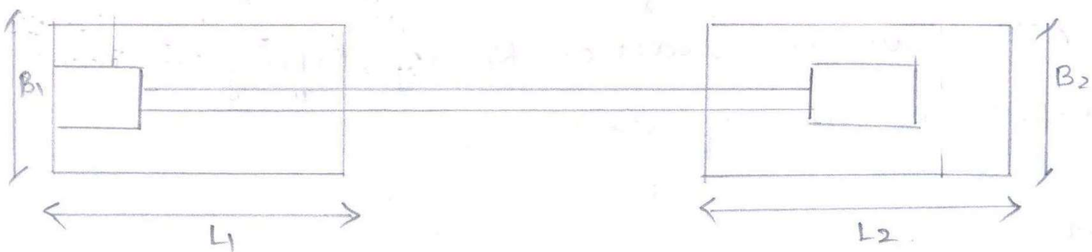
The reaction R_2 is obviously equal to $Q_1 + Q_2 - R$.

5) The width of the footing.

$$B_1 = A_1 / I_1 \quad \text{and} \quad B_2 = \sqrt{A_2}$$

Design the individual footing as in case of spread footing.





Proportion a strap footing for the following data. Allowable soil pressure at DL (WL) + reduced (LL) is 180 KN/m. For DL & LL is 270 KN/m.

Column	DL	LL
A	500KN	400KN
B	660KN	840KN

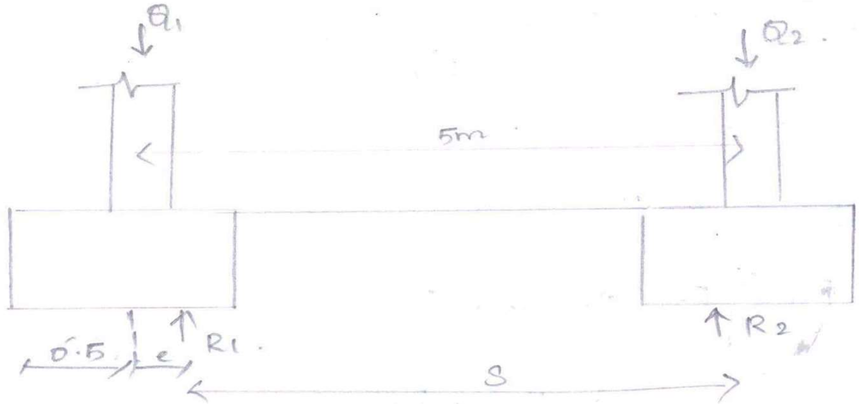
The c/c distance between the column is 5m. Projections beyond the column A, not to exceed 0.5m [April/May 2015]

SOLUTION:

Load on column A = 500 + 50% of LL

= 500 + 0.5 x 400

= 700 KN.



Load on column B $= 660 + (0.5 \times 840)$

$= 1080$ KN.

$q_a = 180$ KN.

Step1: Find the value of S.

Assume the width of footing as 2.1m.

$C = b/2 - 0.5$

$= 2.1/2 - 0.5$

$= 0.55$m.

$S = 5 - 0.55 = 4.55$m.

Step2: Find the value of R_1

$R_1 S = Q_1 x$

$R_1 = Q_1 x / S$

$= 700 \times 5 / 4.45$

$R_1 = 786.5$ KN.

Step3: Find A_1 and L.

$A_1 = R_1 / q_a = 786.5/180$

$= 4.37 m^2$.

$L = \text{Area} / B = \frac{4.37}{2.1}$

$= 2.08\ m.$

$L \simeq 2.1\ m.$

Step 4: Find R_2, A_2 and B_2.

$R_2 = (Q_1 + Q_2) - R_1$

$= (700 + 1080) - 786.5$

$R_2 = 993.5\ KN.$

$A_2 = R_2 / q_a$

$= 993.5/180 = 5.52 m^2.$

$B_2 = \sqrt{A_2}$

$= 2.34 m.$

Step 5: To find q_{a1} and q_{a2}.

$q_{a1} = R_1 / A_1.$

$R_1 = \frac{Q_1 x}{S} = \frac{900 \times 5}{4.45}.$

$= 1011.2\ KN.$

$R_2 = Q_1 + Q_2 - R_1$

$= 1388.8\ KN.$

$q_{a1} = R_1 / A_1 = \frac{1011.2}{2.1 \times 2.1}$

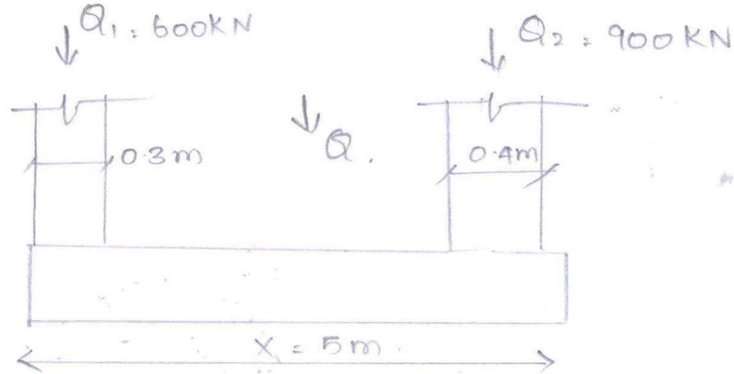
$= 229.3 < 270\ KN/m^2.$

$q_{a2} = R_2 / A_2 = \frac{1388.8}{5.52}.$

$= 251.59 < 270\ KN/m^2.$

Hence safe.

6. Design a rectangular combined footing for 2 columns as shown in figure. Take allowable soil pressure as 100 KN/m². (Nov/Dec 2013) (May/June 2011)



GIVEN :-

$Q_1 = 600\ KN$; $Q_2 = 900\ KN$.

$Q = Q_1 + Q_2 = 600 + 900$

$Q = 1500\ KN$.

$X = 5m$

$b_1 = 0.3m$; $b_2 = 0.4m$.

SOLUTION:-

STEP 1 : FIND $\bar{X}$

$$\bar{X} = \frac{Q_2 X}{Q} = \frac{900 \times 5}{1500} = 3.$$

$\bar{X} = 3m$.

STEP 2 : FIND 'L'

$$L = 2(\bar{x} + b_1/2)$$

$$= 2(3 + 0.3/2)$$

$L = 6.3\ m$.

STEP 3 : AREA OF FOOTING :

$$A = Q / q_o .$$

$$= 1500 / 100 .$$

$$A = 15 \, m^2 .$$

STEP 4 : FIND 'B'

$$B = \frac{Area}{Length} = \frac{15}{6.3}$$

$$B = 2.38 \, m .$$

STEP 5 : ACTUAL PRESSURE 'q_a'

$$q_a = Q / A = 1500/15$$

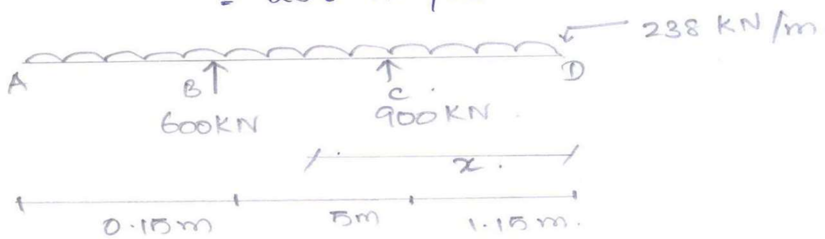
$$= 100 \, KN / m^2 .$$

STEP6 : INTENSITY OF LOADING :

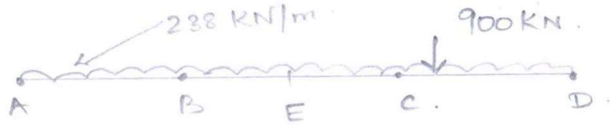
$$\text{Loading intensity} = q_a \times B$$

$$= 100 \times 2.38$$

$$= 238 \, KN/m \simeq 240 \, KN/m .$$



POINT OF CONTAFLEXURE .



$$238 \cdot x \cdot x/2 = 900(x-1)$$

$$900 = 238 . X$$

$$X = 3.78 \, m .$$

$E_c = 3.78 - 1.15$. (upward +)

$E_c = 2.63m$. (downward -)

SHEAR FORCE ::

Shear force at 'D' = 0

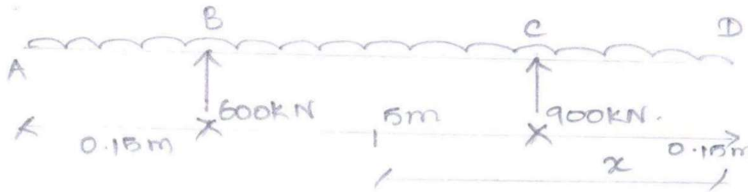
Shear force at 'A' = 0

Shear force at C = - 238 × 1.15 = - 273.7 KN.

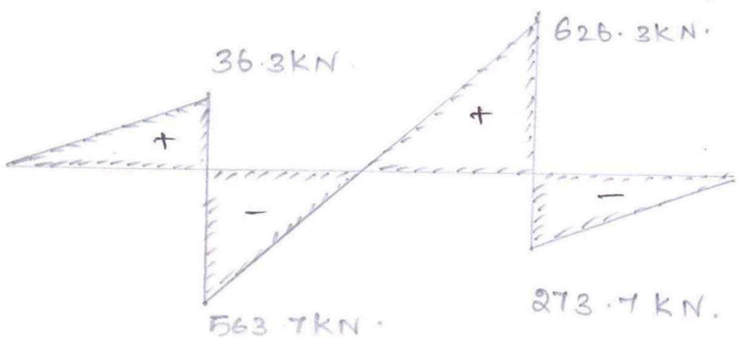
Shear force at C = 900 - 273.7 = 626.3 KN.

Shear force at B = 626.3 - (238 × 5) = - 563.7 kN

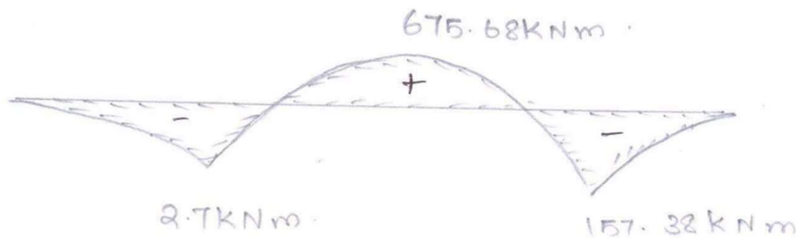
Shear force at B = -563.7 + 600 = 36.3 KN.



