PIE Tech

POLLACHI INSTITUTE OF ENGINEERING AND TECHNOLOGY

(Approved by AICTE and Affiliated to Anna University)

sky is the limit

Department of Civil Engineering

Regulation 2021

III Year – V Semester

CE3503- Foundation Engineering

CE3503 **FOUNDATION ENGINEERING** L T P C
3 0 0 3

COURSE OBJECTIVE:

- To impart knowledge to plan and execute a detail site investigation programme, to select geotechnical design parameters and type of foundations. Also to familiarize the students for the geotechnical design of different type of foundations and retaining walls

UNIT I SITE INVESTIGATION AND SELECTION OF FOUNDATION 9

Scope and objectives – Methods of exploration – Auguring and boring – Wash boring and rotary drilling – Depth and spacing of bore holes – Soil samples – Representative and undisturbed – Sampling methods – Split spoon sampler, Thin wall sampler, Stationary piston sampler – Penetration tests (SPT and SCPT) – Data interpretation - Strength parameters and Evaluation of Liquefaction potential - Selection of foundation based on soil condition- Bore log report.

UNIT II BEARING CAPACITY OF SHALLOW FOUNDATION 9

Introduction – Location and depth of foundation – Codal provisions – Bearing capacity of shallow foundation on homogeneous deposits – Terzaghi's formula and BIS formula – Factors affecting bearing capacity – Bearing capacity from in-situ tests (SPT, SCPT and plate load) – Allowable bearing pressure – Seismic considerations in bearing capacity evaluation. Determination of Settlement of foundations on granular and clay deposits – Total and differential settlement – Allowable settlements – Codal provision – Methods of minimizing total and differential settlements.

UNIT III FOOTINGS AND RAFTS 9

Types of Isolated footing, Combined footing, Mat foundation – Contact pressure and settlement distribution – Proportioning of foundations for conventional rigid behaviour – Minimum depth for rigid behaviour – Applications – Floating foundation – Special foundations – Seismic force consideration – Codal provision

UNIT IV PILE FOUNDATION 9

Types of piles and their functions – Factors influencing the selection of pile – Carrying capacity of single pile in granular and cohesive soil – Static formula – Dynamic formulae (Engineering news and Hileys) – Capacity from insitu tests (SPT, SCPT) – Negative skin friction – Uplift capacity- Group capacity by different methods (Field's rule, Converse – Labarra formula and block failure criterion) – Settlement of pile groups – Interpretation of pile load test (routine test only), Under reamed piles – Capacity under compression and uplift – Codal provision.

UNIT V RETAINING WALLS 9

Plastic equilibrium in soils – Active and passive states – Rankine's theory – Cohesionless and cohesive soil – Coulomb's wedge theory – Condition for critical failure plane – Earth pressure on retaining walls of simple configurations – Culmann Graphical method – Pressure on the wall due to line load – Stability analysis of retaining walls – Codal provision.

TOTAL: 45 PERIODS

COURSE OUTCOMES:

- On completion of the course, the student is expected to be able to

CO1 Graduate will demonstrate an ability to plan and execute a detailed site investigation to select geotechnical design parameters and type of foundation

CO2 Graduate will demonstrate an ability to design shallow foundations, its component or process as per the needs and specifications.

CO3 Graduate will demonstrate an ability to design combined footings and raft foundations, its component or process as per the needs and specifications.

CO4 Graduate will demonstrate an ability to design deep foundations, its component or process as per the needs and specifications.

CO5 Graduate will demonstrate an ability to design retaining walls, its component or process as per the needs and specifications.

TEXTBOOKS:

1. Murthy, V.N.S., "Soil Mechanics and Foundation Engineering", CBS Publishers and Distributers Ltd., New Delhi, 2015

FOUNDATION ENGINEERING

UNIT-1 : SITE INVESTIGATION AND SELECTION OF FOUNDATION

Scope and objectives - Methods of exploration - Auguring and boring - Wash boring and rotary drilling - Depth and spacing of bore holes - Soil samples - Representative and Undisturbed - Sampling methods - Split spoon sampler, Thin wall sampler, Stationary piston sampler - Penetration test (SPT and SCPT) - Data interpretation - Strength parameters and Evaluation of Liquefaction potential - Selection of foundation based on soil condition - Bore log report.

Scope of Site Investigation:

It refers to the procedure of determining surface and subsurface condition in the area of proposed construction. The purpose of site investigation is to get clear information about the soil and hydrological conditions at the site.

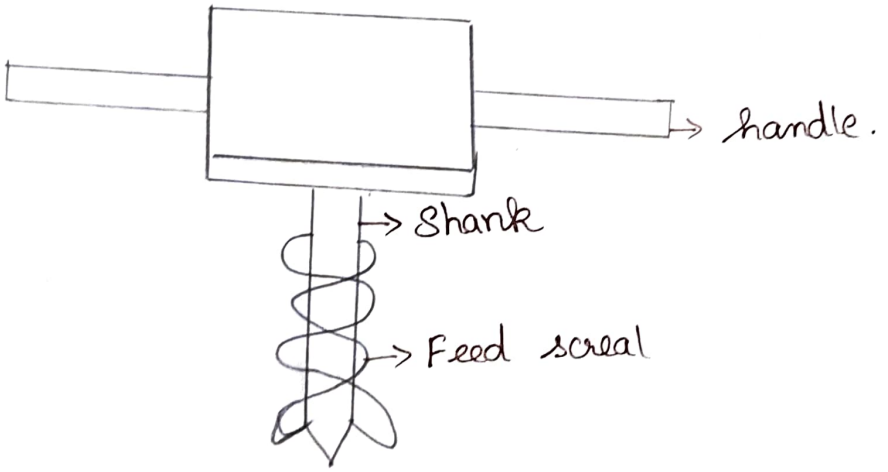
Objectives of Site Investigation :

* To access the general suitability of the site.
* To achieve safe and economical design of foundation and temporary works.
* To know the nature of each stratum and engineering properties of the soil and rock, which may affect the design and mode of construction of proposed structure and foundation.
* To find out the source of construction material and selection of sites for disposal of water or surplus material.
* To ensure the safety of surrounding existing structures.
* To design for the failed structures or remedial measures for the structures deemed to be unsafe.
* To locate the ground water level and possible corrosive effect of soil and water on foundation material.

Methods of Exploration:

* Direct method → Open Excavation (1.2 x 1.2m)
* Semi direct method → Boring methods
  - ↳ Anchor boring
  - ↳ Wash boring
  - ↳ Rotary boring
* Indirect method → Penetration test (SPT, SCPT, DCPT).
  - ↓
  - Geophysical method
    - Electrical resistivity
    - Seismic refraction

⇒ Anchor Boring:



It cannot be used effeciently soils, rocks and the soil containing obstraction.

Advantages of Anchor Boring:

* It is generally used in soils which can stay open without casing or filling mud clay, silt and partially saturated sand.

* Useful for subsurface investigation of highways, railways and airfields and depth of exploration is very small.

* Investigation are done very rapidly and economically.

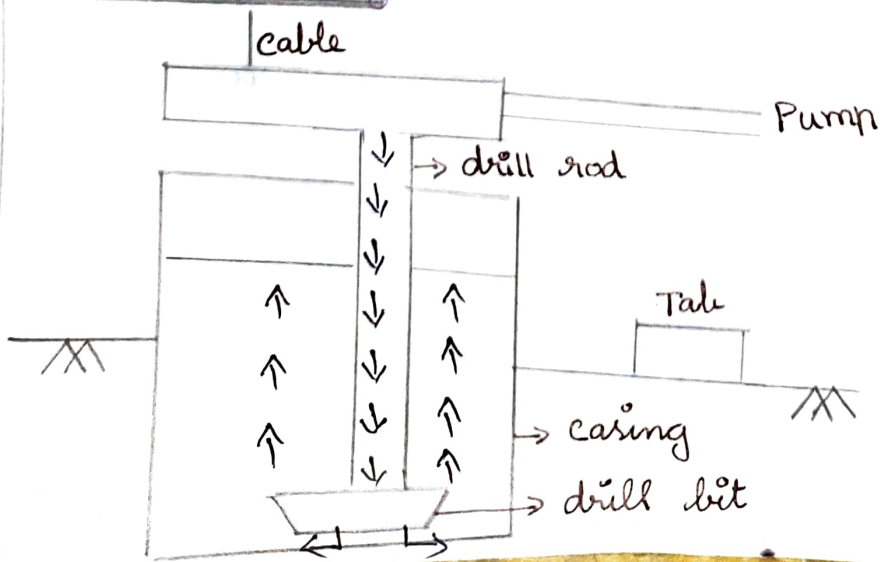
Disadvantages of Anchor Boring:

* The main advantages of soil samples are it is highly distributed.

It becomes difficult to exact change in soil extract.

It cannot be used when there are obstructions prevents drilling of holes.

⇒ Wash Boring:



ROTARY DRILLING:

Rotary boring or rotary drilling is useful if the soil highly resistance to augering or wash boring.

The method can be outside used also can be in case of sands and clay.

This method boring is effected by the the cutting action of rotating bit which is kept in the firm conduct with the bottom of the whole.

The bit is attached to the lower end of hollow drill rod which is rotated by a suitable chuck.

Drilling mull is continuously forced below down the hollow drill rods. The mud returing upwards through the annular space between the drill rods and the

side of the hole brings the cutting to the surface. The method is also known as Mud rotary drilling.

Core barrels with rotary drilling are enable to simelteneously obtaining of rock cores. This method is known as Core drilling. For large diameter hole (Cover 150mm), short drill is used.

The system is different from other types of core drilling because the coarser cutting do not return to the surface but are accumulated in a chip immediately above the bit. Chilled short is used as an abrasive instead of the drilling head.

Advantages of Wash Boring:

* The wash boring is used for advancing a hole in the ground.
* The equipment used in the wash boring is releatively light and inexpensive.

Disadvantages of Wash Boring:

* The wash samples collected in the tube and does not represent soil in its true condition, there is a complete breakdown of particles by chopping actions.
* It is slow in stuff soils and coarse grain soils.

Soil samples and types:

i) Distributed soil sample
ii) Undistributed soil sample.

i) Disturbed soil sample:

It contains the different particle size distrubution as in the insitustratum from which it is collected.

The soil structure may be seriously disturbed. The water contact also have changed.

Disturbed samples can be used for identification of determining alterberg limit test,

specific gravity of soil, organic and carbon content and compaction test.

ii) Undisturbed Sample:

It is the one that preserves the particle size distribution as well as the soil structure of insitu stratum.

The moisture content is also tried to be preserved to its original insitu value.

The undisturbed samples are used to determine the shead strength and consolidation characterstics.

Depth and Spacing of bore holes:

Depth: The depth of exploration depends on the types of structures, intensity of loading, shape and disposition of loaded area, soil propile and physical properties of the soil.

* It is not possible to fix the no. and depth of borings without making a few preliminary boring at the site.

Spacing (or) Lateral extent of exploration:

* Mainly depends upon the variation of strata in horizontal direction.

* For small and less imp building one bore or a trial pit in the centre is sufficiant.

* For compact buildings, covering an areas of about 0.4 hecters, there should be 5 bore holes. one at the centre and four near the corners.

* For large multistoryed building the bore holes should be made at all the corners and also at imp locations. The spacing between the bore hole is 10-30 m depending upon the subsurface conditions and loading.

* For highways carried out doing the proposed centre line at 150-300 m. spacing.

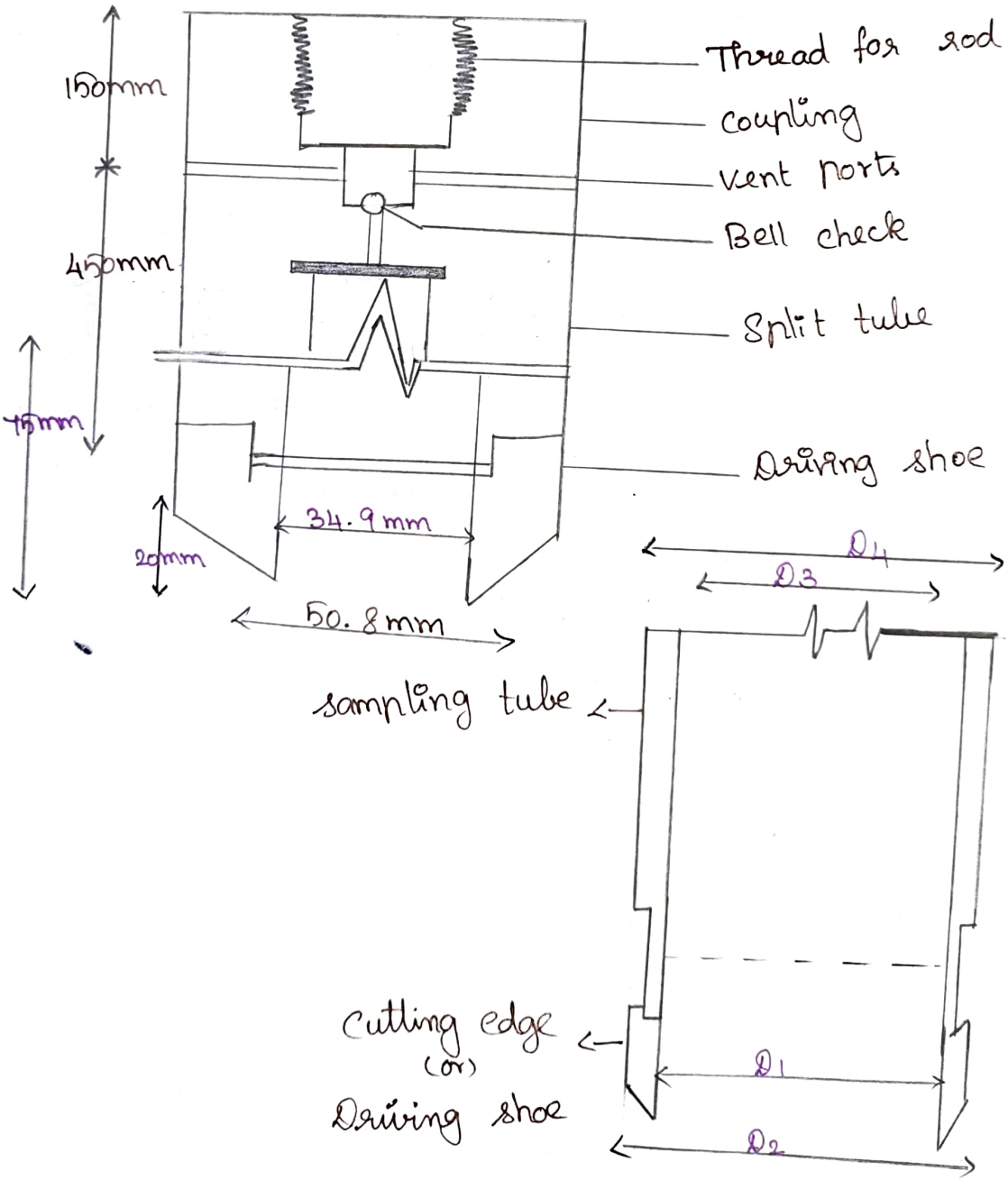
Incase of concrete dams, the spacing should be 40-80 m

Sampling Methods:

Types of Samplers:

1. Open drive samplers
2. Split spoon samplers
3. Thin walled samplers
4. Piston samplers.
5. Rotary samplers
6. Hard cursed samplers.

→ Split spoon Samplers:



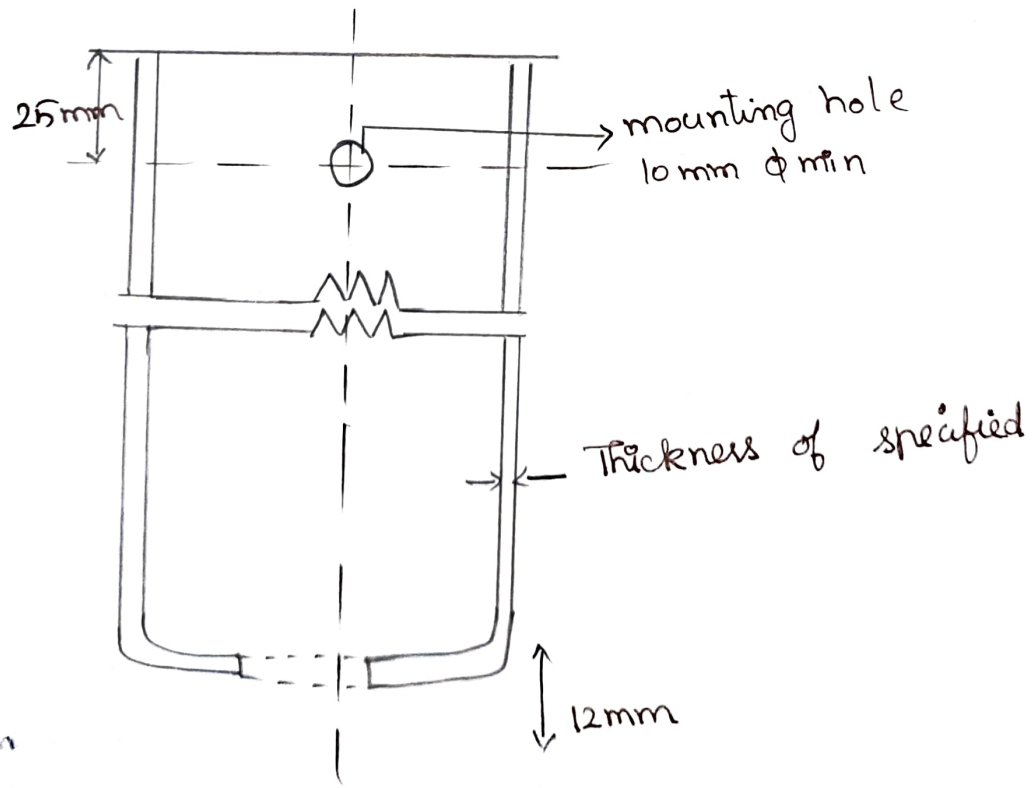
It has a drive shoe serves as a cutting edge. It is 34.9mm internal dia & 15.9 mm ext. dia.

The samplers are collected to the bottom of the bore hole by attaching it to the drill rod.

The samplers is driven into the soil by blows of the hammer.

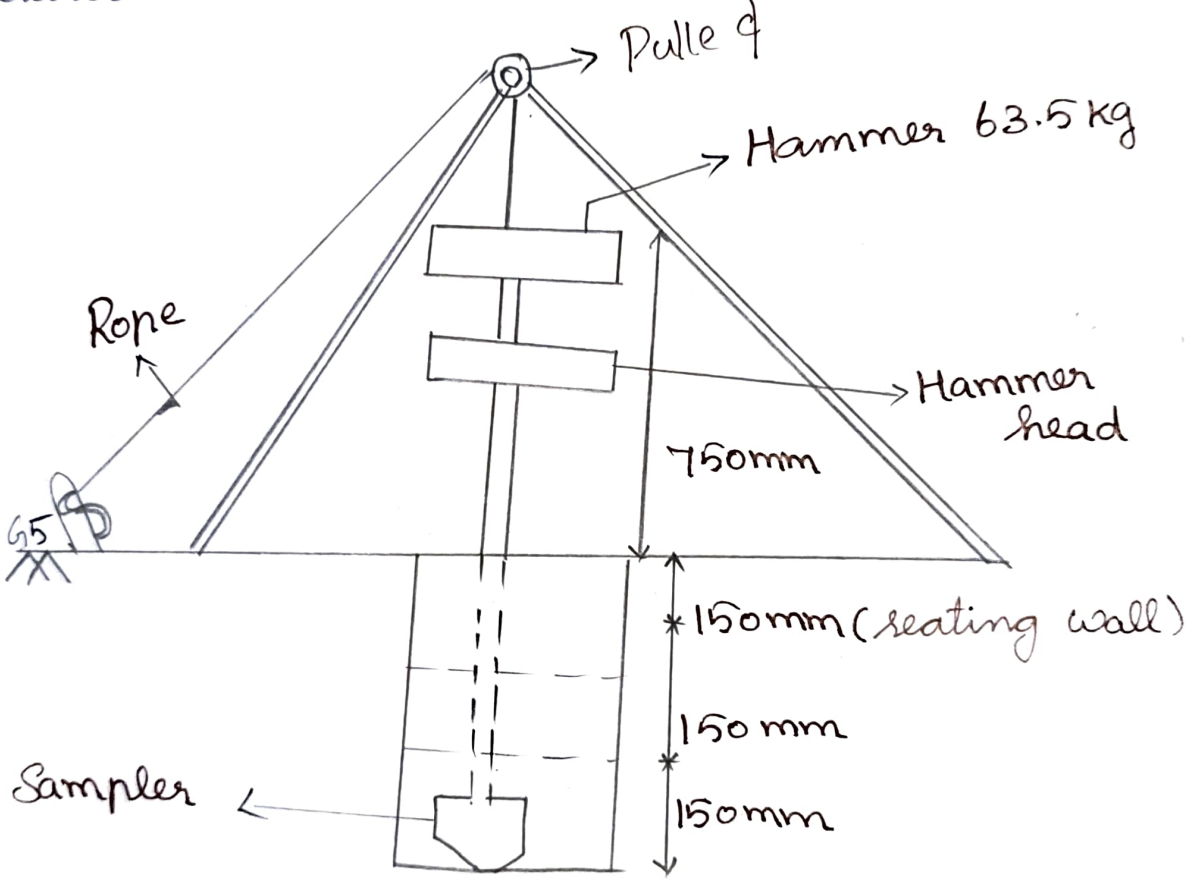
The samplers is extracted and two halfs of the panel are seperated and the sample is served in a glass jar.

→ Thin Walled Sampler:



The sampling tube is made of steel. The lower end is reduced to form cutting edge, this taper reduces hole jection the extraction of sample is same as top split spoon sampler.

Standard Penetration Test.



The standard code for this test is IS2131 : 1981.

This test is meant for cohelonless soil.

It can able to find density index, angle of shearing resistance and standard penetration no'n)

This test is conducted out in bore hole while DCPT & SCPT are carried out without a bore hole.

Procedures:

The bore hole is advanced to the required depth and the bottom is clean.

The split spoon sampler attached to the std drill rod of required length is lowered into the bore hole and rested at bottom.

The split spoon sample is driven into the soil for a depth of 450mm by blows of a drop hammer of 63.5 kg falling vertically and freely to form the height of 750mm.

The no. of blows required to penetrate every 150mm is recorded while driving.

The no. of blows required to drive a sampler into the ground surface for 300mm beyond the seating grade of 150mm must be recorded and this is called as Std penetration number 'N.

Precautions:

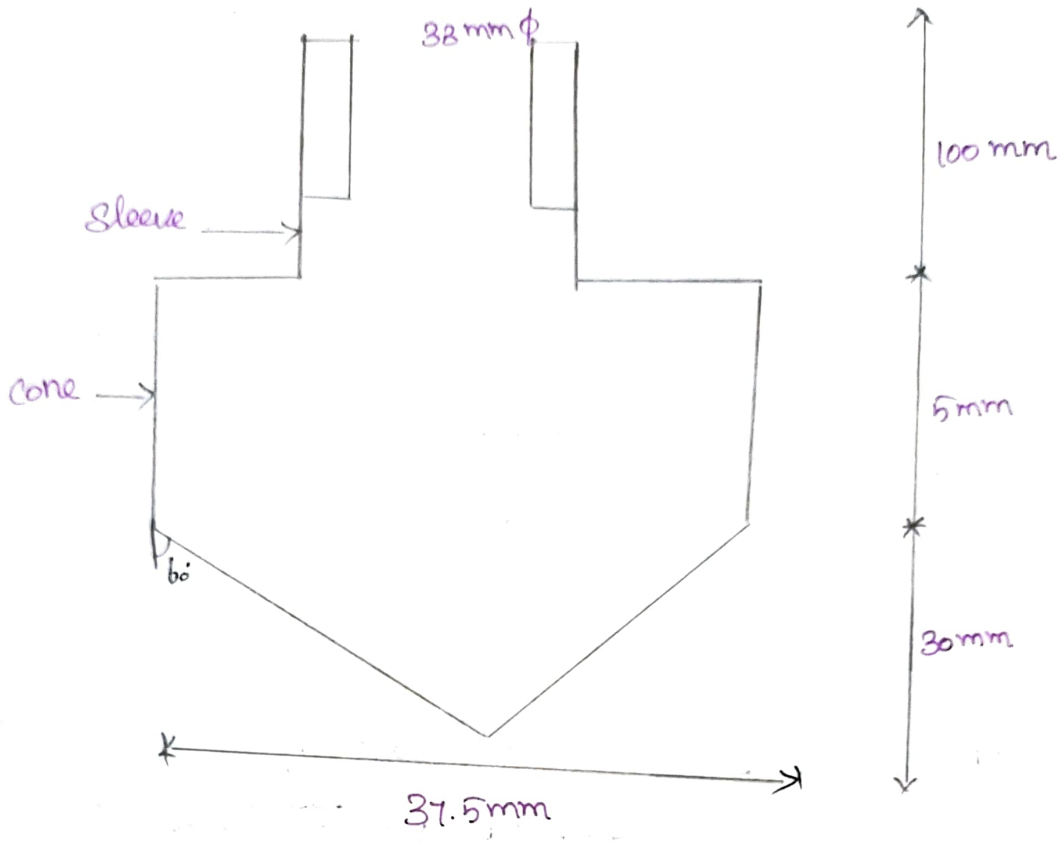
The drill rod should be in std specification and should not be in bend condition.

The drop hammer must be of Std grade, the fall should be free and vertical.

The height of the fall should be exactly 750 mm.

The bottom of the borehead must be properly cleaned before the test is carried out, if this is not done, it can't be able to get undisturbed soil sample.

Static Cone Penetration Test :



Procedure:

Static cone penetration test is also called as Dutch Cone test (DCT).

This test is standard by IS4968:1076

This test can be used to determine bearing capacity of soil. It consists of sleeve cone with an angle 60° and sleeve length 100mm

Dia of cone 37.5mm, dia of sleeve is 33mm.

for obtaining the cone resistance, the cone is pushed downwards at a steady rate of 10mm/sec to a depth of 8cm and max. value of resistance is recorded.

The cone is pushed down by applying thrust and not by drawing.

After the cone resistance has been determined the cone is withdrawn.

The sleeve is pushed on to the cone and both are drawn into the soil. The combined resistance is also determined. Sleeve resistance = combined resistance – Cone resistance

Limitation:

This test is not suitable for graval soil and the soils having SPT 'N' value geter than 50.

SELECTION OF FOUNDATION BASED ON SOIL CONDITION:

⇒ Loads of Structure:

Loading Condition Coming from the structure is one of the factor that govern foundation selection. It is influenced by type of building, type of construction material, environmental factors, earthquake susceptibility.

Selection of material such as bricks, stone, steel and concrete for construction affect the

However, for high - rise building, deep foundation is provided because ground at greater depth are highly compacted.

⇒ Soil Bearing capacity:

It is one of the most crucial criteria that affect the selection of the appropriate type of foundation. Decision can be made to choose shallow or deep foundation based on the soil bearing pressure.

An allowable bearing pressure of atleast 100 KN/m^2 or higher is effective for shallow foundation up to 4 stories.

⇒ Soil Type:

There are different types of soil like sandy soil, loose soil, clayey soil & expansive soil. It is specifically bearing capacity of both topsoil & subsoil layers would to a great extent controls the foundation selection.

- Clay soil
- Peat soil
- Silt
- Sand and Gravel
- Rock
- Loam
- Mountain soil & Forest soils

⇒ Ease of Construction:

The Choice of foundation should be made keeping in mind the ease with which it can be executed / constructed at the site. The construction of different types of foundation requires labour of different skillset and different level of skills.

⇒ Water Table Level:

Water table is another significant that affect the foundation selection. foundation should not be placed on soil that undergo expansion and contraction due to water table fluctuation. If this option cannot be executed, then deep foundation like micro piles or bored piles should be considered to provide the necessary uplift resistance.

⇒ Adjoining structure / water bodies / slopes:

Waterbodies, slopes are parmeters that need to be considered for selection of foundation types. The selection of proper foundation type would be more challenging if the neighbouring structure is high rise building or an old property.

⇒ Natural Disaster and Extreme weather:

If the area has a history of severe natural events or extreme weather, then these parameters into consideration. The past record or data of natural disasters and extreme weather should be employed as a better base for foundation selection.

⇒ Economic Design:

When there are number of feasible foundation choices for the given project site, economical factors may influences the selection of the foundation. Nonetheless, choosing an economical foundation should not compromise the safety, workmanship, strength and durability of the foundation.

BORE LOG REPORT:

* The access obtained from the soil samples sited beam of a bore hole, generally representative form is called bore hole log or report.

* It must consists of,
  i) Scope of the investigation
  ii) Description of proposed structure,

Description of strata	Soil Classification	Texture of strata	Depth from GL	Samples				Test result UCC, ϕ
				Type	No	Depth of Sample	GWL	
Fine Sand Medium Sand	Sand	- -	3m	Un-disturbed	1	2m	—	5 KN
Silty Clay.	clay	//////// //////// //////// ////////	5m	Un-disturbed	1	2m	—	5 KN

Recommendation : 5m → BC → 250 KN/m² → Shallow isolated testing

iii) Description of proposed structure, the location and geological condition of the site.

* Details about the method of exploration
* General discription of subsoil condition has obtained from instu test like penetration test.
* Details of laboratary test conducted on soil sample collected and results obtained.
* Depth of watertable and change in water level.
* Analysis and discussion of the results obtained.
* Recommendation about the allowable bearing presences and type of foundation structure.
* Calculation for determining bearing capacity.
* Drawings which include the site plan, test results ploited in the form of graphs.

* In the conclusion the main finding of the investigation should clearly denote

↳ Name of the client

↳ Boring for type of structure.

↳ Type of boring.

↳ diameter of boring

↳ Location

↳ Boring number

↳ Soil sample used.

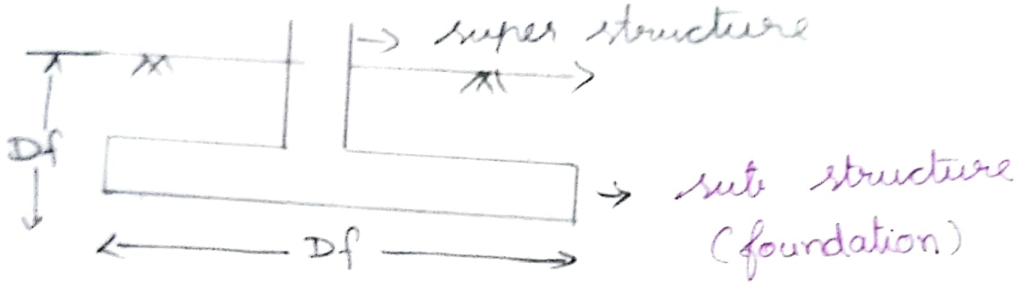
UNIT-II

BEARING CAPACITY OF SHALLOW FOUNDATION

Introduction - location and depth of foundation - codal provisions - Bearing capacity of shallow foundation on homogeneous deposits. Terzaghi's formula and BIS formula - Factor affecting bearing capacity - Bearing capacity from in-situ tests (SPT, SCPT and plate load) - Allowable bearing pressure - seismic considerations in bearing capacity evaluation. Determination of settlement of foundation on granular and clay deposits - Total and differential settlement - Allowable settlements - codal provisions - Methods of minimizing total and differential settlements.

