2.2.2. Earth Pressure and Retaining Structures

→ A retaining structure is required to provide lateral support to the soil mass.

→ The design of the retaining structure requires the determination of the magnitude and line of action of the lateral earth pressure.

→ The magnitude of the lateral earth pressure depends upon the mode of the movement of the wall, the flexibility of the wall, the properties of the soil, the drainage conditions etc.

Different Types of Lateral Earth Pressure.

→ Depending upon the movement of the retaining wall with respect to the soil retained, Lateral earth pressure can be grouped into 3 categories.

(i) At rest pressure
(ii) Active pressure
(iii) Passive pressure.

* The soil retained is also known as backfill.

(1) At rest pressure.

→ The lateral earth pressure is called at-rest pressure when the soil mass is not subjected to any lateral yielding or movement. This case occurs when the retaining wall is firmly fixed at its top and is not allowed to rotate or move laterally.

→ The at-rest condition is also known as the elastic equilibrium, as no part of soil mass has failed and attained the plastic equilibrium

eg. basement retaining walls which are restrained against the movement by the basement slab provided at their tops, bridge abutment wall which is restrained at its top by the bridge slab.

(2) Active pressure

→ A state of active pressure occurs when the soil mass yields in such a way that it tends to stretch horizontally. It is a state of plastic equilibrium as the entire soil mass is on the verge of failure.

CamScanner

→ A retaining wall when moves away from the backfill, there is a stretching of the soil mass and the active state of earth pressure exists.
In fig (b), the active pressure develops on the right-hand side when the wall moves towards left.

(3) Passive Pressure

→ A state of passive pressure exists when the movement of the wall is such that the soil tends to compress horizontally.
→ In fig (b), the passive pressure develops on the left-side of the wall below the ground level, as the soil in this zone is compressed when the movement of the wall is towards left.
⇒ eg. pressure acting on the anchor block.

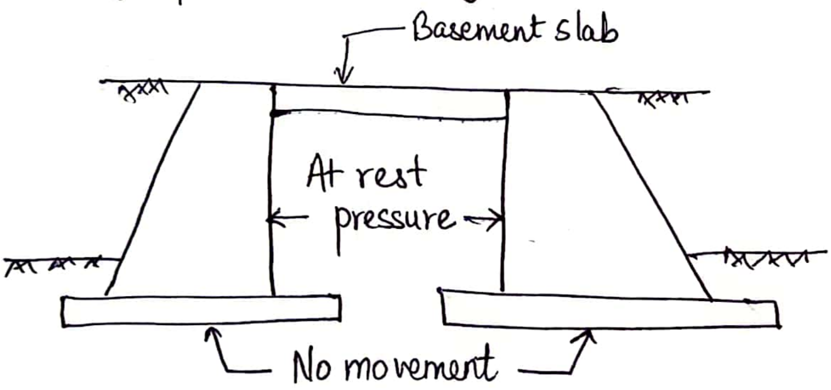


fig (a) At-rest pressure

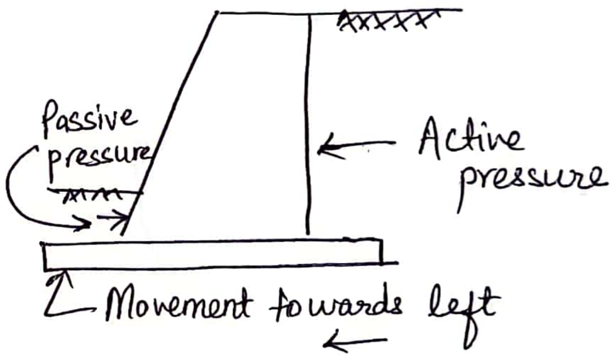


fig (b)

Variation of Pressure

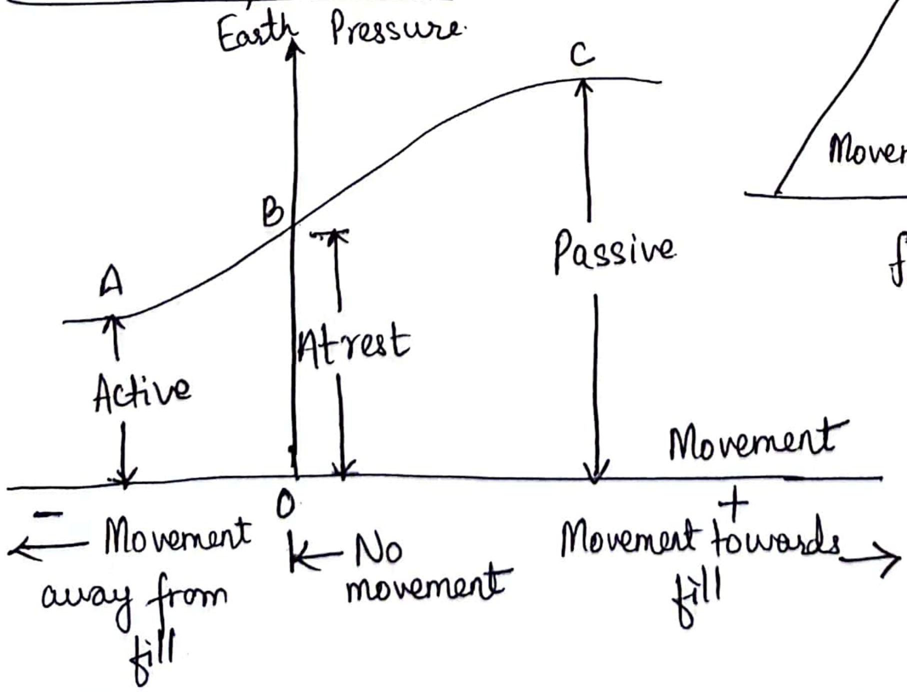


fig: Variation of earth pressure with wall movement.

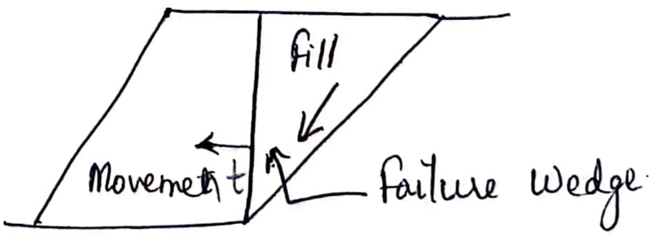


fig.(Active)

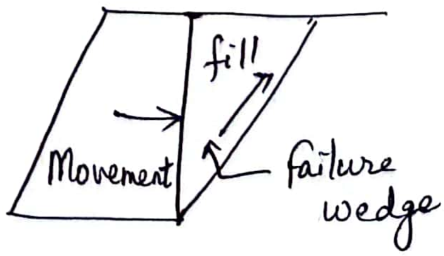


fig: Passive

CamScanner

Earth Pressure At Rest

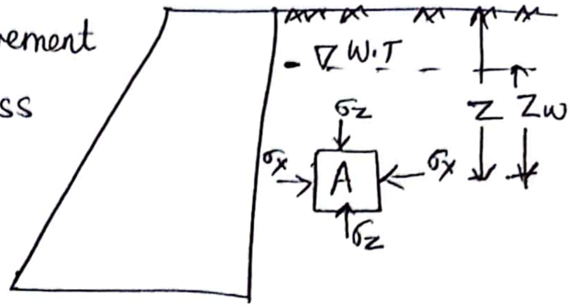


fig. shows a retaining wall in which no movement takes place. The vertical effective stress at point A at a depth z is given by

$$\bar{\sigma}_z = \gamma z - \gamma_w z_w$$

The horizontal intergranular (effective) stress can be obtained using the coefficient of earth pressure at rest (K_o) which is equal to the ratio of the horizontal stress to vertical stress,

Thus, $$K_o = \frac{\bar{\sigma}_x}{\bar{\sigma}_z}$$

$$\therefore \quad \bar{\sigma}_x = K_o \cdot \bar{\sigma}_z = K_o(\gamma z - \gamma_w z_w)$$

The stress $\bar{\sigma}_x$ is usually represented as p_o, indicating the lateral pressure at rest.

Thus, $$p_o = K_o(\gamma z - \gamma_w z_w)$$

The coefficient of lateral pressure at rest (K_o) relates the effective stresses. The total lateral pressure (p_h) is equal to the sum of the intergranular pressure (p_o) and the pore water pressure (u).

Thus, $$p_h = p_o + u$$

In fig, the lateral pressure at depth z is, therefore.

$$p_h = K_o(\gamma z - \gamma_w z_w) + \gamma_w z_w \quad \text{——— (i)}$$

The eq^n (i) indicates, the pressure distribution is triangular with zero pressure at the top ($z = 0$), and the maximum pressure at the bottom of the wall.

CS CamScanner

pressure distribution when the soil is dry.

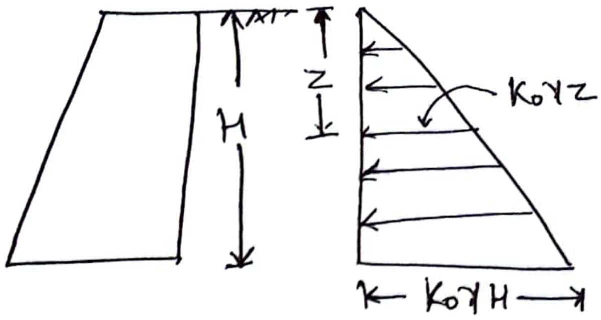
→ The pressure at the bottom of the wall at depth H is given by

$$P_h = K_0 \gamma H$$

The total pressure force per unit length of the wall is given by

$$P = \int_0^H K_0 \gamma z \cdot dz$$

$$P = \frac{1}{2} K_0 \gamma H^2 .$$



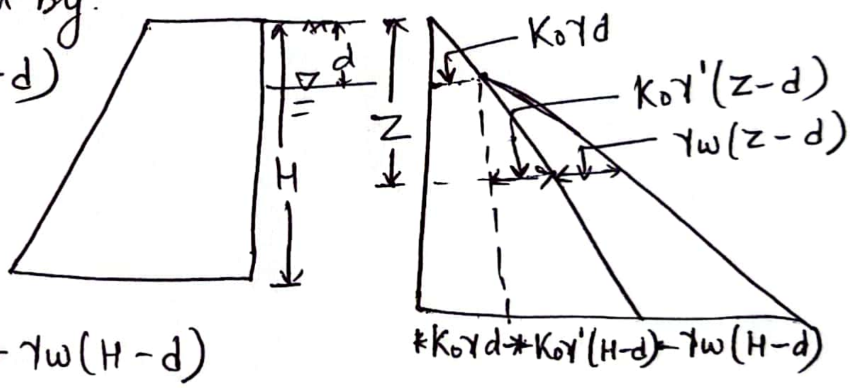
depth of water table is at depth d below the surface.

Pressure at depth $z > d$ is given by.

$$P_h = K_0 \gamma d + K_0 \gamma'(z-d) + \gamma_w(z-d)$$

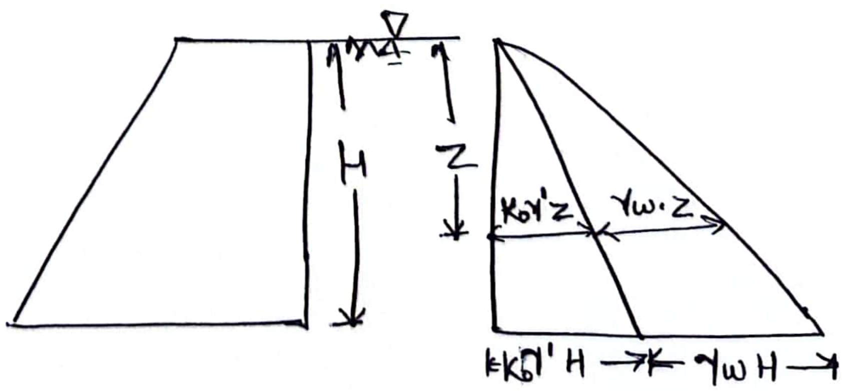
The pressure at the bottom $(z = H)$ of the wall is given by.

$$P_h = K_0 \gamma d + K_0 \gamma'(H-d) + \gamma_w(H-d)$$



If the water table is at the ground surface.

Pressure at bottom of wall, $(d=0)$ $P_h = K_0 \gamma' H + \gamma_w H$



CamScanner

* Rankine's Earth Pressure Theory

→ Rankine considered the equilibrium of a soil element within a soil mass bounded by a plane surface.

The following assumptions were made for the derivation of earth pressure.

(i) The soil mass is homogeneous and semi-infinite.

(ii) The soil is dry and cohesionless.

(iii) The ground surface is plane, which may be horizontal or inclined.

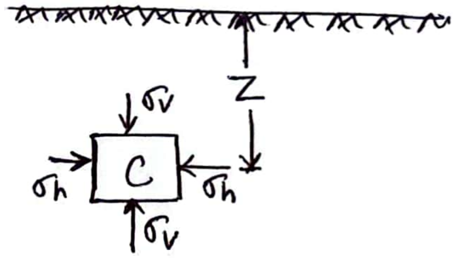
(iv) The back of the retaining wall is smooth and vertical.

(v) The soil element is in a state of plastic equilibrium ie. at the verge of failure.