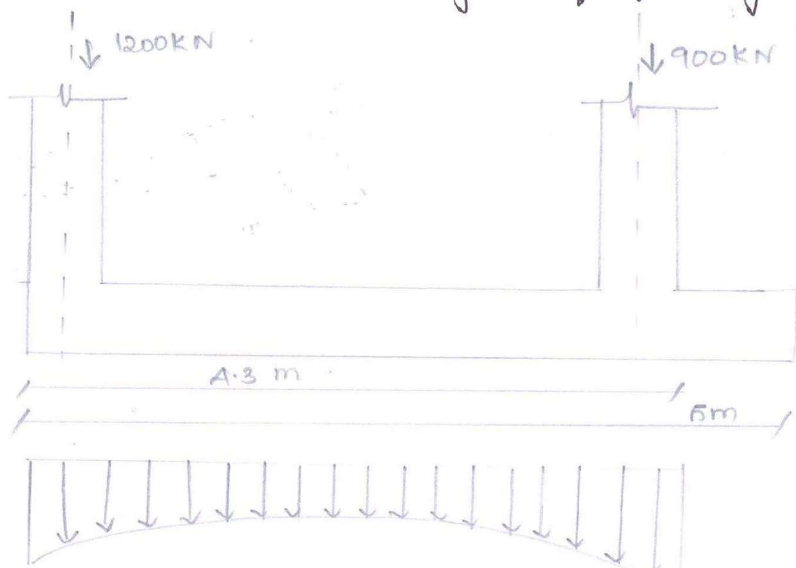
SFD.



BMD.



7. Design a trapezoidal combined footing for 2 columns of 30 cm × 30 cm, carrying column load of 1.2 MN and 0.9 MN. If the spacing between the 2 columns is 4 m, and allowable soil pressure is taken as 200 kN/m². Length of footing is 5 m. (Nov/Dec 2011)



GIVEN::

$Q_1 = 1.2\ MN = 1200\ KN.$

$Q_2 = 0.9\ MN = 900\ KN.$

$q_{na} = 200\ KN.$

$x_2 = 4 \ ; \ L = 5m.$

SOLUTION::

STEP 1;

$Q = Q_1 + Q_2.$

$= 1200 + 900$

$Q = 2100\ KN.$

STEP 2:

$$A = Q / q_{na}$$

$$= 2100 / 200$$

$$A = 10.5 \ m^2.$$

STEP 3:

$$\bar{x} = \frac{Q \cdot x_2}{Q}$$

$$= \frac{900 \times 4}{2100}$$

$$\bar{x} = 1.714 m.$$

STEP 4:

$$x' = \bar{x} + b_1/2.$$

$$= 1.714 + 0.3/2.$$

$$x' = 1.864 m.$$

STEP 5:

$$L_1 = x_2 + b_1/2 + b_2/2.$$

$$= 4 + 0.3/2 + 0.3/2$$

$$L_1 = 4.3 m.$$

Condition: $L/3 < x' < L/2$.

$$\Rightarrow 1.67 < 1.86 < 2.5$$

Hence safe.

STEP 6:

Width of footing,

$$B_2 = \frac{2A}{L}\left[\frac{3x'}{L} - 1\right]$$

$$= \frac{2 \times 10.5}{5}\left[\frac{3 \times 1.864}{5} - 1\right]$$

$$= 4.2 (0.1184)$$

$$= 0.497 \simeq 0.5 m.$$

$$B_1 = \frac{2A}{L} - B_2.$$

$$= \frac{2 \times 10.5}{5} - 0.5.$$

$$B_1 = 3.7.$$

STEP 7 :-

$$q_o = Q / \text{Actual area}.$$

$$= 2100 / 10.5.$$

$$= 200 \, KN/m^2.$$

STEP 8 :- SFD.

$$V_A = 0.$$

$$V_B(L) = \frac{740 + 720.8}{2} \times 0.15$$

$$= 109.56 \, KN.$$

$$V_B(R) = 109.56 - 1200$$

$$= -1090.44 \, KN.$$

$$V_C(L) = \frac{208.8 + 100}{2} \times 0.85 - 1200.$$

$$= -1068 \, KN.$$

$$V_C(R) = -\left[\frac{208.8 + 100}{2} \times 0.85\right]$$

$$= -131.24 \, KN.$$

$$V_D = 0.$$

$$V_C(L) = \left(\frac{208.8 + 100}{2} \times 0.85\right) - 1200.$$

$$= 768.76 \, KN.$$

$$V_C(R) = \frac{208.8 + 100}{2} \times 0.85.$$

$$= 131.24 \, KN.$$

$$V_D = 0.$$

Force

$$= 100 + \frac{(740 - 100)}{5} \times 0.85.$$

$$= 208.8 \, KN$$

UNIT IV

PILE FOUNDATION

1.What are methods to determine the load carrying capacity of a pile?

(Nov/Dec 2015)

- Dynamic formulae
- static formula
- pile load test
- penetration test

2.Define negative skin friction. **(May/June 2014), (Nov/Dec 2012), (May/June 2011), (Nov/Dec 2011)**

When the soil layer surrounding a portion of the pile shaft settles more than a pile, a downward drag occurs on the pile. The downward drag is known as negative skin friction.

3.Define group efficiency of pile. **(May/June 2016), (May/June 2011)**

The ratio of resting capacity of a pile group to the sum of individual capacities of piles in the group is termed as group efficiency.

$$\text{Group efficiency , } n = \frac{Q_g}{N_p \times Q_p}$$

Where, Q_g - Group capacity

Q_p – Pile load on single pile

N_p – Number of piles

4.What are the conditions where a pile foundation is more suitable than a shallow foundation?

- Huge vertical load with respect to capacity
- Very weak soil
- Huge lateral loads

- For fills having very large depth
- Uplift situation
- Urban areas for future and huge construction near the existing building.

5.What are the limitations of dynamic pile load test?

- It is largely depend on the nature of the ground through which the pile was driven to get down to finished level.
- It takes very little account of the effect of friction on sides of pile, and this friction tends only to develop later.

6.List the piles based on materials of installation. **(Nov/Dec 2013)**

- End bearing pile
- Friction pile
- Compaction pile
- Tension pile
- Anchor pile
- Fender pile and dolphins
- Batter pile
- Sheet pile

7.What are the factors governing selection of pile? **(Nov/Dec 2012)**

- Soil condition
- Type of structure or building
- Adjacent site condition
- Construction techniques availability
- Location of ground water table
- Durability etc.

8.Define end bearing pile.

End bearing piles are used to transfer load through water or soft soil to a suitable bearing stratum. The end bearing pile is driven through poor soil strata and rests on a firm incompressible stratum such as rock, developing the bearing pressure of its base and passing it to that firm stratum.

9.How is the selection of pile carried out? **(Nov/Dec2014)**

The selection of the type, length and capacity is usually made from estimation based on the soil condition and magnitude of the load.

10.For identical soil conditions, the load permitted on bored pile is lesser than driven pile of identical shape and dimensions, why?

The load carrying capacity of bored cast in situ pile will be much smaller than that of a driven pile in sand. The angle of shearing resistance of the soil is reduced by 30, to account for the loosening of the sand due to the drilling of the hole.

11.What are fender piles? **(May/June 2013)**

Fender piles are the type of the piles which are used to protect water front structures against impact from ships or other floating objects.

12.What is meant by friction pile? **(May/June 2014)**

Friction piles are used to transfer loads to a depth of a friction load carrying material by means of skin friction along the length of the pile.

PART-B (16MARK)

1.Explain the under reamed pile foundation with neat sketch. **(Nov/Dec2015) (May/June 2015), (May/June 2013), (Nov/Dec2012)**

Under – reamed pile foundation

- Under reamed piles are bored cast in-situ concrete piles having one or more bulbs formed by enlarging the bore hole for the pile stem by an under reaming tool.
- These piles find applications in widely varying situations in different types of soils where foundation are required to be taken down to a certain depth to avoid the undesirable effect of seasonal moisture changes as in expansive soils or to reach strata or to obtain adequate capacity for downward, upward and lateral loads or to take the foundations below scour level and for moments.
- When the pile has only one bulb, it is known as single under –reamed pile, while the pile with more than one bulb is known as multi –under –reamed

pile. Generally, the diameter of under –reamed bulbs is kept equal to 2.5 times the diameter of pile stem.

- However, it may vary from 2 to 3 times the stem diameter, if required, depending upon the design requirements and feasibility of construction.

Details of pile and under reamed bulb:

- In deep layers of expansive soils, the minimum length of pile required is 3.5 m where the ground movements become negligible.
- In shallow depths of expansive soils and other poor soils depending upon the load poor soil requirements the length may be reduces and the piles may be taken upto at least 50 cm in stable zone pile length may be increased for higher loads.
- The diameter manually bored piles range from 20 cm to 37.5 cm.
- The spacing of the piles of the piles shall be considered in relation to the nature of the ground, the types of piles and the manner in which the piles transfer the loads to the ground.
- Generally, the center to center spacing for under-reamed piles should not be less than 3 D_u.
- It may be reduced to 1.5 D_u when a reduction in load carrying capacity of 10 % should be allowed.
- For the spacing of 2 D_u the bearing capacity of pile group may be taken equal to the number of piles multiplied by the bearing capacity of individual pile.
- If the adjacent piles are of different diameters, an average value for spacing should be taken.
- The maximum spacing of the under-reamed pile should not normally exceed 2 ½ meters so as to avoid heavy capping beams.
- In building, the piles should generally be provided under all wall junctions to avoid point loads on beams.
- Position of intermediate piles are then decided trying to keep the door opening fall in between two piles as far as possible.
- In double and multi-under-reamed piles of size less than 30 cm dia., the center-to-center vertical spacing between the two under reams may be kept equal to 1.5 D_u while for piles of 30 cm and more this distance may

be reduced to 1.25 D_u. the upper bulb should not bulb is 1.5m or 2 D_u whichever is greater.

- Under reamed piles can be made at a better also, for sustaining large lateral loads, thus making them suitable for tower footing, retaining walls and abutments. They have also been found useful for factory buildings, machine foundations and transmission line towers and poles.
- In black cotton soils and other expansive soils, the under reamed pile anchors the structures at a depth where the volumetric changes in soils due to seasonal and other variation is negligible.
- The under reamed pile is nominally reinforced with 10 to 12 mm dia. Longitudinal bars, and 6mm Ø rings. A clear cover of 4 cm is provided.