Foundation:

Foundation is the part of the structure below ground surface which is direct contact with the soil.



↳ Functions of foundations:

1) Transmits the load from super structure & its own self weight to the soil

2) It gives suitability to the super structure.

↳ Requirements of good foundation:

* Location and Depth of foundation:

Foundation must be properly locate in available area at a desired depth.

⇒ Location:

The foundation could be placed or located below the depth of soil erosion, below the depth of top soil or fill, beyond the depth upto which the influence of adjacent

structure, tree and seasonal temperature movement of soil are severe. Foundation is also placed on the ground if firm or hard ground surface with erosion protection surrounding the foundation is made.

Types of elements / Depth	Masonry			Column Footing	Raft	Pile
	Non load bearing	Bearing				
		Class B	Class A			
mm	600	750 Sand 900 clay	1200	1200	1500	4000

The width of foundation shall be minimum of wall width not less than 300mm for concrete work, 450mm for brick masonry.

↳ Depth of Foundation:

NBS 2005 specifies the depth to which foundation should be carried depends upon the following principal factors.

a) The securing of adequate allowable capacity.

b) In the case of clayey soils, penetration below the zone where shrinkage and swelling due to seasonal weather changes.

Bearing capacity of Shallow foundation on homogeneous deposit.

Foundation is one of the element of the structure, the one responsible for transmitting load from superstructure to the soil beneath including its own weight

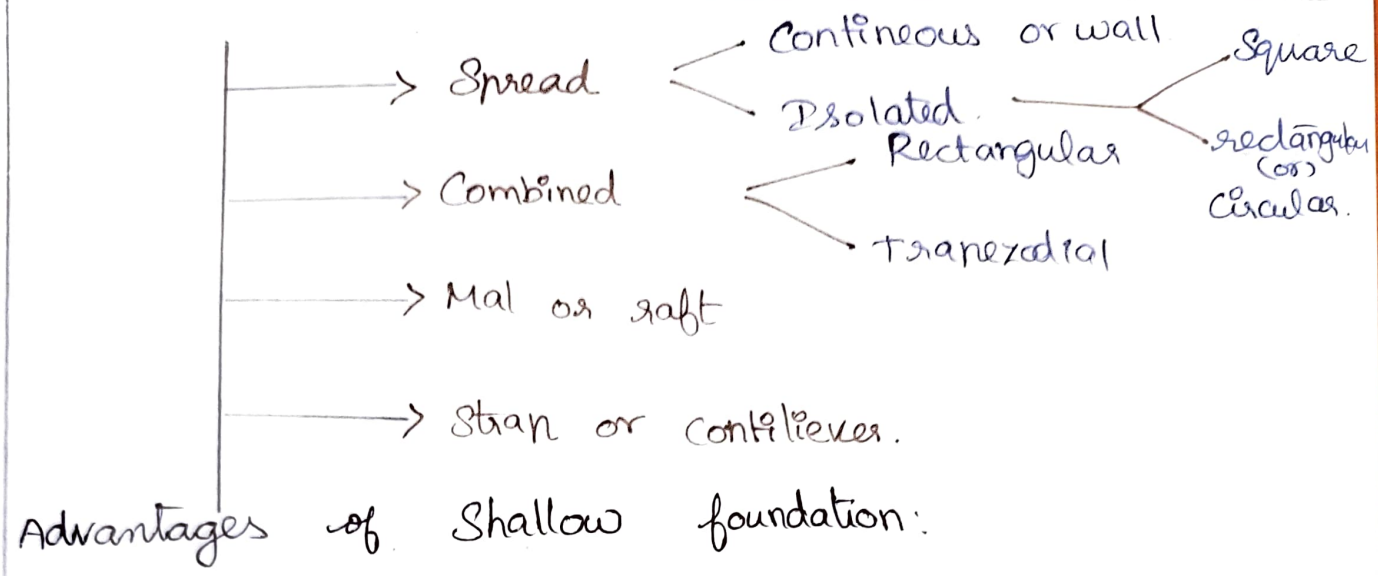
Foundation are designed to have an adequate load capacity depending on the types of subsoil supporting the foundation.

Foundation are generally classified into two types: ⇒ Shallow foundation
⇒ Deep foundation

⇒ Shallow foundation:

* It is a type of foundation which transfers building loads to the nearest earth surface

* It is generally width is greater than the depth i.e $Df/B <= 1$.



Advantages of Shallow foundation:

1) Affordable cost
2) Simple construction procedure
3) Material used is mostly concrete
4) Does not require experience workers.

→ Spread Footing:

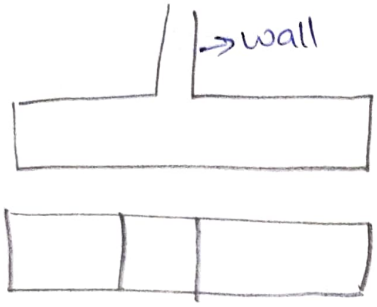
* It is basically a concrete pad spread out the load from the walls or columns to the large area of the soil. It is used good bearing capacity.

* There factory required to support a column means isolated footing.

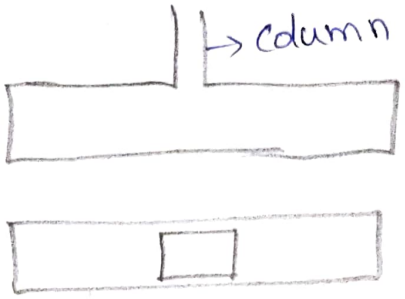
* If the column is square, it is called square column.

* If the column is rectangular, it is called regular isolated footing.

* If the column is circlar, it is called circular isolated footing.



Wall footing



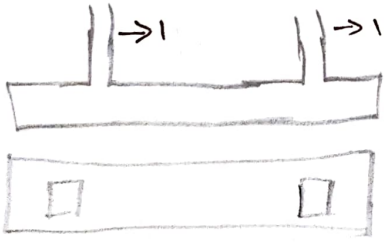
Column Isolated.

* This footing is used when soils are having good bearing capacity.

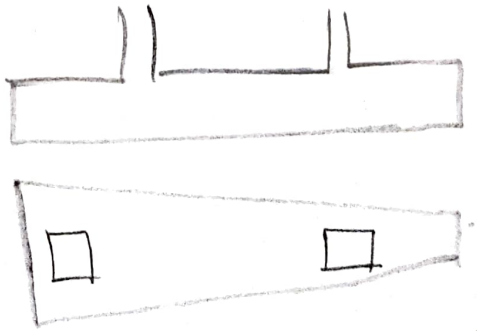
* For good strength soil, the spread footing is most suitable to reduce the cost of construction.

⇒ Combined Footing:

* It supports two or three column on a single rod.



Rectangular Combined footing

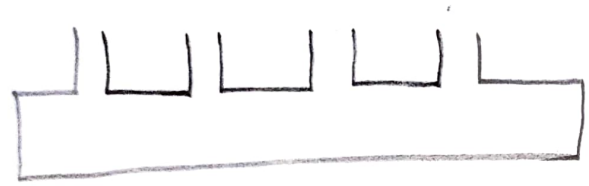


Tranezodial combined footing.

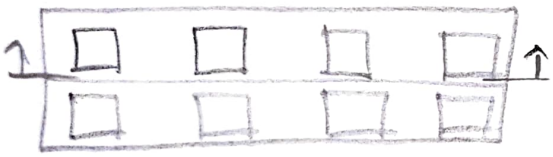
* When individual columns are closed to each other, the combined footings are required.

⇒ Mat or Raft foundation:

* these are large foundation supports number of columns with number of rows.



* It is required the dist b/w the columns are very less & individual columns are very closed.



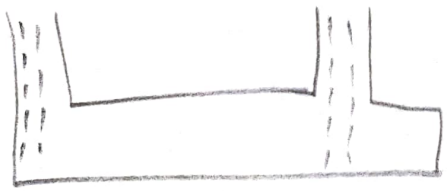
* When the soils is having low bearing capacity mat or raft foundation is adopted.

Advantages:

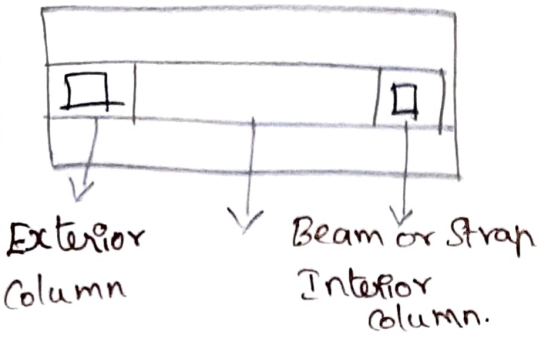
*It reduces the differential settlement.

⇒ Strap or contilever footing:

*It consists of two footings & connected by a beam called strap.



2) Due to end of fixed & other end is free, so it is also called as Cantilever footing.



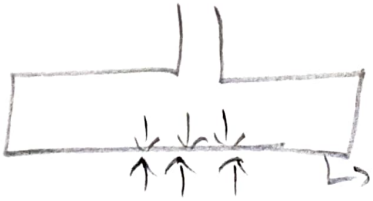
* It is required when the footing of exterior column cannot extent into adjacent property.

* It is more economical than combined footing & it is also required when the allowable soil pressure is releatively high & the distance b/w the column is large.

* **Bearing Capacity:**
Load carrying capacity of the soil is called as bearing capacity.

* **Foundation soil:**
Upper part of the earth mass carrying the loads from the structure is called as foundation soil.

* **Gross Pressure Intensity (q):**
It is the total pressure at the base of the footing due to the weigh of the soil and the wt of the structure.



* Net Pressure Intensity:

It is the difference b/w gross pressure intensity & the overburden pressure:

$$q_n = q - \sigma \qquad \sigma = \gamma D$$

$$\therefore \boxed{q_n = q - \gamma D}$$

* Ultimate bearing Capacity:

It is the minimum gross pressure intensity at the base of footing at which the soils fails

* Net ultimate bearing capacity (q_{nf}):

Net ultimate is the difference b/w ultimate bearing capacity & the the over ~~bross~~ buden pressure.

$$q_{nf} = q_f - \sigma$$

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

* Net Safe bearing capacity (q_{ns}):

It is the ratio of net ultimate bearing capacity to the factor of safety

$$\boxed{q_{ns} = \frac{q_{nf}}{F}}$$

* Safe bearing capacity (q_s):

It is defined as the max pressure which the soil can carry without risk of any failure.

Mathematically, $q_s = q_{ns} + \sigma$

$$\boxed{q_s = q_{ns} + \gamma D}$$

Formula:

$$q_s = q_{ns} + \gamma D$$

$$q_{ns} = \frac{q_{nf}}{F}$$

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

q_f = OBC, By Terzaghi's eq, BIS eq.

Determination of bearing capacity:

The load carrying capacity of soil can be determined by various analysis of method.

1) Building Codes: (IS 1904 - 1986) - Tables value are available.

⇒ These values are called as presumptive B.C values.

⇒ These are not exact value, it is approximately design only.

2) Analytical Method:

i) Terzaghi's equations

ii) BIS equations

iii) Mayerhoff's equations

iv) Vesic's eq.

v) Hansen's eq.

By using these eq. we can find ultimate bearing capacity (qf).

3) Practical Method (or) Approach:

*) Plate load test

*) Penetration test (SPT, SCPT, DCPT)

Bearing capacity failures:

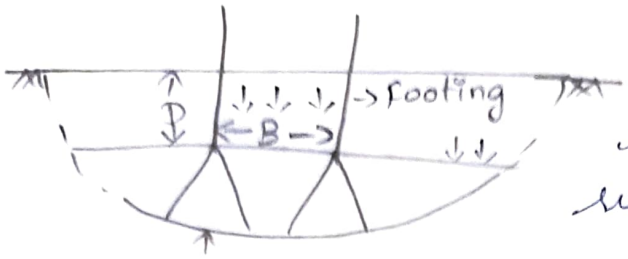
If a footing or foundation fails due to insufficient bearing capacity, the failure patterns are developed. Based on the failure patterns, failures are classified as three types.

1) General shear failure (GSF)

2) Local shear failure (LSF)

3) Punching shear failure (PSF)

9) General Shear Failure (GSF):



→ Continuous failure surface develop blw the edges of the footing & the ground surface.

→ It has well defined failure pattern upto the ground surface.

→ Full failure takes place with tilting of the footing.

→ Failure is sudden.

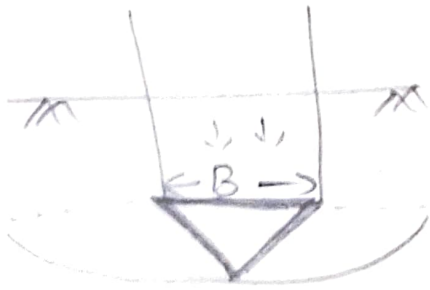
→ Ultimate bearing capacity is well defined.

→ Considerable bulging of soil around the footing.

→ $\underline{\phi > 36^\circ}$ (x), $N > 30$, $e < 1.55$, I.D > 70% Relative Density I.D.

∴ soil is hard.

99) Local Shear failure (LSF):



→ Failure pattern is clearly defined below the base of the footing.

→ Notiting

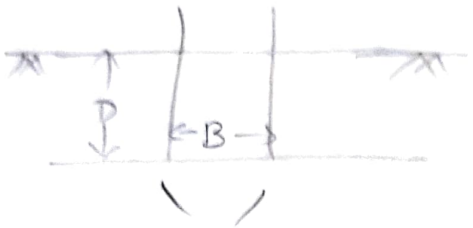
→ Failure is not sudden

→ Ultimate bearing capacity is not well defined.

→ Slight bulging around the footing.

→ $\underline{\phi < 28^\circ}$ (x), $N < 5$, $e > 1.55$, I.D < 20%. (soil is very loose)

8) Punching Shear Failure (PSF):



→ No failure pattern is observed below the base of the footing.

→ No tilting, No bulging.

→ Ultimate bearing capacity (UBC) is not well defined.

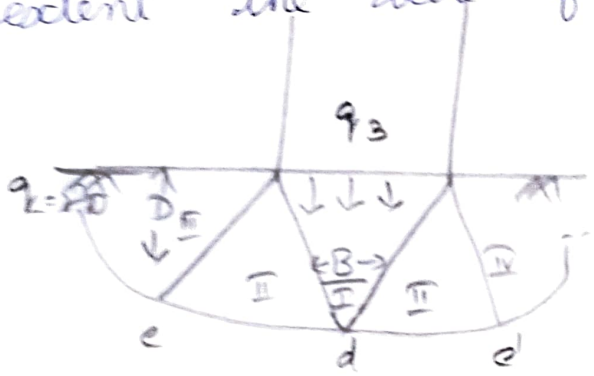
Terzaghi's formula and BIS formula:

⇒ Terzaghi's Analysis of bearing Capacity Theory:

Assumptions:

1) A complete bearing capacity failure is general shear failure ($\phi > 36°$).
2) Soil behaves perfectly plastic material
3) Soil is homogeneous and isotropic
4) Shear strength represented by coloumb's equation
5) Footing is shallow
6) Base of the footing is rough.
7) Load on the footing is vertical.
8) The soil above the base of the footing is replaced by equivalent surcharge ($q = \gamma_1 D_f$)

9. The failure surface is assumed not to extent the base of the footing.

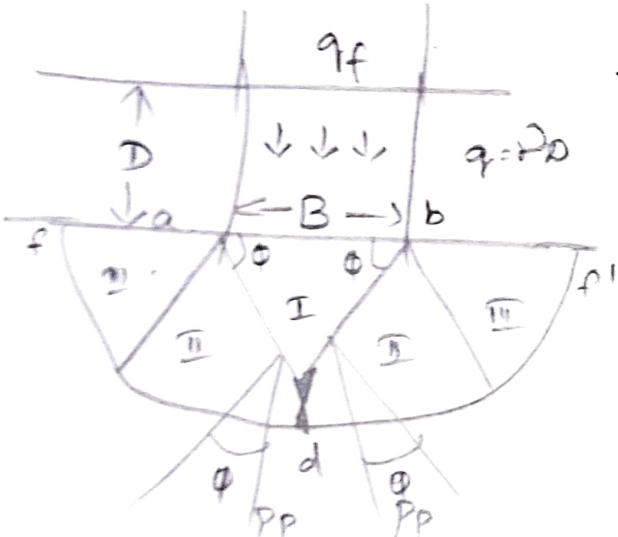


D - Depth of footing

R - Width of footing

q_f - Intensity of loading.

The loaded soil fails along the surface fede 'f' the region of surface of failure divided into five zones. They are zone - I pair of zone - II & pair of zone - III. The surcharge at the base of footing is γD.



→ When the base of the footing ab sinks into ground due to the intensity of loading q_f is prevented by the cohesive forces $c \sin\phi$ along with the surface ad & db.

→ Due to this zone - I remains in the state of plastic equilibrium. The boundaries ad & db is assumed as plane surface.

→ Further application of intensity of loading tends to push ad & db with the lateral displacement of zone-II & zone-III which is resisted by passive forces.

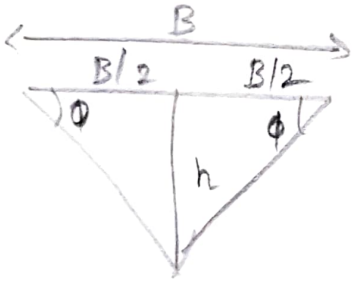
→ the passive forces will be makes an angle ϕ with the normal to the surface ad & db.

Total downward forces:

1. Intensity of loading q_f for the width of B.

(1.0) $q_f \cdot B$.

2. Loading with wt. of the wedge of soil



$W = \text{Area} \times \gamma$

$\tan \phi = \frac{h}{B/2}$

$\boxed{h = \frac{B}{2} \tan \phi}$

$\text{Area} = \frac{1}{2} Bh$

$= \frac{1}{2} \times B \times \frac{B}{2} \tan \phi = \frac{1}{4} B^2 \tan \phi$

$\boxed{W = \frac{1}{4} B^2 \tan \phi \gamma}$

∴ Total downward forces: $\boxed{q_f B + \frac{1}{4} B^2 \tan \phi \gamma 2}$ → (1)

Upward force:

1. Passive force along the surface ad & db will be $2P_p$.

2. Cohesive force along the surface ad & db.

Vertical component of Cohesive forces.

$$\cos\phi = \frac{B/2}{ad(or)\,db}$$

$$\boxed{ad \text{ or } db = \frac{B}{2\cos\phi}}$$

Cohesive force along the surface,

$$= C \sin\phi \times \frac{B}{2\cos\phi}$$

$$\boxed{= \frac{BC}{2}\tan\phi}$$

Total upward forces,

$$\boxed{= 2P_p + \frac{BC}{2}\tan\phi} \longrightarrow (2)$$

Based on equilibrium Conditeon,

Total upward force = Total downward force.

$$2P_p + \frac{BC}{2}\tan\phi = \Phi_f B + \frac{1}{4} B^2 \tan\phi\gamma$$

Mirly for square footing, $q_f = 1.3cN_c + \gamma D N_q + 0.4 B \gamma N_\gamma$

For Circular footing, $q_f = 1.3cN_c + \gamma D N_q + 0.3 B \gamma N_\gamma$

For Rectangular footing. $q_f = cN_c \left[1 + 0.3\frac{B}{2}\right] + \gamma D N_q + 0.4 B \gamma N_\gamma \left[1 + 0.2\frac{B}{2}\right]$

These equations are same for any two types of failure.

Where, c - Cohesion, γ - Unit wt. of the soil,

D - Depth of the footing.

B - Width of the footing.

N_c, N_q, N_γ - Terzaghi's bearing capacity factors depends upon the value ϕ.

ϕ - angle of internal frication

1. A strip footing 1m wide & its base is located at a depth of 0.8m below ground surface Unit wt of soil 18 KN/m², Cohesion 30KN/m², angle of Internal friction 20° Determine safe bearing capacity of the soil with factor of safety 3.

Given: Strip footing

B = 1m, D = 0.8m

Assume the P_p can be resolved into 3 components.

1) $P_{p\gamma}$ ⇒ force produced by wt. of the wedge.

2) P_{pc} ⇒ force produced by cohesion

3) P_{pq} ⇒ force produced by surcharge.

$$q_f B = 2P_p + \frac{Bc}{2}\tan\phi - \frac{1}{4}B^2\tan\phi\gamma$$

$$q_f B = 2\left[(P_p\gamma + P_{pc} + P_{pq})\right] + \frac{Bc}{2}\tan\phi - \frac{1}{4}B^2\tan\phi$$

Let the terms, $2P_p\gamma - \frac{1}{4}B^2\tan\phi\gamma = \frac{1}{2}B\gamma B N_\gamma$ (By Terzaghi's)

$$2P_{pc} + \frac{Bc}{2}\tan\phi = BcN_c \text{ (By Teragi's)}$$

$$2P_{pq} = B\gamma D N_q \text{ (By Teragi's)}$$

$$q_f B = \frac{1}{2}B\gamma B N_\gamma + BcN_c + B\gamma D N_q$$

$$q_f = \frac{1}{2}B\gamma N_\gamma + cN_c + \gamma D N_q$$

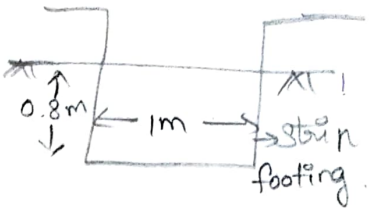
$$\boxed{q_f = cN_c + \gamma D N_q + 0.5 B\gamma N_\gamma.}$$

This is the equation for shallow strip footing. General shear failure.

Local shear failure, Strip footing } $q_f = \frac{2}{3}cN_c + \gamma D N_q + 0.5 B\gamma N_\gamma$

$\gamma = 18\ kN/m^3$, $C = 30\ kN/m^2$

$\phi = 20° < 28°$ ∴ Local shear failure



FOS = 3, SBC = ?

Solution:

$$q_s = q_{ns} + \gamma D$$

$$q_{ns} = \frac{q_{nf}}{F}$$

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

$$q_f = \frac{2}{3} C N_c + \gamma D N_q + 0.5 B \gamma N_\gamma$$

[From the table, Local shear failure

For $\phi = 20° \Rightarrow N_c = 11.8$, $N_q = 3.9$, $N_\gamma = 1.7$]

$$q_s = \frac{2}{3} \times 30 \times 11.8 + 18 \times 0.8 \times 3.9 + 0.5 \times 1 \times 18 \times 1.7$$

$$\boxed{q_f = 307.46\ kN/m^2}$$

$$q_{nf} = q_f - q_s$$

$$= 307.46 - 18 \times 0.8 \quad \boxed{= 293.06\ kN/m^2}$$

$$q_{ns} = \frac{q_{nf}}{F} = \frac{293.06}{3} = \boxed{97.68\ kN/m^2}$$

SBC, $q_s = q_{ns} + \gamma D = 97.68 + 18 \times 0.8$

$$\boxed{q_s = 112.08\ kN/m^2}$$

2. A Square footing 2.5m x 2.5m is in a bed of sand with unit wt. 20 KN/m³, angle of internal friction 35°, depth of footing 1.5m below Ground surface. Calculate the safe load with FOS. 3. Use Terzaghi's analysis.

Given: Square Footing

$B = 2.5m \times 2.5m$

$= 2.5m$

$\gamma = 20 KN/m^3$, $\phi = 35°$

$D = 1.5m$, $FOS = 3$, Sandy Soil $SOC = 0$

Safe load = ?

Solution:

$$\text{Pressure} = \frac{\text{Load}}{\text{Area}}, \quad SBC = \frac{\text{Safe load}}{\text{Area}}$$

$$\text{Safe load} = SBC \times \text{Area}$$

SBC ; $q_s = q_{ns} + \gamma D$

$$q_{ns} = \frac{q_{nf}}{FF}$$

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

$$q_f = 1.3 c N_c + \gamma D N_q + 0.4 B \gamma N_\gamma$$

[From the table, Consider General Shear failure

For $\phi = 35° \Rightarrow N_c = 0$ (Sandy Soil, $N_q = 41.4$, $N_\gamma = 4$

$q_f = 20 \times 1.5 \times 41.4 + 0.4 \times 2.5 \times 20 \times 42.4$

$$q_f = 2090 \text{ KN/m}^2$$

$$q_{nf} = 2090 - 20 \times 1.5 = 2060 \text{ KN/m}^2$$

$$q_{ns} = \frac{2060}{3} = 686.66 \text{ KN/m}^2$$

SBC, $q_c = 686.66 + 20 \times 1.5$

$$q_c = 716.66 \text{ KN/m}^2$$

Safe load = SBC x Area = $716.66 \times 2.5 \times 2.5$

$= 4.47$ KN

3. A Circular footing resting on a clay deposit at a depth of 2m from Ground surface. Determine the diameter of the footing if Column load is 600 KN, FOS 2.5, $\gamma = 20 \text{ KN/m}^3$, Unconfined compression strength 250 KN/m², use Terzaghi's analysis.

Given: Circular footing

Clay soil, $\phi = 0°$

$D = 2m$, $\gamma = 20 \text{ KN/m}^3$, FOS = 2.5

UCC, $q = 250 \text{ KN/m}^2$, Safe load/Column load = 600 KN

diameter, $d = ?$

$$C = \frac{q}{2} = \frac{250}{2} = 1.25 \text{ KN/m}^2.$$

Solution: Safe load = $q_s \times$ Area

$600 = q_s \times \pi d^2/4$

$q_s = q_{ns} + \gamma D$

$q_{ns} = q_{nf}/F$

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

$q_f = 1.3CN_c + \gamma D N_q + 0.3B\gamma N_\gamma$

[From the table,

For $\phi = 0°$, $N_C = 5.7$, $N_q = 1$, $N_\gamma = 0$]

$q_f = 1.3 \times 1.25 \times 5.7 + 20 \times 2 \times 1 + 0.3 \times B \times 20 \times 0$

$\boxed{q_f = 966.25 \text{ KN/m}^2}$

$q_{nf} = 966.25 - (20 \times 2) = \boxed{926.25 \text{ KN/m}^2}$

$q_{ns} = 926.25/50.5 = \boxed{370.5 \text{ KN/m}^2}$

$q_s = 370.5 + (20 \times 2) = \boxed{410.5 \text{ KN/m}^2}$

$600 = 410.5 \times \frac{\pi \times d^2}{4}$

$600 = 322.40 \times d^2$

$\boxed{d = 1.36 \text{m}}$

4. What is ultimate bearing capacity on square footing resting on surface of clay with unconfined compression strength 100 KN/m².

Given:

Square footing, Clay $\Rightarrow \phi = 0^\circ$

UCC, $q = 100\ KN/m^2$, $c = \frac{q}{2} = 50\ KN/m^2$

$UBC = ?$

Solution:

$$UBC,\ q_f = 1.3\ cN_c + \gamma D N_q + 0.4 B \gamma N_\gamma$$

[From the table,

For $\phi = 0^\circ$, $N_c = 5.7$, $N_\gamma = 0$]

$$q_f = 1.3 \times 50 \times 5.7$$

$$\boxed{q_f = 370.5\ KN/m^2}$$

5. Compute the safe bearing capacity of continous footing having 1.8 m width & located at a depth of 1.2 m below GL with unit wt. of soil 20 KN/m³, cohesion = 20 KN/m², FOS = 2.5, $\phi = 20^\circ$. What is the load / metre length of footing.

Given: Continuous footing (i.e, Strip footing).

$B = 1.8\ m$, $D = 1.2\ m$

$\gamma = 20\ kN/m^3$, $C = 20\ kN/m^2$

FOS = 2.5, $\phi = 20° < 28°$ (LSF)

SBC = ?, load / meter length = ?

Solution

$$q_s = q_{ns} + \gamma D, \quad q_{ns} = \frac{q_{nf}}{F}, \quad q_{nf} = q_{nf} - \gamma D$$

$$q_f = \frac{2}{3} C N_c + \gamma D N_q + 0.5 B \gamma N_\gamma,$$

[From the table, LSF

For $\phi = 20°$, $N_c = 11.8$, $N_q = 3.9$, $N_\gamma = 1.7$)

$$q_f = \frac{2}{3} \times 20 \times 11.8 + 20 \times 1.2 \times 3.9 + 0.5 \times 1.8 \times 20 \times 1.7$$

$$\boxed{q_f = 281.53\ kN/m^2}$$

$$q_{nf} = 281.53 - (20 \times 1.2) = \boxed{257.53\ kN/m^2}$$

$$q_{ns} = 257.53 / 2.5 = \boxed{103.01\ kN/m^2}$$

$$q_s = 103.01 + (20 \times 1.2) = \boxed{127.01\ kN/m^2}$$

$$SBC = 127.01\ kN/m^2$$

Load / meter length = 127.01×1.8

$$\boxed{= 228.624\ m}$$

6 A footing of a column is 2.25 m square is founded at a depth of 1m on a Cohesive soil of unit wt. 17.5 KN/m². What is the safe load for the footing. If cohesion is 30kN/m², $\phi = 0°$ & FOS = 3. Terzaghi's bearing capacity factors are $N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$.

Given: $A = 2.25\ m^2$, $D = 1\ m$ — 2.25 m² → Square footing

$\gamma = 17.5\ KN/m^2$, $C = 30\ KN/m^2$

$\phi = 0°$, FOS = 3

$N_c = 5.7$, $N_q = 1$, $N_\gamma = 0$

Safe load = ?

Solution: Safe load = $q_s \times$ Area

$q_s = q_{ns} \times \gamma D$, $q_{ns} = q_{nf}/F$, $q_{nf} = q_f - \gamma D$

$q_f = 1.3\ CN_c + \gamma D N_c + 0.4\ B\gamma N_\gamma$

$= 1.3 \times 30 \times 5.7 + 17.5 \times 1 \times 9$

$\boxed{q_f = 239.8\ KN/m^2}$

$q_{nf} = 239.8 - (17.5 \times 1) = \boxed{222.3\ KN/m^2}$

$q_{ns} = 222.3/3 = \boxed{74.1\ KN/m^2}$

$q_s = 74.1 + (17.5 \times 1) = \boxed{91.6\ KN/m^2}$

Safe load = 91.6×2.25

$= \boxed{206.1\ KN}$

Safe load = $q_s \times$ Area

$= 91.6 \times 2.25 \times 2.25$

$= \boxed{463.725 kN}$

Effect of Water Table on Bearing Capacity:

Water present in soil affects shear parameters & density of soil. The effect of water table on shear parameters is very very small. so, do not consider shear parameters for the calculations of bearing capacity of soil due to the water table. But the effect of water table on density is more. Fore the calculation of B.C, consider the variation in density for the calculation of B.C. or within the base or below the base (or) the depth is equivalent to D+B. The ultimate B.C of soil.

$$q_f = CN_c + \gamma' D N_q R_w + 0.5 B \gamma' N_\gamma R_{w_2}$$

LSF, $q_f = 2/3\, CN_c + \gamma' D N_q R_w + 0.5 B \gamma' N_\gamma R_{w_2}$

Square (LSF or GSF), $q_f = 1.3CN_c + \gamma D N_q R_w + 0.4 \gamma B N_\gamma R_w$

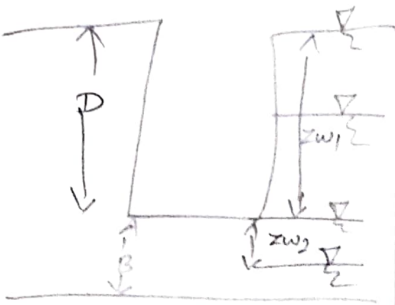
Circular ; $q_f = 1.3\, CN_c + \gamma D N_q R_w + 0.3 B \gamma N_\gamma R_{w_2}$

R_w, R_{w_2} = water reduction factor

$$R_{w_1} = 0.5 \left[1 + \frac{z_{w_2}}{D}\right]$$ z_{w_1} - depth of WT above base

$$R_{w_2} = 0.5 \left[1 + \frac{z_{w_2}}{D}\right]$$ z_{w_2} - " " " below ".