Expressions for the active earth pressure and the passive earth pressure are developed as explained below;

(a) Active Earth Pressure

Let us consider an element of dry soil at a depth z below a level soil surface. Initially, the element is at-rest conditions, and the horizontal pressure is given by



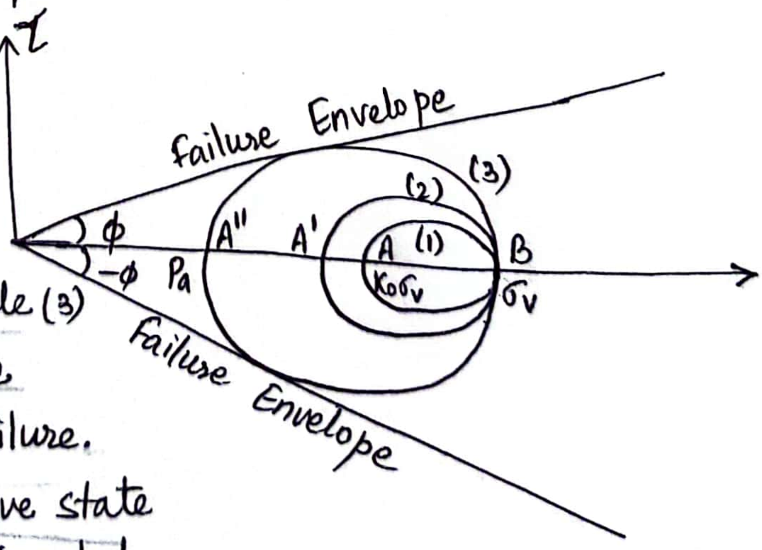
$$\sigma_h = K_0 \cdot \sigma_v$$

where σ_v is the vertical stress at c, and σ_h is the horizontal stress at c.

$$\sigma_v = \gamma z$$

The stresses σ_h and σ_v are, respectively, the minor and major principal stresses, and are indicated by points A and B in the Mohr circle.

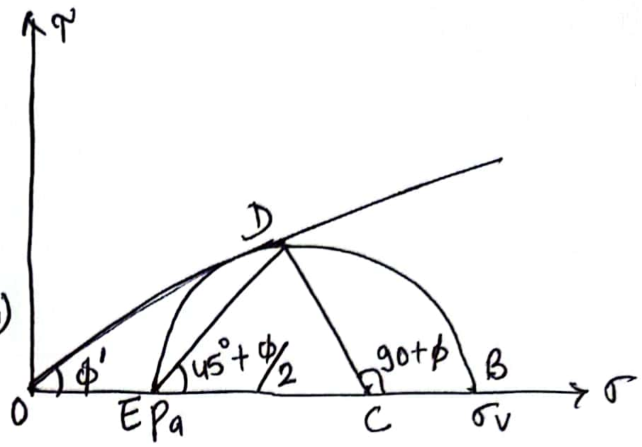
Let us now consider the case when the vertical stress remains constant while the horizontal stress is decreased. The point A shifts to position A'' when the Mohr Circle (3) touches the failure envelope. The soil is at the verge of shear failure. It has attained the Rankine active state of plastic equilibrium. The horizontal stress at that state is the active pressure (p_a)



CS CamScanner

fig shows the Mohr Circle when active conditions are developed. Point E represents the active condition.

From the figure,



$$P_a = OE = OC - CE$$

$$CE = CD = OC \cdot \sin\phi'$$

$$P_a = OC - OC \cdot \sin\phi' = OC(1 - \sin\phi') \quad \text{—(i)}$$

Also, $\sigma_v = OB = OC + CB$

$$= OC + OC \sin\phi'$$

$$\sigma_v = OC(1 + \sin\phi') \quad \text{—(ii)}$$

from eq^n (i) & (ii),

$$\frac{P_a}{\sigma_v} = \frac{1 - \sin\phi'}{1 + \sin\phi'}$$

or,

$$P_a = \left(\frac{1 - \sin\phi'}{1 + \sin\phi'}\right) \cdot \sigma_v$$

$$\therefore \quad P_a = K_a \cdot \gamma z$$

Where Ka is known as the coefficient of active earth pressure.

→ Ka is a function of the angle of shearing resistance (ϕ') and is given by.

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} = \tan^2\left(45 - \frac{\phi'}{2}\right).$$

CS CamScanner

B) Passive Earth Pressure

→ The passive Rankine state of plastic equilibrium can be explained by considering the element of soil at a point at depth z below the soil surface.

→ As the soil is compressed laterally, the horizontal stress (σ_h) is increased, whereas the vertical stress (σ_v) remains constant.

Point A indicates the horizontal stress & Point B indicates the vertical stress. With lateral compressing of the soil, the horizontal stress increases until it reaches a limiting value greater than the vertical stress, indicated by point A'' and the Mohr Circle touches the failure envelope.

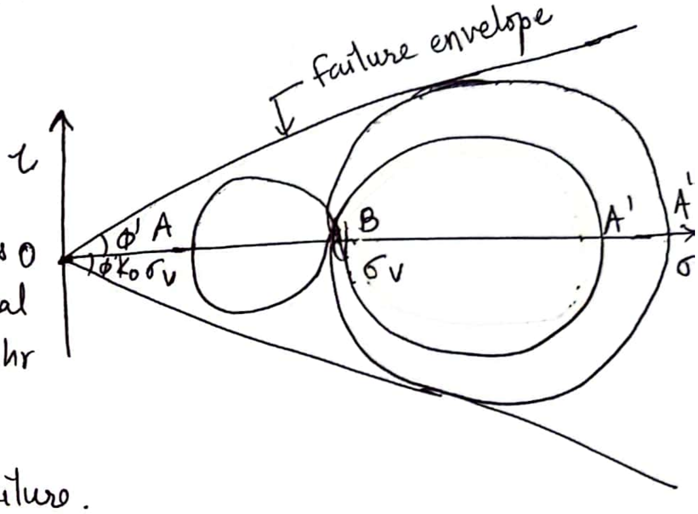
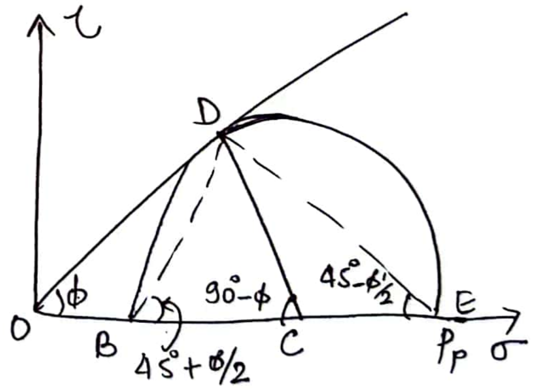


Figure below shows the Mohr circle at failure.

Here,

$$P_p = OC + CE = OC + CD$$
$$= OC + OC\sin\phi'$$
$$= OC(1+\sin\phi') \quad \text{—— (i)}$$



$$OB = OC - BC = OC - CD$$
$$= OC - OC\sin\phi'$$
$$\sigma_v = OC(1-\sin\phi') \quad \text{—— (ii)}$$

From eqⁿ (i) & (ii)

$$\frac{P_p}{\sigma_v} = \frac{1+\sin\phi'}{1-\sin\phi'}$$

$$P_p = \left(\frac{1+\sin\phi'}{1-\sin\phi'}\right)\sigma_v$$

$$P_p = K_p \gamma z$$

where K_p is the coefficient of passive earth pressure, given by

$$K_p = \frac{1+\sin\phi'}{1-\sin\phi'} = \tan^2\left(45^\circ + \phi'/2\right)$$

CS CamScanner

Rankine's Earth Pressure when the Surface is inclined

a) Active Earth Pressure

$$P_a = K_a \gamma z$$

where,

$$K_a = \cos i * \frac{\cos i - \sqrt{\cos^2 i - \cos^2 \phi'}}{\cos i + \sqrt{\cos^2 i - \cos^2 \phi'}}$$

i = angle of surcharge

b) Passive Earth Pressure

$$P_p = K_p \gamma z$$

where,

$$K_p = \cos i * \frac{\cos i + \sqrt{\cos^2 i - \cos^2 \phi'}}{\cos i - \sqrt{\cos^2 i - \cos^2 \phi'}}$$

Rankine's Earth Pressure In Cohesive Soils

→ failure envelope has a cohesion intercept 'c'.

* Active case

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

where,

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} = \tan^2(45° - \phi/2)$$

At $z = 0$,

$$P_a = -2c'\sqrt{K_a}$$

The -ve sign shows that the pressure is negative, ie. it tries to cause a pull on the wall. This tensile stress decreases with an increase in depth, and it becomes zero at a depth z_c, given by.

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

$$\Rightarrow \quad z_c = \frac{2c'}{\gamma\sqrt{K_a}}.$$

* Passive case.

$$\boxed{P_p = \gamma z K_p + 2c'\sqrt{K_p}}$$

#

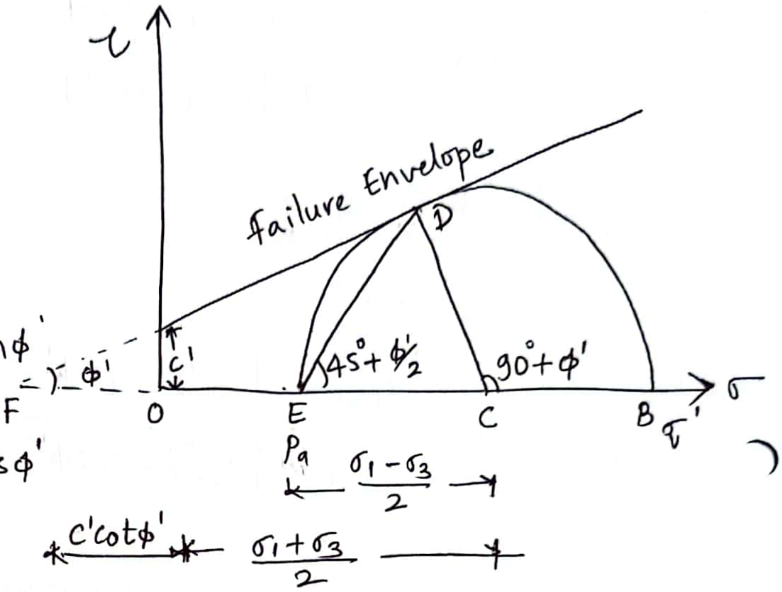
From ΔFCD,

$$\sin\phi' = \frac{CD}{FC} = \frac{CD}{FO+OC}$$

$$\sin\phi' = \frac{(\sigma_1-\sigma_3)/2}{c'\cot\phi' + (\sigma_1+\sigma_3)/2}$$

$$\frac{(\sigma_1-\sigma_3)}{2} = \frac{(\sigma_1+\sigma_3)}{2}\sin\phi' + c'\cot\phi'.\sin\phi'$$

$$\frac{\sigma_1}{2}(1-\sin\phi') = \frac{\sigma_3}{2}(1+\sin\phi') + c'\cos\phi'$$



or, $$\sigma_3 = \frac{1-\sin\phi'}{1+\sin\phi'}\sigma_1 - \frac{2c'\cos\phi'}{1+\sin\phi'} \quad \text{——— (i)}$$

$$= \sigma_1\tan^2(45-\phi'/2) - 2c'\tan(45-\phi'/2) \quad \text{——— (ii)}$$

As σ_3 is equal to active pressure (p_a) and σ_1 is equal to the vertical stress $\sigma_v (= \gamma z)$, eq^n (i) becomes,

$$P_a = \left(\frac{1-\sin\phi}{1+\sin\phi'}\right) * \gamma z - \frac{2c'\cos\phi'}{1+\sin\phi}$$

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

for $z=0$, $$P_a = -2c'\sqrt{K_a}$$ — (-ve pressure, becomes zero at a depth z_c)

Now, $$0 = K_a \gamma z_c - 2c'\sqrt{K_a}$$

$$z_c = \frac{2c'}{\gamma\sqrt{K_a}}$$ 4

CS CamScanner

* Coulomb's Earth Pressure Theory.

→ Coulomb developed a method for the determination of the earth pressure in which he considered the equilibrium of the sliding wedge which is formed when the movement of retaining wall takes place.

Assumptions

(i) The backfill is dry, cohesionless, homogeneous, isotropic and ideally plastic material.

(ii) The slip surface is a plane surface which passes through the heel of the wall.

(iii) The wall surface is rough.

(iv) The sliding edge itself acts as a rigid body.

In coulomb's theory, a plane surface is assumed and the lateral force required to maintain the equilibrium of the wedge is found using the principles of statics. The procedure is repeated for several trials surfaces. The trial surface which gives the largest force for the active case, and the smallest force for the passive case, is the actual failure surface.

→ The method readily accommodates the friction between the wall and the backfill, irregular backfill, sloping wall, surcharge loads, etc. Although the initial theory was for dry, cohesionless soils, it has now been extended to wet soils and cohesive soils as well.

* Coulomb's Active Pressure In Cohesionless Soils

$$P_a = \frac{W \sin(\alpha - \phi')}{\sin(180^\circ - \beta + \delta - \alpha + \phi')}$$

* Retaining Walls

→ Retaining walls are relatively rigid walls used for supporting the soil mass laterally so that the soil can be retained at different levels on the two sides.