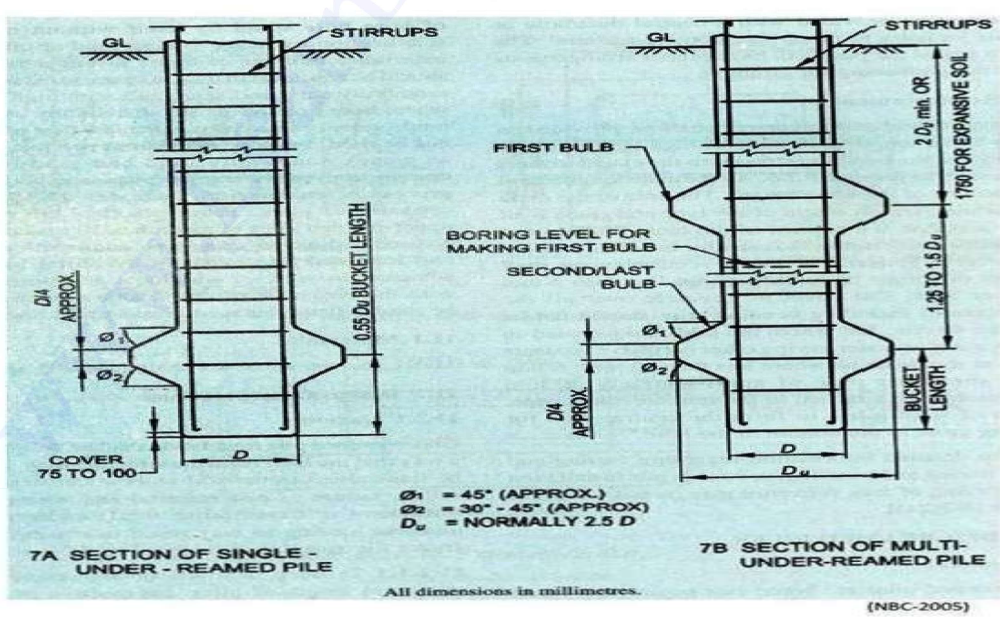
Clayey soils:

$$Q_u = A_p N_c C_p + A_a N_c C_a' + C_a' A_s' + \alpha C_a A_s$$

Sandy soils:

$$Q_u = \pi/2 \, (D_u^2 - D) \, [\ \tfrac{1}{2} D_u \, . \, n \, . \, \gamma \, . \, N_\gamma + \gamma \, . N_q \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots$$

Typical Details of Piles



Typical Details of Bored Cast in-situ Under Reamed Pile Foundation

2.Enumerate the various types of pile in detail. (May/June 2016),(Nov/Dec2015), (May/June 2015), (Nov/Dec2012)

CLASSIFICATION OF PILES

Piles can be classified according to

1. The material used
2. The mode of transfer of load
3. The method of construction
4. The use and
5. Displacement of soil

1. Classification according to material used

There are four types of piles according to materials used

(i) Steel piles
(ii) Concrete piles
(iii) Timber piles
(iv) Composite piles

(i) Steel piles

- Steel piles are generally either in the form of thick pipes or rolled steel H-section. Pipe steel piles are driven into ground with their ends open or closed. Piles are provided with a driving point or shoe at the lower end.
- Epoxy coatings are applied in the factory during manufacture of pipes to reduce corrosion of the steel pipes. Sometimes concrete encasement at site is done as a protection against corrosion. To take into account the corrosion, an additional thickness of the steel section is usually recommended.

(ii) Concrete piles

- Cement concrete is used in the construction of concrete piles. Concrete piles are either precast or cast in- situ. Precast concrete piles are prepared in a factory or a casting yard. The reinforcement is provided to resist handling and driving stresses. Precast piles can also be pre-stressed using high strength steel pre-tensioned cables.

- A cast in-situ pile is constructed by making a hole in the ground and then filling it with concrete. A cast in situ pile may be cased or uncased. A cased pile is constructed by driving a steel casing into the ground and filling it with concrete.
- An uncased pile is constructed by driving to the desired depth and gradually withdrawing casing when fresh concrete is filled. An un-casted pile may have a pedestal.

(iii) Timber piles

- Timber piles are made from tree trunks after proper trimming. The timber used should be straight, sound and free from defects.
- Steel shoes are provided to prevent damage during driving. To avoid damage to the top of the pile, a metal bond or a cap is provided. Splicing of timber piles is done using pipe sleeve or metal straps and bolts. The length of the pipe sleeve should be at least five times the diameter of the pile.
- Timber piles below the water table have generally long life. However above the water table, these are attacked by insects. The life of the timber piles can be increased by preservatives such as creosote oil. Timber piles should be used in massive environment where these are attacked by various.

(iv)Composite piles

- A composite pile is made of two materials. A composite pile may consist of the lower portion of steel and the upper portion of cast in-situ concrete.
- A composite may also have the lower portion of timber below the permanent water table and the upper portion of the concrete.
- As it is difficult to provide a proper joint between two dissimilar materials, composite piles are rarely used in practice.

2. Classification based on mode of transfer of load

Based on the mode of transfer of loads, the pile can be classified into three categories:

(i) End bearing piles

(ii) Friction piles

(iii) Combined end bearing and friction piles

(i) End bearing piles

- End bearing piles transmit the loads through their bottom tips. Such piles act as columns and transmit the load through a weak material to a firm stratum below. If bed rock is located within a responsible depth, piles can be extended to the rock.
- The ultimate capacity of the pile depends upon the bearing capacity of the rock. If instead of bed rock, a fairly compact and hard stratum of soil exists at a reasonable depth, piles can be extended a few minutes piles are also known as “point-bearing piles”.
- The ultimate load carried by the pile (Q_u) is equal to the load carried by the point or bottom end (Q_p)

(ii) Friction piles

- Friction piles do not reach the hard stratum. These piles transfer the loads through skin friction between the embedded surface of the pile and the surrounding soil. Friction piles are used when a hard stratum does not exist at a reasonable depth.
- The ultimate load (Q_u) carried by the pile is equal to the sum of the load carried by the pile is equal to the load transferred by skin friction (Q_s).
- Friction piles are known as floating piles as these do not reach the hard stratum.

(iii) Combined end bearing and friction piles

- The piles transfers loads by a combination of end bearing at the bottom of the pile and friction along the surface of the pile shaft, the ultimate load carried by the pile is equal to the sum of the load carried by the pile point (Q_p) and the load carried by the skin friction(Q_s).

3. Classification based on method of installation

Based on the method of construction, the piles may be classified into the following 5 categories

(i) Driven pile

(ii) Driven and cast in situ piles
(iii) Bored and cast in situ piles
(iv) Screw piles
(v) Jacked piles

(i) Driven piles

- These piles are driven into the soil by applying blows of a heavy hammer on their tops.

(ii) Driven and cast in situ piles

- These piles are formed by drawing a casing with a closed bottom end into the soil. The casing is later filled with concrete. The casing may or may not be withdrawn.

(iii) Bored and cast in situ pile

- These piles are formed by a hole into the ground and then filling it with concrete.

(iv) Screw piles

- These piles are screwed into soil.

(v) Jacked piles

- These piles are jacked into the soil by applying a downward force with the help of a hydraulic jack.

4. Classification based on use

The piles can be classified into the following 6 categories depending upon their use.

(i) Load bearing piles
(ii) Compaction piles
(iii) Tension piles
(iv) Sheet piles
(v) Fender piles
(vi) Anchor piles

(i) Load bearing piles

- These piles are used to transfer the load of the structure to a suitable stratum by end bearing by friction or by both.

(ii) Compaction piles

- These piles are driven into the loose granular soil to increase the relative density. The bearing capacity of the soil is increased due to densification caused by vibrations.

(iii)Sheet piles

- Sheet piles forms a continuous wall or bulk head which are used for retaining earth or water.

(iv)Fender piles

- Fender piles are sheet piles which are used to protect water front structures from impact of ships and vessels.

(v) Anchor piles

- These piles are used to protect anchorage for anchored sheet piles. These piles provide resistant against horizontal pull for a sheet pile wall.

5. Classification based on displacement of soil:

Based on the volume of the soil displacement during installation the piles can be classified into 2 categories

(i) Displacement piles
(ii) Non- displacement piles

(i) Displacement piles

- All driven piles are displacement piles as the soil is displacement laterally when the pile is installed. The soil gets densified. The installation may cause heaving of the surrounding ground.
- Precast concrete pile and closed end pipe pile are high displacement piles. Sheet H- piles are low displacement piles.

(ii) Non- displacement piles

- Bored piles are non- displacement piles. As the soil is removed when the hole is bored, there is no displacement of the soil during installation. The installation of these piles causes very little change in the stresses in the surrounding soil.

3.Explain with neat sketch about pile load test method of determination of load carrying capacity of piles. (May/June 2016), (May/June 2014), (May/June 2013), (Nov/Dec2013)

PILE LOAD TESTS:-

- The pile load test can be performed either on a working pile which form the foundation of the structure or on a test pile.
- The test load is applied with the help of calibrated jack placed over a rigid circular or square plate which in turn is placed on the head of the pile projecting above ground level.
- The reaction of the borne by a truss or platform which have gravity loading or alternatively, the truss can be anchored to the ground with the help of anchor pile. In the later case, under-reamed piles or soil anchor may be used for anchoring the truss.
- The load is applied in equal increments of about one-fifth of the estimated allowable load.
- The settlements are recorded with the help of three dial gauge of sensitivity 0.02mm, symmetrically arranged over the test plate, and fixed to an independent datum bar.
- A remote controlled pumping unit may be used to hydraulic jack. Each load increment is kept for sufficient time till the rate of settlement becomes less than 0.02mm per hour.
- The test pile are loaded until ultimate load is reached. Ordinarily, the test load is increased to a value 2.5times the estimated allowable load or to a load which causes a settlement equal to one-tenth of the pile diameter, whichever occur earlier.
- The results are plotted in the form of load settlement curve. The ultimate load is clearly indicated by load settlement curve approaching vertical. If ultimate load cannot be obtained from the load settlement curve, the allowable load taken as follows:

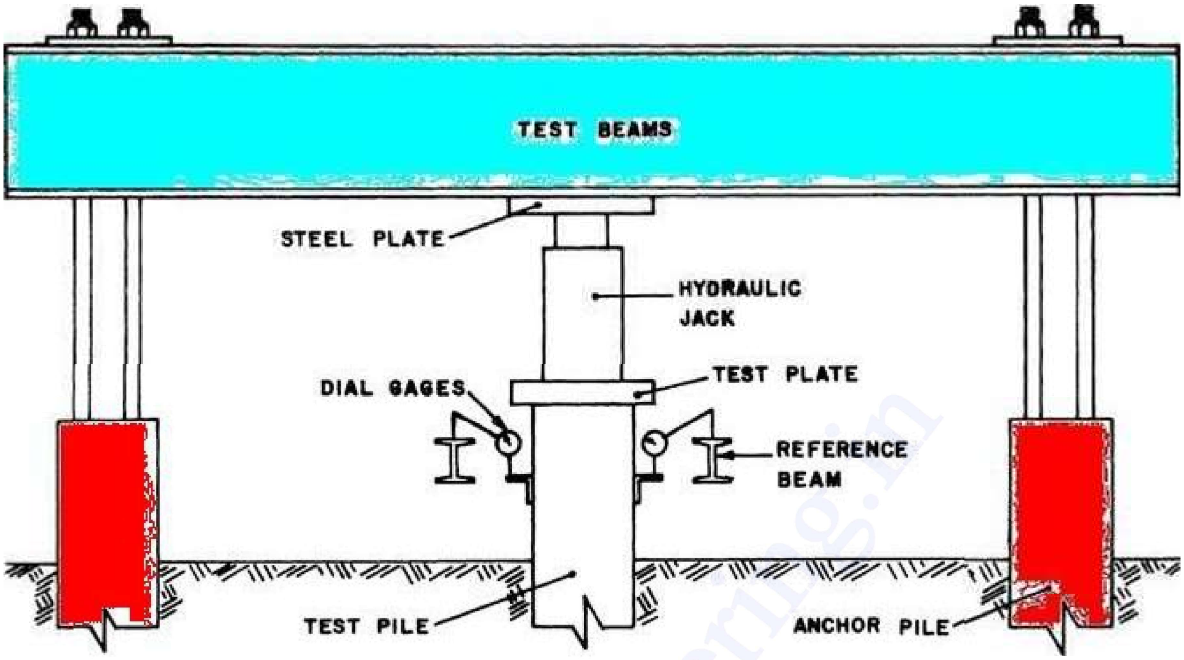
CYCLIC LOAD TEST:

- The cyclic load test is particularly useful in separating the load carried by the pile into the skin friction and point bearing resistance.
- Each load increment is kept on the pile for sufficient time till the settlement decreases the value less than 0.02mm per hour.
- The load is then completely removed and the elastic rebound of the pile top is measured by means of dial gauge. The next load is then applied and the process repeated.
- The cycle of loading and unloading with measurement of settlement and recovery is continued till the final load which causes a marked progressive settlement of the pile is reached.
- The result plotted between loads versus settlement.