If the water table at the G.S or within the base or at the base. Calculate R_{w_2}, assume $R_{w_2} = 0.5$. If the water table is below the depth of footing, calculate R_{w_2}, assume $R_{w_1} = 1$



1. A strip footing 2m wide carries the load intensity of 400 KN/m² at a depth of 1.2m in sand. The saturated unit wt of sand is 19.5 Km/m³. The unit wt above WT is 16.8 KN/m³ $\phi = 35°$, $N_q = 41.4$, $N_\gamma = 42.4$. Determine Fos for the following cases. (i) WT 4m below G.S (ii) WT 1.2m below GS

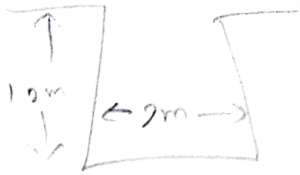
iii) W.T 0.5m below GS vi) W.T is at Ground level.

Given: Strip footing

$B = 2m$, $D = 1.2\,m$

$\gamma_{sat} = 19.5\ kN/m^3$, $\gamma_d = 16.8\ kN/m^3$

Sandy soil, $C = 0$



$\phi = 35° \Rightarrow$ (So GSF)

$N_q = 41.4$, $N_\gamma = 42.4$

Solution:

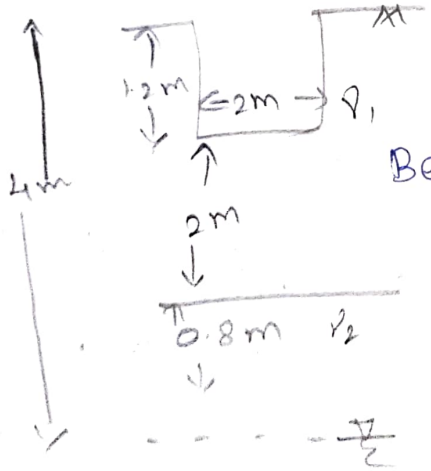
$$F.O.S = \frac{q_f}{\text{Load Intensity}}$$

Load Intensity $= 400\ kN/m^2$

$$q_f = CN_c + \gamma_1 D N_q R_{w_1} + 0.5 B \gamma_2 N_\gamma R_{w_2}$$

Case 9): W.T 4m below GL.

$D = 1.2m$ $B = 2m$



$\gamma_2 = \gamma_2 = \gamma_d = 16.8\ kN/m^3$

Beyond base, assume R_{w_1} = calculate R_{w_2}

$$R_{w_2} = 0.5\left[1 + \frac{2w_2}{B}\right]$$

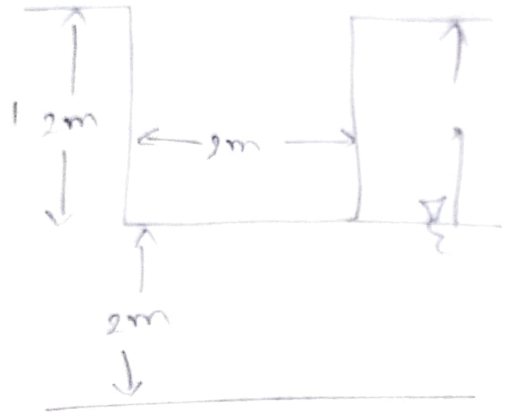
$$= 0.5\left[1 + \frac{2.8}{2.0}\right] = 1.2$$

$$q_f = 16.8(1.2)(41.4)(1) + 0.5(2)(16.8)(42.4)(1.2)$$

$$= 1689.4\ kN/m^2$$

$$FOS = \frac{1689.4}{400} = 4.22$$

Case "ii) W.T 1.2m below G.L



$\gamma_1 = \gamma_d = 16.8\ KN/m^3$

$\gamma_2 = \gamma_{sat} = 19.5\ KN/m^3$

W.T at base, assume

$R_{w_2} = 0.5$ calculate R_{w_1}

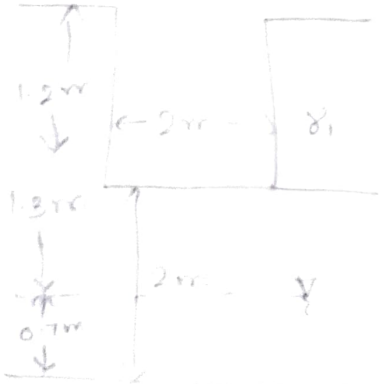
$$R_{w_1} = 0.5\left[1 + \frac{z_{w1}}{D}\right] = 0.5\left[1 + \frac{1.2}{1.2}\right] = 1$$

$$q_f = (16.8)\times 1.2 \times 41.4\times 1) + (0.5 \times 2 \times 19.5 \times 1.2 \times 0.5)$$

$$= 1248\ KN/m^3$$

$$FOS = \frac{1248}{400} = 3.12$$

Case iii) W.T 2.5 m below G.L



$\gamma_1 = \gamma_d = 16.8\ KN/m^3$

$$\gamma_2 = \gamma_{avg} = \frac{1.3\times 16.8 + 0.7 \times 19.5}{2}$$

$$= 17.74\ KN/m^3$$

W.T beyond base, assume $R_{w_1} = 1$ cal R_{w_2}

$$R_{w_2} = 0.5\left[1 + \frac{z_{w_2}}{B}\right]$$

$$= 0.5\left[1 + \frac{1.3}{1.2}\right] = 0.8$$

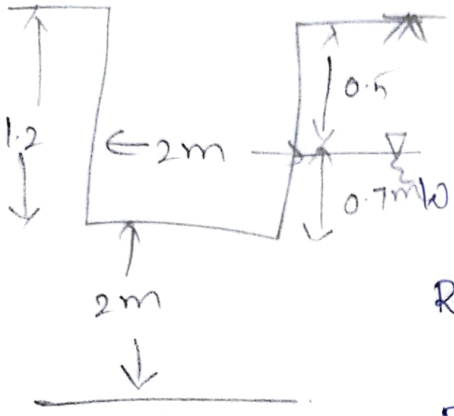
$$q_f = (16.8\times 1.2 \times 41.4\times 1) + (0.5\times 2\times 17.74\times 1.2\times 0.8)$$

$$= 1436.36\ KN/m^2$$

$$FOS = \frac{1436.36}{400} = 3.59$$

Case iv) W.T 2.5 m below GL

$$\gamma_1 = \gamma_{avg} = \frac{0.5 \times 16.8 + 0.7 \times 19.5}{1.2} = 18.31 \text{ kN/m}^3$$



$\gamma_2 = \gamma_{sat} = 19.5 \text{ kN/m}^3$

W.T at base, assume $R_{w_2} = 0.5$ Cal R_w

$$R_{w_1} = 0.5 \left[1 + \frac{Z_{w1}}{D}\right] = 0.5 \left[1 + \frac{0.5}{1.2}\right]$$

$$R_w = 0.7$$

$$q_f = (16.8 \times 1.2 \times 41.4 \times 1) + (0.5 \times 2 \times 17.74 \times 42.4 \times 0.8)$$

$$= 1436.36 \text{ kN/m}^2$$

$$FOS = \frac{1436.36}{400} = 3.59.$$

$$q_f = (18.37 \times 1.2 \times 41.4 \times 0.7) + (0.5 \times 2 \times 19.5 \times 42.05)$$

$$= 1052.23 \text{ kN/m}^2$$

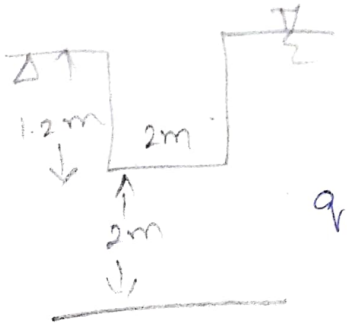
$$F = 05 = 2.63$$

Case v) W.T at GS

$\gamma_1 = \gamma_2 = \gamma_{sat}$ → 19.5 kN/m²

W.T at base

$R_{w_2} = 0.5$ Cal R_w



$$R_{w_1} = 0.5 \left[1 + \frac{0}{1.2}\right] = 0.5$$

$$q_f = (19.5 \times 1.2 \times 41.4 \times 0.5) + (0.5 \times 2 \times 19.5 \times 42 \times 0.5)$$

$$= 894.78 \text{ kN/m}^2$$

$$FOS = 2.24$$

2. Determine safe bearing capacity of strip footing 1.5 m with and 1m deep in a soil with cohesion 35 KN/m², $\phi = 20°$, unit wt. of soil above water Table 1.7 ton/m³, below wat tabe 1.9 ton/m³ depth of water table 1m below Ground Sur FOS = 3.

Given: Strip footing

$B = 1.5m$, $D = 1m$

$C = 35\ KN/m^2$, $\phi = 20° < 28°$ (LSF)

$\gamma d = 1.7\ ton/m^3$

$\gamma_{sat} = 1.9 ton/m^2 = 19\ KN/m^2$

Depth of W.T 1m below G.S, FOS = 3

SBC = ?

Solution:

$$q_s = q_{ns} + \gamma_1 D \quad , \quad q_{ns} = \frac{q_{nf}}{F}$$

$$q_{ns} = q_f - \gamma_1 D$$

$$q_f = \frac{2}{3}\left(N_c + \gamma_1 D N_q R_{w_1} + 0.5 B \gamma_2 N_\gamma R_{w_2}\right)$$

$$\gamma_1 = \gamma d = 17 KN/m^3$$

$$\gamma_2 = \gamma_{sat} = 19 KN/m^3$$

W.T at base, Assume $R_{w_2} = 0.5$, Calculate R_{w_1}

$$R_{w_1} = 0.5\left[1 + \frac{Z_{w_1}}{D}\right] = 0.5\left[1 + \frac{1}{1}\right] = 1$$

From the table,

For $\phi = 28$, $N_c = 11.8$, $N_q = 3.9$, $N_\gamma = 1.7$

$$q_f = \left(\frac{2}{3} \times 35 \times 11.8\right) + (17 \times 1 \times 3.9 \times 1) + (0.5 \times 1.5 \times 19 \times 1.7 \times 0.5)$$

$$q_f = 353.74 \text{ KN/m}^2$$

$$\boxed{q_{nf} = 353.74 \text{ KN/m}^2}$$

$$q_{nf} = 353.74 - 17(1) = \boxed{336.74 \text{ KN/m}^2}$$

$$q_{ns} = \frac{336.74}{3} = \boxed{112.24 \text{ KN/m}^2}$$

$$q_s = 112.24 + 17 \times 1 = \boxed{129.24 \text{ KN/m}^2}$$

3. A column carrying the load of 750 KN has to be supported by a square footing with its base at 1.5 m depth what is the required size of the foundation which will provide a factor 3 against shear failure. Assume $C = 10 \text{ KN/m}^2$, $\phi = 30°$, $\gamma_d = 18 \text{ KN/m}^3$, $\gamma' = 10 \text{ KN/m}^3$, W.T is at 1.5 m depth.

For $\phi = 30°$, $N_c = 37.2$, $N_q = 22.5$, $N_\gamma = 19.7$

Given: Square footing

Load = 750 KN, $D = 1.5$ m

FOS = 3, $C = 10 \text{ KN/m}^2$, $\phi = 30°$

$\gamma_d = 18 \text{ KN/m}^3$, $\gamma' = 10 \text{ KN/m}^3$

$$q_{nf} = 228 - (20 \times 1) = 188 \text{ KN/m}^3$$

$$q_{ns} = \frac{208}{3} = \boxed{62.66 \text{ KN/m}^2}$$

$$q_s = 62.66 + 20(1) = 82.66 \text{ KN/m}^2 \quad \text{SBC}_1$$

(Case 2): Water table rises G.S

$\gamma_1 = \gamma_2 = \gamma_{avg}$ W.T as base, Assume $R_{w_2} = 0.5$

$$R_{w_1} = 0.5\left[1 + \frac{0}{1.5}\right] = 0.5$$

$$q_f = 1.3 \cdot C N_c + D N_q R_{w_1} + 0.4 B \gamma N_\gamma R_{w_2}$$

$$q_f = (20 \times 1 \times 4.4 \times 0.5) + (0.4 \times 1.5 \times 20 \times 5 \times 0.5)$$

$$q_f = \boxed{104 \text{ KN/m}^2}$$

$$q_{nf} = 104 - 20 = 84 \text{ KN/m}^2$$

$$q_{ns} = \frac{84}{3} = 28 \text{ KN/m}^2$$

$$q_s = 28 + 20 = \boxed{48 \text{ KN/m}^2} \Leftarrow \text{SBC}_2$$

$$\% \text{ reduction} = \frac{\text{min. value}}{\text{max. value}} \times 100$$

$$= \frac{48}{82.66} \times 100 = 58.06\%.$$

5. A Square footing located at a depth of 1.5m from the G.S carries a column load of 150KN. The unit wt. of soil is 14KN/m^3, $\phi = 30^\circ$. Find the size

of the footing using Terzaghi's theory.

FOS = 3, $N_q = 10$, $N_\gamma = 6$.

b. A strip footing is to be designed to carry a load of 8000 KN at a depth of 0.4m. The shear parameters are $C = 0$, $\phi = 40^\circ$, Determine the width of the footing, if FOS & Assuming that the W.T may rise to the foundation level. The unit wt. of soil above W.T 17KN/m³ below water Table 19.81 KN/m³.

Determination of Bearing capacity by its Equation:

According to IS 6403 - 1981,

q_f (GSF) = $CN_c S_c d_c i_c + \gamma D N_q S_q d_q i_q + 0.5 \gamma B N_\gamma S_\gamma d_\gamma i_\gamma w'$.

q_f (LSF) = $\frac{2}{3} CN_c S_c d_c i_c + \gamma D N_q S_q d_q i_q + 0.5 B \gamma N_\gamma S_\gamma d_\gamma i_\gamma w'$.

⇒ N_c, N_q, N_γ ⇒ Bearing Capacity Factor.

⇒ For GSF N_c, N_q, N_γ value depends upon angle of Internal friction = ϕ

⇒ In LSF N_c, N_q, N_γ depends upon, $\phi' = \tan' = \tan^{-1}(0.67\phi)$

⇒ For S_c, S_q, S_γ ⇒ Safe factors [Table 24.1 Pg: 616]

⇒ d_1, d_q, d_γ ⇒ Depth factors.

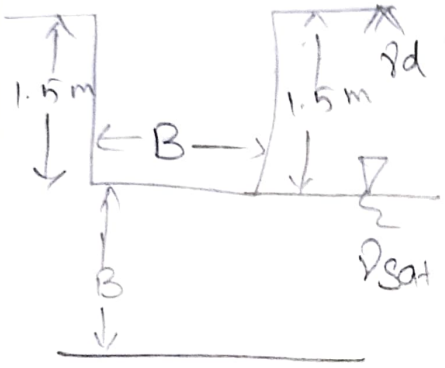
F.S.T = 1.5, $N_c = 37.2$, $N_q = 22.5$, $N_\gamma = 19.7$

width = B^2 = B x B = ?

Solution:

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

$$\gamma_{sat} = \gamma' + \gamma_w = 10 + 9.81 = 19.81\ KN/m^2$$



Safe load = q_s x Area

$$750 = q_e \times B^2$$

$$q_s = q_{ns} + \gamma_1 D, \quad q_{ns} = \frac{q_{nf}}{F}$$

$$q_{nf} = q_f - \gamma_1 D$$

$$q_s = 1.3\ cN_c + \gamma_1 D N_q R_{w_1} + 0.4\ B \gamma_2 N_\gamma R_{w_2}$$

$\gamma_1 = \gamma_d = 18.\ KN/m^3$

$\gamma_2 = \gamma_{sat} = 19.81\ KN/m^3$

$$R_{w_1} = 0.5\left[1 + \frac{1.5}{1.5}\right] = 1$$

$$q_f = (1.3 \times 10 \times 19.81) + (18.$$

$$q_f = 1091.1 + 78.05\ B$$

$$q_{nf} = 1091.1 + 78.05\ B - (18 \times 1.5)$$

$$= 1064 + 78.05\ B$$

$$q_{ns} = \frac{1064 + 78.05\ B}{3} = 354.66 + 26.01\ B$$

$$q_s = 354.66 + 26.01\ B + (18 \times 1.5) = 381.66 + 26.01\ B$$

$$750 = (381.66 + 26.01\ B) \times B^2$$

$$750 = 381.66\ B^2 + 26.01\ B^3$$

$$\boxed{B = 1.34\ m}$$

4. Compute the safe bearing capacity of square footing 1.5m x 1.5m is located at a depth of 1m below G.L. In the soil of avg. density 20 kN/m³, $\phi = 20°$, $N_c = 17.7$, $N_q = 7.4$, $N_\gamma = 5$. Assume suitable FOS & that the W.T is very deep. Also compute the reduction in safe bearing capacity of footing if W.T rises to the G.L.

Given: Square footing

$D = 1m$, $B = 1.5\ m$, $\gamma_{avg} = 20\ kN/m^3$

$\phi = 20°$, $N_c = 17.7$, $N_q = 7.4$, $N_\gamma = 5$

1) W.T is very deep

2) W.T out GS

SBC 1 = ?, SBC = ?

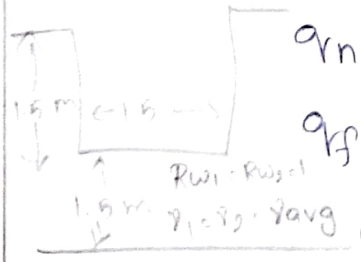
reduction in SBC = ?

Solution:

$$q_s = q_{ns} + \gamma D, \quad q_{ns} = \frac{q_{nf}}{F}$$

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

$$q_f = 1.3\, C N_c^{\circ} + \gamma D N_q R_{w_1} + 0.4 B \gamma N_\gamma R_{w_2}$$



C ⇒ Not given, so assume soil is sandy

C = 0

Case 1): W.T is deep Assume, FOS = 3

$$q_f = (20 \times 1 \times 7.4 \times 1) + (0.4 \times 1.5 \times 20 \times 5)$$

$$\boxed{q_f = 208\ kN/m^2}$$

$d_c = 1 + 0.2 \frac{D}{B} \sqrt{N_\phi}$ $\quad N_\phi = \tan^2\left(45 + \frac{\phi}{2}\right)$

$d_q, d_\gamma = 1$ if $\phi < 10°$.

$d_q, d_\gamma = 1 + 0.1 \frac{D}{B} \sqrt{N_\phi}$ if $\phi > 10°$.

$\Rightarrow i_c, i_q, i_\gamma \Rightarrow$ Inclination factors

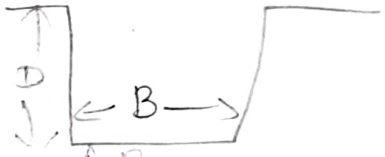
$i_c = i_q = i_\gamma = 1$ $\quad i_c = i_q = \left(1 - \frac{\alpha}{90}\right)^{2}$ α: angle of inclination

$$i_\gamma = \left(1 + \frac{\alpha}{\phi}\right)^2$$

$\Rightarrow w'$ — water reduction factor.

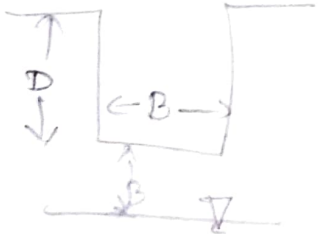
Case 1: If W.T beyond the depth of equivalent to B+D

$w' = 1$



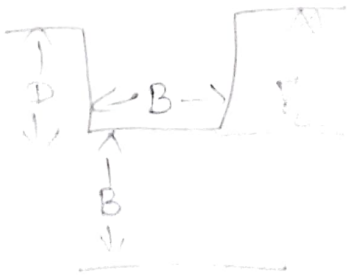
Case 2: If W.T below the base & ~~depth~~ = B+D

$w' = 0.5 + \left[1 + \frac{Z_{we}}{B}\right]$



Case 3: If W.T below the G.S or within the base.

$w' = 0.5$



1. In footing is a size of 1.8m x 3m has to transmit the load of a column at a depth of 1.5m calculate the safe load with FOS 3 & the soil has the following properties prosity - 10% SD Gravity - 2.67, cohesion - 8 KN/m², $\phi = 35°$, Use IS equation

Given: Rectangular footing of size 1.8m x 3m

B = 1.8 m (smaller dimension), L = 3m

D = 1.5 m, FOS = 3

n = 10%, G = 2.67, C = 8 KN/m², $\phi = 35°$ (GSF)

Solution: Safe load = q_s x Area

$q_s = q_{ns} * \gamma D$, $q_{ns} = \frac{q_{nf}}{F}$

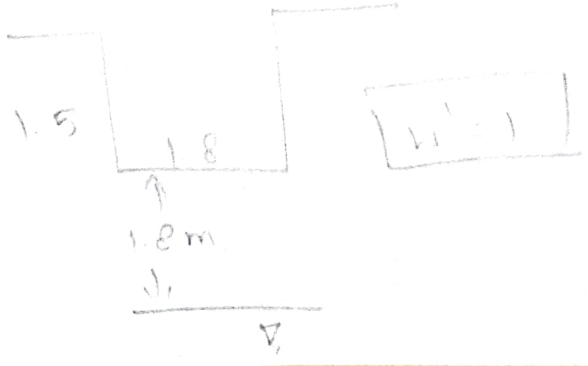
$q_{nf} = q_f - \gamma D$

$q_f = CN_c S_c d_c i_c + \frac{1}{2} D N_q S_q d_q i_q + 0.5 B \gamma N_\gamma d_\gamma S_\gamma i_\gamma$,

C = 8 KN/m²
D = 1.5m
B = 1.8m
γ = ?

$\gamma = \frac{G\gamma_w}{1+e}$ $e = \frac{n}{1-n} = \frac{0.1}{1-0.1} = 0.11$

$= \frac{2.67 * 9.81}{1+0.11} = \boxed{23.59 \text{ KN/m}^3}$



From the table,

For $\phi = 35°$, $N_c = 46.12$, $N_q = 33.3$, $N_\gamma = 48.03$

S_c, S_q, S_γ depends upon type of footing.

Pg no: 8, 9
IS 6403

For rect footing, $S_c = 1 + 0.2 \frac{B}{L} = \boxed{1.12}$

$S_q = 1 + 0.2 \frac{B}{L} = \boxed{1.12}$

$S_\gamma = 1 - 0.4 \frac{B}{L} = \boxed{0.76}$

$\phi > 10°$, $d_c = 1 + 0.2 \frac{D}{B} \sqrt{N\phi}$

$N\phi = \tan^2(45 + 35/2) = 3.690$

$= 1 + 0.2 \frac{1.5}{1.8} \sqrt{\frac{89.08}{3.69}} = \cancel{2.57}$ $\boxed{1.32}$

$d_q, d_\gamma = 1 + 0.1 \frac{D}{2} \sqrt{N\phi} = 1 + 0.1 \left(\frac{1.5}{1.8}\right) \sqrt{\frac{\cancel{89.08}}{3.69}} = \boxed{1.16}$

$i_c = i_q = i_\gamma = 1$

$q_f = (8 \times 46.12 \times 1.12 \times 1.32 \times 1) + (23.59 \times 1.5 \times 33.3 \times 1.16) +$
$(0.5 \times 1.8 \times 23.59 \times 0.3 \times 1.16 \times 0.76 \times 1 \times 1)$

$q_f = 2095 \text{ KN/m}^2$

$q_{nf} = 2975 - (23.59 \times 1.5) = 2939 \text{ KN/m}^2$

$q_{ns} = \frac{2939}{3} = 979 \text{ KN/m}^2$

$q_s = 979 + (23.59 \times 1.5) = 1015 \text{ KN/m}^2$

Safe load $= 1015 \times 1.8 \times 3 = \boxed{5482 \text{ KN}}$

2. Find the dimension of the rect. footing to carry desload of 1000 KN with FOS = 3. A foundation is at a depth 1.2 m below GL in clayey soil with UCC strength 120 KN/m². The width to length ratio of footing can be 0.8 adopt BAS 6403 recommendations.

According to IS 6403 - 1981 ⇒ The ultimate bearing capacity of C-Φ soil,

$$q_f = CN_c s_c d_c i_c + \gamma D N_q d_q s_q i_q + 0.5 B \gamma N_\gamma d_\gamma s_\gamma i_\gamma$$

(C-Φ) sandy soil, C = 0

q_f, q soil, $C=0$, sand } $q = \gamma D N_q d_q s_q i_q + 0.5 B \gamma N_\gamma d_\gamma s_\gamma i_\gamma w'$.

q_f, C soil, $\phi=0$, clay } $q = \gamma D N_q d_q s_q i_q + 0.5 B \gamma N_\gamma d_\gamma s_\gamma w'$.

Given:

Rectangular footing

FOS = 3, design load = 1000 KN Clay soil $\phi=0$

D = 1.2m, B/L = 0.8

UCC, $q = 120 KN/m^2$, $C = \frac{q}{2} = 60 KN/m^2$ ϕ

BAS 6403

Solution: $q_f = C N_c S_c d_c i_c$

$\boxed{i_c = 1}$ $\quad \phi = 0 \Rightarrow \boxed{N_c = 5.14}$

$$S_c = 1 + 0.2 \frac{B}{L} = 1 + 0.2 (0.8)$$

$$\boxed{S_c = 1.16}$$

$$d_c = 1 + 0.2 \frac{D}{B} \sqrt{N_\phi}$$

$$N_\phi = \tan^2 \left(45 + \frac{\phi}{2}\right)$$

$$= \tan^2 \left(45 + \frac{0}{2}\right) = 1$$

$$d_c = 1 + 0.2 \left(\frac{1.2}{B}\right) \times 1$$

$$\boxed{d_c = 1 + \frac{0.24}{B}}$$

$$q_f = 60 \times 5.14 \times 1.16 \times \left(1 + \frac{0.24}{B}\right) \times 1$$

$$q_f = 357.744 \left(1 + \frac{0.24}{B}\right)$$

$$q_f = 357.744 + \frac{85.85}{B} \rightarrow (1)$$

Safe load = Design load × F.O.S

$= 1000 \times 3 = 3000$ KN.

Safe pressure $(KN/m^2) = \frac{3000}{B \times L} \rightarrow (2)$

$\frac{B}{L} = 0.8$

$L = \frac{B}{0.8}$

$(1) = (2)$

$$357.74 + \frac{85.85}{B} = \frac{3000}{B \times L}$$

$$357.74 + \frac{85.85}{B} = \frac{3000 \times 0.8}{B \times B}$$

Factors Affecting Bearing capacity:

I.S code stipulates that the following factors to be taken in arriving final bearing capacity value.

a) The effect of interference of different foundation should be taken into account.

b) The type and condition of the soil or rock to which the foundation transfers the loads.

c) The type of building namely, load bearing, shear wall, tubular etc., the general layout of the columns and load bearing walls showing the estimate loads, including moments and torques due to various loads.

d) The change in ground level.

e) Seismic Zone of the region.

f) Chemical condition of the sub soil water.

g) Any buried elements like water/drainage pipelines influences on the bearing capacity & settlement calculations.

h) Flood erosion, man-made cutting etc, which affect in arriving minimum depth of foundation.

i) Adjacent site conditions like topographs, environment, distance, depth and type of existing surroundings, buildings etc.,

j) In the soil directly below the foundation; if a thin layer of soil having smaller safe bearing capacity is seen, it should be removed or the smaller bearing capacity value of that layer has to be taken as safe bearing capacity at the level of such stratum.

k) when the footing are closely spaced, the pressure transmitted to the underlying soil will overlap. In such cases, the pressure in the overlapped zones will have to be considered.

l) Depth factor, inclination factor, size factor in footing, shape factor.

m) Footing resting on slope, bottom of the footings at different level, footing adjacent to sloping ground all affects the bearing capacity.

Bearing Capacity from in-situ tests (SPT, SCPT and plate load).

SPT:

N value can be determine from std. penetration test. Based on the 'N' value the net ultimate bearing capacity of soil for 25 mm settlement can be determined from the formula.

$$q_{nf} = 35 (N-3) \left[\frac{B + 0.3}{2B} \right]^2 R_{w_2} R_d$$

25 mm settlement

$$R_d = \left(1 + 0.2 \frac{D}{B} \right) \leq 1.2$$

$$R_{w_2} = 0.5 \left[1 + \frac{Z w_2}{B} \right]$$

$Z w_2 \Rightarrow$ depth of W.T below the base of footing.

1. A footing 2m x 2m is a depth of for which the N value is equal to 24, water table is as a depth of 2m from the ground level. Determine the net ultimate B.C. If the permissible settlements is 40mm.

Given: Square footing (2m x 2m)

$B = 2m$, $D = 1.5m$

W.T at 2m from G.S

$N = 27$

$q_{nf} = ?$

40mm settlements.

Solutions:

$$q_{nf} = 35 (N-3) \left[\frac{B+DB}{2B} \right]^3 R_{w_2} R_d$$

25 mms

$$R_d = \left(1 + 0.2 \frac{D}{B}\right)$$

$$= 1 + 0.2 \left(\frac{1.5}{2}\right) = 1.15$$

$$R_{w_2} = 0.5 \left[1 + \frac{Z w_2}{B}\right]$$

$$= 0.5 \left[1 + \frac{0.5}{2}\right] = 0.625$$

$$q_{nf} = 35 (27-3) \frac{2 + 0.83}{2(2)} \times 0.625 \times 1.15$$

$$\boxed{q_{nf} = 199.61 \text{ kN/m}^2}$$

$$q_{nf} = \frac{199.61}{25} \times 40$$

$$\boxed{q_{nf} = 319.38 \text{ kN/m}^2}$$

400 mms

2. Using the following values find the width of the footing N = 20, settlements S = 25mm, square footing, safe load = 800, D = 2m, W.T 1m below G.L, unit wt. of soil = 20 KN/m³, FOS

Solution: Safe load = $q_s \times$ Area B = ?

$$q_s = q_{ns} \times \delta D$$

$$q_{ns} = \frac{q_{nf}}{F}$$

$$q_{nf} = 35 (N-3) \left[\frac{B+0.3}{2B}\right]^2 R_{w_2} R_d$$

25mm S.

$$R_{w_2} = 0.5 \left[1 + \frac{Z w_2}{B}\right]$$

$$= 0.5 \left[1 + \frac{0}{B}\right] = 0.5$$

$$R_d = 1 + 0.2 \frac{D}{B} \leq 1.2 \quad \therefore R_d = 1.2$$

$$q_{nf} = 35 (20-3) \left[\frac{B+0.3}{2B}\right]^2 (0.5)(1.2)$$

$$= 357 \left[\frac{B}{2B} + \frac{0.3}{2B}\right]^2$$

$$= 357 \left[(0.25) + \frac{0.0225}{B^2}\right]$$

$$q_{nf} = 89.25 + \frac{8.0325}{B^2}$$

$$q_{ns} = \frac{q_{nf}}{F} = \frac{89.25 + \frac{8.0325}{B^2}}{3}$$

$$q_{ns} = 29.75 + \frac{2.6775}{B^2}$$

$$q_s = q_{ns} + \gamma D = 29.75 + \frac{2.6775}{B^2} + (20 \times 2)$$

$$q_s = 69.75 + \frac{2.6775}{B^2}$$

$$800 = q_s \times B \times B$$

$$800 = \left(69.75 + \frac{2.6775}{B^2}\right) \times B^2$$

$$800 = 69.75 B^2 + 2.6775$$

$$\boxed{B = 3.38 m}$$

Determination of Bearing Capacity by Plate Load test:

* This test is standardised by IS (1888-1972)
* This is a field test.
* It is used to determine B.C & probable settlement of soil.
* This test consists of bearing plate, loading arrangements. loading arrangments consists of hydraulic jack, Dial gauges and bags or concreate blocks, test pit.