Types

(1) Gravity Retaining Walls.

→ These walls depend upon their weight for stability.
→ The walls are usually constructed of plain concrete or masonry.
→ not economical for large heights. → tension is not developed anywhere in the secti.
"Middle third Rule" - basis of design.

(2) Semi gravity Retaining walls.

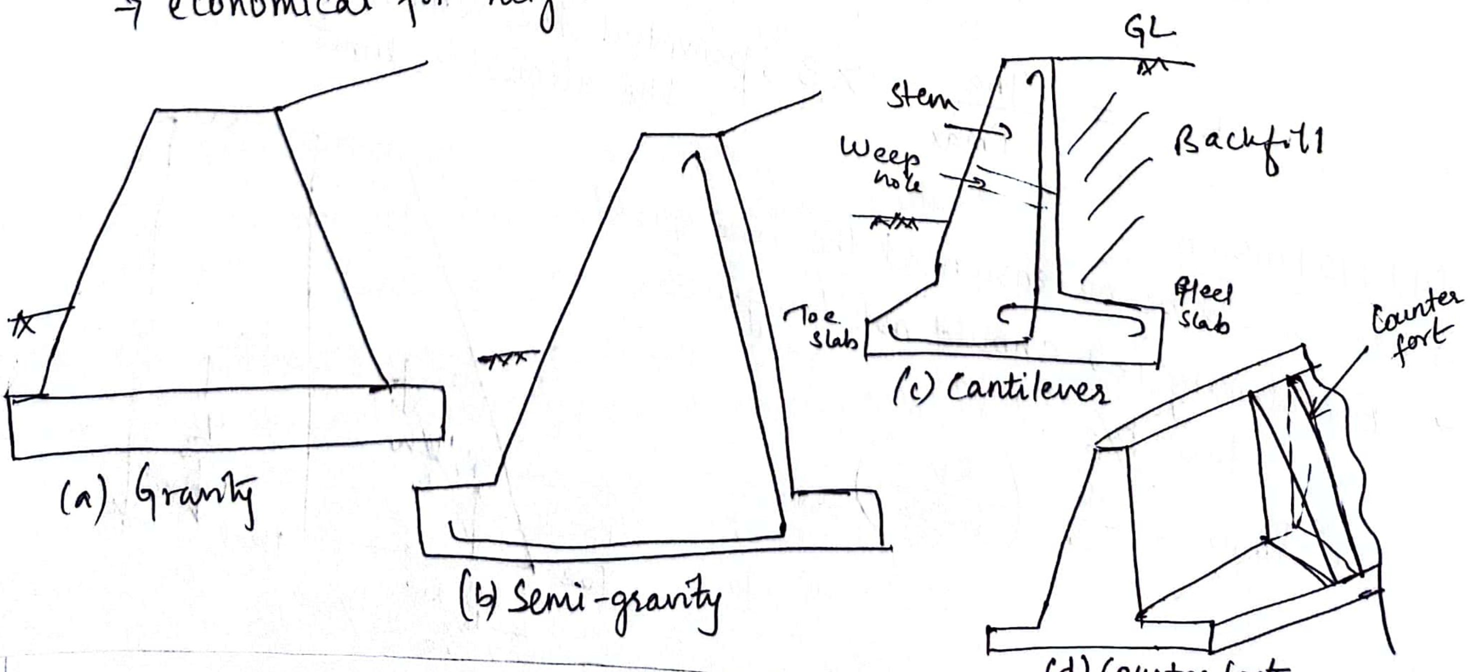
→ reinforcement is provided near the back face to reduce the size of the section of gravity wall.

(3) Cantilever Retaining Walls

→ made of reinforced cement concrete.
→ Consists of a thin stem and a base slab cast monolithically.
→ economical up to height ~~of~~ 6 ~~to 8~~ m. } wt. of earth on heel slab & wt. of retaining wall together provide stability of wall.

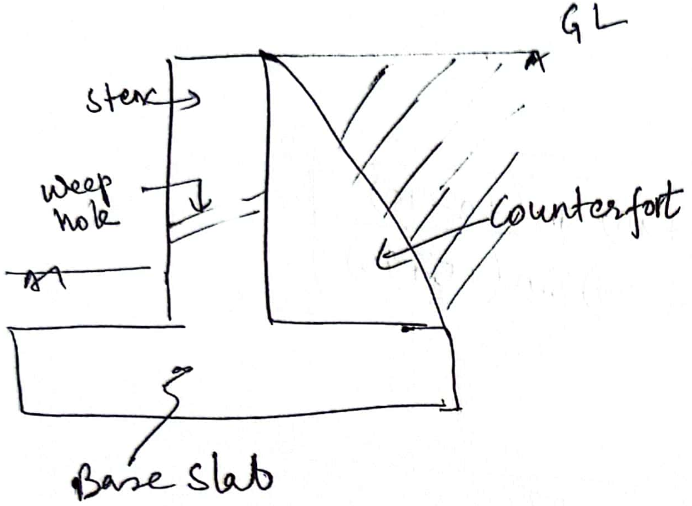
(4) Counterfort Retaining Walls.

→ thin vertical slabs, known as counterforts are spaced across the vertical stem at regular intervals to tie them with base slab.
→ reduces the SF & BM in the vertical stem & the base slab.
→ economical for height more than ~~6 to 8 m~~ 6m.



(a) Gravity (b) Semi-gravity (c) Cantilever (d) Counterfort

CamScanner



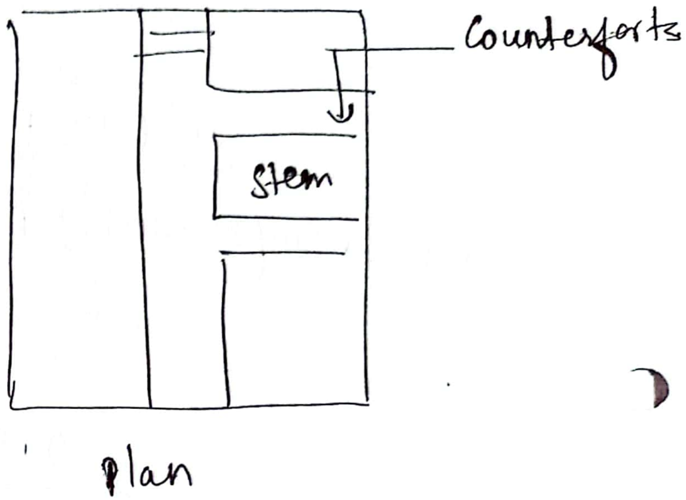


fig: Counterfort retaining wall.

CamScanner

Principles of the Design of Retaining Walls

→ Soil parameters (unit wt. of soil, angle of shearing resistance, cohesion, angle of wall friction) that influence the earth pressure and the bearing capacity of soil must be evaluated.

→ With the earth pressure known, the retaining wall as a whole is checked for stability.

For the safe design, the following requirements must be satisfied

(1) No Sliding. – Must be safe against sliding. ($\mu R_V > R_H$)

$$F_s = \frac{\mu R_V}{R_H} > 1.5$$

μ - Coeff. of friction betn base of wall & soil
R_H – Horizontal components of R
R_V – Vertical " " R.

(2) No Overturning.

↳ must be safe against overturning about toe.

$$F_o = \frac{\Sigma M_R}{\Sigma M_o} > (1.5 - 2.0)$$

M_R – Sum of resisting moment about toe
M_o – " " overturning " " "

(3) No bearing capacity failure.

→ Pressure caused by R_V at toe of wall must not exceed the allowable bearing capacity of soil. (Linear pressure distribution at the base)

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

q_{na} = allowable bearing pressure

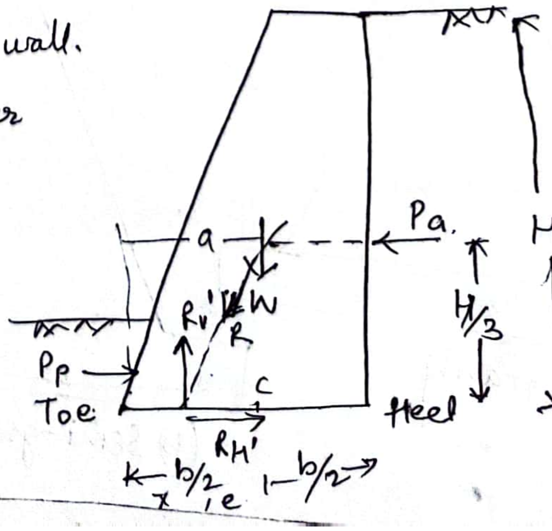
$$F_b = \frac{q_{na}}{P_{max}} > 3$$; provided the settlement is also within the allowable limit.

(4) No tension

→ There should be no tension at the base of wall.

→ Eccentricity (e) should not be greater than b/6.

$$P_{max} = \frac{4}{3}\left(\frac{R_V}{b - 2e}\right).$$



* Bearing Capacity and Settlements

Bearing Capacity of Soil

→ The load or pressure developed under the foundation without introducing any damaging movement in the foundation and in the supporting structure, is called bearing capacity of soil.

Types

(1) Ultimate Bearing Capacity (q_u)

→ gross pressure at the base of foundation at which the soil fails in shear.

(2) Net Ultimate Bearing Capacity (q_{nu})

→ It is the net increase in pressure at the base of foundation that causes shear failure of soil.

$$\boxed{q_{nu} = q_u - \gamma D_f}$$

γ - unit wt. of soil
D_f - Depth of foundation

(3) Net Safe Bearing Capacity (q_{ns})

→ It is the net soil pressure which can be applied to the soil considering only shear failure.

$$\boxed{q_{ns} = \frac{q_{nu}}{FOS}}$$

F - factor of safety
(usually taken as 3)

(4) Gross Safe Bearing Capacity (q_s)

→ It is the maximum gross pressure which the soil can carry safely without shear failure.

$$\boxed{q_s = q_{ns} + \gamma D_f = \frac{q_{nu}}{F} + \gamma D_f}$$

(5) Net Safe Settlement Pressure (q_{np})

→ It is the net pressure which soil can carry without exceeding the allowable settlement. (unit soil pressure or safe bearing pressure)

(6) Net Allowable Bearing Pressure (q_{na})

→ It is the net bearing pressure which can be used for the design of foundations.

$$q_{na} = q_{ns} \quad \text{if } q_{np} > q_{ns}$$

$$q_{na} = q_{np} \quad \text{if } q_{ns} > q_{np}$$

(safe both w.r.t. shear failure & settlement)

CamScanner

Factors influencing the bearing Capacity

(1) Type of Soil

→ Course grained soil has greater bearing capacity than fine grained soil

(2) Physical features of the foundations

→ Types, size, shape, depth, rigidity

(3) The amount of total and differential Settlement

(4) Physical properties of soil (grain size)

(5) Position of ground water level

(6) Fluctuations in the level of GWT

(7) Structural arrangement of soil

(8) Soil erosion and seepage

(9) Earthquake and dynamic motion

(10) Subsurface voids.

(11) Relative density in case of granular soil & consistency in terms of Cohesive soil

(12) Frost action

CamScanner

* Terzaghi's Bearing Capacity Theory.

→ Terzaghi gave a general theory for the bearing capacity of soils under a strip footing, making the following assumptions:

(1) The base of footing is rough.

(2) The foundation is laid at a shallow depth ie. $D_f \leq B$.

(3) The shear strength of the soil above the base of the footing is neglected. The soil above the base is replaced by a uniform surcharge γD_f.

(4) The load on the footing is vertical and is uniformly distributed.

(5) The footing is long ie. L/B ratio is infinite, where B is the width and L is the length of the footing.

(6) The shear strength of the soil is governed by the Mohr-Coulomb equation.

Derivation of Equation

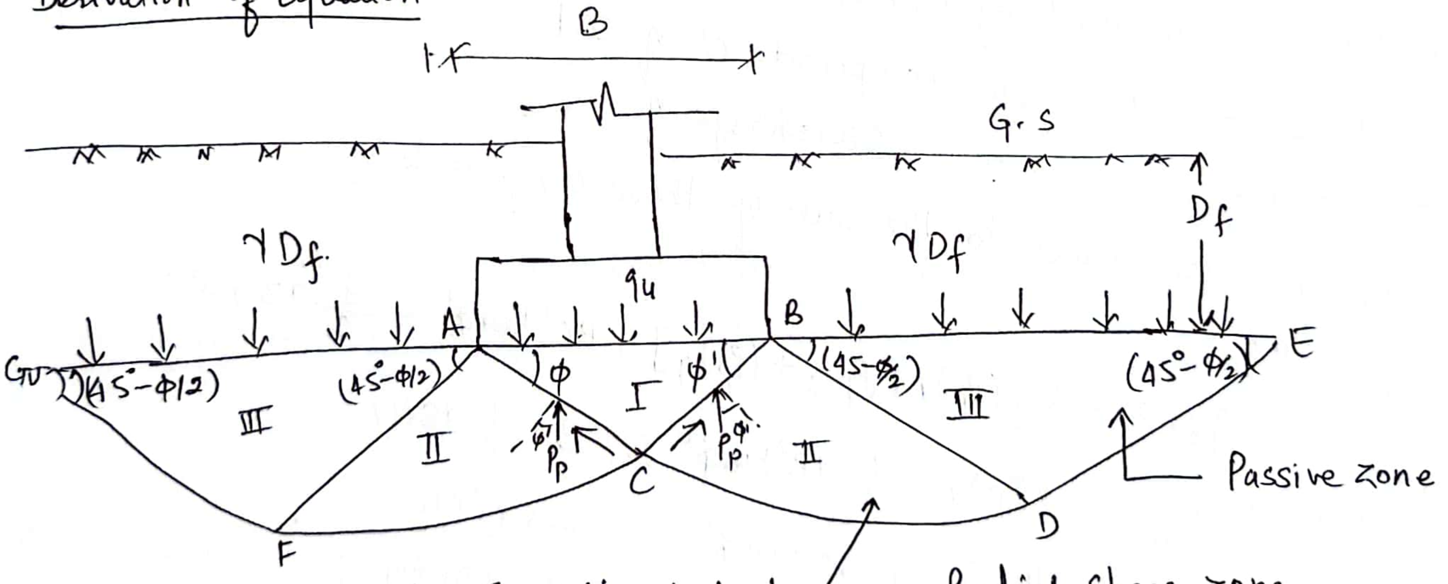


fig: Terzaghi's Analysis

→ Soil in wedge ABC (zone I) remains in a state of elastic equilibrium.