The elastic compression of the pile corresponding to any load Q can be calculated from the following expression based on HOOK's law,

$$\text{Elastic compression} = ((Q-R_f/2)L)/AE$$

- The separation of Q at any stage of loading into R_p and R_f is based on that the load on the pile toe increases linearly with the elastic compression of soil, and that straight line showing the relationship between point resistance and elastic compression of soil is parallel to the straight line portion of the curve drawn between the load on the pile and elastic compression of soil.
- The elastic compression of the soil is equal to the total elastic recovery of the pile top minus the elastic compression of the pile. The procedure described in the following steps:



STEPS:

1. If R_f is not known to start with, it is assumed that the elastic compression of the pile is zero, and hence the elastic compression of the soil is equal to the total elastic recovery of the pile top. A curve OA_1 is then drawn between load Q on pile top as abscissa and the elastic compression of the soil as ordinate.
2. Through origin O, a line OA_1' is drawn parallel to the straight portion of bearing R_p and skin friction R_f.
3. For various loads Q_1, Q_2, Q_3, etc., the skin friction R_{f1}, $R_{f2\ \Delta}$, R_{f3} etc., are determined.
4. Corresponding to each value of R_f, the elastic compression of pile is determined. The elastic compression of the soil is calculated from the relation.

$$\Delta_{soil} = \Delta - \Delta_{pile}$$

Where, Δ = total elastic recovery of the pile top.

5. Knowing Δ_{soil} for each load Q_1 ,Q_2,Q_3etc. A curve is drawn between Q and Δ_{soil}.
6. Through the origin O, line OA_2' is drawn parallel to the straight line portion of curve OA_2'.

7. Step 3, 4, 5 and 6 are repeated to get the final curve and OA’ parallel to the straight line portion of curve OA. The third trial of curves gives sufficiently accurate results. From this two, any load Q can be divided to skin friction and point resistance.
8. The value of skin friction and point resistance corresponding to a load causing a total settlement of one-tenth of the pile diameter are by factors of safety of 2 and 2.5 respectively and added together to give the allowable load for the pile.

4. A concrete pile of diameter 40 cm is to be driven in a stiff clay. Unconfined compressive strength of clay is 180 kN/m^2. What is the length required to be penetrated by the pile to support a safe working load of 350 kN. Take adhesion factor as 0.7. [May/June 2012]

GIVEN:

Diameter = 0.40 m.

q_u = 180 kN/m^2.

α = 0.7

Safe load = 350 kN.

SOLUTION:

Ultimate load = Q_u = F.S × Q_s.

Assume Factor of safety = 2.5.

$\therefore Q_u = 2.5 \times 350$.

$= 875$ kN.

$Q_u = CN_c A_p + \alpha C A_s$.

N_c = 9 (Bearing capacity factor)

A_p = Cross section of pile.

$= \frac{\pi \times 0.4^2}{4}$

$A_p = 0.126\ m^2$.

A_s = Surface area of pile.

$= \pi d l$.

$A_s = \pi \times 0.4 \times l$.

$A_s = 1.26 l$.

Cohesion $C = q_u/2 = 180/2 = 90\ kN/m^2$.

$\alpha = 0.7$

$Q_u = CN_c A_p + \alpha C A_s$.

$= (90 \times 9 \times 0.126) + (0.7 \times 90 \times 1.26 l)$

$875 = 102.06 + 79.38 l$.

$\therefore l = \dfrac{875 - 102.06}{79.38}$

$l = 9.74\ m$.

Length of pile $l = 9.74\ m$.

5. Determine group efficiency of a pile group consists of 16 piles of each 20m long and diameter with c/c distance on both directions equal to 1.0m which are embedded on a clay deposit having cohesive strength of $35\ kN/m^2$ by static method. Feld's rule and converse Labara formula. Take adhesion factor as 0.6. [Nov/Dec 2013]

Data:

Square group of 16 piles $n = 16$.

Length $= 20\ m$.

Diameter $= 1\ m$.

c/c distance $= 1\ m$.

$C = 35\ kN/m^2$.

$\alpha = 0.6$

Solution:

Static Method:

For piles acting individually,

$$Q_{un} = (A_p r_p + A_s r_f)\, n.$$

C/s area of Pile $A_s = \frac{\pi}{4} \times 1^2 = 0.78\ m^2$.

$$r_p = CN_c.$$
$$= 35 \times 9 = 315\ kN/m^2.$$

$$A_s = \pi d l$$
$$= \pi \times 1 \times 20 = 62.83\ m^2.$$

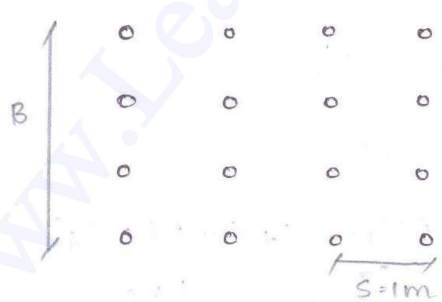
$$r_f = \alpha C$$
$$= 0.6 \times 35 = 21\ kN/m^2.$$

$$\therefore Q_{un} = (0.78 \times 315 + 62.83 \times 21)\, 16.$$

$$Q_{un} = 25042\ kN.$$

Piles acting in a group:

$$Q_{ug} = A_{pg}\, r_p + A_{sg}\, r_f.$$



$$B = 3S + d$$
$$= (3 \times 1) + 1$$
$$= 4.$$

$$A_{pg} = B^2 = 4^2 = 16\ m^2.$$

$$A_{sg} = 4BL = 4 \times 4 \times 20 = 320\ m^2.$$

$$r_p = CN_c = 9 \times 35 = 315\ kN/m^2.$$

$$r_f = 21\ kN/m^2.$$

$$Q_{ug} = (16 \times 315) + (320 \times 21)$$

$$Q_{ug} = 11760 \text{ KN}.$$

Ultimate load is the lesser of two values,

$$\therefore Q_u = 11760 \text{ KN}.$$

Efficiency of pile group $\eta = \dfrac{Q_{ug}}{Q_{un}} = \dfrac{11760}{25042}.$

$$\eta = 0.47 \text{ or } 47\%.$$

Converse Labarre Formula:

$$\eta_g = 1 - \frac{\theta}{90}\left[\frac{(n-1)m + (m-1)n}{mn}\right]$$

$$\theta = \tan^{-1} d/s.$$

d = diameter of pile

n = number of piles in a row

s = spacing of pile.

m = number of rows.

$$\theta = \tan^{-1}(1/1).$$

$$\theta = 57° 17'$$

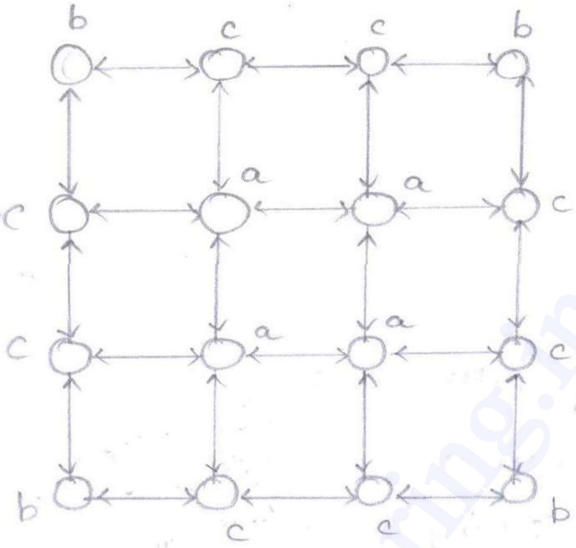
$$\eta_g = 1 - \frac{57°17'}{90}\left[\frac{(4-1)4 + (4-1)4}{4 \times 4}\right]$$

$$= 1 - 0.64\left(\frac{12+12}{16}\right) = 1 - 0.96.$$

$$= 0.04$$

$$\eta_g = 4\%.$$

Feld's Rule:-



Piles 'a' influenced by 8 piles.

'b' influenced by 3 piles

'c' influenced by 5 piles

No of 'a' piles	= 4 @ 8/16		2
b piles	= 4 @ 15/16		3.75
c piles	= 8 @ 11/16		5.50
			11.25.

$$\eta = \frac{11.25}{\text{no of piles}} = 0.7.$$

$$\eta = 70\%.$$

6. In a 16 pile group, pile diameter is 45 cm and c/c spacing of piles in a group (square) 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 10m long. Take $m = 0.70$, for shear mobilisation around each pile. [May / June 2014]

GIVEN:

No of piles = 16.

Pile diameter = 0.45 m.

c/c spacing = 1.5 m.

$c = 50\ kN/m^2$.

Length of pile = 10 m.

$m = 0.70$

Solution:

Width of group = $(3 \times 1.5 + 0.45)$

$= 4.95\ m$.

For Block failure:

$Q_g = C * \text{perimeter} \times \text{length}$.

$= 50 \times 4 \times 4.95 \times 10$.

$Q_g = 9900\ kN$.

Piles acting individually:

$Q_g = n * (mc \cdot A_s)$

$= 16 \times (0.7 \times 50 \times \pi \times 0.45 \times 10)$

$= 7916.8 = 7917\ kN$.

∴ The foundation is governed by piles acting individually and ultimate load capacity is 7917 kN.

7. A group of 9 piles arranged in a square pattern with diameter and length of each pile as 25 cm and 10m respectively, is used as a foundation in soft clay deposit. Taking the unconfined compressive strength of clay as 120 kN/m², and pile spacing as 100 cm center to center, find the load capacity of the group. Assume the bearing capacity factor $N_c = 9$ and adhesion factor = 0.75. FOS = 2.5 may be taken. [NOV/DEC 2012].

Solution:

Piles acting individually:

$$C_a = q_u/2 = 120/2 = 60\,kN/m^2.$$

$$Q_{up} = A_p r_p + A_s r_f.$$

$$A_p = \frac{\pi \times 0.25^2}{4} = 0.04909\,m^2.$$

$$A_s = \pi \times 0.25 \times 10 = 7.854\,m^2.$$

$$r_p = cN_c = 9 \times 60 = 540\,kN/m^2.$$

$$r_f = mc = 0.75 \times 60 = 45\,kN/m^2.$$

$$Q_{up} = (0.04909 \times 540) + (7.854 \times 45)$$

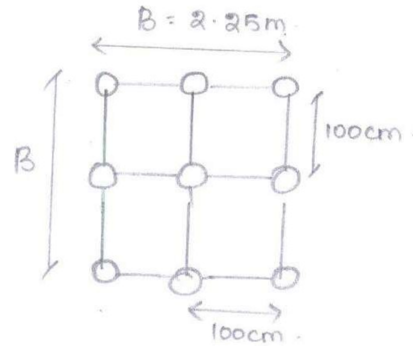
$$= 380\,kN.$$

Load capacity of 9 piles = 9 × 380 = 3419 kN.