* Bearing Plate is a square plate which varies from 30 cm × 30 cm to 75 cm × 75 cm

* Test pit: Width of test pit is 5 times the width of the plate width is small hole may be made in the test pit edge of the hole.

edge of the plate

* Loading arrangement: Load may be applied on the plate by means of hydraulic jack interms of sand bags covered concentrate block The settlements may be measured by means of dial gauges

1) Plate is seated in the hole in test pit

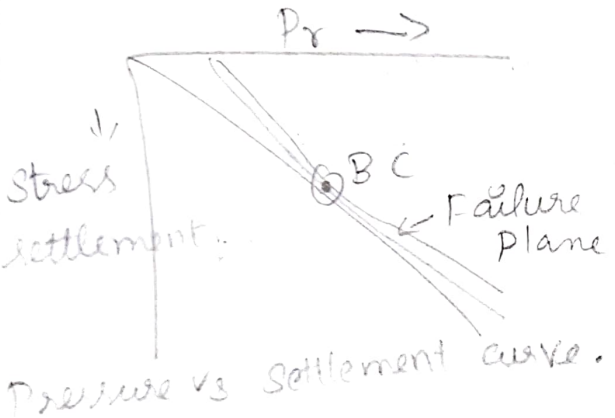
2) Load is applied with the help of hydraulic jack interms of sand bags or concrete blocks

3) As per IS (1888 - 1982) an initial load of 7 kg/cm² is released before the actual test is started

4) Load is applied in convienent increments.

5) The deflection or settlements of plate should be observed for each increment of load after an internal of 1, 5, 20, 20, 40, 60 min & further can interval of one hour.

6) The load intensity & settlements are plotted in graph.



7) By means of this test, Bc can be determined from the graph & also by means of some equations.

8) For sandy soil, $q_f = q_p \left(\frac{B_F}{B_p} \right)$

9) For clay soil, $q_f = q_p$

q_p - ultimate B.C of plate B_f - width of footing

B_p - width of plate.

in soil mass. Reducation in water content increases the B.C

4) Grouting the soil mass ⇒ This method is based on injection chemicals into the soils mass injection of chemical will increases the B.C.

Important Questions:

1. Area ratio problem & defination
2. How will you decide the depth of soil exproration
3. Diff Representative & Non - representive sample.
4. What are the Geophysical method of soil exploration.
5. What are the information req. in general exploration.
6. objectives of soil exploration
7. Define SP Number
8. Corrections of SPT
9. What is bore log?
10. Requirement of good foundation.
11. Imp defination ⇒ Bc, ultimate B.C, SBC

Factors Affecting bearing Capacity of the Soil:

1) Physical and Engg properties of the soil
2) Size & Shape of the foundation
3) Load from structure on foundation
4) Settlement
5) Ground water table.
6) Chemical conditions of subsoil waters
7) Bearing Capacity factors (Depth. safe)
8) Adjacement site Condition.

Methods to improve bearing Capacity of Soil:

1) Increase in depth of the foundation ⇒ This method can be adopted only when increases in the case of the construction.

2) Reducing the air space in the soil mass.

3) Reduction in air voids presents in the soil mass increases the BC of soil ⇒ This method is done by compaction of the soil.

4) Draining sub soil water ⇒ Water present in soil mass is drained by loging open jointed pipes

12. Factors affecting B c

13. Assumptions made in Terzaghi's equations.

14. Define shallow foundation.

15. Types of failures.

16. List the types of shallow footing

17. Methods, to improve B.C of the soil.

Part-B

1. What are the methods of soil exploration

2. Explain SPT

3. Explain plate load Test

4. Problems releated to B.C of soil ⇒ Terazaghi's & ISeq.

SETTLEMENT

Settlement:

→ This is based on the principles of Compressiability (or) Consolidation. This settlement is the gradual sinking of a structure due to compression of the soil.

⇒ In otherwords, the downwards vertical movement of the soil is known as settlement. Its effects on the structure depends upon its structure, uniformity, duration of time over which it takes place.

⇒ The procedure for the computations of anticipated settlement is called as settlement analysis.

⇒ In general settlement is classified into

* Allowable (or) Permissible settlement
* Uniform settlement
* Differential settlement.

Allowable (or) Permissible settlement:

⇒ Foundation satisfies the limiting value of settlement is called as allowable settlement.

⇒ According to IS 1978, the settlement ranging from 200mm 300mm is generally permitted for various structures.

⇒ Eg: For Isolated settlements in sandy soil, the max. permissible settlements is 40mm, it must be uniform. For clay soil - 65 mm.

→ Footing is raft.

Sandy soil → 10 - 65 mm

Clay soil → 65 100 mm

Uniform Settlement

⇒ The magnitude of settlement below the base of the footing is same at all points

⇒ Uniform settlement is possible, if the base of footing.

1) Rest on homogeneous soil

2) Base in Rigid.

Differential settlements.

⇒ The difference b/w magnitude of settlements at any two points is known as differential settlements

⇒ Differential settlements are rigid base & produces large moments leads to structural damage

Cause of settlements

1) Underground erosion

2) Structural collapse of soil

3) Landslides

4) Thermal changes ⇒ Eg: Shrinkage

5) Vibrations in ~~sand~~ cohesionless soil.

6) Creep in clay soils

7) Mining subsides ⇒ Removal of minerals from the soil.

Remidial measures for avoiding settlements:

1) Removal of soil strata.

2) Use of properly designed pile foundation

3) Reduction of contact pressure at the soil.

4) Stabilizing the soil.

Remedial measures for avoiding differential settlements:

1) Provision of raft or mat foundation

2) Laying the foundation at great depth.

3) Deeper foundation (Pile, caution, well) for heavier loads.

4) Increase in size of footing on clay.

Total Settlements: $\boxed{S = S_i + S_c + S_s}$

S_i = Immediate Settlement / Distortion.

S_c = Consolidation "

S_s = Secondary "

Immediate Settlements : (S_i)

* It is also called as distoration settlements (or) contact settlements.

* It is supposed to take place immediately after the application of water.

$$S_i = q B \left(\frac{1-\gamma^2}{E_s}\right) I_t$$

(Rill)

q - Pressure
B - Breath of footing
γ - Poission ratio: 0.5
E_s - Modulus of Elasticity
I_t - Influence ratio based on the ratio L/B.

I_t values:

1) Flexible footing:

L/B	1	2	3	4	5
I_t	0.56	0.76	0.88	0.96	1

2) Rigid Footing:

L/B	1	2	5	10	5
I_t	0.82	1	1.22	1.26	

1. Reinforced concrete foundation of dimension 18 m x 36 m exists a uniform pressure 180 kN/m² on a soil mass with modulus of elasticity 45 MN/m². Determine the immediate settlements.

Given: Size = 18m x 36m

B = 18m, L = 36m

$q = 180\ KN/m^2$

$f_c = 45\ MN/K^2 = 45 \times 10^3\ KN/M^2$

$S_1 = ?$

Solution:

$$S_i = q\,B\left(\frac{1-\gamma^2}{Es}\right) I_t$$

I_t : Assumes footing Rigid

$\frac{L}{B} = \frac{36}{18} = 2$

If $\frac{L}{B}$ is 2, $I_t = 1$.

Assume $\gamma = 0.5$

$$= 180 \times 18\left(\frac{1-0.5^2}{45 \times 10^3}\right) \times 1$$

$S_1 = 0.054$

$= 54mm$

2. Ass. settlements of a footing I = 1.5 m x 1.5m and footing II is 3.2m x 3.2m. 250 KN/m² is 300mm what would be settlement of footing 3.2 x 3.2 carrying a load of 175 KN/m² sub soil conditions are identical for both cases. What is your inferences from the result.

Given:

$$S_i = q\,B\left(\frac{1-\gamma^2}{Es}\right) I_t$$

$$\frac{S_{i1}}{S_{i2}} = \frac{250 \times 1.5\left(\frac{1-\gamma^2}{Es}\right) I_t}{175 \times 3.2\left(\frac{1-\gamma^2}{Es}\right) I_t}$$

$$\frac{0.01}{S_{i2}} = \frac{250 \times 1.5}{175 \times 3.2}$$

$$S_{i2} = 0.0448 = \boxed{44.8\,mm}$$

∴ size of footing increases settlements also increases.

Condition Settlements (S_c):

This component of settlement occurs due to gradual expulsion of water from the voids in the soil mass after the immediate settlements.

$$S_c = \frac{C_c H_0}{1+e_0} \log_{10} \frac{\overline{\sigma_0} + \Delta\overline{\sigma}}{\overline{\sigma_0}}$$

C_c - compression index $\Rightarrow 0.009 (w_L - 10\%)$

H_0 - thickness of clay layer

e_0 - Initial void ratio

$\overline{\sigma_0}$ - Initial effective pressure → Total Pre - Pore water pr ($\sigma - u$).

$\Delta\overline{\sigma}$ - Incremental pr

$S_c = m_v \Delta\overline{\sigma} H_0$

m_v : Coefficient of volume compressibility $= \frac{a_v}{1+e_0}$

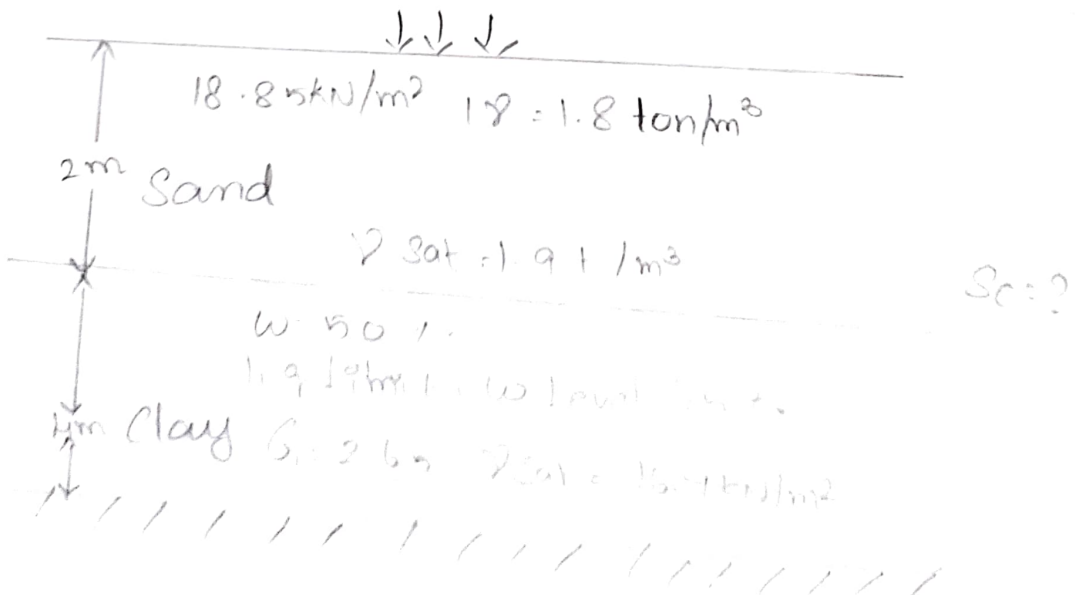
$a_v = \frac{\Delta E}{\Delta\sigma}$ → Δe - Change into void ratio

$\Delta\sigma$ - Change in pressure

a_v - Co.eff of compressibility.

1. The soil profile at a building site consists of dense sand upto 2m depth. Normally soft clay from 2-6m depth & impervious rock below 6m depth. The water table is at 0.4m below ground level sand has a density 1.85 ton/m³ above water table and 1.9 ton/m³ below water Table. For clay water content 50%, liquid limit 65%, SP. gravity 3.65, Calculate the settlements resulting from the uniformly distributed surface load of 4 ton/m² applied over an extensive area of the site.

Given:



Solution:

$$\gamma_{sat} = \left(\frac{G + Se}{1+e}\right)\gamma_w$$

$$S = 100\% = 1$$

$$e = wG$$

$$= 0.5 \times 2.65 = 1.325$$

$$\gamma_w = 9.81 \text{ KN/m}^3.$$

$$\gamma_{sat} = \left(\frac{2.65 + (1).325}{1+1.325}\right) = 9.81$$

$$\gamma_{sat} = 16.4 \text{ KN/m}^3.$$

$$S_c = \frac{C_c H_0}{1+e_0} \log_{10} \frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}$$

$$C_c = 0.009 + (w_E - 10\%)$$

$$= 0.009\ (65\% - 10\%) = 0.495$$

H_0 = thickness of clay layer = 4m

$e_0 = 1.325$

$$\bar{\sigma_0} = \sigma - u$$

$$= \left[(0.4 \times 18.5)\right] + \left[(1.6 \times 1.9)\right] - 1.6 \times 9.81 + (2 \times 15.4) - 2 \times 9.81$$

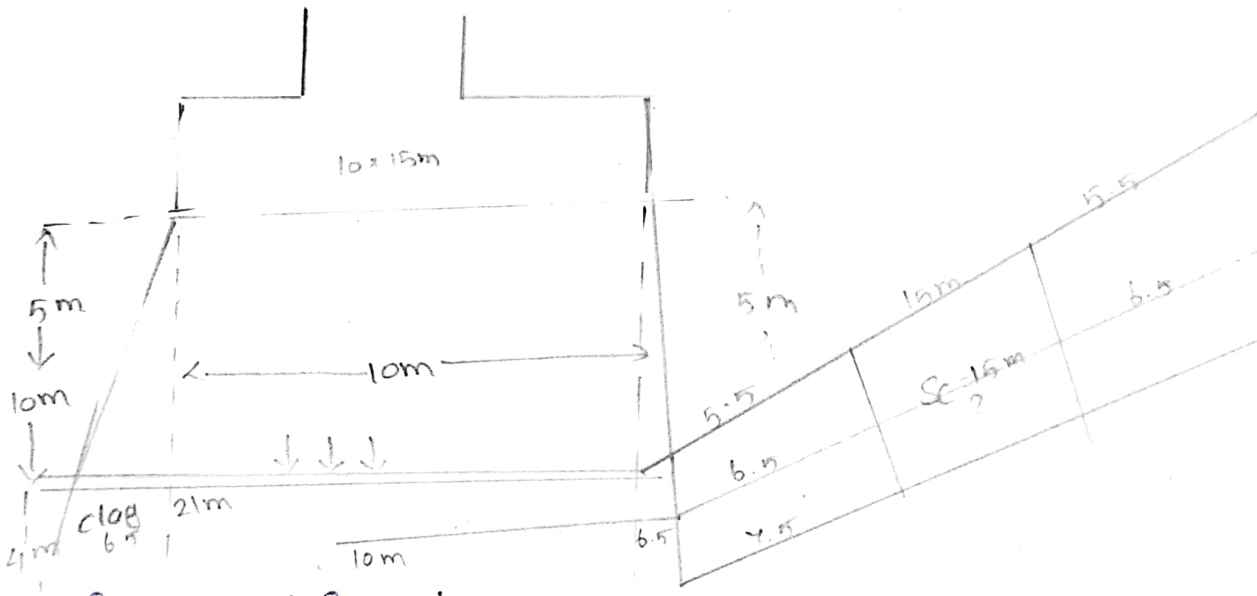
$$\bar{\sigma}_0 = 3.3 \times 2.84 \text{ KN/m}^2$$

$$S_c = \frac{0.495 \times 4}{1+1.325} \log_{10} \frac{35.884 + 4}{35.884}$$

$$S_c = 0.39 = \boxed{39\text{mm}}$$

2. A foundation 10m wide 15m extends a loads of 100 KN the top soil is 1m tk, layers of silt a sandy stratuation 10m tk lies under the silt below which lies clay layer with 1k.4m. The unit wt. of silt 50 KN/m³ for sand 10KN/m³ above W.T & 18KN/m³ below W.T. SP. gravity = 2.7, void ratio 1.2, liquid limit 60%, W.T is at 6m below GS, Pr. distribution 2:1, calculate the settlement.

Given:



$$\gamma_{sat} = \left(\frac{G + Se}{1+e}\right)\gamma_w \qquad S = 100\% = 1$$

$$= \left(\frac{2.7 + 1(1.2)}{1+1.2}\right) 9.81$$

$$\boxed{\gamma_{sat} = 17.39 \text{ KN/m}^3}$$

$$S_c = \frac{C_c H_c}{1+e_o} \log_{10} \frac{\overline{\sigma_o} + \overline{\Delta\sigma}}{\overline{\sigma_o}}$$

$$C_c = 0.009 \ (w.e - 10\%)$$

Estimate the final settlement of the fill.

Given:

$\gamma = 18.84\ KN/m^3$

$H = 3.5m$, Ht of Sand fill $= 3m$

$m_v = 7 \times 10^{-4}\ m^2/KN$ $\quad S_c = ?$

Solution:

$S_c = m_v \Delta\bar{\sigma} H_0$

$H_0 = 3.5$

$\Delta\bar{\sigma} = 3 \times 18.84 = 56.52\ KN/m^2$

$S_c = 7 \times 10^{-4} \times 56.52 \times 3.5$

$S_c = 0.1384m = 138.4mm$

The following date were obtained

Load Intensity KN/m^2	50	100	200	300	400	500	600
Settlement (m)	15	2	4	7.5	12.5	20	40

Plot the Pr. Settlement curve & determine ultimate bearing Capacity of the soil.

Ans: In graph sheet.

4. A large layer of soft clay is 6m and lies under a newly constructed building, a wt of sand overlying the clay layer produces a pressure of $260\ KN/m^2$ & the new construction increases the pressure by $100\ KN/m^2$, if the compression

$H_0 = 4m, \quad e_0 = 1.2$

↳ tk. of clay layer

$\bar{\sigma}_0 = \sigma - u$

$= 1 \times 15 + 5 \times 10 + [(5 \times 18) - 5 \times 9.81] + [(2 \times 17.39) - 2 \times 9.81)]$

$\boxed{\bar{\sigma}_0 = 121.1 \text{ KN/m}^2}$

$\Delta\bar{\sigma} = \frac{\text{Load}}{\text{Area}} \qquad \text{Load} = 100 \text{ KN}$

$A\bar{\sigma} = \frac{1}{6}\left[A\bar{\sigma}_t + 4\bar{\sigma}_m + A\bar{\sigma}_B\right]$

$A\bar{\sigma}_t = \frac{100}{21 \times 26} = 0.183 \text{ KN/m}^2$

$4\bar{\sigma}_m = 4 \times \frac{100}{23 \times 28} = 0.621 \text{ KN/m}^2$

$A\bar{\sigma}_B = \frac{100}{25 \times 30} = 0.133 \text{ KN/m}^2$

$A\bar{\sigma} = \frac{1}{6}\left[0.183 + 0.621 + 0.133\right)$

$\boxed{= 0.156 \text{ KN/m}^2}$

$S_c = \frac{0.45 \times 4}{1 + 1.2} \log_{10} \frac{121.1 + 0.156}{121.1}$

$= 4.57 \times 10^{-4} \text{ m}$

$\boxed{S_c = 0.45 \text{ mm}}$

3. Sandfill compacted with the density of 18.84 KN/m³ is to be placed on a compressible saturated deposit 3.5m to its thickness. ht of the sandfill is to be 3m, if the volume compressibility of the deposit is $7 \times 10^{-4} \text{ m}^2/\text{KN}$.

increases the pressure by 100 KN/m2 if the compression index is 0.5 Compute the settlement. Water content is 40%. SP. gravity is 2.65, soil is in saturated condition.

Given:

Clay - thickness = 6m

Initial pressure = 260 KN/m2.

Incremental pressure = 100 KN/m2.

w = 40%., Sp. Gs = 2.65, Compressibility index = 0.5

S = ?

Solution:

$e = wG/S$ soil is in saturated condition, $\therefore S = 1$

$= 0.4 \times 2.65 \neq 1$

$= 1.06$

$$S_c = \frac{C_c H_0}{1+e_0} \log_{10} \frac{\bar{\sigma_0}}{\bar{\sigma_0}} + \frac{\Delta \bar{\sigma}}{\bar{\sigma_0}}$$

$$= \frac{0.5 \times 6}{1+1.06} \log_{10} \frac{260+100}{260}$$

$= 1.456 \log_{10} (1.3846)$

$= 1.456 \times 0.14132$

$S_c = 0.205\ 1m$

$\boxed{S_c = 205.7\ mm}$

Important questions.

1. What are the different types of settlements
2. Define differential settlements.
3. What are the causes of settlements.
4. Explain the methods of minimizing settlements.
5. Explain plate load test with limitations
6. Problems related to $S_c = \frac{C_c H_0}{1+e_0} \log_{10} \frac{\bar{\sigma_0} + \Delta\bar{\sigma}}{\bar{\sigma_0}}$
7. Define settlements & list the Components.

UNIT- III : FOOTING AND RAFTS

Types of isolated footing, Combined footing, Mat footing - Contact pressure and settlement distribution - Proportioning of foundations for conventional rigid behaviour - Minimum depth for rigid behaviour - Applications - Floating foundations - Special foundations - Seismic force consideration - Codal provision.

⇒ Structural design of footing:

i) Spread footing: It is basically a concrete patch used to transfer the loads from the column to the soil.

If the soil is having sufficient bearing capacity to withstand the load from the structure.

Advantages:

1. More Economical
2. Easy to Construct
3. Require simple tools for construction.
4. Require normal skilled labour.

⇒ Design procedure for Isolated footing:

1. Determination of base area.

$$\text{Base Area, } A_f = \frac{\text{Load}}{S_{BC}(q_a)}$$

2. Determination of size of the footing.

Square ⇒ $A_f = B^2$, Rectangular ⇒ $A_f = B \times L$

Circular ⇒ $A_f = \pi D^2$

3. Determination of effective depth.

(Square) $d^2(q_s - q_4) + d(q_s - q/2) = (B^2 - a^2)\, q/4$.

If footing is rectangular ⇒ B^2 is replaced by

BL (1.2)

$B^2 = BL$

If footing is circular,

$$d^2(q_s + q/4) + d(q_s + q/2) = (D^2 - a^2)\, q/4.$$

According to IS456-2001, If moment is known

$$M_u \text{ ultimate} = 0.138\, f_{ck}\, bd^2.$$

4. Find Bending Movement

$$= q\, \frac{B(B-a)^2}{8}.$$

5. Determination of Area of Steel

6. Assume dia of the rod A_{sg} (Ast)

7. To know single area of the steel (ast)

8. No. of bars.

9. Spacing. 10) Draw Reinforcement.

q_s = Permissible stress

D = Effective depth

q = Pressure = Load/area

a = size of the column.

1. Design a square footing to carry load of 850 KN on column 300 x 300mm. Allowable soil pressure 200 KN/m². Permissible stress 500KN/m². Use M20 & Fe - 400 Steel.

Given:

Design square footing

Load = 850 KN

Size of the column : 300 x 300mm = 0.3 x 0.3m

q_a = 200 KN/m²

q_s = 500 KN/m²

Solution:

1. Area of footing, $A_f = \frac{\text{Load}}{q_a} = \frac{850}{200} = \boxed{4.25 m^2}$,

2. Size of the footing,

Square $\Rightarrow A_f = B^2$

$4.25 = B^2 \Rightarrow B = 2.06 \simeq 2.1m$

$\boxed{B = 2.1m \times 2.1m}$ $\quad q = \frac{850}{2.1 \times 2.1} = 192.74 KN.$

3. Effective depth.

Square $\Rightarrow d^2 (q_s - q/4) + d (q_s - q/2) = (B^2 - a^2) q/4$

$d^2 (500 - 192.74/4) + d (560 - 192.74/2) = (2.1^2 - 0.3^2 \times 192.14)$

$451.81 d^2 + 403.63d = 936.11 \quad 268.45$

$d = 0.370l = 370mm$

D_f = effective depth +

= 370mm + 30mm

= 400mm = 0.4m

Plan dimension of isolated footing,

= 2.1 x 2.1 x 0.4m

4. Bending movement

$$= q \frac{B(B-a)^2}{8}$$

$= 163.93$ KN.m $= 163.93 \times 10^6$ N.mm

5. Ast,

From ES, 456 2000, Annexure G, equation (B) =>

Area of Steel required.

$$BM = 0.87 f_y A_{st} d \left(1 - f_y \frac{A_{st}}{f_{ck} bd}\right)$$

$f_y = 415$; $f_{ck} = 20 N/mm^2$, $b = B = 2.1m = 2100mm$

d = eff depth = 370

$$1.5 \times 163.93 \times 10^6 = 0.87 \times 415 \times A_{st} \times 370 \left(1 - \frac{415 \ A_{st}}{20 \times 2100 \times 370}\right)$$

$$1.5 \times 163.93 \times 10^6 = 1335.88.5 \ A_{st} \ (1 - 2.67 \times 10^{-8} A_{st})$$

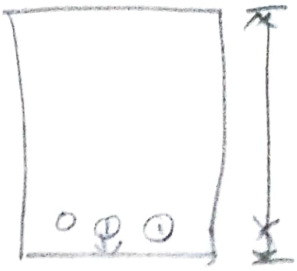
$$1.5 \times 163.93 \times 10^6 = 133588.5 \ A_{st} - 3.5668 1295 \ A_{st}^2$$

$$-3.5665 1295 \ A_{st}^2 + 1335.885 - 1.5 \times 163.93 \times 10^6 - 0.$$

$A_{st} = 1941 mm^2$

6. (Assuming 8mm = 22mm)

Dia, D = 12mm

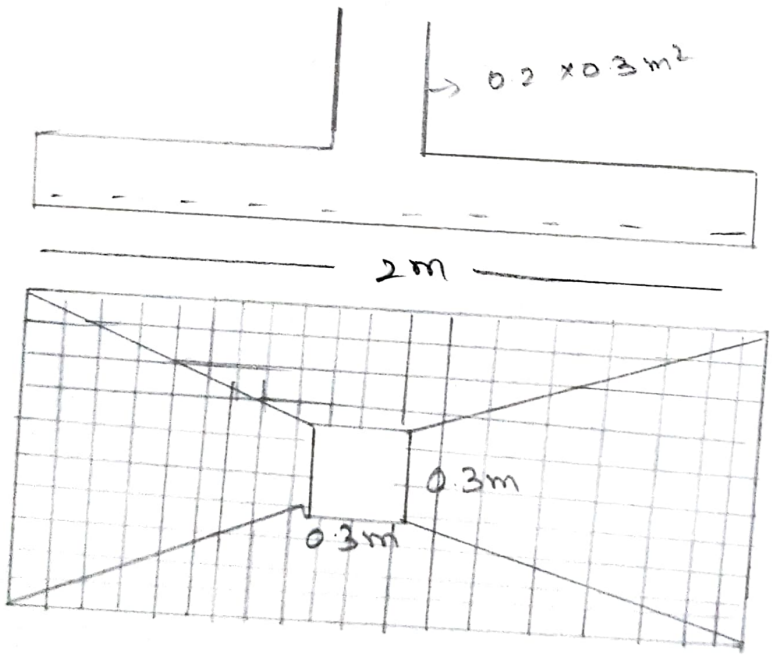


7. $ast = \frac{\pi}{4} \times D^2 = \frac{\pi}{4} \times 12^2 = 113 mm^2$

8. No of bars = $\frac{AST}{ast} = \frac{1941}{113} = 18$ bars

9. Spacing = $\frac{ast}{AST} = \frac{113}{1941} \times 1000 = 58 mm$.

Provide 12mm dia 18 bars at a spacing of 58 mm Centre to centre in both directions.



⇒ Design of combined footing:

↳ A Combined footing supports two or more columns in a single row. It may be trapezoidal or rectangular in plan. It reduces the differential settlements & improve the foundation rigidity.

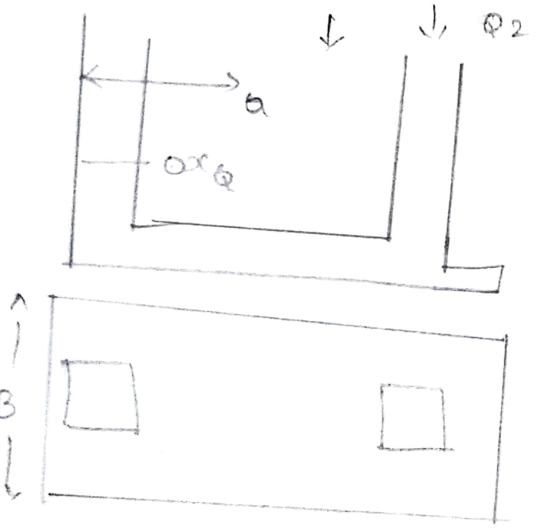
Design of Rectangular Combined footing :

⇒ Footing is perfectly rigid

⇒ Pressure distribution is uniform below the footing.

Procedure :

1. Determine total load ⇒ $Q = Q_1 + Q_2$
2. Determine base area area $A_f = \frac{\text{Total load}}{q_a}$



3. Determine the centroida Column by taking moments

$$\overline{Qx} = Q_2 x_n$$

$$\overline{x} = \frac{Q_2 \times n}{Q}$$

4. Determine size of the fox

$$L = a/2 + x_n + a/2 + c$$

5. Determine width of the footing

$$B = \frac{A_f}{L}$$

6. Determine actual pressure, Actual pressure : $\frac{\text{Load}}{\text{Provided area}}$

7. Draw pressure distribution diagram

8. Compute bending moment & Shear force & draw BMD & SF.

9. Determine area of steel required (MSF)
10. Assume diameter of rods & arr
11. Find no of bars
12. Find spacing of bars
13. Draw reinforcement details.

1. Design a rectangular combined footing to support two adjacent columns at a distance of 5m centre to centre column loads are 600 kN & 900 kN. Heavier column is at a distance of 1m from the property line, allowable soil Pr. 100 kN/m², size of both column is 0.3x0.3m, use M20 concrete, Fe 415 steel.