→ The sloping edges AC and BC of the soil wedge ABC bear against the radial shear zones CBD and CAF (Zone II).

→ Two triangular zones BDE and AFG are Rankine Passive zones (zone III). An overburden pressure $q = \gamma D_f$ acts as a surcharge on the Rankine passive zones.

→ The loading conditions are similar to that on a retaining wall under passive pressure case.

→ The failure occurs when the downward pressure exerted by loads on the soil adjoining the inclined surfaces CB & CA of the soil wedge is equal to the upward ~~cond~~ pressure.

Downward forces : load ($q_u * B$) and the weight of the wedge ($\frac{1}{4}\gamma B^2 \tan\phi'$)

Upward forces : vertical components of passive pressure (P_p) and the cohesion (c') acting along the inclined surfaces.

From equilibrium condition in vertical direction,

$$q_u * B + \frac{1}{4}\gamma B^2 \tan\phi' = 2P_p + 2c' * L_i \sin\phi'$$

where, L_i = Length of inclined surface CB $[= (B/2)/\cos\phi']$

$$\therefore \quad q_u * B = 2P_p + Bc'\tan\phi' - \frac{1}{4}\gamma B^2 \tan\phi' \quad \text{—(i)}$$

The resultant P_p on the surfaces CB & CA constitutes the following 3 components

(1) $(P_p)_\gamma$ - produced by wt. of shear zone BCDE

(2) $(P_p)_c$ - " " component c' of soil

(3) $(P_p)_q$ - " " surcharge q.

P_p is taken equal to the sum of these components.

∴ for eq^n (i),

$$q_u * B = 2[(P_p)_\gamma + (P_p)_c + (P_p)_q] + Bc'\tan\phi' - \frac{1}{4}\gamma B^2 \tan\phi'$$

Substituting, $2(P_p)_\gamma - \frac{1}{4}\gamma B^2 \tan\phi' = B * \frac{1}{2}\gamma B N_\gamma$

and, $2(P_p)_c + Bc'\tan\phi' = B * c' N_c$

and, $2(P_p)_q = B * \gamma D_f N_q$,

we get,

$$q_u B = B * c' N_c + B\gamma D_f N_q + B * \frac{1}{2}\gamma B N_\gamma$$

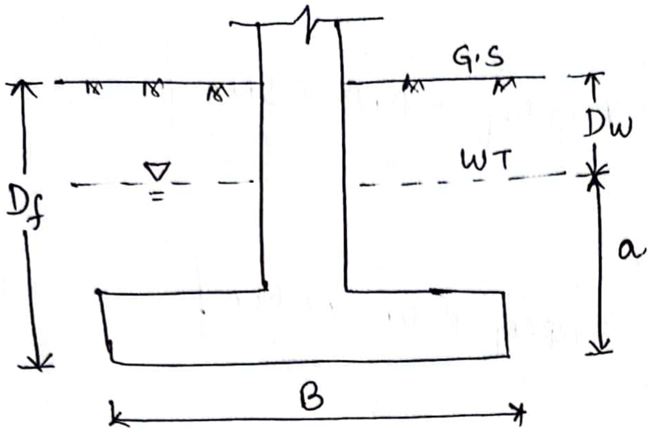
$$\boxed{q_u = c' N_c + \gamma D_f N_q + 0.5\gamma B N_\gamma}$$ – for general shear failure.

N_c, N_q, N_γ are bearing capacity factors and depends upon the angle of shearing resistance (ϕ') of the soil.

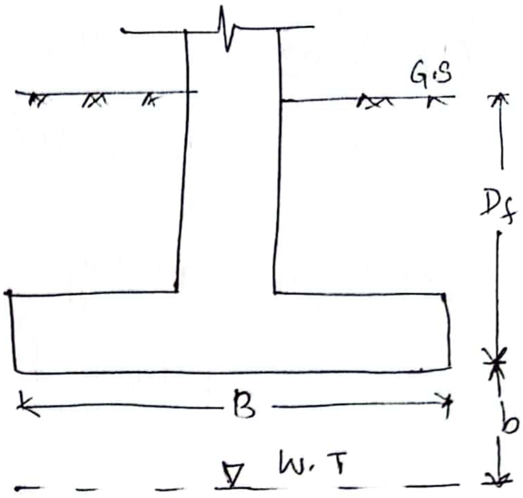
* For local shear failure $\boxed{c_m' = \frac{2}{3}c'}$

CamScanner

* Effect of Water Table on Bearing Capacity:



(a)



(b)

Case I: Water table located above the base of footing.

→ The effective surcharge is reduced as the effective weight below the water table is equal to the submerged unit (γ').

Therefore,

$$q = D_w * \gamma + a * \gamma' = D_w * \gamma + (D_f - D_w) * \gamma'$$
$$= \gamma' D_f + (\gamma - \gamma') D_w$$

Also,

$$q_u = c' N_c + [\gamma' D_f + (\gamma - \gamma') D_w] N_q + 0.5 \gamma' B N_\gamma$$

→ If $D_w = 0$ (ie. $a = D_f$)

$$q_u = c' N_c + \gamma' D_f N_q + 0.5 \gamma' B N_\gamma$$

If $D_w = D_f$ (ie. $a = 0$)

$$q_u = c' N_c + \gamma D_f N_q + 0.5 \gamma' B N_\gamma$$

Case II: Water table located at a depth below base. fig (b)

$$\bar{\gamma} = \gamma' + \frac{b}{B}(\gamma - \gamma')$$

$$q_u = c' N_c + \gamma D_f N_q + 0.5 B \left[\gamma' + \frac{b}{B}(\gamma - \gamma')\right] N_\gamma$$

When $b = 0$ ie WT at base, $q_u = c' N_c + \gamma D_f N_q + 0.5 B \gamma' N_\gamma$

When $b = B$, ie. WT at depth B below base,

$$q_u = c' N_c + \gamma D_f N_q + 0.5 B \gamma N_\gamma$$

CamScanner

Bearing capacity for various footings.

(1) Square Footing.

$$\boxed{q_u = 1.2c'N_c + \gamma D_f N_q + 0.4\gamma B N_\gamma}$$

(2) Circular footing.

$$\boxed{q_u = 1.2c'N_c + \gamma D_f N_q + 0.3\gamma B N_\gamma}$$

(3)

CS CamScanner

* Modes of foundation failure.

→ When a footing is placed on a soil, the soil deforms and foundation settles as shown in figure below. Before failure takes place, soil below foundation undergoes into various stages. The stages of failure of foundation are:

(i) Distortion of Soil

→ lateral bulging of the column of soil directly below the foundation takes place and the soil gets distorted elastically ie. stress is proportional to strain.

(ii) Local cracking

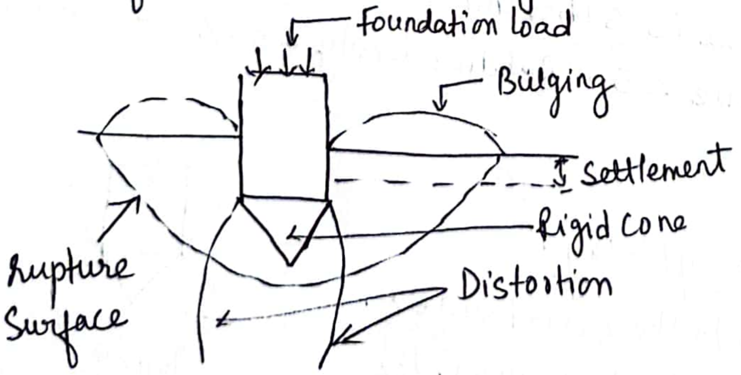
→ Due to gradual increase in foundation load, local shear failure around the perimeter of the foundation takes place in the soil. Load settlement curve does not obey Hooke's law and transitions occur in soil from elastic stage to plastic stage.

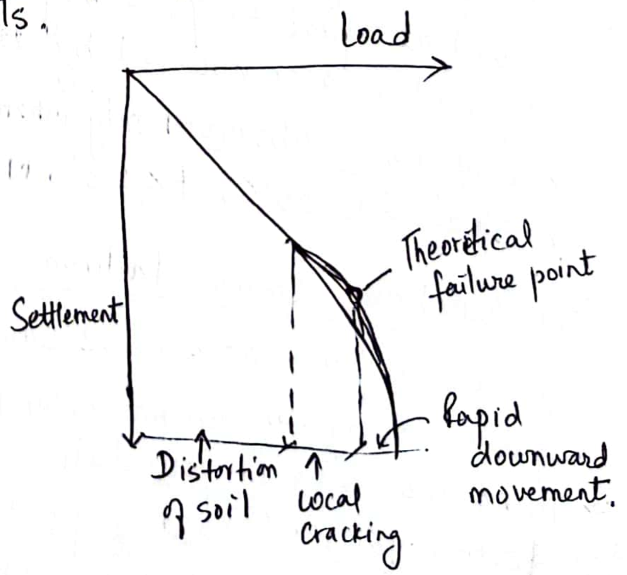
(iii) Formation of rigid cone

→ rigid cone is formed directly below the foundation. With further increase in load, the cone forces the soil to move downward & outward due to which shear of soil is initiated.

(iv) Development of rupture surface

→ In this stage, the shearing of soil below the foundation is continued and well defined shear zone is developed and finally a curved rupture surface is formed along which soil fails.



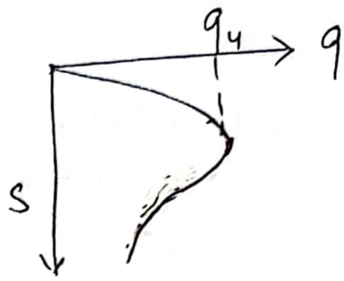


Types of Shear Failure.

(1) General Shear failure

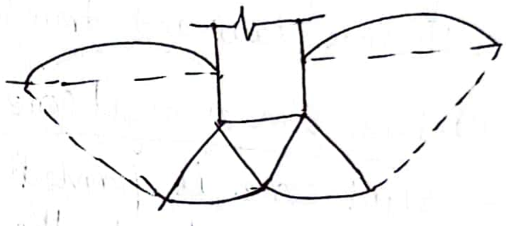
- failure occurs in dense sand or a stiff clay.
- failure takes place at a very small strain.
- At a certain load intensity equal to q_u, the settlement increases suddenly. A shear failure occurs in the soil at that load and the failure surfaces extend to the ground surface.
- A heave on the sides is observed.
- Strain $< 5\%$, $\phi > 36°$, N-Value > 30
  Relative density $> 70\%$, $e < 0.55$

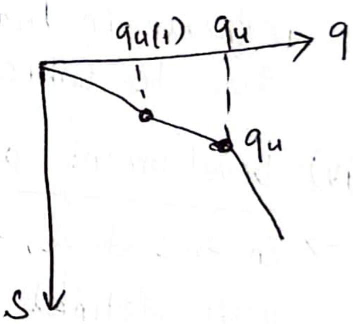
(a)



(2) Local Shear failure

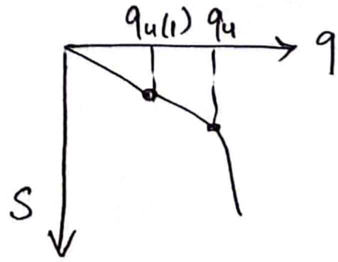
- failure occurs in medium dense sand or on a clay of medium consistency.
- When load is equal to $q_{u(1)}$, the foundation movement is accompanied by sudden jerks. The failure surfaces gradually extend outwards from the foundation. However, a considerable movement of foundation is required for the failure surfaces to extend to ground surface. load at equal to q_u & beyond this point, increase of load is accompanied by large increase in settlement.
- A heave is observed only when there is substantial vertical settlement.
- strain $= 10$–20%, $\phi < 28°$, N-Value < 5, Relative density $< 35\%$, $e > 0.75$





(3) Punching Shear failure

- failure occurs on a loose sand or soft clay.
- failure surfaces do not extend up to the ground surface.
- There are jerks in foundation at a load of $q_{u(1)}$. The footing fails at load of q_u at which stage the load-settlement curve becomes steep & practically linear.
- No heave is observed. There is only vertical movement of footings.