Pile acting as a group:

$$B = 2x + d = (2 \times 1) + 0.25.$$

$$= 2.25\,m.$$



$Q_{ug} = A_p r_p + A_s r_f$.

$A_p = B^2 = 2.25 \times 2.25 = 5.0624 m^2$.

$A_s = 4B \times 10$

$= 4 \times 2.25 \times 10 = 90 m^2$.

$r_p = 9C = 9 \times 60 = 540 \ kN/m^2$.

$r_f = C = 60 \ kN/m^2$.

$Q_{ug} = (5.0625 \times 540) + (90 \times 60)$

$= 8133.8 \ kN$.

$Q_a = \frac{Q_{u\,min}}{F} = \frac{3419}{3.5}$

$Q_a = 1367.6 \ kN$.

UNIT V

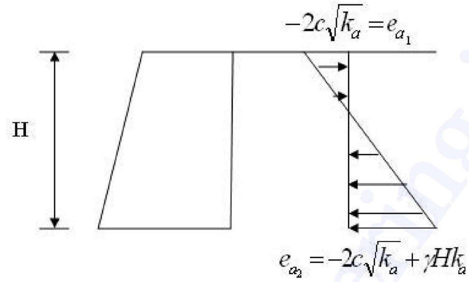
RETAINING WALLS

PART A

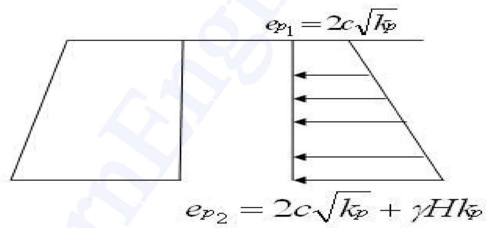
1. **Draw the lateral earth pressure diagram of clay depends for active and passive condition.**

(May/June 2016)

The value of active earth pressure is

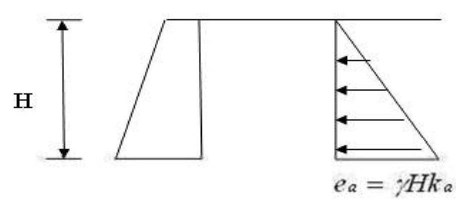


The value of passive earth pressure is

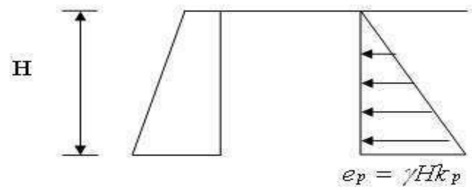


2. **Draw the lateral earth pressure diagram of sand depends for active and passive condition.** **(May/June 2016)**

The value of active earth pressure is



The value of passive earth pressure is



3. **What is surcharge angle? (Nov/Dec 2015), (May/June 2015), (May/June 2013)**The angle of surcharge of a material is the angle to the horizontal, which the urface of the materials assumes, while the material is at rest on a moving conveyor belt. The surcharge angle is generally 5' to 15' less than the angle of repose.

4.What is earth pressure at rest? (May/June 2014), (May/June 2013), (Nov/Dec2011)

The earth pressure at rest is defined as the intensity of lateral earth pressure when the lateral strain is zero and it is expressed as $P_R = K_R \cdot \gamma' Z$, where K_R – coefficient of earth pressure.

5.What are the assumptions in coulomb's theory? (May/June 2011)

- Uniform c – Φ
- Failure plane is straight
- Failure wedge is a rigid body
- Frictional force is developed along the wall boundary during the movement of wedge

6.What is meant by critical depth of vertical cut for a clay soil? (Nov/Dec2013)

Due to negative pressure, a tension crack usually developed in the soil near the top of the wall, upto to a depth Z_0. Also, the total pressure upon a depth $2Z_0$ is zero. This means that a cohesive soil should be able to stand with a vertical face upto a depth $2Z_0$ without any lateral support. The critical height H_c of an unsupported vertical cut in cohesive soil is thus given by,

$$H_c = 2Z_0 = \frac{4\,C}{\gamma} \tan\alpha$$

7.Why retaining walls are usually designed for active earth pressure? (Nov/Dec2013)

From Rankine's assumption, no-existence of frictional forces at the wall face, the resultant pressure must be parallel to the surface of the backfill. The existence of friction makes the resultant pressure inclined to the normal to the wall at an angle between the soil and the wall.

8.What do you understand by plastic equilibrium in soil?

A body of soil is said to be in plastic equilibrium, if every point of it is on the verge of failure.

9.What is critical failure plane?

Critical failure plane defined as the plane along which the failure occurs in which the shear stress on the plane is less than the maximum shear stress.

10.Write the types of retaining wall.
(Nov/Dec2012)

The earth retaining walls are of following types:

(a) Gravity wall

- (i) Mass concrete or masonry wall
- (ii) Wall on wells
- (iii) Precast block wall
- (iv) Two row sheet pile wall
- (v) Crib wall

(b) Reinforced concrete wall

- (i) Cantilever type 'T' wall or 'L' wall
- (ii) Counterforted or butteressed wall

(c) Sheet pile wall

- (i) Cantilever sheet pile wall
- (ii) Anchored sheet pile wall or Anchored bulkhead.

11.Compare Rankine's and Coulomb's theory.

Rankine's theory	Coulomb's theory
The intensity of earth pressure at each depth is known. So point of application of the earth pressure is known at any depth	Only the total earth pressure value acting on the retaining structures can be calculated. The point of application of earth pressure can be calculated from Coulomb's assumption that all points on the back of the retaining wall are essentially considered as feet of failure surface
Wall is smooth and vertical	Wall is rough and sloped
Wall moved sufficiently so soil is i9n plastic failure mass	Wall is rigid, straight failure plane and rigid failure wedge

12.What are the conditions to be satisfied while designing a retaining wall?

Sliding resistance:

$$\text{Factor of safety} = \frac{Sum\ of\ resisting\ force}{Sum\ of\ driving\ force}$$

Factor of safety against sliding should be atleast 1.5 for sandy soil and 2.0 for clayey soil.

Overturning:

To avoid overturning the resultant thrust must fall within the middle third of the wall base.

$$\text{Factor of safety} = \frac{Sum\ of\ resisting\ force}{Sum\ of\ overturning\ force}$$

Factor of safety against overturning should be atleast 1.5 for sandy soil and 2.0 for clayey soil.

Bearing Capacity:

$$\text{Factor of safety} = \frac{Allowable\ bearing\ pressure}{Maximum\ contact\ pressure}$$

Factor of safety against bearing capacity should be atleast 2.5 for sandy soil and 3.0 for clayey soil.

13.Write down any two assumptions of Rankine's theory? (Nov/Dec2012)

- Semi infinite soil
- Cohesion-less backfill
- Homogenous soil
- The top surface is a plane which may be inclined or horizontal.

14. How do you check the stability of retaining walls?

- The wall should be stable against sliding
- The wall should be stable against overturning
- The base of the wall should be stable against bearing capacity failure.

15.Define angle of repose?

Maximum natural slope at which the soil particles may rest due to their internal friction, if left unsupported for sufficient length of time.

PART B

1.Explain Rankine's theory for the cases of cohesion less backfill. (May/June 2016), (Nov/Dec 2015), (May/June 2013)

Rankine's Theory:: GSR 397

- Considers the stress in a soil mass, when it reaches a state of plastic equilibrium. (i.e) when shear failure is imminent at every point within the soil mass.

Assumptions.

1. Backfill is isotropic, homogeneous and cohesionless.
2. In a state of plastic equilibrium during active and passive earth conditions.
3. Rupture surface is a planar surface, obtained by considering plastic equilibrium of soil.
4. Backfill surface is horizontal.
5. Back of wall is vertical & smooth.

Active Pressure: Cohesionless Backfill.

1) Dry / Moist Backfill with no surcharge.
   - Considering element at depth z below ground surface.
   - When wall is at the point of moving outwards (i.e) away from fill, active state of plastic equilibrium is established.
   - Horizontal pressure σ_h is the min principal stress, σ_3 and the vertical pressure σ_v is the major principal stress σ_1.

$$\frac{\sigma_3}{\sigma_1} = \frac{\sigma_h}{\sigma_v} = \frac{1}{\tan^2(45° + \phi/2)} = \cot^2(45° + \phi/2)$$

σ_h = lateral earth pressure = p_a.

σ_v = Vertical pressure on the element = $\gamma . z$.

$\therefore\ p_a = \gamma z \cot^2(45 + \phi/2) = K_a \gamma z$

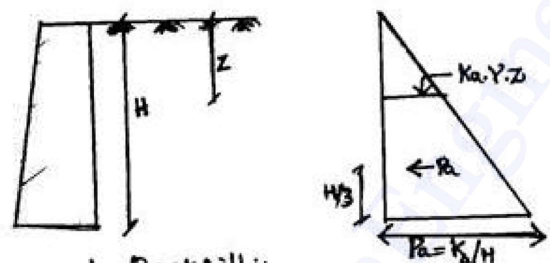
At $z = H$, earth pressure is $p_a = K_a . \gamma H$.

Total active pressure P_a or resultant pressure form a triangular pressure distribution diagram.

$$P_a = \frac{1}{2} K_a \gamma H^2 \text{ acting } H/3 \text{ above the base of wall}$$

γ = dry weight of soil if dry.

. if wet, moist weight of soil.

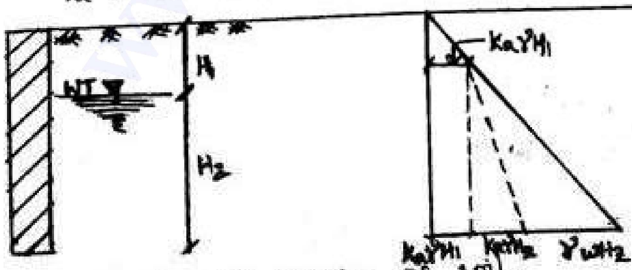


Submerged Backfill:-

If the backfill is submerged by water table at H_1 from top active earth pressure upto H_1 is using above equation. For submerged portion, its sum of earth pressure due to submerged unit weight of soil mass γ' and hydrostatic pressure.

$\therefore$ Lateral pressure intensity at the base of the wall is

$$P_a = K_a \gamma H_1 + K_a \gamma' H_2 + \gamma_w H_2.$$



γ' = submerged unit weight of soil.

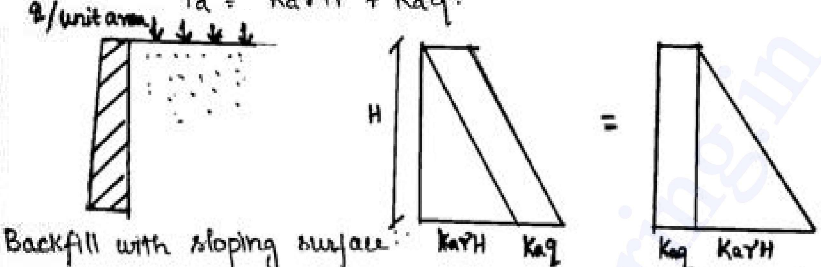
It is assumed that value of ϕ is same for submerged and moist soil.