Given



1) Determination of total load $Q = Q_{P_1} + Q_{P_2}$

$= 600 + 900$

$Q = 1500\ kN$

2) Determine base area

$A_f = \frac{\text{Total load}}{q_a} = \frac{1500}{100} = 15\ m^2$

3) Determine Centroidal Column by taking moments

$$Q\bar{x} = Q_b x_n$$

$$1500 \times \bar{x} = 900 \times 5$$

$$\bar{x} = \frac{4500}{1500} = \boxed{3m}$$

4) Determine the size of the footing

$$L = a/2 + x_n + a/2 + c$$

$$L = \frac{0.3}{2} + 5 + \frac{0.3}{2} + 1 = \boxed{6.3m}$$

5) Determine the width of the footing:

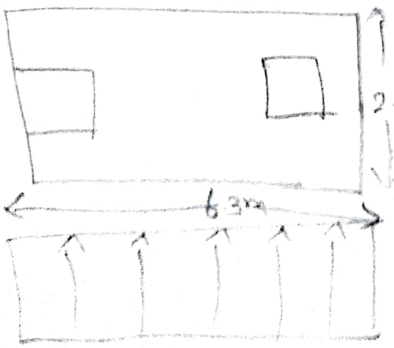
$$B = \frac{A_f}{L} = \frac{15}{6.3} = 2.38 \sim \boxed{2.4m}$$

$$\boxed{B = 6.3 \times 2.4m}$$

6) Determine actual pressure

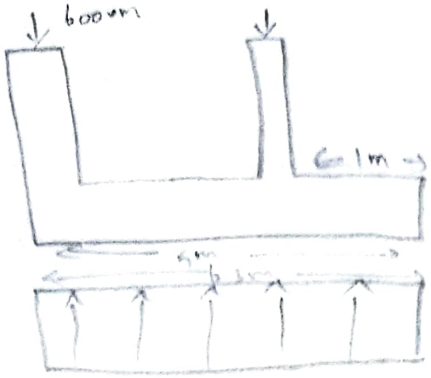
$$\text{Actual pressure} = \frac{\text{Load}}{\text{Provided Area}} = \frac{1500}{6.3 \times 2.4} = \boxed{99.2 kN/m^2}$$

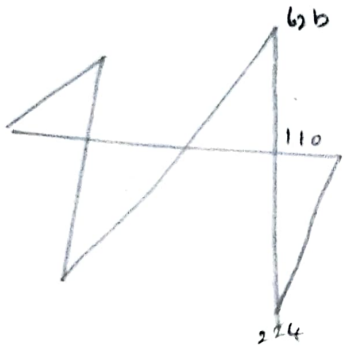
7) Draw pressure distribution diagram.



99.2 x 2.4 = 238.08 kN/m.

8. Compute B.M & S.F & Draw BMD & SFD





Bending Moment:

$$\frac{5-x}{564} = \frac{x}{626}$$

$$x = 2.63$$

$$\Rightarrow 900 \times 2.63 - 238 \times \left(\frac{2.63+1.15}{2}\right) \times (2.63+1.15)$$

$$\Rightarrow 2367 - 1700.31$$

$$= \boxed{666.68 \text{ KN.m}}$$

Max moment = 666 KN.m

Design moment = $1.5 \times 666 = 999$ KN.m

$= 999 \times 10^6$ N.mm.

Effective depth:

According to IS 456 - 2000,

$M_u = 0.138 f_{ck} b d^2$.

$$999 \times 10^6 = 0.138 \times 26 \times 2400 \times d^2$$

$d = 388.34$ mm $\simeq 400$ mm.

Reinforcement in longitudinal direction:

At top: $999 \times 10^6 = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{f_{ck} bd}\right]$

$999 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{A_{st} \times 415}{20 \times 2400 \times 400}\right]$

$999 \times 10^6 = 144420\ A_{st} \left[1 - 2.161 \times 10^{-5} A_{st}\right]$

$999 \times 10^6 = 144420\ A_{st} - 3.1209\ A_{st}^2$

$A_{st} = 8466.27$ $\simeq 8400 mm^2$

$A_{st} = 37808.83$,,

10) Assume diameter of bars = 20mm

$ast = \frac{\pi}{4} \times 20^2 = 314 mm^2$

11) Find no. of bars

No. of bars $= \frac{8400}{314} =$ 27 Numbers

12) Spacing $= \frac{314}{8400} \times 1000 =$ 37mm

Provide 20mm dia bars 27 numbers at a spacing of 37mm Centre to Centre at the top.

At bottom:

$(1.5 \times 157 \times 10^6) = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{f_{ck} bd}\right]$

$2355 \times 10^5 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{A_{st} \times 415}{20 \times 2400 \times 400}\right]$

$2355 \times 10^5 = 124420\,Ast - 3.120\,Ast^2$

$Ast = 1692.5t \approx \boxed{1693\,mm^2}$

$= 244582.5$

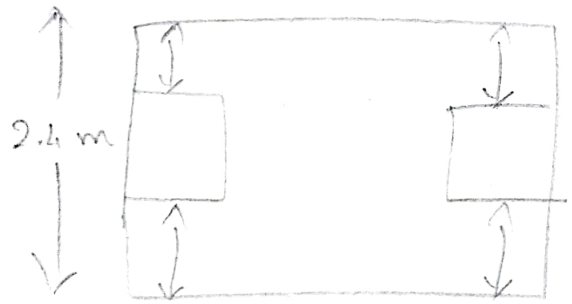
b) Assume dia of bar = 12mm

$ast = \frac{\pi}{4} \times 12^2 = \boxed{113\,mm^2}$

n) No. of bars $= \frac{1693}{113} = 14.9 \approx \boxed{15\ Number}$

(2) Spacing $= \frac{113}{1693} \times 1000 - 66.7 \approx \boxed{67mm}$

Provide 12mm dia of 15 numbers at a spacing of 67mm Centre to centre at bottom.



Under Column no:1

Factor load per unit wt $= \frac{600}{2.4} = 250\ kN/m.$

BM at face of the column $= 250 \times \frac{1.05^2}{2} = 137.8\ kNm.$

Design moment $= 1.5 \times 137.8 \times 10^6 = 206 \times 10^6\ N\text{-}mm.$

Reinforcement under column ①.

$$206 \times 10^6 = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{f_{ck} bd}\right]$$

$$206 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{A_{st} \times 415}{20 \times 2400 \times 400}\right]$$

$$206 \times 10^6 = 144420 A_{st} - 3.1209 A_{st}^2$$

$$A_{st} = 1478 mm^2$$

Assume diameter of bar = 12mm $ast = \frac{\pi}{4} \times 12^2 = 113 mm^2$

No. of bars $= \frac{1478}{113} =$ 13 Numbers

Spacing $= \frac{113}{1478} =$ 76mm

Provide 12mm dia bars of 13 numbers at a spacing of 76mm at column 1.

Under Column ②

Factor load/unit wt $= \frac{900}{2.4} = 375$ kN/m.

BM at face of the column $= 375 \times \frac{1.05^2}{2} = 206$ kN-m

Design moment $= 1.5 \times 206 \times 10^6$ N.mm $= 310 \times 10^6$ N-mm

Reinforcement under column②

$$310 \times 10^6 = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} \times f_y}{f_{ck} bd}\right]$$

$$310 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{A_{st} \times 415}{20 \times 2400 \times 400}\right]$$

$$310 \times 10^6 = 144420\, A_{st} - 3.1209\, A_{st}$$

$$\boxed{A_{st} = 2256\ mm^2}$$

Assume dia of bar = 16mm $a_{st} = \frac{\pi}{4} \times 16^2 = \boxed{201\ mm^2}$

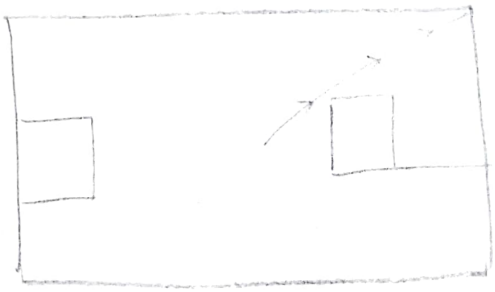
No. of bars $= \frac{2256}{201} = \boxed{11 \text{ numbers}}$

Spacing $= \frac{201}{0.256} \times 1000 = \boxed{89mm}$

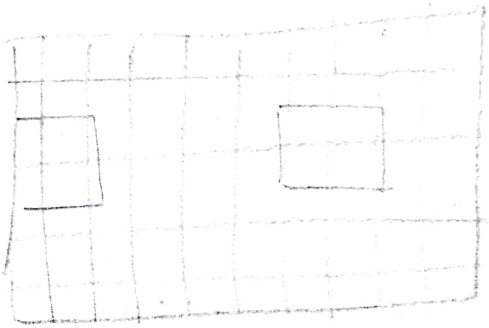
Provide 16mm dia bars of 11 numbers at a spacing of 89mm centre to centre at column 2.

13) Reinforcement diagram:



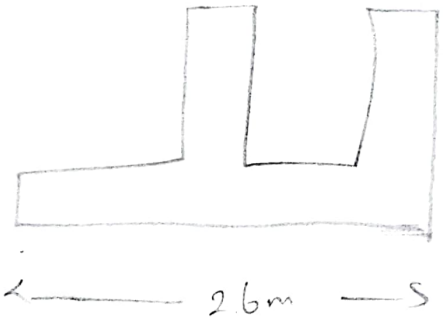


At top



At bottom.

2. Design reinforced concrete rectangular combined footing for two columns A & D. Located at 36m apart. The size of the column is 400 x 400m & 600 x 600 mm & the loads on them are 1000 KN & 1500 KN resp. The projection of the footing beyond the axis of column is restricted to 5mm the limiting bearing capacity of soil is 420 KN/m2. Use M15 Concrete & Fe 415 steel.



Design of Trapezodial Combined footing :

Procedure : $Q_1 > Q_2$

1) Determine total load, $Q = Q_1 + Q_2$,

2) Base area required $A_f = \frac{Q}{q_a}$

3) Determine Centroidal column, $\bar{x} = \frac{Q_2 L}{Q}$ $Q\bar{x} = Q_2 L'$

4) Determine length of the footing 'L'.

5) Check if trapezodal footing is necessary or not.

$\frac{L}{3} > x' < \frac{L}{2}$ $x' = \bar{x} + \frac{a}{2}$ a - width of the column.

6) Determination of width,

$$B_2 = \frac{2Af}{L}\left(\frac{3x'}{L} - 1\right), \quad B_1 = \frac{2Af}{L} - B_2.$$

7) Determine actual pressure.

8) Draw pressure distribution diagram.

9) Determine B.M & S.F & Draw BMD as well as SFD.

10) Determine effective depth.

11) Find Ast in both longitudinal as well as transverse connection.

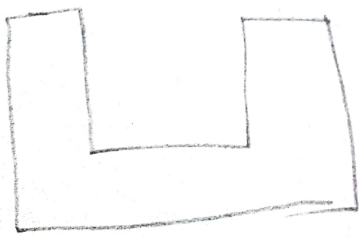
12) Assume dia of bar & determine ast.

13) Find no. of bars.

14) Determine spacing of the bars.

15) Draw reinforcement details.

1. Design a trapezoidal combined footing to support two columns 500 x 500mm carrying the loads of 2000kN & 1500 kN. Allowable soil pr. 200 kN/m² centre to centre dist. b/w column 6m.

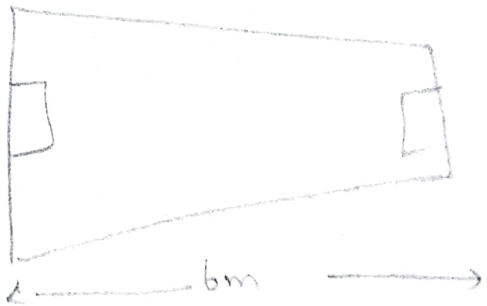


1) $Q = Q_1 + Q_2$

$= 2000 + 1500 = \boxed{3500\ kN}$

2) $Af = \frac{Q}{q_2} = \frac{3500}{200} = \boxed{17.5\ m^2}$

3) $\bar{x} = \frac{Q_2 L'}{Q} = \frac{1500 \times 6}{3500} = \boxed{2.57m}$



4) $2/3 < x' < 1/2$

$x' = \bar{x} + 0/2$

$= 2.57 + \frac{0.5}{2} = \boxed{2.82m}$

$L = 0.25 + 6 + 0.25 = \boxed{6.5m}$

5) $\frac{L}{3} < x' < L/2 \Rightarrow \frac{6.5}{3} < 2.82 < \frac{6.5}{2}$

$= 2.16 < 2.82 < 3.25$

∴ Trapezoidal footing is required

$B_2 = \frac{2Af}{L}\left(\frac{3x'}{L} - 1\right)$

$= \frac{2 \times 17.5}{6.5}\left(\frac{3 \times 2.85}{6.5} - 1\right) = \boxed{1.623m}$

$B_1 = \frac{2Af}{L} - B_2 = \frac{2 \times 17.5}{6.5} = 1.623 = \boxed{3.16m}$

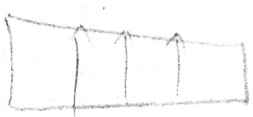
7) Actual area $= \frac{1}{2}(a+b)h = \frac{1}{2} \cdot (3.76 + 1.623) \times 6.5$

$= 17.5 m^2$

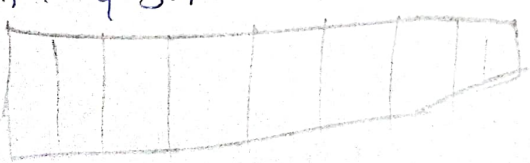
Actual pressure $= \frac{\text{Load}}{\text{actual area}} = \frac{3500}{17.5} = 200 \text{ kN/m}^2.$



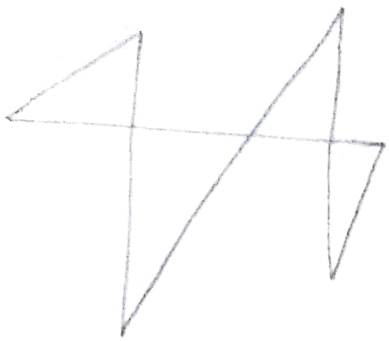
8) Pressure distribution diagram.



9) B.M & S.F



$$\frac{427}{6.5} \times 0.25 = 16.46$$

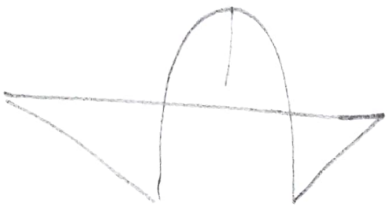


$$\frac{427}{6.5} \times 6.25 = 410$$

$$= 325 \times 0.25 + \left(\frac{1}{2} \times 16 \times 0.25\right)$$

$$= \boxed{83.5\ KN}$$

$$1500 - 83.5 = \boxed{1416.5\ KN}$$



$$= 324 \times 0.25 + \frac{1}{2} \times 17 \times (410 \times 0.25) +$$

$$\left(\tfrac{1}{3} \times 17 \times 0.25\right)$$

$$= \boxed{1856\ KN}$$

$$2000 - 1856 = \boxed{1814.4\ KN}$$

$$Z = \frac{427}{6.5} \times x \qquad \frac{x}{1416.5} = \frac{6-x}{1814}$$

$$\boxed{x = 2.63\ m}$$

$$Z = 189.6\ KN/m$$

Right side:

$$= \left(324 \times \frac{0.25^2}{2}\right) + \left(\frac{1}{2} \times 0.25 \times 16 \times \frac{0.25}{3}\right) = \boxed{10.291\ KN/m}$$

Left Side:

$$= \left(324 \times \frac{0.25^2}{2}\right) + \left(410 \times \frac{0.25^2}{2}\right) + \left(\frac{1}{2} \times 2.63 \times 189 \times \frac{2.63}{3}\right) +$$

$$+ (1500 \times 2.6)$$

$$= 2606.58 = 2600\ KN-m.$$

10) Effective depth:

$$M_u = 0.138\ f_{ck}\ bd^2$$

$$1.5 \times (2600 \times 10^6) = 0.138 \times 20 \times 3760 \times d^2$$

$$d = 613 \simeq \boxed{620mm}$$

11) Ast :

Logitudinal reinforcement at top:

B = 3.76m = 3760mm

According to IS 456-2000,

$$Mu = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{f_{ck} bd}\right)$$

$$1.5 \times (2600 \times 10^6) = 0.87 \times 415 \times A_{st} \times 620 \left(1 - \frac{A_{st} \times 415}{20 \times 3760 \times 620}\right)$$

$$39 \times 10^8 = 223851 A_{st} (1 - 8.90 \times 10^{-6} A_{st})$$

$$39 \times 10^8 = 223851 A_{st} - 1.9924 A_{st}^2$$

12) Assume dia of bar, d = 22mm

$$a_{st} = \frac{\pi d^2}{4} \times \frac{\pi \times 22^2}{4} = 380 mm^2$$

13) No of bars = $\frac{21550}{380}$ = 57 numbers

Provide 22mm dia bar 57 numbers at a spacing of 18mm centre to centre at top

Logitudinal reinforcement at bottom:

$$Mu = 0.87 f_y A_{st} d \left[1 - \frac{A_{st} f_y}{f_{ck} bd}\right]$$

$$1.5 \times 23 \times 10^6 = 0.87 \times 415 \times A_{st} \times 620 \left\{\frac{1 - A_{st} \times 415}{25 \times 3760 \times 620}\right\}$$

$$345 \times 10^5 = 223851 A_{st} [1 - 8.90 \times 10^{-6} A_{st})$$

$$345 \times 10^5 = 223851 A_{st} - 1.9924 A_{st}^2$$

$$A_{st} = 154.33 \sim 155 mm^2.$$

Assume dia of bar = 10mm

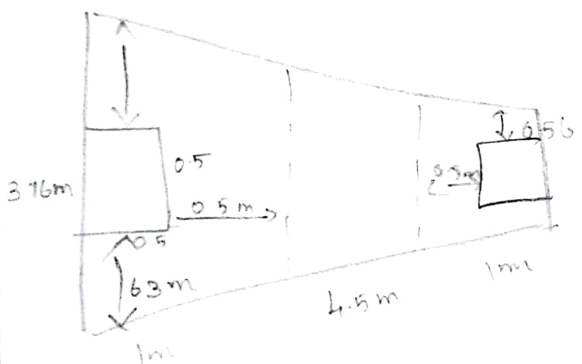
$$ast = \frac{\pi \times 10^2}{4} = 78.5\ mm^2$$

$$No.\ of\ bars = \frac{155}{78} = 2\ bars$$

$$Spacing = \frac{78}{155} \times 1000 = 500\ mm$$

Provide 10mm dia of 5 number at a spacing of 500mm centre to centre at bottom.

Transverse reinforcement:



$$\frac{5.76 - 0.5}{2} = 1.63m$$

$$\frac{1.62 - 0.5}{2} = 0.56m$$

$$Factor\ load = \frac{2000}{3.76} = 532\ kN/m$$

$$B.M\ at\ face\ of\ column = 532 \times \frac{1.63^2}{2} = 706\ kN.m$$

Reinforcement under column ①

$$1.5 \times 706 \times 10^6 = 0.87\ fy\ Ast\ d \left[1 - \frac{Ast * fy}{fck\ bd}\right]$$

$$Ast = 4948mm.$$

Assume dia of bar = 20mm

$$ast = \frac{\pi}{4} \times 20^2 = 314\ mm^2$$

$$No.\ of\ bars = \frac{4948}{314} = 16\ bars$$

$$\text{Spacing} = \frac{314}{4948} \times 1000 = 64\ \text{mm}$$

Provide 20mm dia bar of 16 number at a spacing of 64mm Centre to Centre under Column ①

Under column ②

$$\text{Factor load} = \frac{1500}{1.623} = 924\ \text{KN/m}$$

$$\text{B.M at face of column} = 924 \times \frac{0.56^2}{2} = 145\ \text{KN.m}$$

Reinforcement under column ②

$$1.5 \times 145 \times 10^6 = 0.87\, fy\, Ast\, d \left[1 - \frac{Ast \times fy}{fck\ bd}\right]$$

$$Ast = 982\ mm^2$$

Assume dia of bar = 12 mm

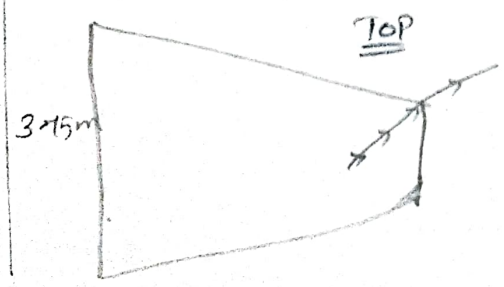
$$ast = \frac{\pi}{4} \times 12^2 = 113\ mm^2$$

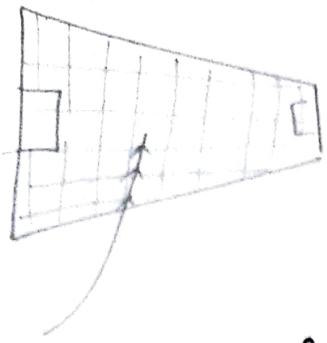
$$\text{No. of bars} = \frac{982}{130} = 8\ \text{bars.}$$

$$\text{Spacing} = \frac{113}{982} \times 1000 = 133\ \text{mm}$$

Provide 12mm dia bar of 8 numbers at a spacings of 133m Centre to centre under column ②.

15) Reinforcement details:





Mat or Raft foundation:

→ It consists of no. of columns with no of rows

→ The bearing capacity of the soil is less, this f. foundation is used.

→ columns are very close mat foundation is used.

→ By using this foundation differential settlement can be avoided.

Procedure:

→ Determination of total load of the column

→ Determination of total load with self wt. of footing

→ Determination of length & width of the footing.

→ Determination of actual pressure.

→ Draw pr distribution diagram.

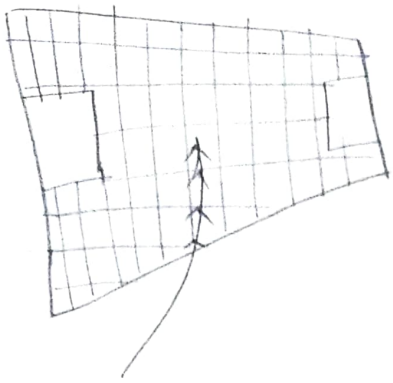
→ Find S.F & B.M & draw SFD & BMD

→ Determine Ast.

→ Assume dia of bar & find ast

→ Find No. of bars

→ Find Spacing of bars → Draw reinforcement details.



Mat or Raft foundation:

→ It consists of no. of columns with no. of rows 

→ The bearing capacity of the soil is less, this foundation is used.

→ Columns are very close mat foundation is used.

→ By using this foundation differential settlements can be avoided.

Procedure:

→ Determination of total load of the column.

→ Determination of total load with self wt. of footing

→ Determination of base area = $\frac{\text{Total load}}{\text{allowable soil pr}(q_a)}$

→ Determination of length & width of the footing.

→ Determination of actual pressure.

→ Draw pr. distribution diagram

→ Find S.F and B.M & draw SFD and BMD

→ Determine Ast.

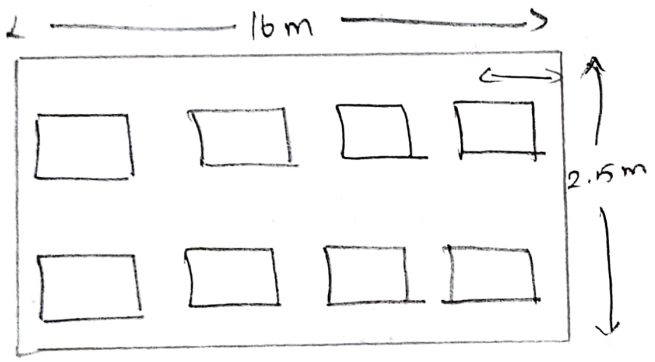
→ Assume dia of bar & Find ast.

→ Find No. of bars

→ Find spacing of bar.

→ Draw reinforcement details.

2. Design a Mat foundation for system of columns shown in fig. All columns are 500×500m. Allowable soil Pr. 120 KN/m².



Step 1: Determination of total load.

= (4×850) + (4×7000)

= 7400 KN.

2) Total load with self wt.

= 7400 + $\frac{10}{100}$ × 7400

= 8140 KN..

= $\frac{8140}{120}$ = 67.83m².

3) Base area f².

4) Length & width of the footing

L = 0.5 + 5 + 5 + 5 + 0.5 = 16m

B = $\frac{Af}{L}$ = $\frac{67.83}{16}$ = 4.23 ≃ 5m

5) Actual pressure = $\frac{Load}{Area}$ = $\frac{7400}{16 \times 5}$ = 92.5 KN/m²

9) Assume diameter of bar is 20mm,

$$ast = \frac{\pi}{4} \times 20^2 = \boxed{314 mm^2}$$

~~10) Assume diameter of bar~~

10) No. of bar $= \frac{13420}{314} = \boxed{43 \text{ number}}$

11) Spacing $= \frac{314}{13420} \times 1000 = \boxed{25 mm}$

Provide 20mm dia bars of 43 number at a spacing of 25mm centre to centre.

Longitudinal reinforcement in transverse direction:

B.M at face of column $= 231.25 \times \frac{2.5^2}{2} = 722.65$ KN.m.

$$Mu = 0.87 fy Ast d \left[1 - \frac{Ast \times fy}{fck\ bd} \right]$$

$$1.5 \times 722 \times 10^6 = 0.87 \times 415 \times Ast \times 540 \left[1 - \frac{Ast \times 415}{20 \times 5000 \times 540} \right]$$

$$1.5 \times 722 \times 10^6 = 205798.5\ Ast - 1.4983\ Ast^2$$

$$\boxed{Ast = 5742.5\ mm^2}$$

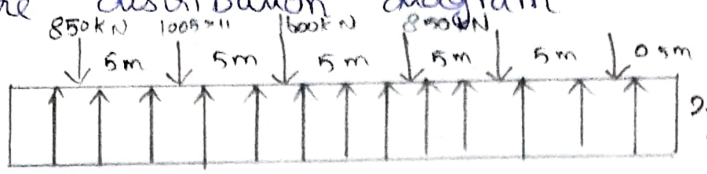
9) Assume diameter of bar 16mm ; $ast = \frac{\pi}{4} \times 16^2 = \boxed{201.6 mm^2}$

10) No. of bars $= \frac{5742.5}{201.6} = \boxed{29 \text{ bars}}$

11) Spacing $= \frac{201.6}{5742.5} \times 1000 = \boxed{35mm}$

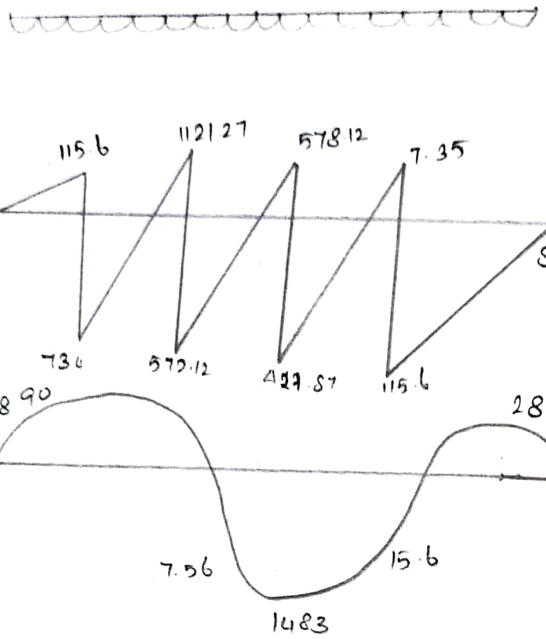
Provide, 16mm dia bars of 29 no. at Centre to Centre of 35mm spacing.

6) Pressure distribution diagram



92.5×2.5
$= 231.25$ kN/m

7) SF & BM and SFD & BMD



$231.25 \times 0.5 = 115.6$ kN

$850 - 115.62 = 735$ kN

$231.25 \times 5.5 - 850 = 421.87$ k

$1000 - 421.87 = 578.125$ kN

$B.M = \frac{wt^2}{2} = 231.25 \times \frac{0.5^2}{2}$

$= 28.9$

$= \frac{wt^2}{2} = 231.25 \times \frac{5^2}{2}$

Max. BM at 8m,

$231 \times \frac{8^2}{2} - 850 \times 4.5 - 1000 \times 2.5$

$= 1483$ KN-m.

8) Ast:

Longitudinal reinforcement at column ①

$M_u = 0.138 \ f_{ck} \ bd^2$

$1.5 \times 148.3 \times 10^6 = 0.138 \times 20 \times 2500 \times d^2$

$d = 567.7m \simeq \boxed{570mm}$ $B = 5m = 5000mm$

$M_u = 0.87 \ f_y \ A_{st} \ d \left[1 - \frac{A_{st} \ f_y}{f_{ck} \ bd}\right]$

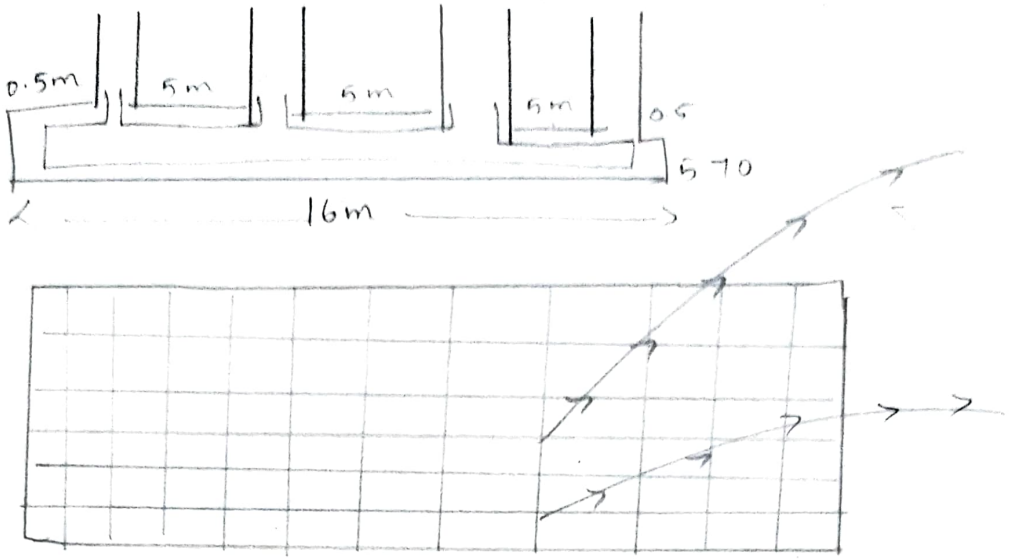
$1.5 \times 1483 \times 10^6 = 0.87 \times 415 \times A_{st} \times 570 \left[1 - \frac{A_{st} \times 415}{20 \times 5000 \times 570}\right]$

$22245 \times 10^6 = 442348.5 \ A_{st} \left[\right.$

$22245 \times 10^5 = 442348.5 \ A_{st} - 3.22 \ A_{st}^2$

$\boxed{A_{st} = 13420 \ mm^2}$

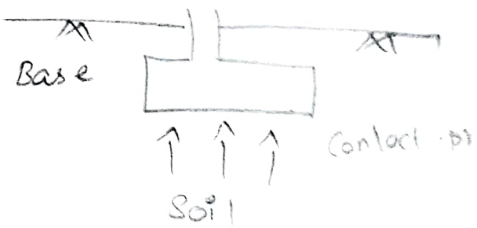
12) Reinforcement details:



Side of footing (or) foundation is called contact pressure.

=> It depends upon the elastic properties of foundation material (Rcc, steel, cement, fine, coarse, egg of soil and also of footing.

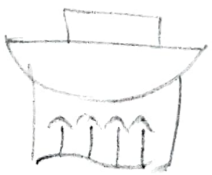
=> Contact Pressure on clay:



Flexible



Rigid

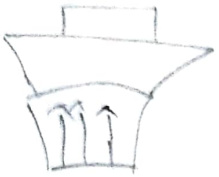


Contact pr. distribution under flexible & rigid footing resting of clay soil & subjected to udl. In flexible footing almost pr. distribution is uniform.

→ But rigid footing, the pr. distribution is min. at centre & max at edges.

→ In case of sandy soil:

Flexible



rigid



→ The contact pr. distribution for flexible is uniform for clay and sand.

→ The contact pr. distribution for rigid footing max at edges on the footing for clay but for sand min at edges.