* Types of foundation and their suitability in Context of Nepal

→ A foundation is that part of a structure which transmits the weight of the structure to the ground.
All structures constructed on land are supported on foundations,

→ The foundation should be designed such that (1) the soil below does not fail in shear and (2) settlement is within the safe limits.

Foundations may be classified into two categories. (depending upon type of structure, distribution of loads, type & capacity of subsoil, presence & level of water table).

(1) Shallow foundations (2) Deep Foundations.

Types of Shallow foundations.

A shallow foundation, is one whose width is greater than its depth (ie. $D/B \leq 1$)
These are located just below the lowest part of the wall or a column which they support.
Footings are structural members, made of brick work, masonry or concrete, that are used to transmit the load of the wall or column such that the load is distributed over a large area.
The footings are of following types.

(1) Strip footing / continuous footing. ($l >>> w$)

→ It is provided for a load-bearing wall.
→ It is also provided for a row of columns which are so closely spaced that their spread footings overlap or nearly touch each other.
→ used where the building loads are carried by entire walls in load bearing masonry buildings.

(2) Spread or Isolated footing (l/w = bet^n 1 & 2)

→ It is provided to support an individual column.
→ most simple and common type of footings used for RCC column.
→ It is circular, square or rectangular slab of uniform thickness.
→ employed when columns are not closely spaced, masses on footings are less, safe bearing capacity of soil is usually high.
→ Sometimes, it is stepped to spread the load over a large area.

CamScanner

(3) Strap or Cantilever footing.

- → It consists of two isolated footings connected with a structural strap.
- → The strap connects the two footings such that they behave as one unit. It is designed as a rigid beam.
- → The individual footings are so designed that their combined line of action passes through the resultant of the total load.
- → A strap footing is more economical than a combined footing when the allowable soil pressure is relatively high and the distance between the columns is large.

(4) Combined footing.

- → It supports two columns.
- → It is used when the columns are so close to each other that their individual footings would overlap.
- → It is also provided when the property line is so close to one column that a spread footing would be ~~even~~ eccentrically loaded when kept entirely within the property line.
- → may be rectangular or trapezoidal in plan. (B.C of soil is low)

(5) Mat or raft foundations.

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

Types of Deep Foundations

→ When the soil at or near the ground surface is not capable of supporting a structure, deep foundations are required to transfer the loads to deeper strata.

→ These are used when surface soil is unsuitable for shallow foundation, and a firm stratum is so deep that it cannot be reached economically by shallow foundations.

→ provided when load of the super structure is heavy, poor bearing capacity of soil, sub soil water level is high, fluctuation in subsoil water level & the structure is situated at seashore or river bed.

Deep foundations are of following types:

(1) Pile foundation.

→ common type of deep foundation

→ A pile is a slender structural member made of steel, concrete or wood. A number of piles is either driven into the soil or formed in-situ by excavating a hole and filling it with concrete.

→ provided when loads are heavy, soil has poor bearing capacity.

(2) Pier foundation.

→ A pier is a vertical column of relatively larger cross-section than a pile.

→ A pier is installed in a dry area by excavating a cylindrical hole of large diameter to the desired depth and then backfilling it with concrete.

(3) Caisson foundation

→ It is a hollow, water tight box or chamber, which is sunk through the ground for laying foundation under water.

(4) Well foundation

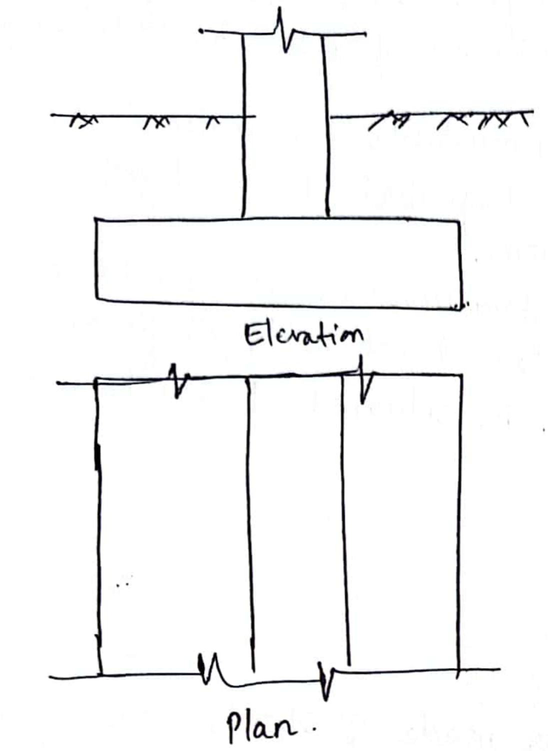
→ can be constructed on the dry bed or after making a sand island.

→ At locations where the depth of water is greater than 5m to 6m. and the velocity of water is high, wells can be fabricated on the river bank and then floated to the final position and grounded.

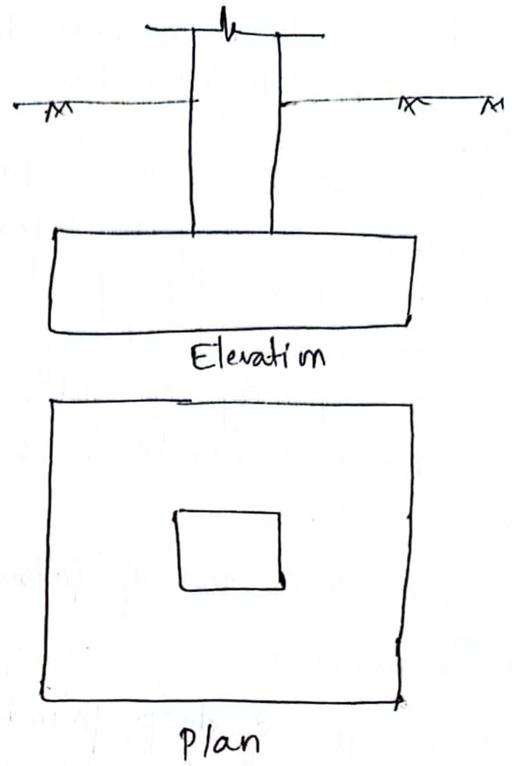
→ provided for bridges.

CamScanner

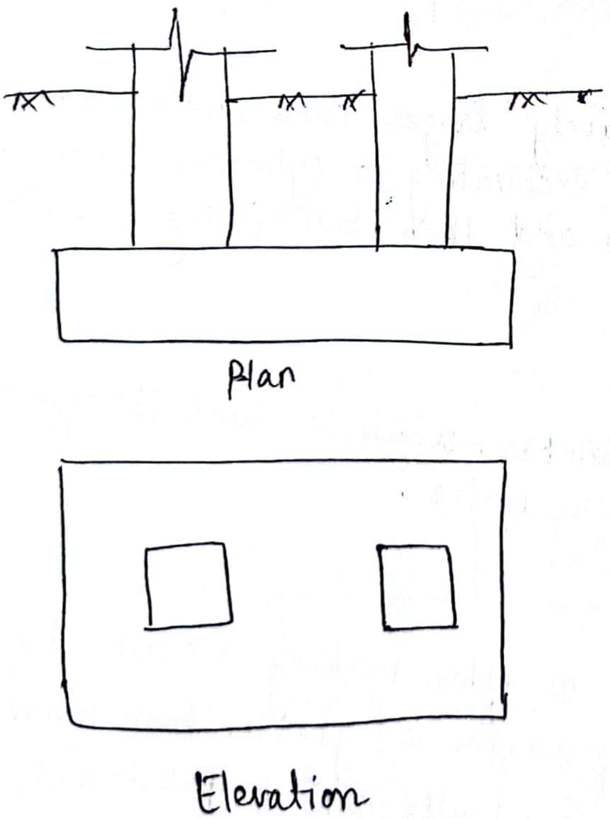
figures of different types of foundations



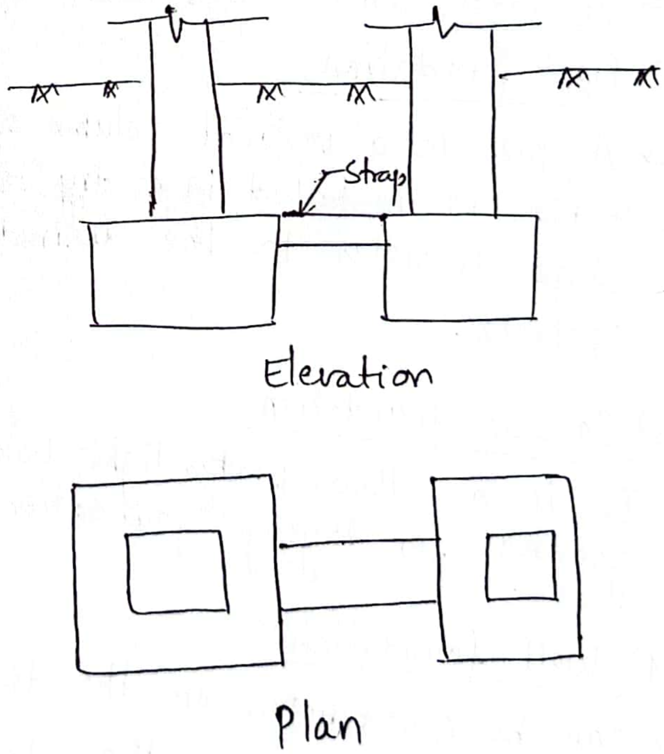
(a) Strip footing.



(b) Spread Footing.



(c) Combined footing



(d) Strap Footing.

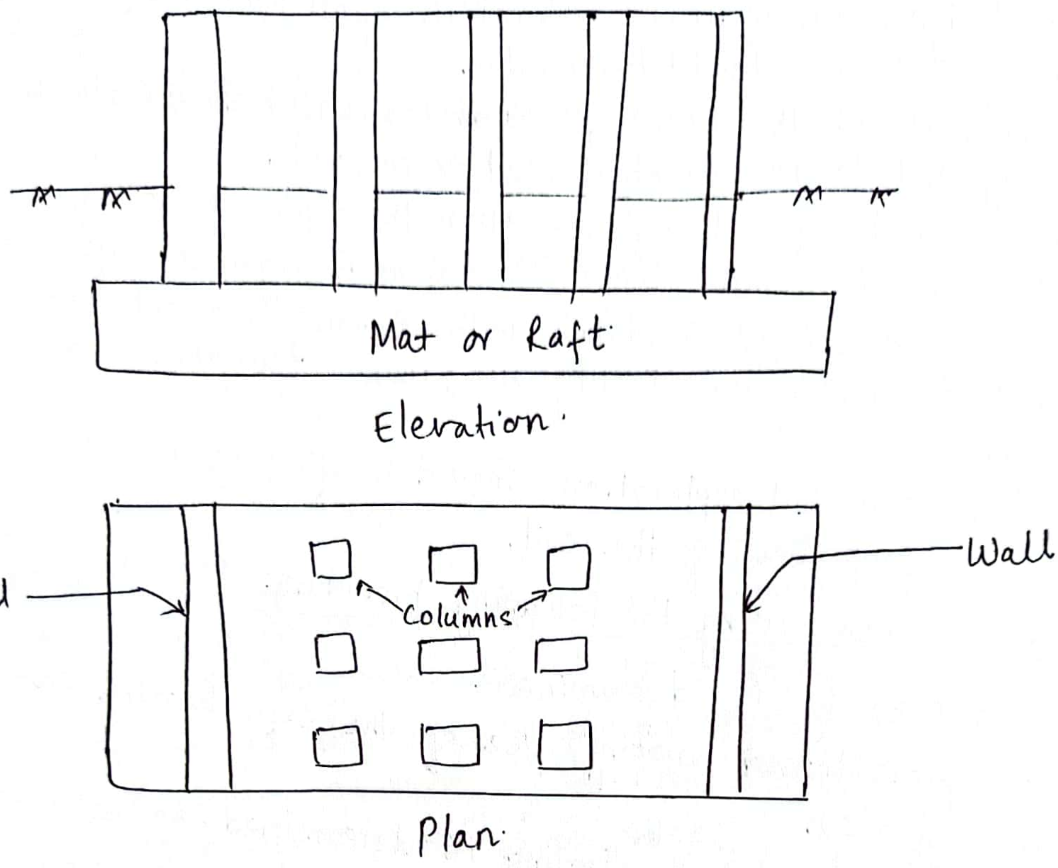


fig: Mat foundation

2

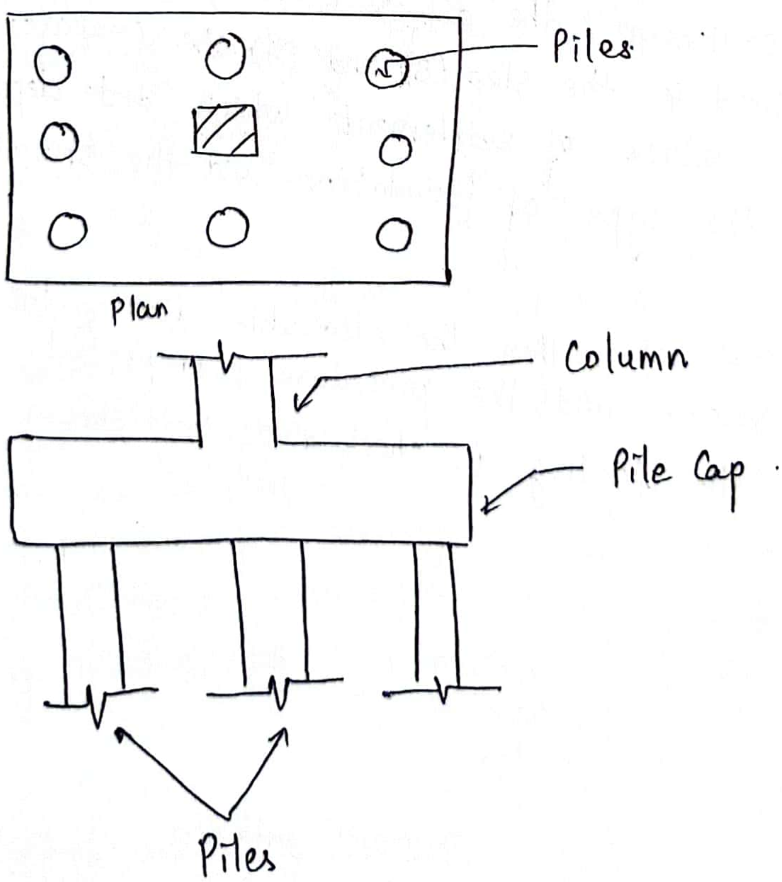


fig: Pile foundation

Principles of Design of Footings.