3) Backfill with uniform surcharge :

- Backfill is horizontal and carries a surcharge of uniform intensity q per unit area, Increase in lateral pressure due to this is $K_a q$.
- Lateral pressure for any depth Z is $P_a = K_a \gamma z + K_a q$.

Pressure intensity at base of the wall is

$$P_a = K_a \gamma H + K_a q.$$



4. Backfill with sloping surface :

- Sloping surface behind the wall be inclined at an angle β with the horizontal. β is surcharge angle
- Assumption : Vertical and lateral stresses are conjugate

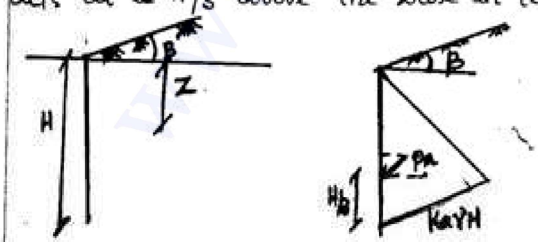
Let σ, p be conjugate stresses, σ - vertical, p - parallel to sloping backfill. $\frac{\sigma_1 - \sigma_3}{\sigma_1 + \sigma_3} = \sin\phi$.

$$P_a = K_a \gamma z \cos\beta .$$

$$K_A = \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

Active pressure for wall of height $H = \frac{1}{2} K_a \gamma H^2$.

acts at H/3 above the base in direction parallel to the surface

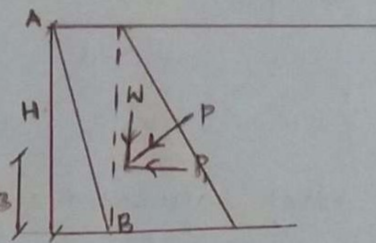


5. Inclined Back & surcharge.

- Inclined back supporting backfill with horizontal ground surface. Total pressure is resultant of horizontal pressure and of wedge ABC.

$P = \sqrt{P_1^2 + W^2}$

$P_1 = \frac{1}{2} K_a \gamma H^2$



Resultant of P is the vector sum of P_1 & W.

Active Earth Pressure of Cohesive Soils:-

- Principal stress relationship, $\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha$.
- At any depth z, $\sigma_1 = \gamma z$ and σ_3 = lateral pressure P_a.

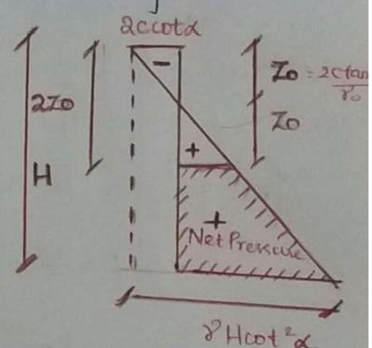
$$\gamma z = P_a \tan^2\alpha + 2c\tan\alpha. \qquad (\alpha = 45° + \phi/2)$$

$$P_a = \gamma z \cot^2\alpha - 2\cot\alpha . c$$

At $z = 0$, $\quad P_a = -2c\cot\alpha$.

When $P_a = 0$, $\quad z = z_0 = \frac{2c}{\gamma}\tan\alpha = \frac{2c}{\gamma}\frac{1}{\sqrt{K_a}}$.

∴ Negative pressure is developed at top level of retaining wall and decreases to 0 at a depth $z_0 = \frac{2c}{\gamma}\tan\alpha$.



For depth > z_0, Pressure P_a is positive.

At $z = H$, $P_a = \gamma H\cot^2\alpha - 2c\cot\alpha$.

If $c = 0$, pressure at the base $= \gamma H\cot^2\alpha$.

Total net pressure $P_a = \int_0^H P_a . dz = \int_0^H (\gamma z\cot^2\alpha - 2c\cot\alpha)dz$

$$P_a = \frac{1}{2}\gamma H^2\cot^2\alpha - 2cH\cot\alpha.$$

Critical height of unsupported Vertical cut in cohesive soil is

$$H_c = 2z_0 = \frac{4c}{\gamma}\tan\alpha.$$

Total lateral thrust on wall $= P_a \int_{z_0}^{H} \gamma z\cot^2\alpha - 2c\cot\alpha \, dz$.

$$= \frac{1}{2}\gamma(H^2 - z_0^2)\cot^2\alpha - 2c(H - z_0)\cot\alpha.$$

$$P_a = \frac{1}{2}\gamma H^2\cot^2\alpha - 2cH\cot\alpha + 2c^2/\gamma \quad \left(\text{where } z_0 = \frac{2c\tan\alpha}{\gamma}\right)$$

Backfill with surcharge:-

If backfill carries a surcharge of uniform intensity q/unit area lateral pressure is increased by $K_a q$. or $q\cot^2\alpha$.

$$P_a = \gamma z\cot^2\alpha - 2c\cot\alpha + q\cot^2\alpha. \qquad (\because \text{at depth } z_0 \; P_a = 0)$$

At $z_0 = 0$; $P_a = q\cot^2\alpha - 2c\cot\alpha$. $\quad \therefore z_0 = \frac{2c}{\gamma}\tan\alpha - q/\gamma$.

Submerged Backfill:

- Water table exists at a depth H_1 below the top of wall

$$P_a = \left[\gamma H_1 + \gamma' (z - H_1)\right]\cot^2\alpha - 2c\cot\alpha + \gamma_w (z - H_1)$$

Backfill of intact saturated clay:

It is assumed $\phi = \phi_u = 0$ immediately after construction

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 1.$$

$$p_a = \gamma_{sat} z - 2c_u \quad P_a = \frac{1}{2}\gamma_{sat}(H^2 - z_0^2) - 2c_u (H - z_0)$$

$$z_0 = 2c_u / \gamma_{sat}.$$

Passive Earth pressure:

1. cohesionless backfill:

- Lateral pressure is major principal stress and vertical pressure is minor principal stress.

$$\sigma_h = p_p = \sigma_1 \text{ and } \sigma_v = \sigma_3 = \gamma z$$

Principal stress relationship : $\sigma_1 = \sigma_3 \tan^2\alpha$

$$p_p = \gamma z \tan^2\alpha = K_p \gamma z$$

p_p = passive earth pressure intensity. K_p = Rankine's coefficient of passive earth pressure.

$$K_p = \tan^2\alpha = N_\phi = \frac{1 + \sin\phi}{1 - \sin\phi} = \frac{1}{K_a}.$$

$$\frac{K_p}{K_a} = \tan^4(45° + \phi/2) \quad \text{if } \phi = 30° \text{ then } K_p = 9K_a.$$

∴ Passive earth pressure distribution is triangular with maximum value of $K_p\gamma H$ at base of retaining wall of height H. Total pressure P_p for depth H is $P_p = \int_0^H K_p \gamma z \, dz = \frac{1}{2} K_p \gamma H^2$.

i) Uniform surcharge of intensity q / unit area acts:

Passive pressure intensity at depth z $p_p = K_p(\gamma z + q)$.

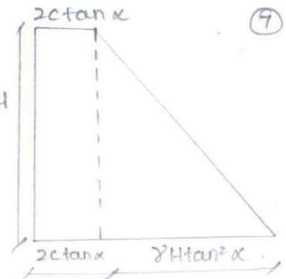
ii) Backfill having top surface inclined at angle β,

$$p_p = K_p \gamma z \; ; \; K_p = \cos\beta \cdot \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}$$

Passive Earth Pressure - cohesive Backfill ::

- Principal stress relationship at failure is

$$\sigma_1 = \sigma_3 \tan^2\alpha + 2c\tan\alpha .$$



$\sigma_1 = \sigma_n = p_p$ and $\sigma_3 = \sigma_v = \gamma z$.

$$\therefore \quad p_p = \gamma z \tan^2\alpha + 2c\tan\alpha .$$

$$p_p = \gamma z N_\phi + 2c\sqrt{N_\phi}$$

$$z = 0 \text{ and } p_p = 2c\tan\alpha .$$

At $z = H$, $p_p = \gamma H \tan^2\alpha + 2c\tan\alpha$.

Total pressure $P_p = \int_0^H p_p \cdot dz = \frac{1}{2}\gamma H^2 \tan^2\alpha + 2cH\tan\alpha$.

$$= \frac{1}{2}\gamma H^2 N_\phi + 2cH\sqrt{N_\phi} .$$

2.Explain with neat sketch the culmann's method of calculating active earth pressure. (Nov/Dec2015), (May/June 2016), (Nov/Dec2012)

CULMANN'S GRAPHICAL METHOD FOR ACTIVE PRESSURE:

Culmann (1866) also gave a graphical solution to evaluate the active pressure and can be conveniently used for ground surface of any shape, for various types of surcharging loads, and for a layered backfill of different densities.

PROCEDURE:

1. Draw the ground line ₵ line and the ψ line as usual
2. Take a slip plane BC1 .calculate the weight of the wedge ABC1and plot it as BE1to some scale on the ₵ line.
3. Through E1, draw E1,F1 parallel to the line ψ, to cut the slip plane BC_2 IN F_1.
4. Similarly take another slip plane BC_2 , calculate the weight of wedge ABC_2 and plot it as BE_2 on the line. Draw E_2F_2 parallel to the line cut the slip plane BC_2 in F_2
5. Take number of such slip planes BC_3, BC4. Plot the weight of the corresponding wedge s on the ψ line and obtain point's f3, f4.
6. Draw a smooth curve through points B, F1, F2, F3, F4 etc. This curve is known as the culmann's line.
7. Draw a tangent to the culmann's line parallel to the ₵ line .the maximum value of the earth pressure is represented by the intercept EF, on the adopted scale. EF

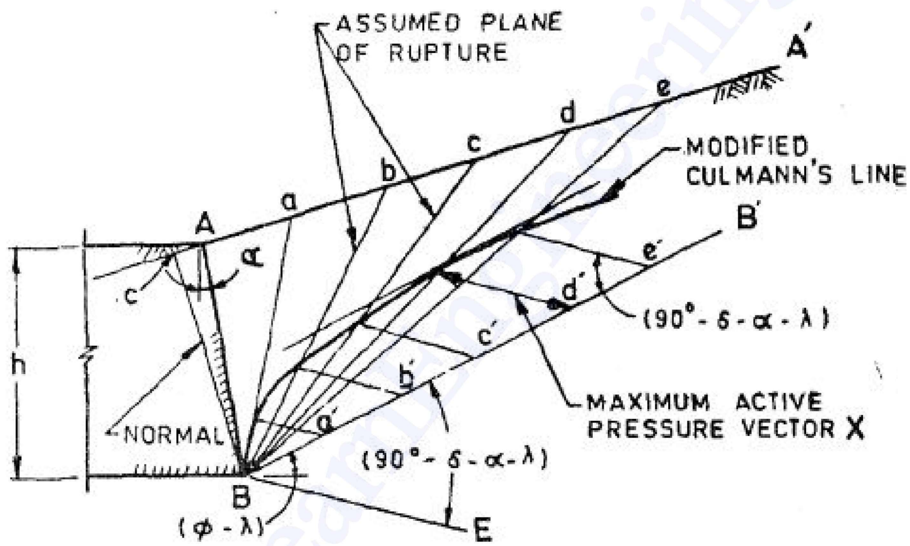
being drawn through the points of tangency parallel to the line ψ line. BFC represents the critical slip plane.