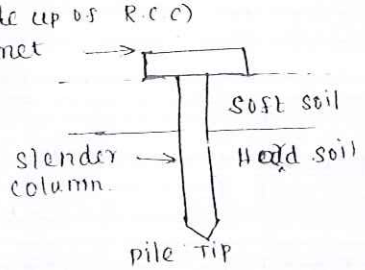
→ For convenience the contact pr is assumbled to be uniform & pr. distribution is uniform for all types of soil.

UNIT-4

PILE FOUNDATION

2m **Pile foundations** are employed when the soil strata immediately below the structure or not capable of supporting with allowable settlement & adequate safe bearing capacity against shear failure.

Piles are long slender columns driven into the hard strata through soft soil strata.



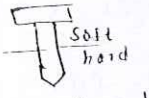
2m **Uses of the pile:**

1) To carry the vertical compressive loads.
2) It will resist the tensile forces
3) It resist horizonal and inclined loads.

16m **classification of piles:**

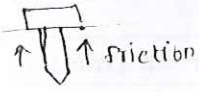
* classification based on action or friction
* classification based on material
* classification based on Installation

1. Based on action or Friction:

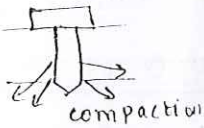


The piles are further classified into

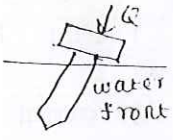
1. **End bearing pile:** These piles are used to transfer the load to the hard soil strata through soft soil strata or water.
2. **Friction pile:** It is used to transfer the load to the suitable strata by means of friction.



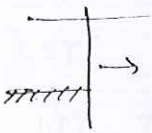
3. **compaction pile:** Used to compact the gravel or soil or sand to increase the bearing capacity of the soil is called as compaction pile.



4. **Anchor pile:** These piles are used in water front structures & is used to provide anchorised against horizontal pull from water.



5. **Fender pile:** It is used in water front structures against impact from floating agents.



6. **Sheet pile:** These piles are used to reduce the seepage in hydraulic structures.

7. Batter pile: It is used to resist horizontal & inclined forces in water front structures. It is in inclined position.

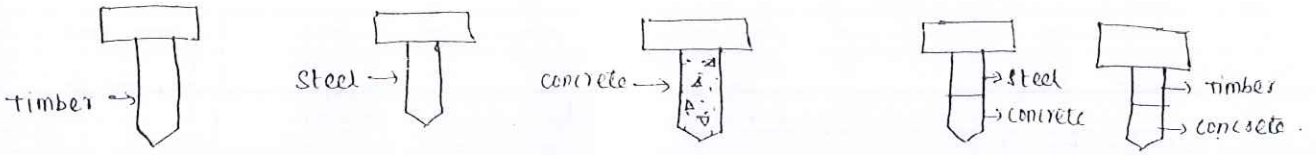


8. Laterally loaded piles: This piles used to support retaining walls, bridges and dams.

2) Based on Material:

The piles are classified into

1. Timber pile 2) Steel pile 3) Concrete pile 4) Composite pile



3) Based on Installation:

1. Driven piles: Timber, steel, concrete piles may be driven into the position either vertical or Inclined is called driven piles.
2. Cast in situ piles: Only concrete pile can be cast instu piles. In this pores are drilled & filled with concrete.
3. Driven & cast Insitu piles: It is the combination of both driven and cast insitu piles.

PILE DRIVING:

piles are commonly driven into the ground by means of a hammer supported by a grid or by a special device is known as pile driver.

Types of hammer:

There are three types of hammer for driving pile into the G.S

1. Drop hammer ⇒ Raising by hammer, falling by Gravity.
2. Single acting hammer → Raising by steam, falling by Gravity.
3. Double acting hammer ⇒ Both by Steam.

Drop hammer: Raised action by means of crane.

If the hammer is raised by vinch or crane & allowed to fall by means of gravity on the top of the pile is called drop hammer

2. Single acting hammer

or compressed air & allow to fall by means of gravity on the top of pile is called as single acting hammer.

3. Double acting hammer : It employs steam & air for raise and fall of the hammer on the top of the pile is called as double acting hammer.

Load carrying capacity of pile :

The load carrying capacity can be determined from

1) Dynamic formulae } Analytical approach.
2) Static formulae }
3) pile load test } practical approach.
4) SPT }

1. Dynamic formulae

There are number of fe dynamic formulas are used to determine load carrying capacity

1) Engineering news formulae
2) Hilley's formulae

27/8 Engineering news formulae

$$Q_a = \frac{WH}{f(s+c)}$$

Q_a - allowable load on the pile
W - wt of the hammer
H - Ht of wall
S - penetration per blow / final set
f - factor of safety = 6, c constant
c = 2.5 cm (drop), 0.25 cm (single, double

If we use drop hammer,

$$Q_a = \frac{WH}{6(S+2.5)}$$

For single acting hammer,

$$Q_a = \frac{WH}{6(S+0.25)}$$

For double acting hammer, $Q_a = \frac{(W+ap)H}{6(S+0.25)}$

a - area of piston
p - effective pressur

If a and p are not given, $(W+ap) = W$

1. A wooden pile is being driven with a drop hammer weighing 20kN & having a free fall of 1m. penetration in the last blow (i.e) final set is 5mm. Determine the load carrying capacity according to Engineering news formulae

Given: Drop hammer. $\Rightarrow f=6$, $c=2.5$cm.

$W = 20$kN, $H = 1m = 100$cm.

$S = 5mm = 0.5$cm

Solution:

$$Q_a = \frac{WH}{f(S+C)}$$

$$Q_a = \frac{20 \times 100}{6(0.5+2.5)} \Rightarrow \boxed{Q_a = 111 \text{ KN}}$$

---

2. Determine safe bearing capacity (allowable load) of pile driven by a double acting hammer weighing 30KN falling through 750mm. If the total penetration for the final set of blow is 400mm. Use Engineering news formula.

Given:

Double acting hammer $\Rightarrow f = 6$, $C = 0.25$ cm.

$W = (W + ap) = 30$ KN.

$H = 750$ mm $= 75$ cm.

$S = 400$ mm $= 40$ cm.

Solution:

$$Q_a = \frac{(W + ap)H}{f(S+C)} = \frac{30 \times 75}{6(40+0.25)}$$

$$\boxed{Q_a = 9.316 \text{ KN}}$$

Hilley's formulae:

$$Q_f = \frac{\eta_h H W \eta_b}{S + C/2}$$

$$Q_a = \frac{Q_f}{FS}$$

$$\eta_b = \frac{W + e^2 P}{W+P} \text{ if } W > ep$$

$$\eta_b = \frac{W + e^2 P}{W+P} - \frac{W - eP}{W+P} \text{ if } W < ep$$

Q_f - ultimate bearing capacity of pile

η_b - efficiency of the hammer

η_h = 100% for drop hammer.

= 65% for double acting hammer.

H - Ht. of fall, W - Wt. of hammer

S - final set, c - total elastic compression. $\Rightarrow \boxed{C = C_1 + C_2 + C_3 \cdots}$

η_b - efficiency of the hammer blow

P - wt. of pile including pile cap.

e - co.efficient of restitution.

---

1. Using Hilley's formula determine the dynamic pile load capacity if wt. of the hammer 30KN, ht. of fall 2m, wt. of the pile 20KN, final set 8mm, efficiency of hammer 75%, $C_1 = 6$mm, $C_2 = PL/A.E$, $E = 20 \times 10^6$ KN/mm^2, Length of the pile = 11m, Dia of the pile = 300mm & co.efficient of restitution 0.5, $C_3 = 2.5$mm.

Given

$W = 30\,kN$, $P = 20\,kN$ $d = 300\,mm$

$H = 2m = 200cm$ $S = 8\,mm = 0.8\,cm$.

$\eta_h = 75\%$ $C_1 = 6mm$ ~~$= 0.6cm$~~ , $C_2 = \frac{PL}{AE}$

$C_3 = 2.5mm$ ~~$= 0.25cm$~~ , $E = 20 \times 10^6\ kN/mm^2$

$L = 11\,m = 1100cm$.

Solution:

$$\eta_b = \frac{W + e^2P}{W+P}$$

W	eP
30	0.5×20
30	> 10

$$= \frac{30 + 0.5^2 \times 20}{30+20}$$

$$\boxed{\eta_b = 0.7}$$

$A = \frac{\pi}{4} d^2 = \frac{\pi}{4} \times 300^2$

$A = 70685.83\,mm^2$

$$C_2 = \frac{PL}{AE} = \frac{20 \times 11000}{70685.83 \times 20 \times 10^6}$$

$$\boxed{C_2 = 1.55 \times 10^{-7}\,mm}$$

$C = C_1 + C_2 + C_3 = 6 + 1.55 \times 10^{-7} + 2.5$

$$\boxed{C = 8.5\,mm = 0.85cm}$$

$$Q_s = \frac{\eta_h H W \eta_b}{S + C/2} = \frac{0.75 \times 200 \times 30 \times 0.7}{0.8 + 0.85/2}$$

$$\boxed{Q_s = 2571.42\,kN}$$

---

2. A concrete pile 40cm x 40cm & 20m long is driven by drop hammer having wt. 40kN Ht of fall 1m, penetration 6mm, co-eff of restitution 0.4, Elastic compression = 25mm, Determine i) ultimate bearing capacity of pile ii) Determine the penetration to which the pile should be driven under above condition. If $Q_s = 1600\,kN$, iii) find max. ultimate bearing capacity. Take unit wt. of concrete 24

Given

Drop hammer $\Rightarrow ff = 6$, $C = 0.25cm$, $\eta_h = 100\% = 1$.

concrete pile $\Rightarrow$ 40cm x 40cm

$L = 20\,m = 2000cm$. , $W = 40\,kN$.

$H = 1m = 100\,cm$, $S = 6mm = 0.6\,cm$

$e = 0.4$, $C = 25mm = 2.5\,cm$ ~~$= 0.25cm$~~ , $\gamma_{con} = 24\,kN/m^3$.

$Q_s = ?$, η_h S if $Q_s = 1600\,kN = ?$, $Q_{s\,max} = ?$

Wt. of pile = volume × γ_{con}

$= 0.4 \times 0.4 \times 20 \times 24$

$\boxed{P = 76.8 \text{ kN}}$

W	ep
40	0.4×76.8
40	> 30.72

$$\eta_b = \frac{W + e_p P}{W + P} = \frac{40 + (0.4^2 \times 76.8)}{40 + 76.8}$$

$\boxed{\eta_b = 0.44}$

i) $$Q_f = \frac{\eta_h H W \eta_b}{S + C/2} = \frac{1 (100) \times 40 \times 0.44}{0.6 + \phi 25/2}$$

$Q_f = 951.35 \text{ kN}$

ii) $$1600 = \frac{1 (100) 40 (0.44)}{0.6 S + 2.5/2}$$

$$1600 = \frac{1760}{S + 1.25} \Rightarrow S + 1.25 = 1.1$$

$\boxed{S = -0.15 \text{ cm}}$

iii) $Q_{f\max} \Rightarrow \boxed{S = 0.}$

$$Q_{f\max} = \frac{1 \times 100 \times 40 \times 0.44}{0 + 2.5/2} \Rightarrow \boxed{Q_f = 1408 \text{ cm}}$$

Determination of ultimate Bearing Capacity of pile by static formulae.

In General, $Q_a = \frac{1}{F} (Q_{pu} + Q_s)$

A_p - area of pile

Related with Bearing resistance $\Rightarrow Q_{pu} = q_{pu} \cdot A_p$

q_{pu} - bearing resistance

" frictional resis $\Rightarrow Q_f = f_s A_s$

Q_f - ultimate point resistance

A_s - Surface area of the pile, f_s - unit skin friction resistance.

$$\boxed{Q_a = \frac{1}{F} [q_{pu} \cdot A_p + f_s A_s]}$$

For sandy soil,

$$Q_a = \frac{1}{F} (q_{pu} A_p + f_s A_s)$$

γ - unit wt of soil

$$q_{pu} = \gamma D N_q$$

N_q - B.C factor

D - Depth or length of the pile

$$f_s = k \tan \delta \sigma_{av}$$

k, δ - constants depends upon the material of the pile

Material	δ	k	
		loose sand	dense sand
steel	20°	0.5	1
concrete	0.75 ϕ	1	2

ϕ - angle of internal friction.

$\sigma_{av} = \frac{\gamma D}{2}$, $D = \pi d$

$$Q_F = [\gamma D N_q A_p + k \tan\delta \sigma_{av} A_s] \rightarrow (1)$$

$$Q_a = \frac{Q_F}{F}$$

For clay soil:

$$Q_a = \frac{1}{F} (q_{pu} A_p + f_s A_s)$$

c - cohesion , N_c - B.C factor.

$q_{pu} = C N_c$

α - mobilisation factor.

$f_s = \alpha C$

$\alpha = 0.3$ to 1 assume

$$Q_F = \frac{1}{F} [C N_c A_p + \alpha C A_s] \rightarrow (2)$$ $Q_a = \frac{Q_F}{F}$

Eq (1) & (2) can be used to determine the load carrying capacity of the single pile.

---

1. A concrete pile is used as a foundation for a column in a sandy soils. Angle of internal friction 32° with 300m dia & 10m in length. Calculate the load carrying capaci of the pile. If $\gamma = 18\ kN/m^3$ & $N_q = 27$

Given: concrete pile , sandy soil

$\Phi = 32°$, $d = 300mm = 0.3m$, $L = 10m$.

$\gamma = 18\ kN/m^3$, $N_q = 27$, $Q_F = ?$

$A_s = \pi d l = \pi \times 0.3 \times 10 = 9.42$

$A_p = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 0.3^2 = 0.0706$

Assume loose sand concrete, $K = 0.75$ $\delta = \Phi$

$\delta = 0.75\Phi = 0.75(32) = 24°$.

$\sigma_{av} = \frac{\gamma D}{2} = \frac{18 \times 10}{2} =$

Solution:

$Q_F = q_{pu} A_p + f_s A_s$

(i.e) $Q_F = \gamma D N_q A_p + k \tan\delta \sigma_{av} A_s$

$= (18 \times 10 \times 27 \times 0.0706) + (1 \times \tan 24° \times 90 \times 9.42)$

$Q_F = 720\ kN$

---

2. A 450mm wide square concrete pile 15m length is driven in deep deposit of uniform clay, cohesion - $37.5\ kN/m^2$, $N_c = 9$, $\alpha = 0.7$. Find ultimate load capacity of the pile.

Given: concrete pile , clay soil , square pile

$l = 15m$, Size of pile = 450 mm × 450 mm = 0.45 × 0.45m.

$C = 37.5\ kN/m^2$, $N_c = 9$, $\alpha = 0.7$, $Q_F = ?$

Solution.

$$Q_F = CN_c A_p + \alpha C A_s$$

$$= (37.5 \times 9 \times 0.45 \times 0.45) + (0.7 \times 37.5 \times \pi \times 0.45 \times 15)$$

$$\boxed{Q_F = 777.09 \text{ kN}}$$

---

3. A concrete pile $30cm^2$ is driven into clay & is to carry a load of 300 kN, $C = 0.15 \text{ N/mm}^2$. Determine the required length of the pile to carry the load. FOS = 2.5, mobilisation factor = 0.7.

Given:

concrete pile, size of pile = $30cm^2$ (Area) $30cm \times 30cm = 0.3 \times 0.3 m$

$P = 300$ kN, $C = 0.15 \text{ N/mm}^2$, For clay soil $N_c = 9$,

$C = 0.15 \times 10^6 \times 10^{-3} \text{ kN/m}^2$ FOS = 2.5

$= 150 \text{ kN/m}^2$ $\alpha = 0.7$, $l = ?$

Solution:

$$Q_a = \frac{CN_c A_p + \alpha C A_s}{F}$$

$$300 = \frac{(150 \times 9 \times 0.3 \times 0.3) + (0.7 \times 150 \times \pi \times 0.3 \times l)}{2.5}$$

$$300 = \frac{121.5 + 126l}{2.5}$$

$$300 = 48.6 + 50.5l$$

$$251.4 = 50.5l$$

$$\boxed{l = 4.98 \text{ m}}$$

4. A pile 1m dia is installed at a depth of 15m in a deep deposit of clay ucc strength 120 kN/m², Estimate allowable load on pile, FOS = 3, $\alpha = 0.6$.

Given:

$D = 15m$ $C = \frac{UCC}{2} = \frac{120}{2} = 60 \text{ kN/m}^2$, $d = 1m$.

FOS = 3 $\alpha = 0.6$, For clay, $N_c = 9$

$$A_p = \frac{\pi}{4} \times d^2 = \frac{\pi}{4} \times 1 = 0.785 \text{ m}^2$$

$$A_s = \pi d l = \pi \times 15 \times 1 = 47.12 \text{ m}^2$$

Solution:

$$Q_a = \frac{CN_c A_p + \alpha C A_s}{F}$$

$$Q_a = \frac{60 \times 9 \times 0.785 + 0.6 \times 60 \times 47.12}{3}$$

$$\boxed{Q_a = 707.7 \text{ kN}}$$

Test Questions

1. Under what circumstances the different types of footing is provide
2. Factors considered for selection of foundation.
3. Types of settlement.
4. What is mat foundation
5. Define floating "
6. Define various types of piles
7. Methods to determine load carrying cap. of pile.
8. Expansive soil - Define
9. Remedial measures to avoid settle
10. classification of pile.
11. Uses of piles
12. What is pile
13. What is footing

Part-B:

1. combined footing design (or) settlement problem.
2. Hilley's formula (or) Engineering use formula

10/9

Group action of piles:

A single pile is not used under the column to support the structure.

A single pile is not used under the column because it is extremely difficult to ~~train~~ drive the pile absolutely vertical & place the column exactly over a centre line, the pile is not in a vertical position there is some B.M & eccentricity transmitted to the pile cap by the colum The eccentric loading results the connection blw the pil & column in actual practise the structur loads are supported by number of piles acting as a group. The loads are usually transfered to the pil group through a Rcc slab structurally filled on the top of the pile. Such that piles acts as a unit. This slab is known as pile cap. The load from the structu transferred to the pile cap which distributes the loads to the piles. For the safe design of pile foundation, we must calculate efficiency of pile groups and capacity of pile group.

Efficiency of pile Group:

Efficiency of pile can be calculated by the formulae,

1. $\eta = \frac{Q_{ug}}{n \cdot Q_u}$

Q_{ug} - capacity of group,
n - no. of piles in group
Q_u - single pile capacity.

2. Converse Labbree's formula:

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

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

d - dia of pile
s - centre to centre dist b/w the piles.
m - no. of rows
n - no. of piles in a row

3 Field Rule:

---

1. A group of 9 piles 300mm dia arranged in three rows are spaced at 1m centre to centre. Determine the capacity of the pile group and efficiency of the pile group. If the single pile capacity is 300 kN.

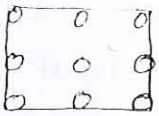
Given:

no. of piles = 9
dia = 300mm , 3 rows.
s = 1m c/c.
Q_u = 300 kN. $Q_{ug}, \eta = ?$

Solution:

$Q_{ug} = \eta \cdot n Q_u$

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



$\theta = \tan^{-1} d/s = \tan^{-1} 0.3/1 \Rightarrow \boxed{\theta = 16°41'}$

$m = 3, n = 3.$

$$\eta = 1 - \frac{16°41'}{90}\left[\frac{(3-1)3 + (3-1)3}{3 \times 3}\right]$$

$\boxed{\eta = 0.75 = 75\%}$

$Q_{ug} = 0.75 \times 9 \times 300$

$\boxed{Q_{ug} = 2025 \text{ kN}}$

Determination of efficiency by Feldo rule

$$\eta = \frac{\Sigma Q_R}{n \cdot \Sigma Q_P}$$

$\Sigma Q_P = 16$.

Q_R - reduced capacity.

$Q_R = 16$ - pile influences the ~~next~~ neighbouring pile.

---

1. A pile group has 9 piles. Estimate efficiency of pile gr

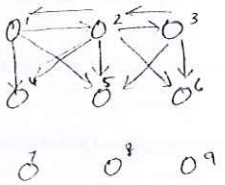
Given:

$n = 9$

$\eta = ?$

Solution:

$$\eta = \frac{\Sigma Q_R}{n \cdot \Sigma Q_P}$$



$Q_R = 16$ - pile influences neighbouring pile

$Q_{R_1} = 16 - 3 = 13$

$Q_{R_2} = 16 - 5 = 11$

$Q_{R_3} = 16 - 3 = 13$

$Q_{R_4} = 16 - 5 = 11$.

$Q_{R_5} = 16 - 8 = 8$

$Q_{R_6} = 16 - 5 = 11$.

$Q_{R_7} = 16 - 3 = 13$.

$Q_{R_8} = 16 - 5 = 11$.

$Q_{R_9} = 16 - 3 = 13$.

$\Sigma Q_R = 104$.

$$\eta = \frac{104}{9 \times 16} \Rightarrow \boxed{\eta = 72\%}$$

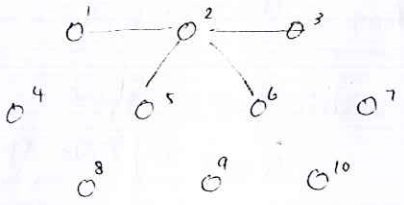
---

2. A pile group has 10 piles are arranged in 3+4+3 pattern. Find efficiency using field rule.

Given: $n = 10$.

pattern : 3+4+3

$\eta = ?$



Solution:

$$\eta = \frac{\Sigma Q_R}{n \Sigma Q_P}$$

$Q_R = 16$ - pile influences the neighbouring pile.

$Q_{R_1} = 16 - 3 = 13$

$Q_{R_2} = 16 - 4 = 10$.

$Q_{R_3} = 16 - 3 = 13$.

$Q_{R_4} = 16 - 3 = 13$.

$Q_{R_5} = 16 - 6 = 10$.

$Q_{R_6} = 16 - 6 = 10$.

$Q_{R_7} = 16 - 3 = 13$.

$Q_{R_8} = 16 - 3 = 13$

$Q_{R_9} = 16 - 4 = 12$.

$Q_{R_{10}} = 16 - 3 = 13$.

$\Sigma Q_R = 121$.

$$\eta = \frac{121}{10 \times 16} \Rightarrow \boxed{\eta = 75\%}$$

Capacity of piles:

Group of piles may ~~fite~~ in one of the following,

1) Block failure.

2) pile individual failure.

By block failure, $Q_{ug} = CN_c A_b + P_b L C$

By individual failure, $Q_{ug} = n \left[CN_c A_p + \alpha C A_s \right]$

A_b - Area of block C - cohesion

P_b - perimeter of block N_c - B.C factor.

α - mobilisation factor = 0.5 to 1.

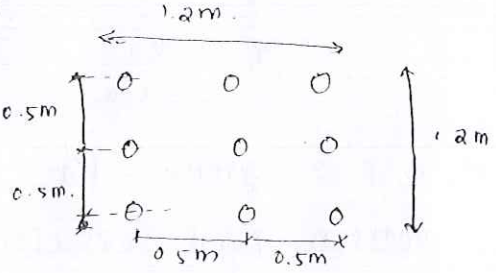
A_p - area of the single pile, A_s = surface area of the pile.

9 A 200mm dia & 8m long used as a foundation for a column in a uniform deposit of clay spacing b/w the piles is 500mm. There are 9 piles in a group arranged in a square pattern. Calculate the ultimate load capacity of the group. Take $\alpha = 0.9$, Ucc strength = 100 kN/m², $N_c = 9$.

Given:

dia = 200mm = 0.2m , L = 8m , S = 500mm = 0.5m , n = 9 , square pattern,

Ucc = 100 kN/m² , $N_c = 9$, $\alpha = 0.9$, $Q_{ug} = ?$



Solution:

(i) Block failure:

$Q_{ug} = CN_c A_b + P_b L C$

$= 50 \times 9 \times (1.2 \times 1.2) + (4 \times 1.2) \times 8 \times 50$

$\boxed{Q_{ug} = 2568 \text{ kN}}$

$A_b = L^2$

$P_b = 4L$

(ii) Individual failure:

$Q_{ug} = n \left[CN_c A_p + \alpha C A_s \right]$

$= 9 \left[50 \times 9 \times \pi/4 \times 0.2^2 + 0.9 \times 50 \times \pi \times 0.2 \times 8 \right]$

$\boxed{Q_{ug} = 2163 \text{ kN}}$

Least of these, $\boxed{Q_{ug} = 2163 \text{ kN}}$

2. An n pile has to be proposed in a uniform pattern of soft clay with equal spacing in all directions. Assuming the value of c. Determine the optimum value of spacing in group. Take n = 25, $\alpha = 0.7$, Assume pile is circular. Neglect bearing resistance

Given:

n = 25 , $\alpha = 0.7$, C = C , circular , dia = d , S = ?

... bearing resistance

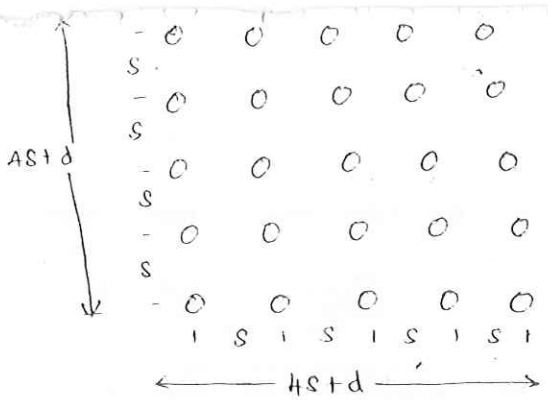
i) Block failure:

$Q_{ug} = CN_c A_b + P_b L C$

Neglect bearing resistance

$Q_{ug} = P_b L C$

$= 4(4S+d) \times L \times C \longrightarrow (1)$



ii) Individual failure:

$Q_{ug} = n\left[CN_c A_p + \alpha C A_s\right]$

$= n \alpha C A_s$

$Q_{ug} = 25 \times 0.7 \times C \times \pi \times d \times L \longrightarrow (2)$

(1) = (2)

$(16S + 4d)LC = 54.97\,\pi\, dL$

$16SLC + 4dLC = 54.97\,dLC$

$16SLC = 50.97\,dLC$

$SLC = 3.186\,dLC$

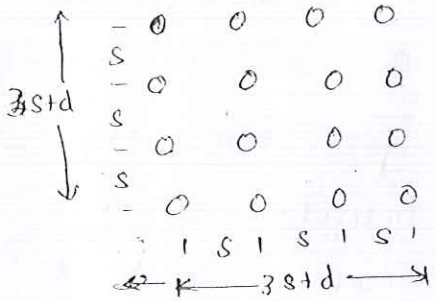
$\boxed{S = 3.186\,d}$

3. A 16 pile group has to be arranged in a form of square pattern with soft clay in uniform spacing. Determine optimum value of spacing in terms of d. $\alpha = 0.6$, neglect bearing resistance.

i) Block failure:

$Q_{ug} = CN_c A_b + P_b L C$

$Q_{ug} = 4(3S+d) \times L \times C \longrightarrow (1)$



ii) Individual failure:

$Q_{ug} = n\left[CN_c A_p + \alpha C A_s\right]$

$Q_{ug} = 16 \times 0.6 \times C \times \pi \times d \times L \longrightarrow (2)$

(1) = (2) $\Rightarrow$ $12SLC + 4dLC = 30.159\,dLC$

$12SLC = 26.159\,dLC \Rightarrow \boxed{S = 2.18\,d}$

4. Design a pile group to carry 400kN in clay UCC strength 60 kN/m², $\alpha = 0.6$ FOS = 3, dia of pile = 0.3m, length of pile = 6m.

Given:

Total load = 400kN, UCC = 60kN/m² $\Rightarrow C = 30$ kN/m², $\alpha = 0.6$, FOS = 3, $d = 0.3$m, $L = 6$m. Design the pile group.

Solution:

Design of pile group includes the determination of no. of piles in a group, efficiency of pile group & capacity of grop.

capacity of pile > Total load group.

(i.e) Q_{ug}

i) no. of piles:

$n = \dfrac{\text{Total load}}{\text{Individual pile load}}$, $Q_a = \dfrac{Q_u}{FOS}$

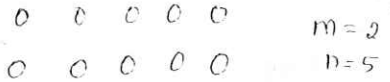
$Q_u = CN_c A_p + \alpha C A_s$

$= 30 \times 9 \times \pi/4 \times 0.3^2 + 0.6 \times 30 \times \pi \times 0.3 \times 6 = 120.81$ kN

$Q_a = \dfrac{120.87}{3} = 40.29$ kN $\qquad n = \dfrac{400}{40} = 10$

ii) Efficiency of pile:

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



Assume, $S = 3d$

$= 3 \times 0.3$

$= 0.9\,m$

$$\theta = \tan^{-1} d/s = \tan^{-1}\left(\frac{0.3}{0.9}\right) \Rightarrow \boxed{\theta = 18^\circ 26'}$$

$$\eta = 1 - \frac{18^\circ 26'}{90}\left[\frac{(2-1)5 + (5-1)2}{2 \times 5}\right]$$

$$\eta = 0.733 = \boxed{73.3\%}$$

(iii) capacity of pile:

$$Q_{ug} = \eta . n\, Q_u = 0.733 \times 10 \times 120.87$$

$$\boxed{Q_{ug} = 886\ kN} \Rightarrow 400\ kN$$

5. Determine the capacity of the group consisting of 15 piles with 300mm dia arranged in three rows. Take $L = 8m$, $c = 25\ kN/m^2$, $S = 0.8\,m$, $N_c = 9$, $\alpha = 0.9$

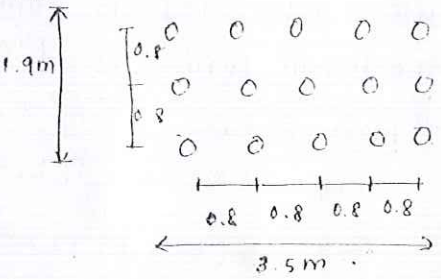
Solution

(i) Block failure:

$$P_b = 2(a+b), \quad A_b = a \times b$$

$$Q_{ug} = CN_c A_b + P_b L c$$

$$Q_{ug} = 25 \times 9 \times 1.9 \times 3.5 + (2(1.9+3.5)) \times 8 \times 25$$



$$\boxed{Q_{ug} = 3656.25\ kN}$$

(ii) Individual failure:

$$Q_{ug} = n\left[CN_c A_p + \alpha C A_s\right]$$

$$= 15\left[25 \times 9 \times \pi/4\,(0.3)^2 + 0.9 \times 25 \times \pi \times 0.3 \times 8\right]$$

$$\boxed{Q_{ug} = 2783.25\ kN}$$

least value.

$\therefore Q_{ug} = 2783.25\ kN$

2019

Settlement of pile Groups in clay soil:

Assumptions:

1) clay contain b/w top of the pile & upto 2/3rd of L is incompressible.

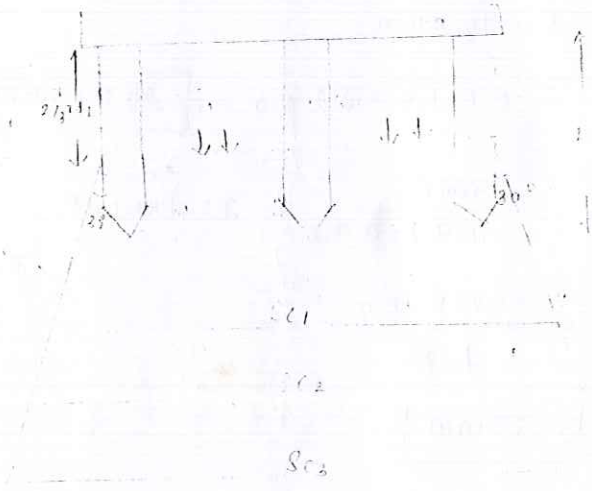
2) Load is applied to the soil at a depth of 2/3rd of L from G.L

3) Load is assumed spread 30° with vertical.