- Before actual design, an accurate estimation of all loads (DL + LL + others) acting on the foundation should be made.
- The bending moment at the base of column (or wall) should also be determined if it is subjected to an eccentric load or moment.
- As the bearing capacity of soil depends upon the depth of footing and its length, and width, an estimate about these dimensions is required before the actual design.
- For members carrying axial load combined with bending moment that do not change direction, a rectangular footing is more suitable than a square footing.
- Site Investigation and soil exploration should be first carried out to determine the engineering properties of the soil.

<u>The footing is designed using the following procedure:</u>

(1) The safe bearing capacity is determined.

(2) The footing is proportioned making use of the safe bearing capacity determined in step (1).

(3) The maximum settlement of the footing is determined. An estimate of the differential settlement between various footings is made.

(4) Angular distortion is determined between various parts of the structure.

(5) The maximum settlement, the differential settlement and the angular distortion obtained in the step (3) and (4) are compared with the given allowable values of settlement which ~~deto~~ depends upon the type of soil, the type of foundation and the structural framing system.

(6) If the values are not within the allowable limits, the safe bearing capacity is revised and the procedure is repeated.

(7) The stability of the footing is checked against sliding and overturning.

CS CamScanner

* Design of Spread foundation / Isolated

→ Spread footings are used for distributing concentrated column loads over a large area so that the bearing pressure is less than or equal to allowable soil pressure.

The area of the footing is given by

$$A = Q/q_{na} \qquad Q = \text{Column load}$$
$$q_{na} = \text{allowable soil pressure}$$

If the area actually provided is more, the actual pressure is given by

$$q_0 = \frac{Q}{\text{Actual area}}$$

(a) Plain Concrete Footings.

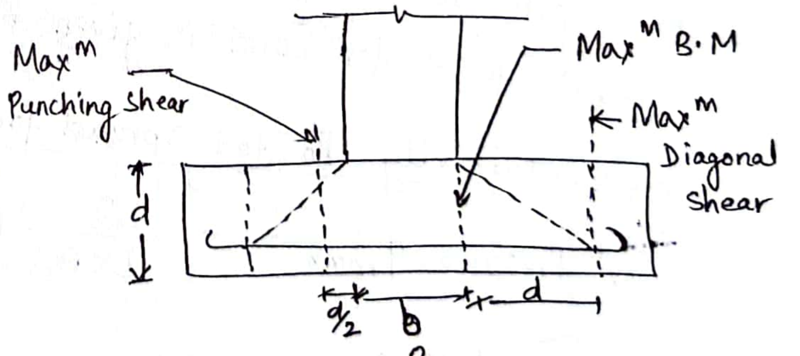
→ The footing is designed so that the contact pressure on the soil does not exceed the allowable bearing pressure.

→ The thickness at the edge of footing should be atleast 15 cm. On cohesive soils, generally minimum thickness of 30cm is specified in order to resist swelling pressure.

→ The thickness of the footing is fixed from the consideration of preventing tension on the underside.

→ The thickness is kept equal to twice the projection or alternatively, the width of the footing is determined by the normal practice of 45° distribution of loading.

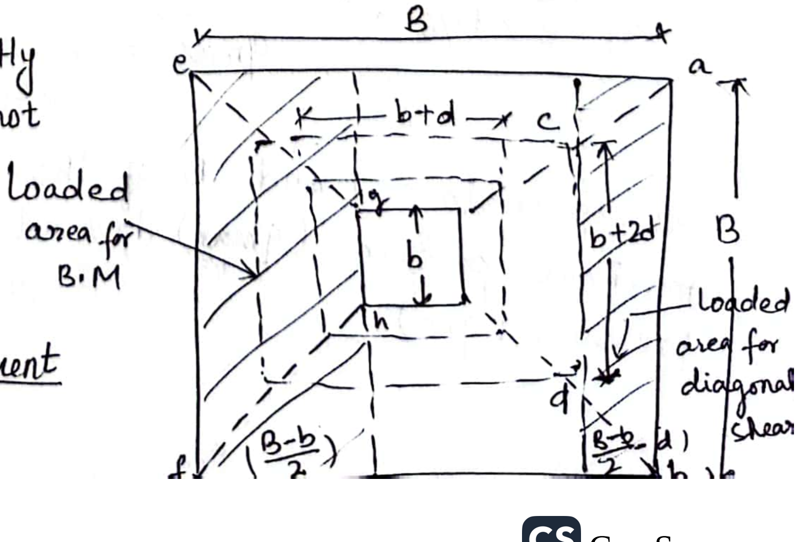
(b) Reinforced Concrete Footings.



→ Area of footing, $A = Q/q_{na}$

→ Shape of the footing may be square, Circular or rectangular.

→ If the column is centrally placed, the upward pressure is uniform. As the weight of the footing is directly transferred to the soil below, it does not affect the bending moment and Shear force.



The critical section for bending moment is taken as under:

CS CamScanner

(i) At the face of the column or pedestal ~~monothic~~ monolithic with the footing when no metal plate is used.
(ii) Halfway between the face of the column or pedestal and the edge of the metal plate on which the column or pedestal rests.

The maximum B.M for the case (i) is given by.

$$M = \frac{q_0 B (B-b)^2}{8}$$

For checking the diagonal shear F, the critical section is taken at a distance equal to the effective depth (d) of the footing from the face of the column.

$$F = q_0 B \left[\frac{(B-b)}{2} - d \right]$$

For punching shear, the critical section is taken at a distance of d/2 from the face of the column.

Generally the overall depth (do) of the footing is determined from the punching shear considerations.

$$d_0 = \frac{q_0 \left[B^2 - (b+d)^2 \right]}{4(b+d)\,\sigma_{sp}}$$

where, q_0 = actual pressure, B = width of footing, b = width of column, σ_{sp} = safe punching shear.

The depth provided is checked for Bending moment, shear & bond.

Max^m force for bond is given by, $F_b = q_0 B \frac{(B-b)}{2}$

For eccentrically loaded spread footings.

Max^m Pressure, $q_{max} = \frac{Q}{L \times B}\left(1 + \frac{6e}{B}\right)$; e is the eccentricity.

If $e > B/6$,

$$q_{max} = \frac{4Q}{3L(B-2e)}$$

The dimensions L and B of the footing are choosen such that the maximum pressure (q_{max}) does not exceed the allowable bearing pressure (q_{na}).

CamScanner

* Design of Strap Footings.

A strap footing is required in the following two cases:

(1) When $x' < L/3$, where x' is the distance of the resultant of column loads from the exterior face of the exterior column, and L is the length of the footing.

(2) When the distance between the two columns is so large that the combined footing becomes excessively long and narrow.

Assumptions.

(a) The soil pressure is uniform beneath each individual footing.

(b) The strap is perfectly rigid.

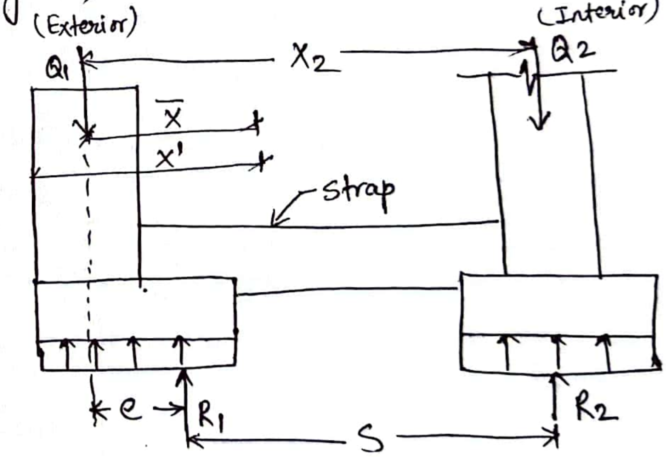
(c) The strap is weightless.

(d) The interior footing is centrally loaded.

The strap footing consists of two spread footings joined by a rigid beam known as strap. The strap is not subjected to any direct soil pressure from below. Its main function is to transfer the moment from exterior footing to the interior footing.

Design Procedure.

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



(2) Determine the length of the footing of the exterior column.

$$L_1 = 2(e + 0.5 b_1)$$

where, b_1 = width of the exterior column.

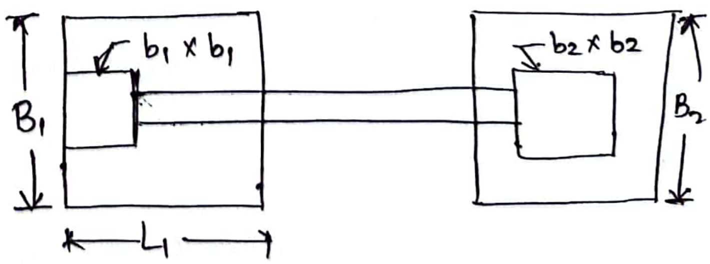
(3) Compute the reaction R_1, by taking moments about the line of action of R_2.

$$R_1 = Q_1 * \frac{X_2}{S}$$

where,

X_2 = distance between loads Q_1 & Q_2

S = distance between reactions R_1 & R_2



(4) Compute areas A_1 and A_2.

$$A_1 = R_1 / q_{na}$$

$$A_2 = R_2 / q_{na}$$

and reaction R_2 is equal to $(Q_1 + Q_2) - R_1$.

CamScanner

(5) find the widths of footings.

$$B_1 = A_1/L_1$$

$$B_2 = \sqrt{A_2}$$

(6) Design the individual footings.

(7) Determine the depth of the strap for diagonal shear and bending moment.

CS CamScanner

* Design of Combined Footings.

(A) Rectangular Combined Footings — provided if sufficient space is available beyond each column.

→ The design consists of selecting length and width of the footing such that the centroid of the footing and the resultant of the column loads coincide.

→ Footing is designed as a continuous beam supported by two columns in the longitudinal direction.

The procedure consists of the following steps.

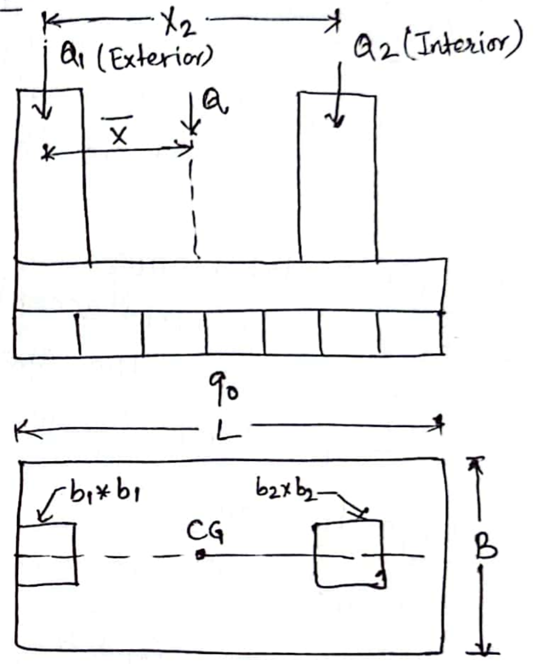


fig: Rectangular Combined Footing

(1) Determine the total column loads.

$$Q = Q_1 + Q_2$$

where, Q_1 & Q_2 are the exterior and interior columns, respectively.

(2) Find the base area of the footings.

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

Where, q_{na} is the allowable soil pressure.

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

$$\bar{x} = \frac{Q_2 * x_2}{Q}$$

where, x_2 is the distance between columns loads.

(4) Determine the total length of the footings.