8. To locate the points of application of the resultant pressure, draw a line parallel to the critical slip plane BC, through the centre of gravity of the sliding wedge ABC and obtain its intersection on the back AB.

When the ground line is a plane, the weights of the wedges ABC1, AC1 (=L3), etc. since the height of soil wedge is constant being equal to H1, Hence the weights of these wedges are plotted as their base lengths L1, L2, L3, etc. on the ₵ line.

P_a=1/2ṛH_1(EF)

If the backfill also carries a surcharge of intensity q, ṛ1



Determination of Active Earth Pressure by Graphical Method

EFFECT OF LINE LOAD:

Culmann's graphical method can also be used to take into account the running parallel to the retaining wall. A line load of intensity q per unit length, acting at a point C1, distant from the top of the wall. BEF1, F_n shows the culmanns line and BC is the failure plane in absence of the line load. Let w1 be the weight of the wedge ABC1 which is plotted as BE1 on the line ₵ and point F1 is obtained if there were no line load. However when the line load is there the weight of the wedge ABC1 increases by q. thus BE represents and a point change in the culmann line the change being proportional to q. for all other failure wedges to the right , the weight q is added to the weight of the wedge and then plotted on the ₵ line. The modified culmann's line is thus represents by BFF_1FF_n. when the slip plane is BC the pressure on the wall is represented by EF and when the slip plane is BC1, the pressure is represented by E,aF. if E^IF^I<EF slip occurs along BC^I and the pressure on the wall is increased

The culmann line BFF_2 is plotted by ignoring the line load .the modified culmann line $BF^IF^I_2$ is then plotted by taking into account the line load, when the load q is added to the weight of each soil wedge considered. By drawing tangents to two culmann's lines parallel to ₵ line, intercepts FE and F^IE^I are obtained. The intercept E^IF gives the greatest value of pressure due to backfill acted upon by q, whereas FE gives the maximum pressure in the absence of the line load. If the tangent at F is prolonged to meet the modified culmann line in F^I_2 the intercept $E^I_2F^I_2$ equals to FE. This means that if the line is placed beyond C_2, there is no effect of the line load on the pressure .for the other plotted. it will be seen that is maximum when the load is just at face of the wall , it remains constant with the position of q up to point c1 and then decreases gradually to zero at C_2. For load positions beyond C_2 .the pressure on the wall is not due to q. This method is very much used in locating the position of the railway line or the footing of building on the backfill at such a safe distance that the earth pressure on the (existing) wall does not increase.

3.Explain the coulomb's Wedge theory of earth pressure with a neat sketch. (May/June 2014), (May/June 2013)

COULOMB'S WEDGE THEORY.

- Coulomb considered the equilibrium of whole of the material supported by a retaining wall when the wall is on the point of moving slightly away from the filling.
- In the case of active earth pressure, the sliding wedge moves downwards and outwards on slip surface.
- In case of passive earth pressure, the sliding wedge moves upwards and inwards on slip surface.

- The pressure on the wall is a force of reaction to keep the sliding wedge in equilibrium.
- Factors such as well friction, irregular soil surfaces and different soil strata can be taken into account in this method.

Following are the basic **Assumptions** of the wedge theory:

1. The backfill is dry, cohesionless, homogenous, isotopic and elastically underformable but breakable.
2. The slip surface is plane which passess through the heel of the wall.
3. The sliding wedge itself acts as a rigid body and the value of earth pressure is obtained by considering the limiting equilibrium of the sliding wedge as a whole.
4. The position and direction of the resultant earth pressure are known.
5. The resultant pressure acts on the back of the wall at one-third the height of the wall from the base and is inclined at an angle δ (called the angle of wall friction)

The forces acting on a wedge of soil are:

(i) Its weight W,

(ii) The reaction R along the plane of sliding

(iii) Active thrust Pa against the retaining wall. R will act at an angle Ø to the normal of the plane of sliding. The pressure P is inclined at an angle of wall friction δ to the normal which is considered positive. Both R and P will be inclined in a direction so as to oppose the movement of the wedge.

Condition for maximum pressure from a sliding wedge.

- BD shows a plane inclined at an angle ϕ to the horizontal at which the soil is expected to stay in the absence of any lateral support.
- The line BD, therefore, is called the natural slope line, repose line or the Ø – line. AD, inclined at β to the horizontal, is called the ground line or surcharge line.
- Plane BC, inclined at angle λ (to be determined) is the line or rupture plane or slip plane; the angle λ is called the critical slip angle.
- The reaction R inclined at an angle ϕ to the normal to the slip line; R is also inclined at an angle (λ-ϕ) to the vertical.
- The wall reaction P_a is inclined at an angle   to the normal to the wall.

- The inclination of P_a to vertical is represented by angle $\psi = 90^\circ - \theta - \delta$ (= constant for given value of θ and δ).
- The value of P_a depends upon the slip angle λ. P_a is zero when $\lambda = \phi$. As λ increases beyond ϕ, P also increases and after reaching a maximum value it again reduces to zero when λ equals $90 + \theta$. Thus, the critical slip plane lies between the line and back of the wall.

In order to derive the condition for maximum active pressure P_a from the sliding wedge, draw line CE at an angle ψ to the ϕ –line. Let x and n be the perpendicular distance of points C and A from the ϕ -line, and m be the length of line BD. It will be seen triangle BCE and the force triangle similar.

Hence $\frac{Pa}{W} = \frac{CE}{BE}$ ———————————— (1)

Now, CE = x cosec ψ = A_1 x (where A_1 = cosec ψ = constant)

BE = BD – (DF - FE) = m – x {cot (ϕ – β) – cot ψ} = m – A_2 x

A_2 = [cot (ϕ – β) – cot ψ] = constant.

W = $\gamma(\Delta ABD) = \gamma(\Delta ABD - \Delta BCD) = 0.5\, \gamma\, m\, (n - x)$

Subsituting the value of CE, BE and W in (1), we get

$P_a = 0.5m\, A_1 \frac{nx - x2}{m - A2x}$ ———————————— (2)

In the above expression x is the only variable which depends upon the position of slip plane BC. For maxima $dP_a/dx = 0$

$(n-2x)(m- A_2\, x) = - A_2\, (n\, x - x^2)$

$mn - mx = mx - A_2\, x^2 = x\, (m - A_2\, x)$

$\Delta ABC = \Delta BCE$

Thus the criterion for maximum active pressure is that the slip plane is so chosen that ΔABC and ΔBCE are equal in area.

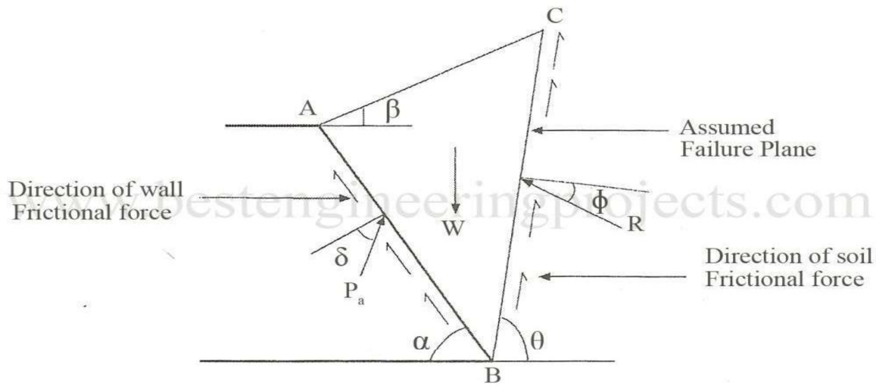


Fig 1 Coulomb's Active Earth Pressure

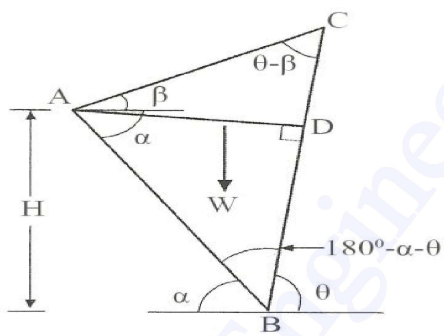
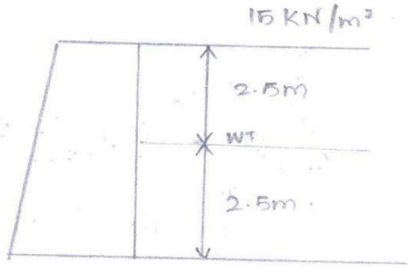


Fig 2 Weight of Wedge ABC

4. For retaining wall shown below, draw the earth pressure diagram, for the active case and find the total active earth pressure per unit length of the wall and point of application from the base of the wall. $\gamma_w = 10\ KN/m^3$. (April/May 2011)



$\gamma = 17\ KN/m^3$ $\phi = 30°$ $c = 0$

$\gamma_{sat} = 18\ KN/m^3$
$\phi = 38°$ $c = 0$

Solution:

$$K_a = \frac{1 - \sin 30°}{1 + \sin 3°} = \frac{1}{3} \text{ (dry soil)}$$

$$K_a = \frac{1 - \sin 38°}{1 + \sin 38°} = 0.236 \text{ (saturated soil)}$$

$$P_1 = K_a q = \frac{1}{3} \times 15 = 5\ KN/m^2.$$

$$P_2 = K_a \gamma_d H_1 = \frac{1}{3} \times 7 \times 2.5$$

$$= 14.167\ KN/m^2.$$

$$P_3 = P_2 = 14.167\ KN/m^2.$$

$$P_5 = \gamma_w H_2 = 10 \times 2.5 = 25\ KN/m^2.$$

$$P_1 = 5 \times 5 = 25\ KN/m.$$

Acting at $Z_1 = 5/2 = 2.5\ m$ (above base)

$$P_2 = 1/2 \times 14.167 \times 2.5 = 17.708\ kN/m.$$

Acting at $Z_2 = 2.5 + \frac{2.5}{3} = 3.3$ above base.

$P_3 = 14.167 \times 2.5 = 35.42$ KN/m.

Acting at $Z_3 = \frac{2.5}{2} = 1.25$ m above base.

$P_4 = \frac{1}{2} \times 4.72 \times 2.5 = 5.9$ KN/m.

Acting at $Z_4 = \frac{2.5}{3} = 0.83$ m above base.