4) Below 2/3rd of L, the soil deposit divided into no. of layer and initial pressure, incremental pressure are calculated at the middle of the each layer. Then the settlement of each layer can be calculated by using the expression,

$$S_c = \frac{C_c H_0}{1+e_0} \log_{10} \frac{\bar{\sigma}_0 + \Delta\bar{\sigma}}{\bar{\sigma}_0}$$

C_c - compression index
H_0 - tk. of clay layer
e_0 - void ratio
$\bar{\sigma}_0$ - Initial pr. } effective pr.
$\Delta\bar{\sigma}$ - Incremental pr. }



1.) calculate the settlement of the pile group to carry a load of 3000 kN at total depth of 20m. Length of pile 10.5m, unit wt. of pile 16 kN/m³, ϕm of pile 0.5m, no. of pile 16, liquid limit = 60%, spacing 3 times the dia of pile

Gn: total load = 3000 kN, L = 10.5m

total depth = 20m, γ = 16 kN/m³

d = 0.5m, n = 16, LL = 60%

Spacing = 3 × d = 3 × 0.5 = 1.5m

Solution:

$$S_c = \frac{C_c H_0}{1+e_0} \log_{10} \frac{\bar{\sigma_0} + \Delta\bar{\sigma}}{\bar{\sigma_0}}$$

$C_c = 0.009 (60 - 10)$

$= 0.45$

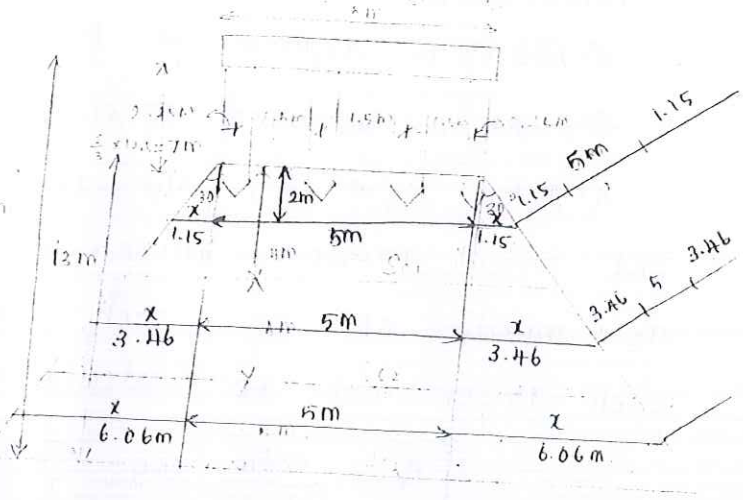
$e_0 = 1, \quad H_0 = 4m$

$\bar{\sigma_0} = \sigma - u$

$\bar{\sigma_0} = \sigma$

$= (7+2) \times 16$

$= 144 \ kN/m^2.$



$\tan 30° = x/2$ $\quad \boxed{x = 1.15m}$

$5 + 1.15 + 1.15 = 7.3 \ m$

$\Delta\bar{\sigma} = \frac{Load}{area} = \frac{3000}{7.3 \times 7.3} = 56.8 \ kN/m^2.$

$S_c = \frac{0.45 \times 4}{1+1} \log_{10} \frac{144 + 56.8}{144} \Rightarrow S_{c_1} = 0.128 \ m \Rightarrow \boxed{S_{c_1} = 128 mm}$

S_{c_2}: $C_c = 0.45, \ e_0 = 1, \ H_0 = 4m.$

$\bar{\sigma_0} = \sigma - u = (7 + 4 + 2) \times 16 = \boxed{208 \ kN/m^2}$

$\Delta\bar{\sigma} = \frac{load}{area} = \frac{3000}{11.92 \times 11.92} = 21.11 \ kN/m^2.$

$\tan 30° = x/6$ $\quad \boxed{x = 3.46m}$

$3.46 + 3.46 + 5 = 11.92$

$S_{c_2} = \frac{0.45 \times 4}{1+1} \log_{10} \frac{208 + 21.11}{208}$

$S_{c_2} = 0.038 \ m \ \boxed{= 38 mm}$

S_{c_3}: $C_c = 0.45, \ e_0 = 1, \ H_0 = 4m$

$\tan 30° = \frac{x}{10.5}$ $\quad \boxed{x = 6.06m}$

$6.06 + 6.06 + 5 = 17.12$

$\bar{\sigma_0} = \sigma - u = (7 + 4 + 4 + 2.5) \times 16 = 280 \ kN/m^2$

$\Delta\bar{\sigma} = \frac{load}{area} = \frac{3000}{17.12 \times 17.12} = 10.24 \ kN/m^2.$

$S_{c_3} = \frac{0.45 \times 4}{1+1} \log_{10} \frac{280 + 10.24}{280}$

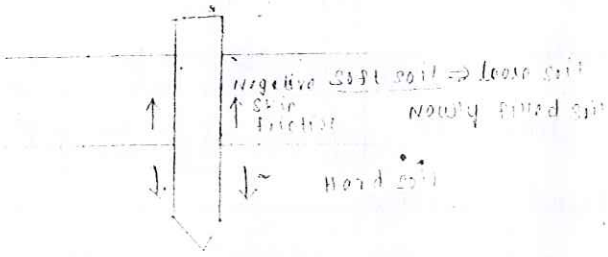
$S_{c_3} = 0.014 \ \boxed{= 14 mm}$

$S_c = S_{c_1} + S_{c_2} + S_{c_3}$

$= 128 + 38 + 14$

$\boxed{S_c = 185 mm}$

Negative Skin Friction:



If the piles are installed in a compressible or newly placed soil the consolidation of soil develops drag on the pile due to relative movement of soil with respect to piles resulting in reduction of pile loads. this reduction is termed as negative skin reduction & the friction is termed as negative skin friction.

It develops by 1) Lowering Ground table
2) Increase in effective stress.

Special types of piles:

1) Under reamed pile 2) Bored compaction pile.

Under reamed pile: (or) single under reaming pile ⇒ purpose: Incr the capacity of pil



1) Boring 2) Under reaming 3) Reinforcing 4) Concreting

If the pile having more than one or two bulk is known as multi under reaming pile.

If pile having two bulk is known as single under reaming pile.

Bored compaction pile:

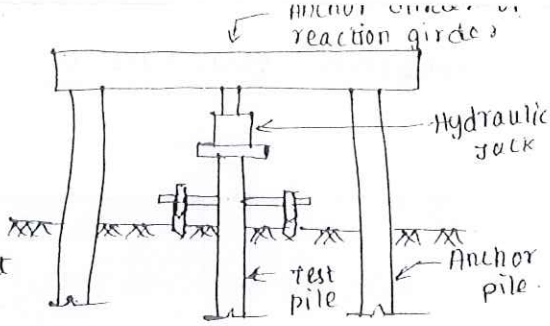


1) Boring 2) Under reaming 3) concreting 4) Reinforcing.

Pile Load Test:

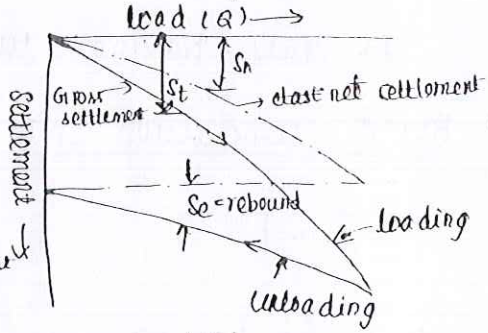
The most reliable method for determining the load carrying capacity of a pile is the pile load test. the set-up generally consists of two anchor piles provided with an anchor girder or a reaction girder at their top. The test pile is installe b/w the anchor piles in the manner in which the foundation piles are to be installed. The test pile should be at least 3B or

2.5m clear from the anchor pile



The load is applied through a hydraulic Jack resting on the reaction girder. The measurements of pile movement are taken with respect to a fixed reference mark. The test is conducted after a rest period of 3 days after installation in sandy soils & a period of one month in silts and soft clays. The load is applied equal increment of about 20% of allowable load. Settlements should be recorded with three dial gauges. Each stage of the loading is maintained till the rate of movement of the pile top is not more than 0.1 mm/hr in sandy soils & 0.02mm/hr in clayey soils or max. of 2 hrs (IS: 2911-1974). Under each load increment, settlements are ~~obtained~~ observed at 0.5, 1, 2, 4, 8, 12, 16, 20, 60 min. The loading should be continued upto twice the safe load or the load at which the total settlement reaches a specified value. The load is removed in the same decrements at 1hr interval and the final rebound is recorded 24 hrs after the entire load has been removed.

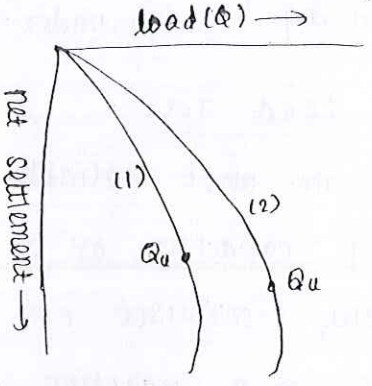
Draw a typical load-settlement curve for loading as well as unloading obtained from a pile load test.



The net pile settlement, $S_n = S_t - S_e$

S_t - total settlement, S_e - elastic settlement

The following curve shows two load net-settlement curves obtained from a pile load test on two different soils. At the ultimate load (Q_u), the load-net settlement curve becomes either linear as curve or there is a sharp break as usually taken as one-half of the ultimate load.



This test is also known as slow maintained test.

2119 **Determination of pile load capacity by penetration test:**

Use of N value, N - Std. penetration number.

$$Q_u = q_p A_p + f_s A_s$$

$$q_p = 40 N \left(\frac{D}{B}\right) < 400 N \quad , \quad f_s = 2N$$

1. A square concrete pile 30cm wide, 10m long is driven into the soil deposit. Determine allowable load on the pile. N = 20, FOS = 3.

Given:

square, 30cm x 30cm.

$L = 10m$, $B = 0.3m$, $N = 20$, $FOS = 3$, $Q_a = ?$

Solution:

$$Q_a = \frac{Q_u}{FOS}$$

$$Q_u = q_p A_p + f_s A_s$$

$$q_p = 40 N \left(\frac{D}{B}\right) < 400 \frac{N}{\wedge} \Rightarrow 40 \times 20 \left(\frac{10}{0.3}\right) < 400 \times 20$$

$$= 26666.66 > 8000$$

$$\therefore q_p = 8000 \text{ kN}.$$

$$f_s = 2N = 2 \times 20 = 40.$$

$$A_s = 4aL = 4 \times 0.3 \times 10$$

$$Q_u = 8000 \times (0.3 \times 0.3) + (40 \times 4 \times 0.3 \times 10)$$

$$Q_u = \cancel{7680}^{1200} \text{ kN}.$$

$$Q_a = \frac{\cancel{7680}^{1200}}{3} = \cancel{2560}^{400} \text{ kN}.$$

2. A concrete pile of square 35 x 35 cm is driven into layered Sandy strata upto a depth of 13m. The top layer has a depth of 9m with avg. N value 12. Bottom layer is 4m depth with avg. N value 40. Determine the allowable load on the pile. FOS = 3.

Solution:

$$Q_a = \frac{Q_u}{FOS}$$

$$Q_u = Q_{u1} + Q_{u2}$$

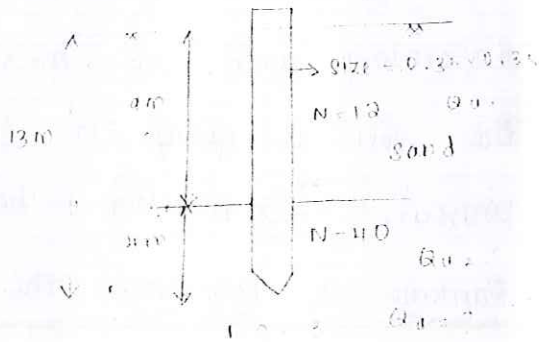
$$Q_{u1} = q_p A_p + f_s A_s$$

$$q_p = 40 N \left(\frac{D}{B}\right) < 400 N$$

$$= 40 \times 12 \left(\frac{9}{0.35}\right) < 400 \times 12$$

$$12\,342.85 > 4800$$

$$\therefore q_p = 4800 \text{ kN}$$



$$Q_{u1} = 4800 \times (0.35 \times 0.35) + (\text{illegible})$$

$$\boxed{Q_{u1} = 890.4 \text{ kN}.}$$

$Qu_2 = q_p A_p + f_s D_s$

$q_p = 40N \left(\frac{D}{B}\right) \leq 400N$

$= 40 \times 40 \left(\frac{H}{0.35}\right) \leq 400 \times 40$

$18285.71 \leq 16000 \quad \therefore q_p = 140 kN.$

$Qu_2 = (16000 \times 0.35 \times 0.35) + (2 \times 40 \times 4 \times 0.35 \times 4)$

$\boxed{Qu_2 = 2408.45 kN}$

$Qu = Qu_1 + Qu_2 = 3298.4 / 1355.55 kN$

$Qa = \frac{3298.4}{3} = \boxed{1099.46 kN}$

22/9

UNIT - V

RETAINING WALLS

Retaining structure supports the body of earth soil mass is called as RETAINING WALLS

The soil retained by retaining structure is called BACKFILL.

Lateral Earthquake pressure:

When a large soil mass supported against the vertical wall there is no soil on other side, soil mass exerts horizontal or lateral pressure on the wall if the section & wt. of retaining wall is not sufficient to compo counteract the moments caused by lateral pressure the wall should turn and collapse. Inorder to design the safe dimension of retaining wall we must evaluate the lateral pressure which the soil is going to exert on the structure this lateral pressure distribution behind the retaining wall was evaluated by Rankine & Coloumb. this theories are called as Rankine's earth pr. theory & Coloumb's earth pr. theory.

Two classical Earth pr. Theory:

The lateral earth pr. can be grouped into 3 categories.

1. Earth pressure at rest.
2. Active Earth pressure.
3. passive " ".

These catogries depends upon the movement of the wall.

1. Earth pr. at rest:

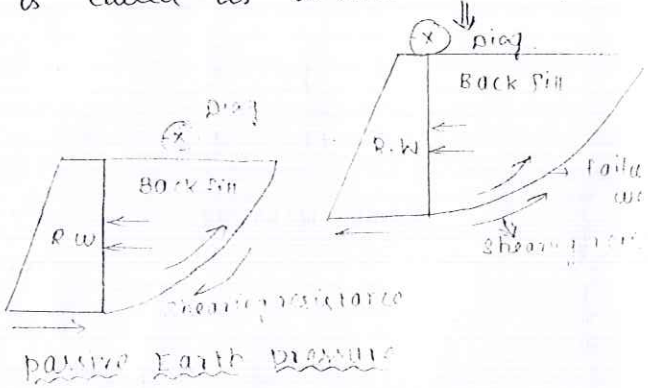
Lateral earth pr. is said to be at rest when retaining wall is not subjected to any movement.

2. Active Earth pressure:

If the retaining wall moves away from the backfill due to lateral earth pr. this pr. is called as active earth pr.

3. passive Earth pressure:

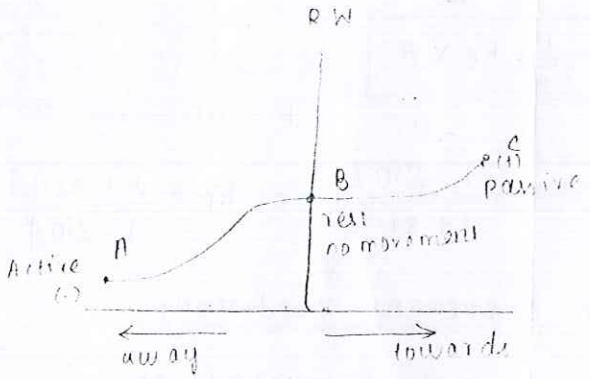
In this case, retaining wall moves towards the backfill due to the lateral earth pr. this pr. is called as passive earth pressure.



passive Earth pressure

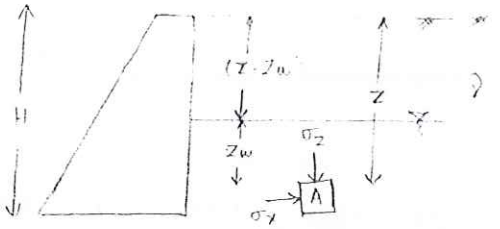
Note: Wedge shaped portion of the backfill tending to mov with the wall is called as failure wedge. (Active Earth pr)

Variation of Earth pr. on wall movement:



* point B represents the case wh there is no movement of wall. I indicates pr. at rest.
* point A indicates active earth pr Due to lateral earth pr. wall moves away from the back fill. Backfill moves downwards & shearing resistance develops upwar this shows decrease in lateral stress.
* point C indicates passive earth pr. Due to lateral earth pr. wo moves towards backfill, backfill moves upwards & shearing resistance develops downwards. this shows increase in lateral stress.

Earth pressure at rest condition :



vertical stress at A at a depth of z $\Big\} = \gamma z$

total stress, $\sigma = \gamma z$

porewater pressure, $u = \gamma_w \cdot z_w$

Effective stress = $\sigma - u$

$$\boxed{\sigma_z = \gamma z - \gamma_w \cdot z_w}$$

k_o - co-efficient of earth pressure at rest.

$$k_o = \frac{\sigma_x}{\sigma_z} \Rightarrow \sigma_x = k_o \sigma_z$$

$$\sigma_x = P_o \qquad \boxed{P_o = k_o \sigma_z}$$

$$P_o = k_o \sigma_z$$

$$\boxed{P_o = k_o (\gamma z - \gamma_w z_w)}$$

total pressure, $\sigma = P_o + u$

$$= k_o (\gamma z - \gamma_w \cdot z_w) + \gamma_w z_w$$

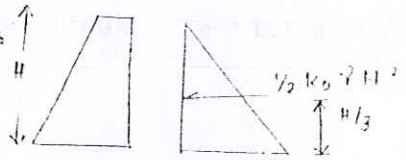
$$= k_o \gamma z - k_o \gamma_w \cdot z_w + \gamma_w z_w$$

pressure at A, $\boxed{\sigma = P = k_o \gamma z}$

∴ pressure at whole element, $P_o = \int_0^H k_o \gamma z$

$$P_o = k_o \gamma \frac{H^2}{2}$$

$$\boxed{P_o = \frac{1}{2} k_o \gamma H^2}$$



p_r distribution diagram

Note:

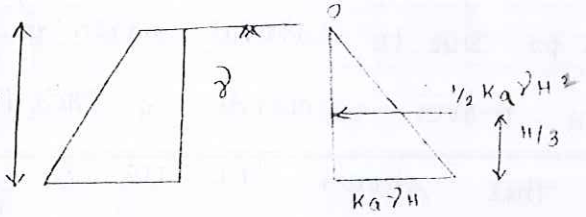
$$P_a = \frac{1}{2} k_a \gamma H^2$$

$$P_p = \frac{1}{2} k_p \gamma H^2$$

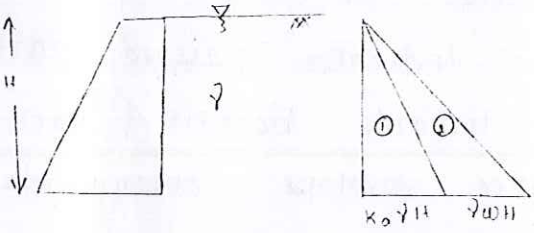
$$k_a = \frac{1 - \sin\phi}{1 + \sin\phi}, \quad k_p = \frac{1 + \sin\phi}{1 - \sin\phi}$$

pressure distribution diagram for different conditions:

1) Dry soil



2) Fully saturated soil

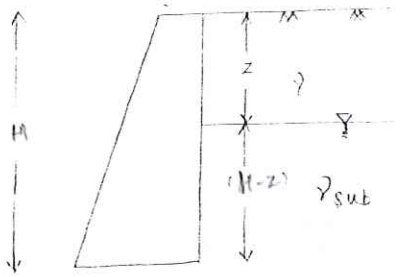


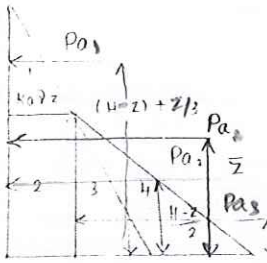
$P_{o1} = \frac{1}{2} k_o \gamma H^2$ at H/3 from base

$P_{o2} = \frac{1}{2} \gamma_w H$ at H/3 from base

$P_o = P_{o1} + P_{o2}$

3) partially saturated soil:





$Pa_1 = K_a \gamma z$

$Pa_1 = \frac{1}{2} K_a \gamma z^2$ at $(H-z) + \frac{z}{3}$ from base

$Pa_2 = K_a \gamma z$

$Pa_2 = K_a \gamma z \times (H-z)$ at $\frac{H-z}{2}$ from base

$Pa_3 = K_a \gamma_{sub} (H-z)$

$Pa_3 = \frac{1}{2} K_a \gamma_{sub} (H-z)^2$ at $\frac{H-z}{3}$ from base

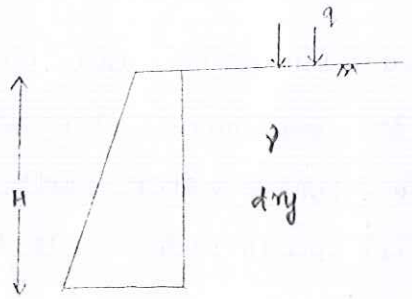
$Pa_4 = \gamma_w (H-z)$

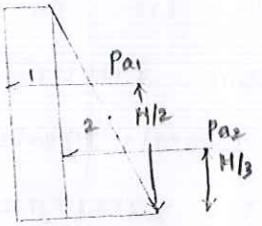
$Pa_4 = \frac{1}{2} \gamma_w (H-z)^2$ at $\frac{H-z}{3}$ from base

$$Pa = Pa_1 + Pa_2 + Pa_3 + Pa_4$$

$$\bar{z} = \frac{Pa_1 \times (H-z) + z/3 + Pa_2 \times \frac{H-z}{2} + Pa_3 \frac{H-z}{3} + Pa_4 \frac{H-z}{3}}{Pa}$$

4) Soil with surcharge





$Pa_1 = Ka\, q$

$Pa_1 = Ka\, q \times H$ at $H/2$ from base

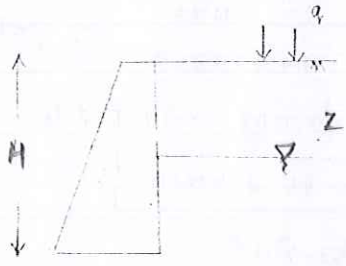
$Pa_2 = Ka\, \gamma H$

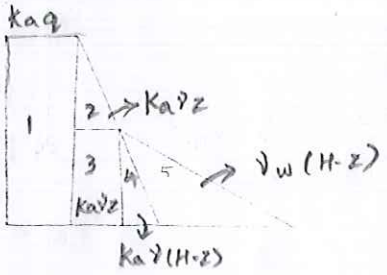
$Pa_2 = \frac{1}{2} Ka\, \gamma H^2$ at $H/3$ from base

$Pa = Pa_1 + Pa_2$

$$\bar{z} = \frac{Pa_1 \times \frac{H}{2} + Pa_2 \times H/3}{Pa}$$

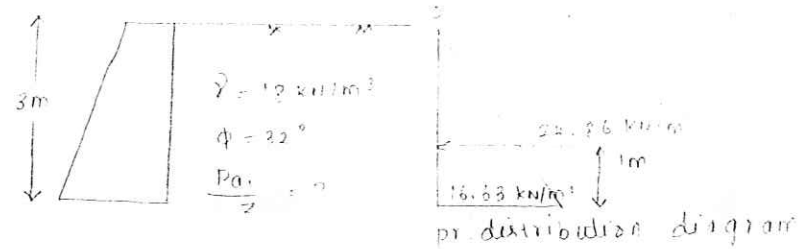
5) po Surcharge with partially saturated soil:







1. A retaining wall with smooth vertical back retains backfill to the ht. of 3m. The density of backfill of sand is 18 kN the angle of shearing resistance is 32°. Estimate the total active earth pressure of wall and point of application of total pressure.



pr. distribution diagram

At top, pressure = 0.

At Bottom, $P_a = k_a \gamma H$

$$K_a = \frac{1-\sin\phi}{1+\sin\phi} = \frac{1-\sin 32^\circ}{1+\sin 32^\circ} \Rightarrow K_a = 0.307$$

$$P_a = 0.307 \times 18 \times 3 \Rightarrow \boxed{P_a = 16.63\ kN/m^2}$$

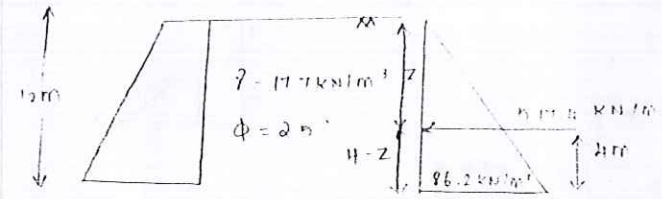
$$P_a = \frac{1}{2} K_a \gamma H^2$$

$$= \frac{1}{2} \times 0.307 \times 18 \times 3^2 \quad \boxed{= 24.86\ kN/m}$$

$$\bar{z} = \frac{z}{3} = \frac{3}{3} = 1m \text{ from base}$$

2. The retaining wall retains 12m of backfill with unit wt. of 17.7 kN/m³, angle of internal friction 25°. Determine i) magnitude and point of application of total pressure. ii) If water table at a height of 6m from G.L. Determine its magnitude & pt. of application of total pressure.

Case 1:



$$P_a = K_a \gamma H$$

$$K_a = \frac{1-\sin 25^\circ}{1+\sin 25^\circ} = 0.406$$

$$P_a = 0.406 \times 17.7 \times 12$$

$$\boxed{P_a = 86.2\ kN/m^2}$$

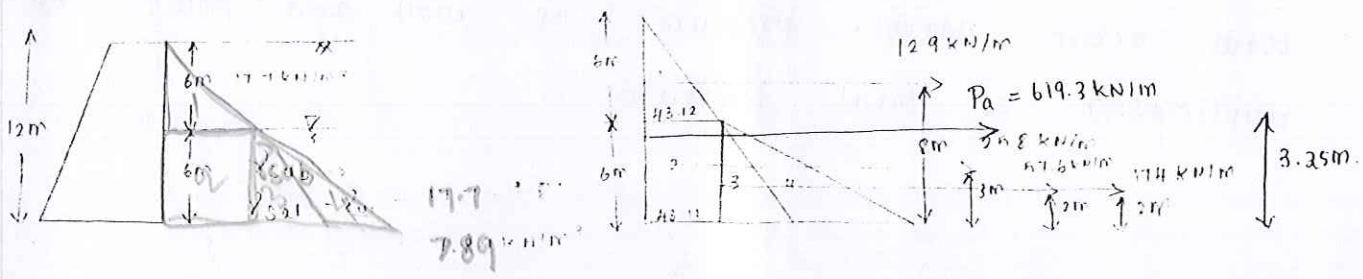
$$P_a = \frac{1}{2} K_a \gamma H^2$$

$$= \frac{1}{2} \times 0.406 \times 17.7 \times 12^2$$

$$\boxed{P_a = 517\ kN/m}$$

$$\bar{z} = \frac{z}{3} = \frac{12}{3} = 4m \text{ from base.}$$

Case 2:



$P_{a_1} = k_a \gamma H z$

$= 0.406 \times 17.7 \times 6$ $\boxed{= 43.12 \text{ kN/m}^2}$

$P_{a_1} = \frac{1}{2} k_a \gamma H^2$

$= \frac{1}{2} \times 0.406 \times 17.7 \times 6^2$ $\boxed{= 129 \text{ kN/m}}$

$\bar{z} = (H - z) + \frac{z}{3} = (12 - 6) + \frac{6}{3} = 8\text{m}$ from base.

$P_{a_2} = k_a \gamma z$

$= 0.406 \times 17.7 \times 6 = \boxed{43.12 \text{ kN/m}^2}$

$P_{a_2} = k_a \gamma z \times (H - z)$

$= 0.406 \times 17.7 \times 6 \times (12 - 6)$ $\boxed{= 258.7 \text{ kN/m}}$

$\bar{z} = \frac{H - z}{2} = \frac{12 - 6}{2} = 3\text{m}$.

$P_{a_3} = k_a \gamma_{sub} (H - z)$

$= 0.406 \times 7.89 \times (12 - 6) = \boxed{19.22 \text{ kN/m}^2}$

wt of
gnitude
e at
of

$P_{a_3} = \frac{1}{2} k_a \gamma_{sub} (H - z)^2$

$= \frac{1}{2} \times 0.406 \times 7.89 (12 - 6)^2$ $\boxed{= 57.6 \text{ kN/m}}$

$\bar{z} = \frac{H - z}{3} = \frac{12 - 6}{3} = 2\text{m}$ from base

$P_{a_4} = \gamma_w (H - z)$

$= 9.81 (12 - 6)$ $\boxed{= 58 \text{ kN/m}^2}$

$P_{a_4} = \frac{1}{2} \gamma_w (H - z)^2$

$= \frac{1}{2} \times 9.81 (12 - 6)^2$ $\boxed{= 174 \text{ kN/m}}$

$\bar{z} = \frac{H - z}{3} = \frac{12 - 6}{3} = 2\text{m}$ from base

$P_a = P_{a_1} + P_{a_2} + P_{a_3} + P_{a_4}$

$\boxed{P_a = 629.3 \text{ kN/m}}$

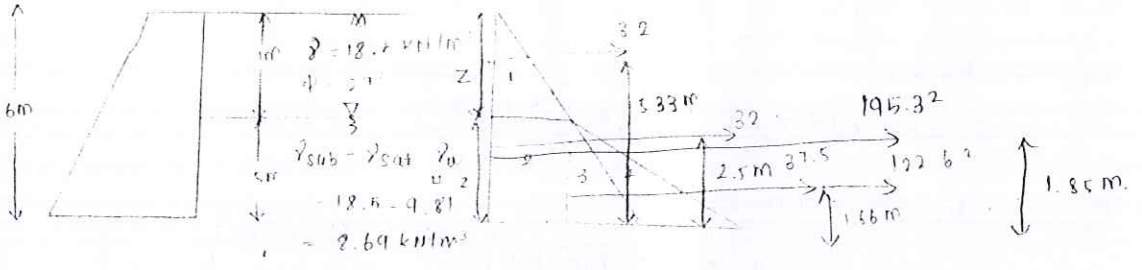
$\bar{z} = \dfrac{P_{a_1} \times (H - z) + z/3 + P_{a_2} \times \frac{H - z}{2} + P_{a_3} \frac{H - z}{3} + P_{a_4} \frac{H - z}{3}}{P_a}$

$= \dfrac{129 (12 - 6) + 6/3 + 258.7 \times \left(\frac{12 - 6}{2}\right) + 57.6 \left(\frac{12 - 6}{3}\right) + 174 \left(\frac{H}{}\right.}{629.3}$

3.25m

$\boxed{\bar{z} = 3.25\text{m}}$.