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

where, b_1 = width of the exterior column.

(5) Find the width of the footing.

$$B = A/L$$

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

$$q_0 = Q/A_0$$

where, A_0 is the actual area.

CamScanner

(7) Draw the shear force and the bending moment diagrams along the length of the footing, considering the pressure q_0.

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

(9) find the thickness of footing for the maximum BM.
Check for diagonal shear and punching shear.
Check for bond at the point of contraflexure.

(10) Determine the longitudinal reinforcement for the max^m BM.
For transverse reinforcement, assume a width of $(b+d)$ to take all the bending moment in the short direction, where b is the column side and d is the effective depth.

(B) Trapezoidal Footing.

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

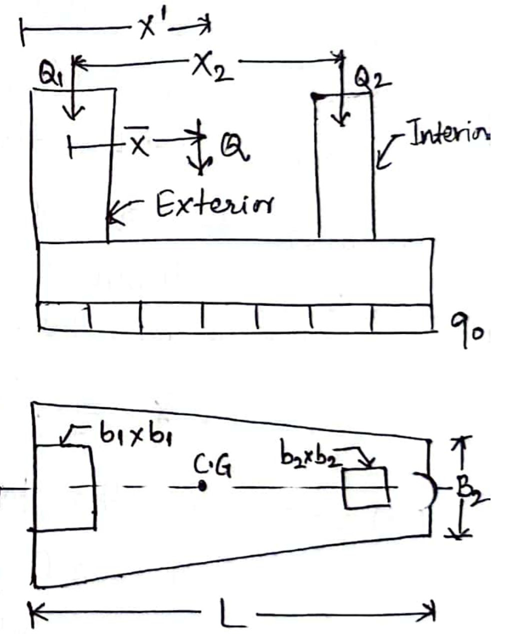


fig: Trapezoidal Combined footing

Design Procedure.

(1) Determine the total column loads.

$$Q = Q_1 + Q_2$$

(2) find the base area of the footing.

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

(3) Locate the line of action of resultant of the column loads.

$$\bar{x} = X_2 * \frac{Q_2}{Q}$$

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

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

A trapezoial footing is required if $1/3 < x' < 1/2$; $L = 2(\bar{x} + b_1/2)$
If $x' = 1/2$; rectangular footing is provided. If $x' < 1/3$, Combined footing cannot be provided. A strap footing is suitable.

(5) Determine the widths B_1 & B_2.

$$B_1 = \frac{2A}{L} - B_2 \qquad A = \left(\frac{B_1 + B_2}{2}\right) * L$$

$$B_2 = \frac{2A}{L}\left(\frac{3x'}{L} - 1\right) \qquad x' = \frac{L}{3}\left(\frac{B_1 + 2B_2}{B_1 + B_2}\right)$$

(6) Once the dimensions B_1 & B_2 have been found, the rest of the design can be done as in the case of rectangular combined footings.

CS CamScanner

Mat foundation.

- A mat (or raft) is a thick reinforced concrete slab which supports all the load-bearing walls and column loads of a structure or a large portion of structure.
- A mat is required when the loads are heavy and the soil is very weak or highly compressible.
- A mat is more economical than individual footings when the total base area required for the individual footings exceeds about one-half of the area covered by the structure.
- It should be safe against shear failure and the settlements should be within the allowable limits.

Types of Mat Foundations.

(1) Flat Plate Type

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

(2) Flat Plate Thickened Under Columns

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

(3) Beam and Slab Construction

- In this type, the beams run in two perpendicular directions and a slab is provided between the beams.
- The columns are located at the intersection of beams.
- This type is suitable when the bending stresses are high because of large column spacing and unequal column loads.

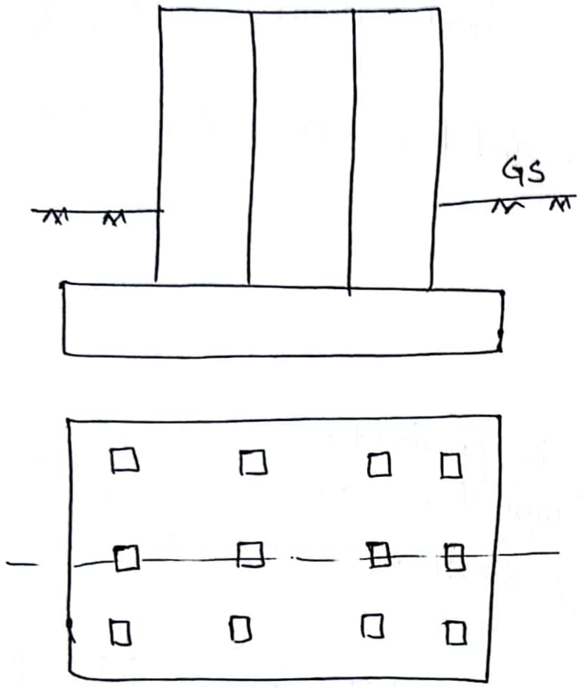
(4) Box Structures

- In this type, a box structure is provided in which the basement walls acts as stiffeners for the mat.
- This type of mat foundation can resist very high bending stresses.

CS CamScanner

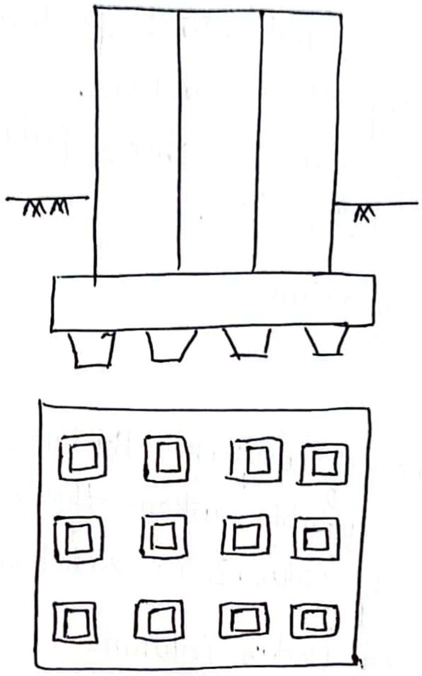
(5) Mats placed on Piles :

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



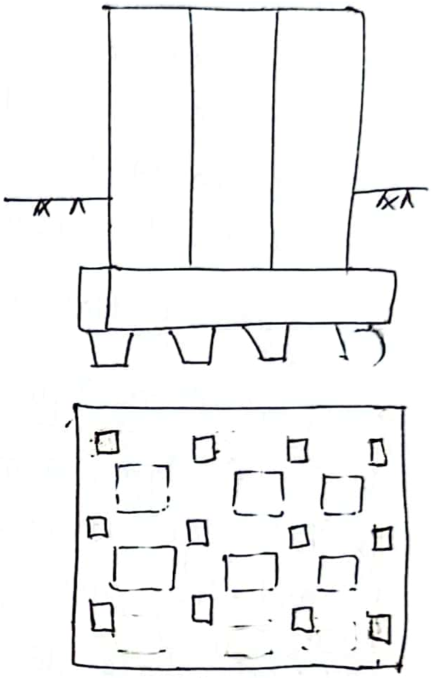
Plan

fig : Flat Plate Type Mat



Plan

fig : Flat Plate thickened under Column



Plan

fig : Beam & Slab Construction

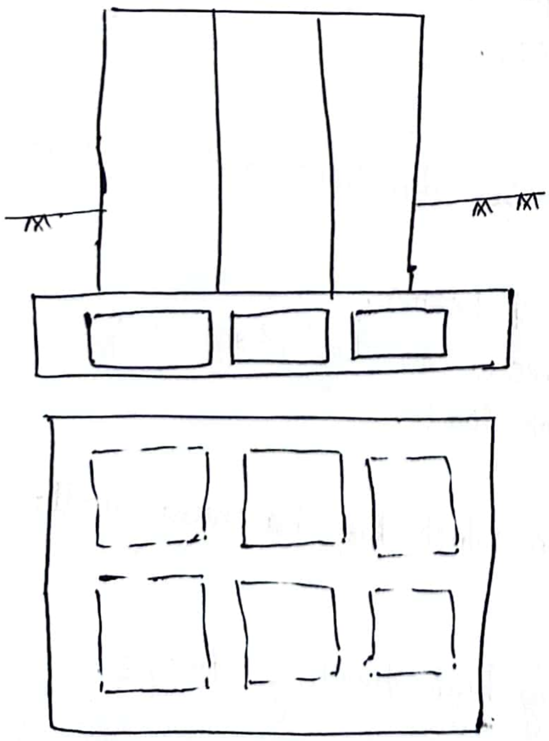


fig : Box Structure

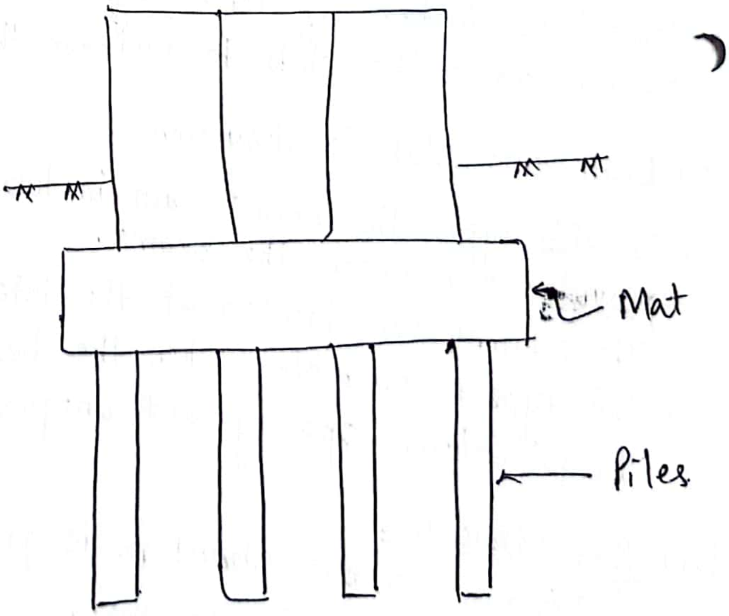


fig : Mats placed on piles

* Floating Mat

→ It is a type of foundation constructed by excavating the soil in such a way that the weight of the structure built on the soil is nearly equal to the total weight of the soil excavated from the ground including the weight of water in the soil before the construction of structure.

→ It is also called balancing raft and causes zero settlement to the structure

→ The main principle of floating foundation is to balance the weight of removed soil by a structure of same weight which causes zero settlement to the structure.

- **Suitability**

→ These are desirable for the following types of soils :

(1) Soils which are having good shear strength but having a problem of large settlements and differential settlements under heavy loads. In this, floating foundation can reduce the settlement values to greater extent.

(2) Soils which are having low shear strength and there is no hard layer of soil at reasonable depth.

- Problems ~~aris~~ arising during construction of floating foundation :

(i) Excavation

(ii) Dewatering

(iii) Critical depth

(iv) Bottom heave.

CamScanner

* Compensating Mat

→ Net pressure increase in the soil under a mat foundation can be ~~increased by~~ reduced by increasing the depth of mat. This approach is generally referred to as compensated foundation design and is extremely useful when structures are to be built on very soft clays.

→ In clays, Factor of Safety against bearing capacity failure can be increased and settlements can be reduced by increasing the depth of embedment.

Net bearing pressure for a mat foundation is given by.

$$q_{net} = Q/A - \gamma D_f$$

Increasing D_f such that $q_{net} = 0$ results in fully compensated foundation, where D_f is given by,

$$\gamma D_f = Q/A$$

or,

$$D_f = \frac{Q}{\gamma A}$$

For compensating foundations, the settlement is (theoretically) zero, and factor of safety against bearing capacity failure becomes theoretically infinite.

CamScanner

Construction Approach of Mat foundations.

(1) In mat foundation construction, the whole area is dug out to the specified depth and 30 cm more wide than the area to be covered.

(2) The bed is compacted and sprinkled over with water.

(3) Water proof plastic sheet is laid over earth.

(4) ~~Ao~~ After that, pour a layer of PCC to create a perfectly flat and level base for foundation.

(5) Reinforcement is laid on spaces over foundation bed. Reinforcement are provided in both directions in the form of steel mesh. Two meshes are reinforced at top and bottom of foundation.

(6) Then, Concrete is poured to desired thickness, usually in the range of 200 mm to 300 mm thick for lower loads and much thicker if heavy loads are to be carried. A minimum rebar cover of 50 mm should be maintained.

(7) Finally, a suitable curing regime should be used, to make sure that concrete achieves the designated compressive strength.

Design Methods for Mat foundations

(1) Rigid Beam Method.

→ Slab is considered to be infinitely rigid as compared with subsoil.

(2) Simplified Elastic Method.

→ Soil behaves like an infinite number of individual independent elastic springs.

→ The springs are assumed to take compression as well as tension.

(3) Elastic Method

→ Soil is considered as homogeneous, linearly elastic half space.

(4) Non-linear Elastic Method.

→ Soil is considered to be a non-linearly elastic solid.

CS CamScanner

* Design of Mat foundations

Procedure

(1) Determine the line of action of all the loads acting on the mat. The self weight of the mat is not considered, as it is taken directly by soil.