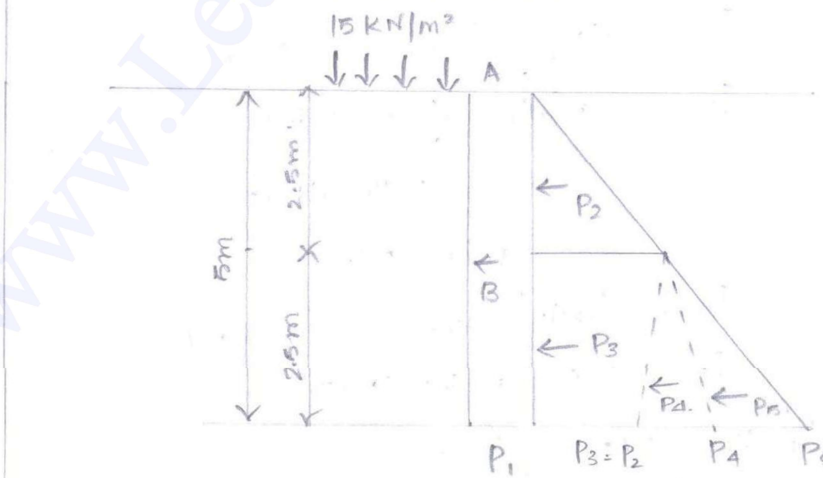
$P_5 = \frac{1}{2} \times 2.5 = 12.5$ KN/m.

Acting at $Z_5 = \frac{2.5}{3} = 0.83$ m above base.

$\therefore$ Total $P = 25 + 17.708 + 35.42 + 5.9 + 12.5$.

$$Z = \frac{2.5 \times 2.5 + 17.708 \times 3.3 + 35.42 \times 1.25 + 5.9 \times 0.83 + 12.5 \times 0.83}{96.528}$$

$Z = 1.88$ m above base.



5. A retaining wall 4 m high support a back fill (c = 20) kN/m²; $\varphi = 30°$, $\gamma = 20\ kN/m^3$ with horizontal top flush with the top of the wall. The backfill carries a surcharge of 20 kN/m². If the wall is pushed towards the backfill compute the total passive pressure on the wall, and its point of application. (Nov/Dec, 2012)

Solution:

$$C = 2\ kN/m^2.$$

$$\varphi = 30°.$$

$$KP = \varphi = \tan^2(45 + \varphi/2)$$

$$= \tan^2 60° = 3.$$

Passive pressure intensity due to surcharge

$$P_1 = kp \cdot q = N_\varphi q$$

$$= 3 \times q = 3 \times 20$$

$$= 60\ kN/m^2.$$

Passive pressure intensity due to backfill,

$$P_1 = 2c\sqrt{N\varphi} + \gamma H N\varphi = P_2 + P_3.$$

$$P_2 = 2c\sqrt{N\varphi} + 2 \times 20\sqrt{3} = 69.28\ kN/m^2.$$

$$P_3 = \gamma H N\phi = 20 \times 4 \times 3 = 240\ kN/m^2$$

$$P_1 = H(q \cdot N\varphi) = 4 \times 60 = 240\ kN/m.$$

acting 200 m above base.

$$P_2 = H(2c \cdot \sqrt{N\varphi}) = 4 \times 69.28$$

$= 277.1$ kN/m, acting 2m above base.

$$P_3 = \frac{1}{2} M(\gamma H N_\phi) = \frac{1}{2} \times 4 \times 240 = 480 \text{ KN/m}.$$

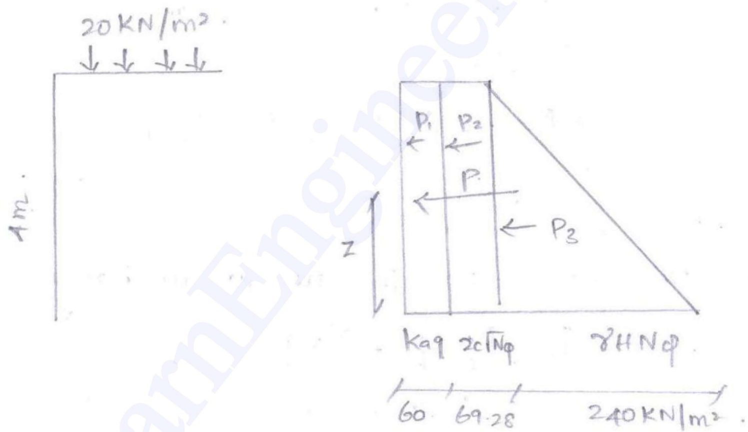
acting 4/3 m above base.

$$P = P_1 + P_2 + P_3 = 997.1 \text{ KN/m}.$$

Point of application above base is given by,

$$\bar{z} = \frac{(240 \times 2) + (277.1 \times 2) + (480 \times 4/3)}{997.1}$$

$$\bar{z} = 1.68 \text{ m}.$$



6. A smooth wall of 6m high retains sand. In the loose state the sand has a void ratio of 0.76 and angle of internal friction of 28°, while in the dense state, the corresponding values are 0.48 and 42° respectively. Find the ratio of active and passive earth pressure at the base in the two cases. Assume G of solids as 2.7. (May/June 2012)

Data:

$h = 6m$.

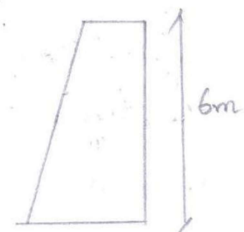
Loose state $e = 0.76$, $\phi = 28°$.

Dense state $e = 0.48$, $\phi = 42°$.

$G = 2.7$

$\gamma_w = 9.81 \, KN/m^3$.

Solution:



i) Unit Weight:

Loose state $\gamma_d = \dfrac{G\gamma_w}{1+e}$

$= \dfrac{2.7 \times 9.81}{1+0.76}$.

$\gamma_d = 15.05 \, KN/m^3$.

Dense state $\gamma_d = G\gamma_w / 1+e$.

$= 2.7 \times 9.81 / 1+0.48$.

$= 17.90 \, KN/m^3$.

ii) Lateral earth pressure coefficient:

Loose state, $K_a = 1 - \sin\phi / 1+\sin\phi$.

$= 1 - \sin 28° / 1+\sin 28°$.

$= 0.36$.

$K_p = 1/K_a$

$= 2.77$.

Dense state,

$$K_a = \frac{1-\sin 42^\circ}{1+\sin 42^\circ}.$$

$$K_a = 0.20.$$

$$K_p = 1/K_a = 5.$$

Active earth Pressure at base,

Loose state $P_a = K_a \cdot \gamma \cdot H.$

$$= 0.36 \times 15.05 \times 6.$$

$$P_a = 32.51 \text{ KN/m}^2.$$

Dense state $P_a = K_a \cdot \gamma \cdot H.$

$$= 0.2 \times 17.9 \times 6.$$

$$= 21.48 \text{ KN/m}^2.$$

Passive Pressure at Base:

Loose state: $P_p = K_p \cdot \gamma \cdot H.$

$$= 2.77 \times 15.05 \times 6.$$

$$= 250.13 \text{ KN/m}^2.$$

Dense state, $P_p = K_p \cdot \gamma \cdot H.$

$$= 5 \times 17.90 \times 6$$

$$= 537 \text{ KN/m}^2.$$

Ratio of active & passive earth pressure:

Loosestate: $\frac{P_a}{P_p} = \frac{32.51}{250.13} = \frac{1}{7.7} = 1:7.7.$

Dense state $\frac{P_a}{P_p} = \frac{21.48}{537} = \frac{1}{25} = 1:25.$

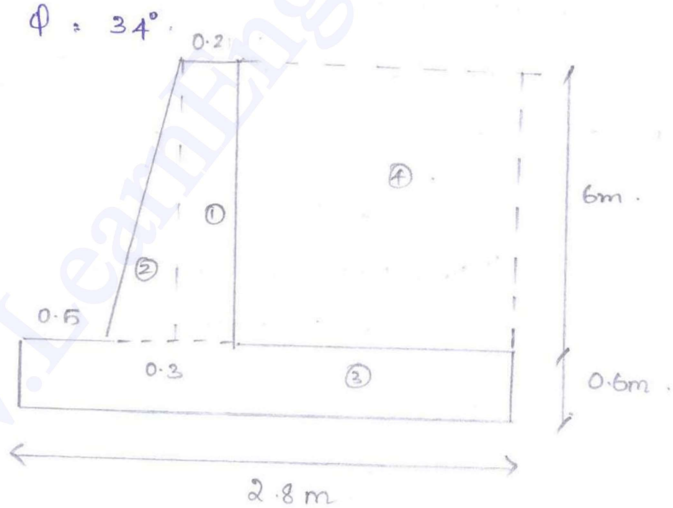
7. Check the stability of a cantilever retaining wall of smooth vertical back of 6m height and 0.2m thick at top and 0.3m at bottom. Foundation base of retaining wall of depth 0.6m projected on left side as 0.5m and 2m on right side. It supports a sandy backfill with unit weight 18 kN/m³ levelled to the top of wall. The angle of internal friction of soil is 34°. Use Rankine theory. [Nov/Dec 2013].

Data:

$\gamma = 18\ kN/m^3$.

$h = 6m$.

$\phi = 34°$.



1) Determination of lateral earth pressure:

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 34°}{1 + \sin 34°}$$

$$= 0.28.$$

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

$= \frac{1}{2} \times 0.28 \times 18 \times 6^2$.

$= 90.72\ kN/m$.

acts at a height of $h/3$ from base.

$Z = 2.2m$ from base.

ii) Determination of vertical forces:

Rectangular component ①

$W_1 = 0.2 \times 6 \times 24 = 28.8\ kN/m$.

acts at a distance, $= 0.5 + 0.1 + (0.2/2)$

$= 0.7m$ from toe.

∴ Moment $= 28.8 \times 0.7 = 20.16\ KN$.

Triangular component ②.

$W_2 = \frac{1}{2} \times 0.1 \times 6 \times 24 = 7.2 kN/m$.

acts at a distance $= 0.5 + \frac{2}{3}(0.1)$

$= 0.57\ m$.

Moment $= 7.2 \times 0.57$

$= 4.10\ KN$.

Base ③ $W_3 = 2.8 \times 0.6 \times 24$

$= 40.32\ kN/m$.

acts @ 1.4m from toe.

Moment $= 40.32 \times 1.4$

$= 56.45$.

Weight due to surcharge ④.

$W_4 = 2 \times 6 \times 18 = 216\ kN/m$.

acts at $0.5 + 0.3 + 1 = 1.8$ m from toe.

Moment $= 216 \times 1.8 = 388.8$.

$\Sigma V = 28.8 + 7.2 + 40.32 + 216$

$= 292.32$ KN.

$\Sigma M_v = 20.16 + 4.10 + 56.45 + 388.8$

$= 469.51$ KN.m.

$$\bar{x} = \frac{\Sigma M}{\Sigma V} = \frac{469.51}{292.32}$$

$= 1.6$ m.

$e = b/2 - \bar{x}$

$= 1.4 - 1.6$

$= -0.20$ m.

For no tension, $e < b/6$.

$0.2 < 2.8/6$

$0.2 < 0.47$.

Hence safe.

Factor of safety against sliding:

Assume $\mu = 0.45$.

R_h = horizontal force.

$F_s = \mu \cdot R_v / R_h$.

$$= 0.45 \times \frac{292.32}{90.72}$$

$F_s = 1.45$.

Fos against overturning:

$$F_o = \frac{\text{Resisting moment}}{\text{Over turning moment}}$$

$$= \frac{469.51}{90.72 \times 2.2}$$

$$= 2.35$$

$FOS = 2.35 > 1.5$

Hence Safe.