3. A retaining wall retains cohesionless soil of unit wt. 18.5 kN/m³ & $\phi = 29°$, ht. of retaining wall is 6m, WT is 5m above the base of the wall. Find the magnitude and position of active earth pressure.



$$Pa_1 = K_a \gamma z \qquad K_a = \frac{1-\sin 29°}{1+\sin 29°} = 0.346$$

$$= 0.346 \times 18.5 \times 1$$

$$= 6.401 \text{ kN/m}^2$$

$$\bar{z} = (H-z) + \frac{z}{3}$$

$$= 6 - 1 + \frac{1}{3}$$

$$= 5.33 \text{ m}$$

$$Pa_1 = \frac{1}{2} K_a \gamma z^2 = \frac{1}{2} \times 0.346 \times 18.5 \times 1^2 = 3.20 \text{ kN/m}$$

$$Pa_2 = K_a \gamma z = 0.346 \times 18.5 \times 5 \times 1 = 6.401 \text{ kN/m}^2$$

$$Pa_2 = K_a \gamma z \times (H-z) = 6.401 \times (6-1) = 32.0 \text{ kN/m}$$

$$\bar{z} = \frac{H-z}{2} = \frac{6-1}{2} = 2.5 \text{ m from base.}$$

$$Pa_3 = K_a \gamma_{sub} (H-z) = 0.346 \times 8.69 \times \frac{(H-z)}{5} \times (6-1) = 15.03 \text{ kN/m}^2$$

$$Pa_3 = \frac{1}{2} K_a \gamma_{sub} (H-z)^2 = \frac{1}{2} \times 15.03 \times 5^2 = 37.5 \text{ kN/m}$$

$$\bar{z} = \frac{H-z}{3} = \frac{6-1}{3} = 1.66 \text{ m from base.}$$

$$Pa_4 = \frac{1}{2} \gamma_w (H-z)^2 = \frac{1}{2} \times 9.81 \times 5^2 = 122.62 \text{ kN/m}$$

$$Pa_4 = \gamma_w (H-z) = 9.81 \times 5 = 49.05 \text{ kN/m}^2$$

$$\bar{z} = \frac{H-z}{3} = \frac{5}{3} = 1.66 \text{ m from base.}$$

$$Pa = Pa_1 + Pa_2 + Pa_3 + Pa_4 = 195.32 \text{ kN/m.}$$

$$\bar{z} = \frac{(3.2 \times 5.33) + (32 \times 2.5) + (37.5 \times 1.66) + (122.62 \times 1.66)}{195.32}$$

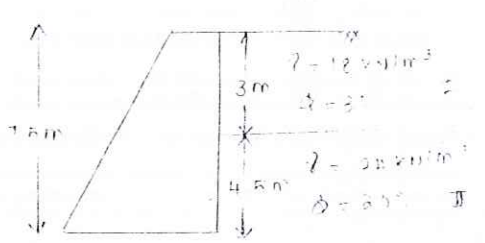
$$\bar{z} = 1.85 \text{ m}$$

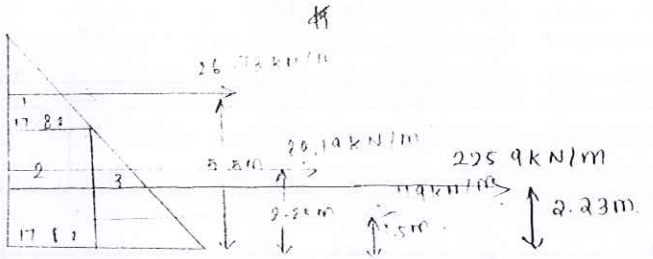
5 kN/m³ 28.19

bove

tion

4) The retaining wall 7.5m ht retains a cohesionless backfill at the top 3m of backfill has unit wt of 18 kN/m³ & $\phi = 30°$, the rest has a unit wt of 24 kN/m³ & $\phi = 20°$. Determine the total pressure & point of application.





$$K_{a_1} = \frac{1 - \sin 30°}{1 + \sin 30°} = 0.33 \qquad K_{a_2} = \frac{1 - \sin 20°}{1 + \sin 20°} = 0.49$$

1-z) + z/3

-1 + 1/3

5.33m

At top : 0,

$P_{a_1} = K_{a_1} \gamma H^z = 0.33 \times 18 \times 3$

at 3m $= 17.82 \text{ kN/m}^2$

N/m²

$P_{a_1} = \frac{1}{2} K_{a_1} \gamma H^2 = \frac{1}{2} \times 0.33 \times 18 \times 3^2 = 26.73 \text{ kN/m}^2$

$\bar{z} = (H - z) + \frac{z}{3} = (7.5 - 3) + \frac{3}{3} = 5.5 \text{ m}$ from base.

$P_{a_2} = K_{a_1} \gamma H = 0.33 \times 18 \times 3 = 17.82 \text{ kN/m}^2$

$P_{a_2} = K_{a_1} \gamma z \times (H - z) = 0.33 \times 18 \times 3 \times (7.5 - 3) = 80.19 \text{ kN/m}$

$\bar{z} = \frac{H - z}{2} = \frac{7.5 - 3}{2} = 2.25$ from base

$P_{a_3} = K_{a_2} \gamma H$

$= 0.49 \times 24 \times 4.5 = 52.92 \text{ kN/m}^2$

$P_{a_3} = \frac{1}{2} K_{a_2} \gamma H^2 = \frac{1}{2} \times 52.92 \times 4.5 = 119 \text{ kN/m}$

$\bar{z} = \frac{H - z}{3} = \frac{7.5 - 3}{3} = 1.5$ from base

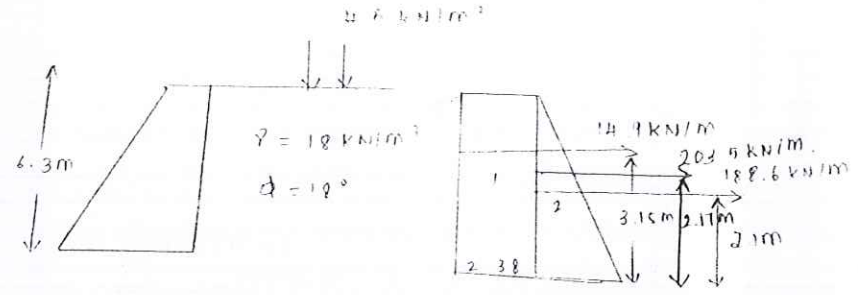
$P_a = P_{a_1} + P_{a_2} + P_{a_3}$

$= 225.9 \text{ kN/m}$

$\bar{z} = \frac{26.73 \times 5.5 + 80.19 \times 2.25 + 119 \times 1.5}{225.9}$

$\bar{z} = 2.23 \text{ m}$

3) A smooth vertical wall is 6.3m ht & retains the soil with unit wt 18 kN/m³ & $\phi = 18°$. If the soil surface carries a udl (surcharge) of 14.5 kN/m² Determine the total active pr & pt of application.



$$K_a = \frac{1-\sin 18°}{1+\sin 18°} = 0.528$$

$P_{a_1} = K_a q = 0.528 \times 4.5 = 2.38 \text{ kN/m}^2$

$P_{a_1} = K_a q \times H = 2.38 \times 6.3 = 14.9 \text{ kN/m}$ $\bar{z} = \frac{6.3}{2} = 3.15\text{m}$ from base

$P_{a_2} = K_a \gamma H = 0.528 \times 18 \times 6.3 = 59.87 \text{ kN/m}^2$

$P_{a_2} = \frac{1}{2} \times K_a \gamma H^2 = \frac{1}{2} \times 59.87 \times 6.3$

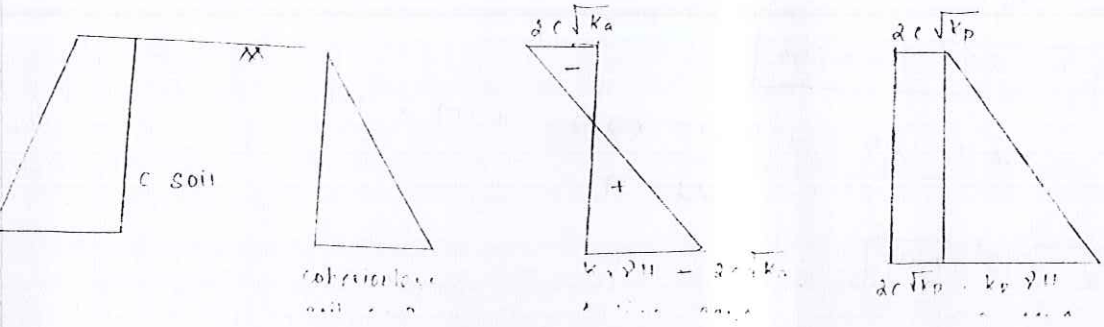
$P_{a_2} = 188.6 \text{ kN/m}$ at $\bar{z} = \frac{H}{3} = \frac{6.3}{3} = 2.1\text{m}$ from base

$P_a = P_{a_1} + P_{a_2} = 203.57 \text{ kN/m}$

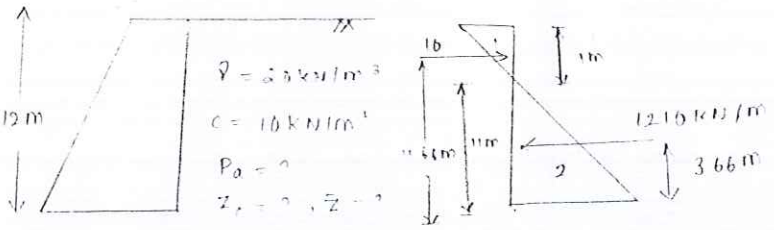
$\bar{z} = \frac{(14.9 \times 3.15) + (188.6 \times 2.1)}{203.5} = 2.17\text{m}.$

Rankine's Theory for Cohesive Soil

Rankine's original theory is meant for cohesionless soil only but the theory is extended for cohesive soils by scientists Resal & Bell. In cohesive soils the failure envelope & pr. distribution diagram has a cohesion intercept in active case. Whereas in cohesionless soils, the cohesion intercept will be equal to zero



1 A retaining wall with a smooth vertical back retains a cohesive back fill, the ht of wall is 12m, unit wt. of soil is 20 kN/m³, cohesion 10 kN/m². What is the total active earth pr on the wall At what depth is the intensity of pressure will be zero & where the resultant pressu would act



cohesive, $\phi = 0$

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

$$z_c = \frac{2c}{\gamma\sqrt{K_a}} = \frac{2\times10}{20\sqrt{1}} \quad \boxed{= 1m}$$

$$P_{a_1} = 2c\sqrt{K_a} = 2\times10\sqrt{1} \quad \boxed{= 20\ kN/m^2}$$

$$P_{a_1} = \frac{1}{2}\times20\times1 \quad \boxed{= -10\ kN/m} \quad \text{at} \quad \bar{z} = (H-z) + \frac{2}{3}(z)$$

$$= (12-1) + \frac{2}{3}(1) = 11.66m \text{ f bas}$$

$$P_{a_2} = K_a\gamma H - 2c\sqrt{K_a}$$

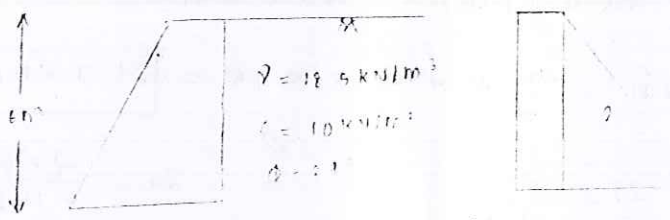
$$= 1\times20\times12 - 2\times10\sqrt{1} \quad \boxed{= 220\ kN/m^2}$$

$$P_{a_2} = \frac{1}{2}\times220\times11 \quad \boxed{= 1210\ kN/m} \quad \text{at} \quad \frac{H-z}{3} = \frac{12-1}{3} = 3.66m \text{ from bas}$$

$$P_a = -P_{a_1} + P_{a_2} = -10 + 1210 \quad \boxed{= 1200\ kN/m.}$$

$$\bar{z} = \frac{(-10\times11.66) + (1210\times3.67)}{1200} \Rightarrow \boxed{3.6m}$$

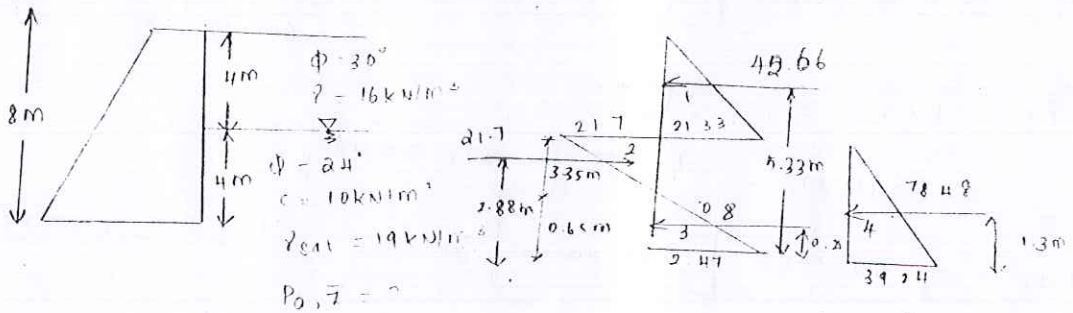
2. A retaining wall 6m ht. retains c-ϕ soil of unit wt 18.5 kN/m³, cohesion 10 kN/m², $\phi = 29°$. Find the total pressure (magnitude) & position of total passive pressure.



K_p =

2419

3. Determine the Rankine active pressure and point of application for the problem given in figure.



$$K_{a_1} = \frac{1-\sin 30^\circ}{1+\sin 30^\circ} = 0.33, \qquad K_{a_2} = \frac{1-\sin 24^\circ}{1+\sin 24^\circ} = 0.42.$$

$$P_{a_1} = K_{a_1}\gamma H^2 = 0.33 \times 16 \times 4 \quad \boxed{= 21.33\ kN/m^2}$$

$$P_{a_1} = \frac{1}{2} \times 21.33 \times H^2 = \frac{1}{2} \times 21.33 \times 4 = \boxed{42.66\ kN/m}$$

$$\bar{Z} = (H-Z) + \frac{Z}{3} \Rightarrow (8-4) + \frac{4}{3} = 5.33\ m \text{ from the base.}$$

$$P_{a_2} = 2c\sqrt{K_{a_2}} = 2 \times 10\sqrt{0.42} = 12.96\ kN/m^2.$$

$$P_{a_2} = -\frac{1}{2} \times 12.96 \times H^2 = -\frac{1}{2} \times 12.96 \times 3.35 = \boxed{-21.7\ kN/m}$$

$$Z_c = \frac{2c}{\gamma'\sqrt{K_{a_2}}} \qquad Z_c = \frac{2 \times 10}{9.19\sqrt{0.42}} = 3.35\ m.$$

$$\gamma' = \gamma_{sat} - \gamma_w = 19 - 9.81 = 9.19\ kN/m^3.$$

$\bar{z} = 0.65 + \frac{2}{3} \times 3.35 = 2.88$ m from base

$Pa_3 = Ka_2 \gamma' H - 2c\sqrt{Ka_2}$

$= 0.42 \times 9.19 \times 4 - 2 \times 10\sqrt{0.42} = 2.47$ kN/m²

$Pa_3 = \frac{1}{2} \times 2.47 \times 0.65$ = 0.8 kN/m

$\bar{z} = \frac{0.65}{3} = 0.216$ m from base.

$Pa_4 = \gamma_w H = 9.81 \times 4 = 39.24$ kN/m²

$Pa_4 = \frac{1}{2} \times 39.24 \times 4$ = 78.48 kN/m

$\bar{z} = \frac{4}{3} = 1.3$ m from base

$Pa = Pa_1 - Pa_2 + Pa_3 + Pa_4$ = 99.83 kN/m

$\bar{z} = \frac{(42.66 \times 5.33) + (-21.7 \times 2.88) + (0.8 \times 0.216) + (78.48 \times$

99.83

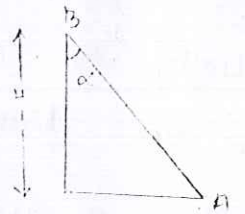
$\bar{z} = 2.67$ m

ication

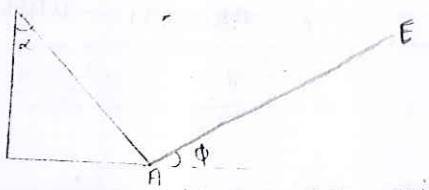
(X) BULMANN'S GRAPHICAL METHOD FOR THE DETERIMINATION OF ACTIVE EARTH PRESSURE:

(X) ** 16m Procedure:

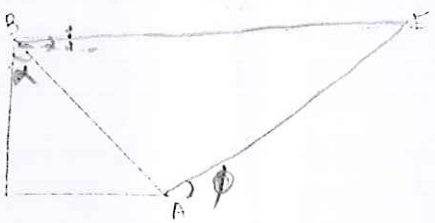
Step1: Draw the retaining wall for the given angle α or β and for given height with suitable scale



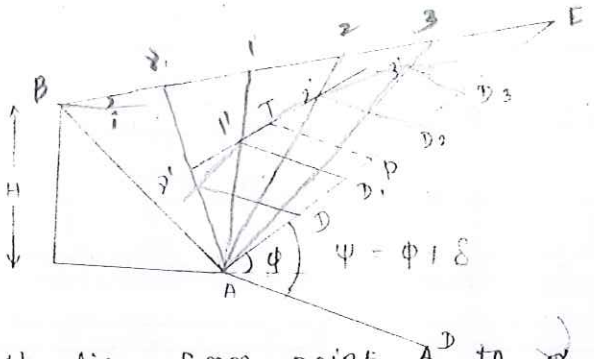
Step2: Draw the line AE with an angle ϕ with the horizor This line is called as weight line.



Step3: Draw the line BE with an angle i with the horizontal.



Step 4: Divide the line BF equally & note down the points. The points are 0, 1, 2, 3.



AB0
AB1
AB2
AB3

Step 5: Draw the line from point A to 0, 1, 2, 3.

Step 6: Find the area of wedges (AB0, AB1, AB2, AB3).

Step 7: Find the wt. of the wedges by multiplying the unit wt. of soil.

Step 8: Draw the line AD with an angle ψ to the weight line (AE).

Step 9: Lay of AE distance to a suitable scale to represent the wt. of wedges.

Step 10: Draw the lines parallel to the AD from the points D, D1, D2, D3.

Step 11: Join the points 0', 1', 2', 3' by means of a smooth curve. This curve is called as Culmann's curve.

Step 12: A line is drawn tangentially to the culmann curve and that must be parallel to the line AE.

Step 13: Find the point of tangent of the culmann curve.

Step 14: Draw the line parallel to the AB from the point T. the magnitude of TP in its natural unit gives the maximum active pressure.

1. For a retaining wall system [illegible]

of the retaining wall = 7m, γ = 1.6 g/cc, $\delta = 20°$, $\beta = 20°$, $\phi = 35°$

Back of the retaining wall is inclined with angle 20° with vertical. Backfill surface sloping 1:10, Determine the magnitude of active earth pressure by culmann's method. Take $\psi = 50°$, 1cm = 2m

3019 Given:

$H = 7m$, $\gamma = 1.6$ g/cc, $\delta = 20°$, $\beta = 20°$, $\phi = 35°$, $\alpha = 20°$

$i = \frac{1}{10} = 6°$ $\psi = 50°$ 1cm = 2m.

Solution:

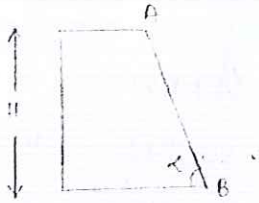
In Graph Sheet

4/10

(8) REBHANN'S GRAPHICAL METHOD FOR THE DETERMINATON OF

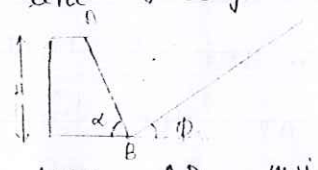
16M ACTIVE EARTH PRESSURE:

Step 1: Draw the R.W with suitable scale.



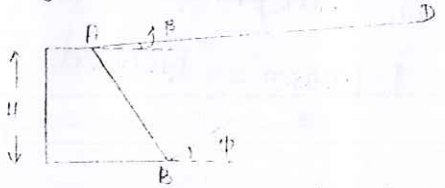
ints

Step 2: Draw the line freely inclined at an angle $\phi°$ horizo



h curve

Step 3: Draw the line AD with an angle β.



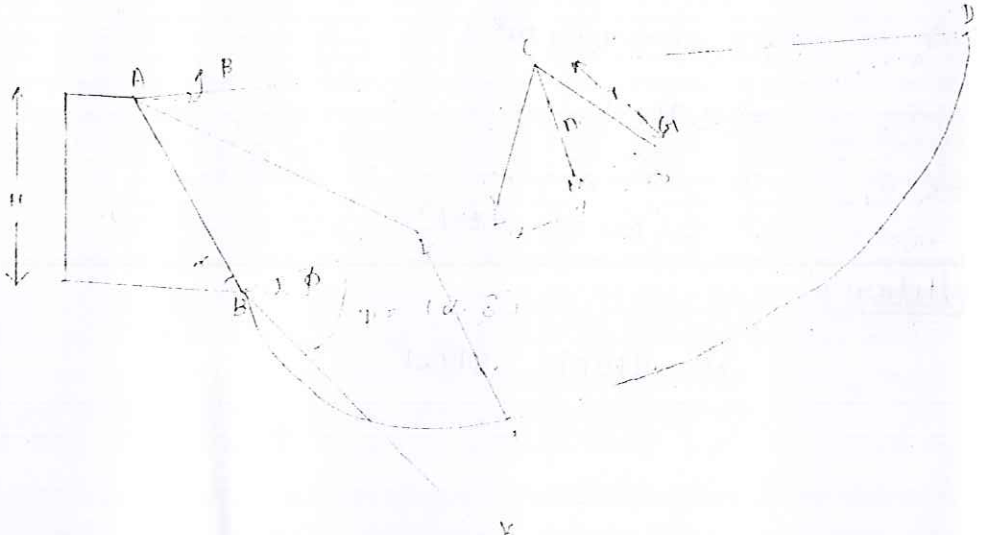
curve

ave.

Step 4: Draw the line BK inclined at an angle ψ with B.

int

$\psi = (\alpha - \delta)$



$P_a = \frac{1}{2} \gamma x^2 \sin\psi$

$P_a = \frac{1}{2} \gamma \times H.$

e

Y

Step 5: Through A draw the line AE parallel to the ψ line to meet BD in E.

Step 6: Describe a semicircle on BD as a diameter

Step 7: Erect a perpendicular to BD at E to meet the semicircle at F.

Step 8: With B as centre, BF as radius draw an arc to meet BD in G_1

Step 9: Through G_1, draw a parallel to ψ line to meet AD in C.

Step 10: With G_1 as centre, G_1C radius draw an arc to cut BD in L.

Join the point CL

Step 11: Also draw a $\perp^r$ CM from C on to LG. Consider this distance $LG_1 = x$, $CG_1 = x$, $CM = n$.

---

1. A retaining wall 3.6m height supports a dry cohesionless backfill with a plane ground surface sloping upwards at a surcharge angle of 10° from the top of the wall. Back of the wall is inclined to the vertical at a +ve batter angle of 9°. The unit wt. of the backfill is 18.9 kN/m³ and $\phi = 30°$. Wall friction angle $\delta = 12°$. Determine the total active pressure by Rebhann's method.

Given:

$H = 3.6m$, $\phi = 30°$

$\delta = 12°$, $\beta = 10°$

$\gamma = 18.9\ kN/m^3$

$\alpha = 90 - 9 = 81°$.

$\psi = \alpha - \delta$

$= 81 - 12 = 69°$.

Solution:

In graph sheet

7/10

St

1) T

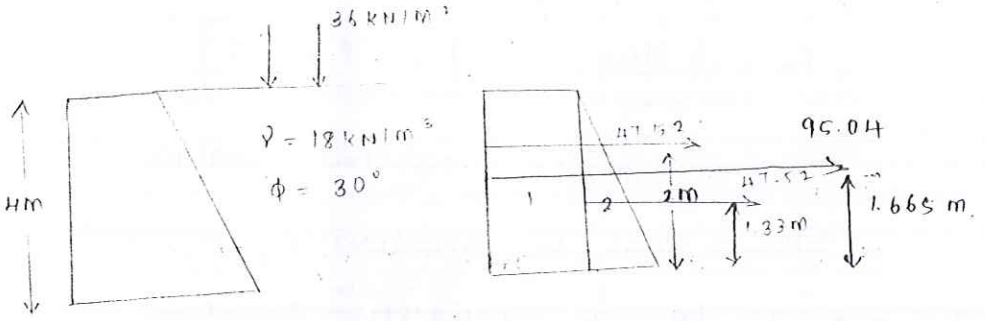
2) D.

3) D.

4) C

∴ d

2) A smooth back vertical wall is 4m height and retain the soil with a bulk unit wt. of 18 kN/m², angle of internal friction 30° If soil surface there is a udl of 36 kN/m². Determine the total active pressure per me length of the wall & also find its point of applicatio



$$K_a = \frac{1 - \sin 30^\circ}{1 + \sin 30^\circ} = 0.33$$

$$P_{a_1} = k_a q = 0.33 \times 36 = 11.88 \text{ kN/m}^2$$

$$P_{a_1} = k_a q \times H = 11.88 \times 4 = 47.52 \text{ kN/m}$$

$$\bar{z} = \frac{H}{2} = 2\text{m} \text{ from base}$$

$$P_{a_2} = k_a \gamma H = 0.33 \times 18 \times 4 = 23.76 \text{ kN/m}^2$$

$$P_{a_2} = \frac{1}{2} \times k_a \gamma H^2 = \frac{23.76}{2} \times 4 = 47.52 \text{ kN/m}$$

$$\bar{z} = \frac{H}{3} = 1.33\text{m} \text{ from base}$$

$$\boxed{P_a = 95.04 \text{ kN/m}}$$

$$\bar{z} = \frac{(47.52 \times 2) + (47.52 \times 1.33)}{95.04} = 1.665 \text{ m}.$$

7/10

Stability of retaining wall:

1) Determination of coefficient of active earth pressure
2) Determination of total active pressure.
3) Determination of horizontal & vertical components.

$$P_H = P_a \cos(\delta + \phi') \qquad P_V = P_a \sin(\delta + \phi')$$

4) Calculation of forces (i.e.) vertical & Horizontal forces
leaver arm moments
↓
(point of application)

5) check the stability against sliding $\boxed{\mu \frac{R_v}{R_h} \geq 1.5}$

$\mu = \tan \delta$.

R_v = vertical force.

R_h = horizontal force.

6) check factor of safety against over turning.

$$F_o = \frac{\Sigma M_R}{\Sigma M_o} \qquad \boxed{F_o = 1.5 \text{ to } 2}$$

ΣM_R = Sum of resistive moment.

ΣM_o = sum of overturning moment.

7) check against bearing capacity failure.

$$F_s = \frac{q_{na}}{P_{max}} > 3$$

q_{na} = allowable soil pressure.

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

b = width of RW at bottom

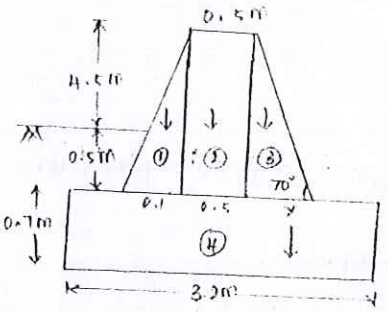
e = eccentricity $= \frac{b}{2} - \bar{x}$

$$\bar{x} = \frac{\Sigma M}{\Sigma R_v} \qquad \Sigma M = \text{Sum of moments.}$$

8) check against tension. $e < \frac{b}{6}$

Check the stability of RW as shown in fig.

unit wt. of concrete = 24 KN/m³

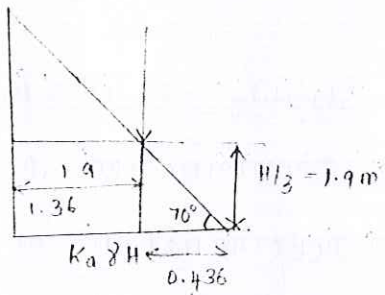


$\delta = 24°$

$\phi = 36°$

$\phi' = 26°$

$\gamma = 23 \text{ KN/m}^3$



1) co-efficient of active earth pressure

$$K_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.259 \simeq 0.26.$$

) total active pressure $= \frac{1}{2} K_a \gamma H^2 = 92.92$ KN/m at 1.9m from base.

3) $P_H = P_a \cos(\delta + \phi') = 66.84$ kN/m.

4) $P_V = P_a \sin(\delta + \phi') = 64.54$ kN/m.

tan 70° = .

x = i.

4) Forces:

S.No	Description	Forces		lever arm	moments	
		vertical kN/m	Horizontal		M_R	M
1	$W_1 = \downarrow$ $\frac{1}{2} \times 0.1 \times 5 \times 24$	6	–	$0.4 + 2/3(0.1)$ $= 0.467$	6×0.467 $= 2.8$	–
2.	$W_2 = \downarrow$ $0.5 \times 5 \times 24$	60	–	$0.5 + \frac{0.5}{2}$ $= 0.75$	60×0.75 $= 45$	–
3.	$W_3 = \downarrow$ $\frac{1}{2} \times 1.8 \times 5 \times 24$	108	–	$1 + 1/3(1.8)$ $= 1.6$	108×1.6 $= 172.8$	–
4	$W_4 = \downarrow$ $3.2 \times 0.7 \times 24$	53.76	–	$3.2/2$ $= 1.6$	53.76×1.6 $= 86.01$	
5.	vertical component	64.54	–	$1 + 1.36$ $= 2.36$	64.54×2.36 $= 152.31$	
6.	Horizontal component	–	66.84	1.9	–	126

$\Sigma R_V = 292.3$ kN/m

$\Sigma R_H = 66.84$ kN/m

$\Sigma M_R = 458.92$

$\Sigma M_o = 126.9$

$\Sigma M = \Sigma M_R - \Sigma M_o = 332.02$

Etc = 24 KN/m³

1.9m

5) Check for sliding

$$\mu \frac{R_V}{R_h} = 0.445 \left(\frac{292.3}{66.84}\right) = 1.94 > 1.5$$

Hence safe against sliding.

6) Check FOS

$$F_o = \frac{\Sigma M_R}{\Sigma M_o} = \frac{458.92}{126.9}$$

from base.

$F_o = 3.61 > 1.5$ to 2 : Hence safe against overtu

$F_S = \frac{q_{na}}{P_{max}}$

Take $q_{na} = 600\ kN/m^2$ (given)

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

$= \frac{292.3}{3.2}\left(1 + \frac{6(0.465)}{3.2}\right)$

$P_{max} = 170.98$

$e = \frac{b}{2} - \bar{x} \qquad \bar{x} = 1.135$

$e = 0.465$

$F_S = \frac{600}{170.98} = 3.5 > 3.$

Safe against BC failure.

8) $e < \frac{b}{6}$

$0.47 < 0.53$

Hence safe against tension.

$CGPA = \frac{(3\times10) + (4\times8) + (4\times8) + (3\times9) + (3\times9) + (5\times8) + (2\times10) + (2\times7)}{3+4+4+3+3+5+2+2}$

sedim

sedi

Objec

i) To

2) To

Theo

matic

plain

They

their

for

are

form

colle

in

result

of

partic

charg

Sum

chan

stab

on

the

eqn o

from

the

pass