(2) Determine the contact pressure distribution as under

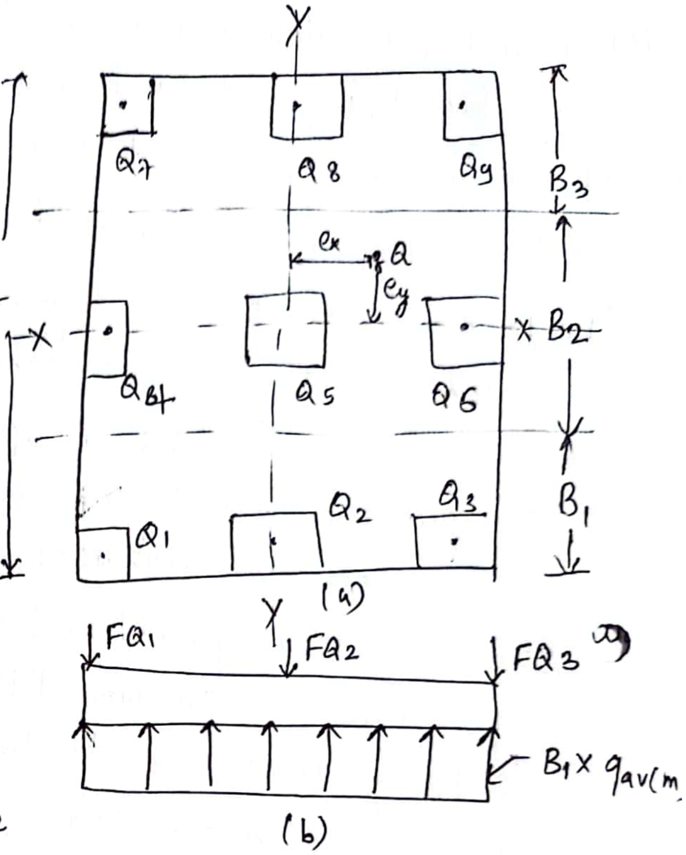
(a) If the resultant passes through the centre of the raft, the contact pressure is given by,

$$q = Q/A$$

(b) If the resultant has an eccentricity of ex and ey in X & y-directions (fig a)

$$q = \frac{Q}{A} \pm \frac{(Q \cdot e_x)}{I_{yy}} x \pm \frac{(Q \cdot e_y)}{I_{xx}} y$$

The maximum contact pressure should be less than the allowable soil pressure.



(3) Divide the slab into strips (bands) in X and y- directions. Each strip is assumed to act as independent beam subjected to the contact pressure and the column loads.

(4) Draw SF and BM diagrams for each strip.

(5) Determine the modified column loads as explained below.
Let us consider the strip carrying column loads Q_1, Q_2 & Q_3 (fig a).
Let B_1 be the width of the strip. Let the average soil (contact) pressure on the strip be q_{av}. Let B be the length of the strip.

Average load on the strip, $Q_{av} = \frac{1}{2}$ (downward load + upward force)

$$= \frac{1}{2}(Q_1 + Q_2 + Q_3 + q_{av} B_1 B)$$

Modified average pressure, $\overline{q_{av}} = q_{av}\left(\frac{Q_{av}}{q_{av} * B_1 B}\right)$

Column Load modification factor, $F = \frac{Q_{av}}{Q_1 + Q_2 + Q_3}$; All the column loads are multiplied by F for that strip.

(6) The BM & SF diagrams ~~for~~ are drawn for modified column loads and modified average soil pressure.

(7) Design the individual strips for the BM & SF found in step 6. The mat is designed as an inverted floor supported at columns.
(Reinforcement = twice the computed value)

CS CamScanner

* Pile Foundations

- It is a type of deep foundation which transfers the loads to deeper strata.
- A pile is a slender structural member made of steel, concrete or wood. A pile is either driven into the soil or formed in-situ by excavating a hole and filling it with concrete.

Necessity of Pile Foundations

Pile foundations are used in following conditions:

(1) When the strata at or just below the ground surface is highly compressible and very weak to support the load transmitted by the structure.

(2) When the plan of the structure is irregular relative to its outline and load distribution. It would cause non-uniform settlement if a shallow foundation is constructed. A pile foundation is required to reduce differential settlement.

(3) These are required for the transmission of structural loads through deep water to a firm stratum.

(4) These are used to resist horizontal forces in addition to support the vertical loads in earth-retaining structures and tall structures that are subjected to horizontal forces due to wind and earthquake.

(5) Piles are required when the soil conditions are such that a wash out, erosion or scour of soil may occur from underneath a shallow foundation.

(6) Piles are used for the foundations of some structures, such as transmission towers, off shore plateforms, which are subjected to uplift.

(7) In case of expansive soils, such as black cotton soil, which swell or shrink as the water content changes, piles are used to transfer the load below the active zone.

(8) Collapsible soils, such as loess, have a breakdown of structure accompanied by a sudden decrease in void ratio when there is increase in water content. Piles are used to transfer the load beyond the zone of possible moisture changes in such soils.

CS CamScanner

* Classification of Piles

Piles are classified according to (1) the material used, (2) the mode of transfer of load, (3) method of construction, (4) the use, or (5) the displacement of soil, as described below.

(1) Classification according to material used - four types.

(i) Steel Piles.

- generally in the form of thick pipes or rolled steel H-sections.
- piles are provided with a driving point or shoe at the lower end.
- Epoxy coatings are applied in the factory during manufacture of pipes to reduce corrosion of steel piles.

(ii) Concrete Piles

- Cement concrete is used in the construction of concrete piles.
- These are either pre cast or cast-in situ.

(iii) Timber Piles

- These are made from tree tunks after proper trimming.
- The timber used should be ~~straight~~ straight, sound and free from defects.

(iv) Composite Piles

- made of two materials.
- may consist of lower portion of steel & the upper portion of cast in situ Concrete.
- rarely used in practice.

(2) Classification based on mode of transfer of loads - 3 types.

(i) End bearing piles.

- transmits the loads through their bottom tips. Such piles act as columns and transmit the load through a weak material to a firm stratum below.
- The ultimate load carried by pile (Q_u) ~~dis~~ is equal to the load carried by the point or bottom end (Q_p) which ~~may be~~ rock or the fairly compact & hard stratum of soil. depends upon bearing capacit of

(ii) Friction Piles

→ These piles transfer the load through skin friction between the embedded surface of the pile and the surrounding soil.

→ These are used when a hard stratum does not exist at a reasonable depth.

→ The ultimate load (Q_u) carried by the pile is equal to the load transferred by skin friction (Q_s).

(iii) Combined end bearing and friction piles

→ These piles transfer loads by a combination of end bearing at the bottom of the pile and friction along the surface of the pile shaft.

→ The ultimate load carried by the pile is equal to the sum of the load carried by the pile point (Q_p) and the load carried by the skin friction (Q_s).

(3) Classification based on method of installation – 5 types.

(i) Driven piles.

→ driven into the soil by applying blows of a heavy hammer on their tops.

(ii) Driven and cast-in situ piles.

→ These piles are formed by driving a casing with a closed bottom end into the soil. The casing is later filled with concrete.

(iii) Bored and cast-in situ piles

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

(iv) Screw piles

→ These piles are screwed into the soil.

(v) Jacked piles

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

(4) Classification based on use – 6 types

(i) Load bearing piles.

→ used to transfer the load of the structure to a suitable stratum by end bearing, by friction or by both.

CS CamScanner

(ii) Compaction piles

→ These piles are driven into loose granular soils to increase the relative density.

(iii) Tension piles

→ These are used to anchor down structures subjected to hydrostatic uplift forces or overturning forces.

(iv) Sheet piles

→ These form a continuous wall or bulkhead which is used for retaining earth or water.

(v) Fender piles

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

(vi) Anchor piles

→ These piles are used to provide anchorage for anchored sheet piles.

(5) Classification based on displacement of soil – 2 types.

(i) Displacement piles

→ All driven piles are displacement piles as the soil is displaced laterally when the pile is installed.

(ii) Non-displacement piles

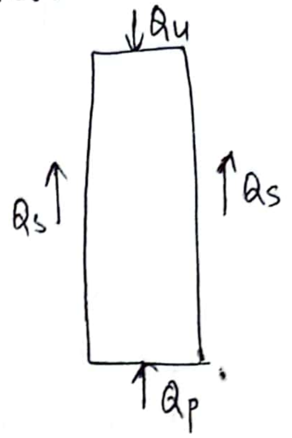
→ Bored piled are non-displacement piles as there is no displacement of soil during installation.

CamScanner

* Load Carrying Capacity of Piles

⇒ A pile foundation should be safe against shear failure and also the settlement should be within the permissible limits.

The methods for estimating the load carrying capacity of a pile foundation can be grouped into the following 4 categories.



(1) Static Methods

→ The static methods give the ultimate capacity of an individual pile, depending upon the characteristics of soil. The ultimate load capacity is given by

$$Q_u = Q_p + Q_s$$

Where,

Q_u = ultimate failure load

Q_p = point (or base or tip) resistance of the pile.

Q_s = Shaft resistance developed by friction (or adhesion) between the soil and the pile shaft.

(i) For driven piles in sand.

$$Q_p = q_p A_p \quad \& \quad Q_s = f_s A_s$$

Where,

q_p is the ultimate bearing capacity of soil at the pile tip.

A_p - area of the pile tip

f_s - average unit skin friction between the sand & the pile surface.

A_s - effective surface area of the pile in contact with soil.

$$Q_p = \bar{q} N_q A_p + \sum_{i=1}^{n} K (\bar{\sigma}_v)_i \tan\delta (A_s)_i$$

where,

$\bar{q}$ = effective vertical pressure at the pile tip

N_q = bearing capacity factors for pile.

n = number of layers in which the pile is installed

$(\bar{\sigma}_v)_i$ = effective normal stress in ith layer

$(A_s)_i$ = surface area of pile in ith layer.

$\tan\delta$ = Coefficient of friction between sand & pile material.

CamScanner

(ii) for driven piles in saturated clay.

$$Q_u = c N_c A_p + \alpha \bar{c} A_s$$

where,

c = cohesion of clay in the zone surrounding the pile tip

N_c - bearing capacity factor

α - adhesion factor

$\bar{c}$ - average cohesion along the shaft length

(2) Dynamic Formulas

→ In certain types of soil, the ultimate capacity of piles driven is related to the resistance against penetration developed during driving operation.

→ It is assumed that the load-carrying capacity of the pile is equal to the dynamic resistance during driving.

→ not much reliable.

(3) In-Situ Penetration Tests.

→ Pile capacity can be determined from the results of in-situ standard penetration test.

→ The N-value is related to the angle of shearing resistance.

(4) Pile Load Tests

→ It is the most reliable method of estimating the pile capacity.

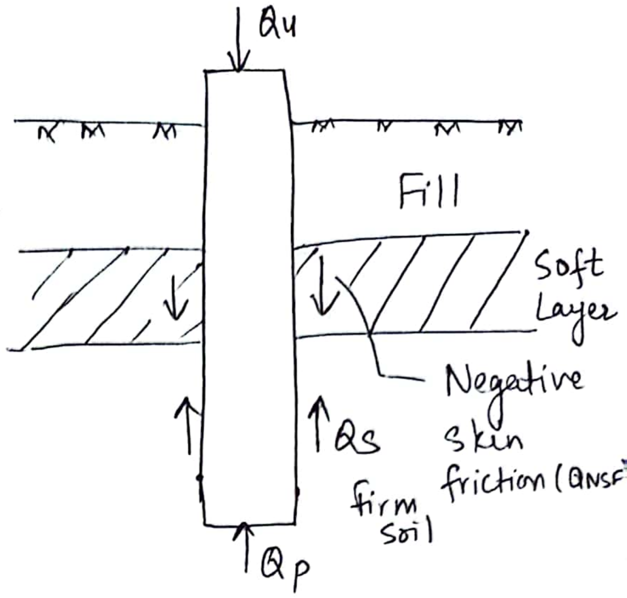
→ The test pile is driven and loaded to failure.

→ The pile capacity is related to the ultimate load or the load at which the settlements do not exceed the permissible limits.

CamScanner

* Negative Skin Friction (NSF)

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



→ NSF develops when a soft or loose soil surrounding the piles settles after the pile has been installed.

→ The NSF occurs in the soil zone which moves downward relative to the pile.

→ It imposes an extra downward load on the pile.

- The net ultimate load carrying capacity of the pile is given by:

$$Q_u' = Q_u - Q_{nsf}$$

where,

Q_{nsf} = negative skin friction

Q_u' = net ultimate load.

→ Where it is anticipated that negative friction would impose undesirable, large downward drag on a pile, it can be eliminated by providing a protective sleeve or a coating for the section which is surrounded by the settling soil.

- For individual piles, the magnitude of NSF, Q_{nsf} may be taken as.

i) For cohesive soils; $Q_{nsf} = p * c * L_f$

ii) For granular soils; $Q_{nsf} = \cancel{2} L_f^2 \, p \gamma K f$

where,

p = perimeter of pile

L_f = depth of fill or soil which is moving vertically

c = cohesion of soil in zone L_f

K = earth pressure coefficient

γ = unit weight of soil

f = coefficient of friction, $\tan\phi$