5.1 Lateral Earth Pressure: Types and Derivation

When a soil mass is retained at a higher level by a retaining wall, the retained mass of the soil tends to slide and assume a flat slope for equilibrium, which is resisted by the retaining wall. This exerts pressure on the retaining wall, which is known as lateral earth pressure. Usually, the retaining wall is constructed first and then the soil behind the wall is backfilled; hence, the retained soil is often called backfill. The back of the wall is either vertical or slightly inclined to the vertical and the lateral earth pressure is slightly inclined to the horizontal due to wall friction and inclination of the back of the wall.

The magnitude of the lateral earth pressure depends on the following factors:

i. Type and extent of the movement of the wall and the resulting horizontal strain in the backfill.

ii. Properties of the backfill material, including the density (γ), cohesion (c), and angle of shearing resistance (ϕ).

iii. Groundwater conditions in the backfill such as depth of water table and provision for drainage.

iv. Degree of roughness of the surface of the back of the retaining wall.

v. Slope of the back of the retaining wall.

vi. Depth of the retaining wall, that is, the height of the backfill to be retained.

vii. Inclination of the backfill surface with the horizontal.

viii. Additional loads on the backfill surface such as traffic loads or additional constructions, if any.

Types of Lateral Earth Pressure:

There are three basic types of lateral earth pressure.

They are:

1. Active earth pressure.
2. Passive earth pressure.
3. Earth pressure at rest.

These three basic types of lateral earth pressures are discussed below:

1. Active Earth Pressure:

Figure 1(a) shows a retaining wall of height H with a backfill having a horizontal surface. If the retaining wall were not there, the backfill would assume a stable flat slope. We know that cohesion less soils assume a stable slope equal to the angle of internal friction without any lateral support. Hence, when a backfill is retained, the wedge of soil above a certain slope tends to slide and move away from the rest of the backfill for equilibrium. This tends to push or rotate the wall away from the backfill if the wall is free to move or rotate.

The movement of the wall away from the backfill causes expansion of the backfill, resulting in stress release, thereby reducing the lateral earth pressure. Thus, the more is the movement of the wall away from the backfill, the more is the horizontal strain in the backfill, in the form of expansion, and the less is the lateral earth pressure. Initially when the wall is in a state of rest, a typical element of backfill at any depth is subjected to vertical stress due to self-weight of soil above the element and lateral earth pressure in the horizontal direction. The state of stress for the soil element is represented by Mohr's circle (I) in Fig. 1(b), where OB is the vertical stress and OA1 is the lateral earth pressure at rest.

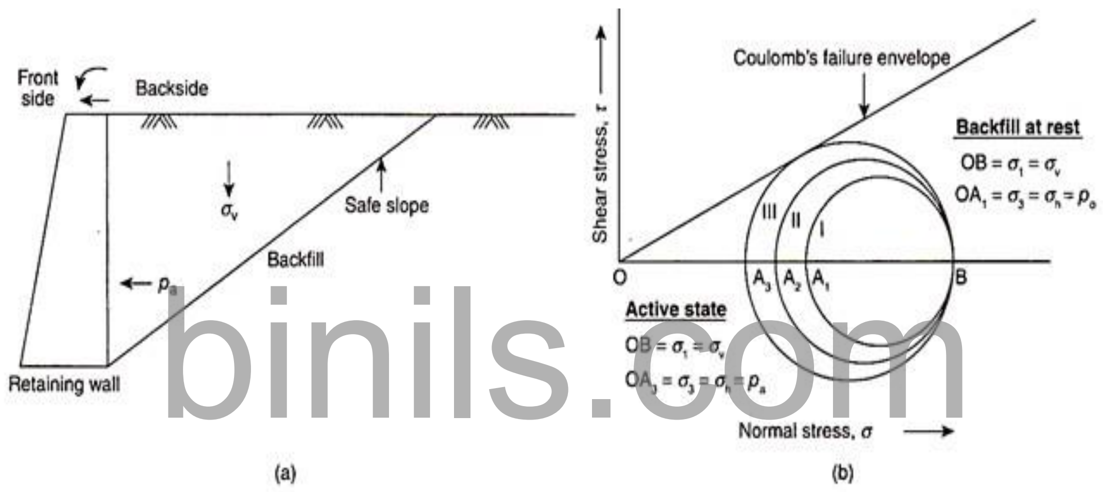


Fig 1(a)Retaining wall with a backfill Fig(b)Mohrs circle showing the gradual decrease of lateral earth pressure in active case

[Fig 1 https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925]

When the lateral earth pressure tends to push or rotate the wall away from the backfill, the movement of the wall away from the backfill causes expansion of the backfill, resulting in stress release, thereby reducing the lateral earth pressure. Thus, the more is the movement of the wall away from the backfill, the more is the horizontal strain in the backfill, in the form of expansion, and the less is the lateral earth pressure.

This is shown in Fig.1 (b), by Mohr's circle (II), in which $\sigma_h = \sigma_3 = OA_2$ is the reduced lateral earth pressure while the vertical stress, equal to $\sigma_v = \sigma_1 = OB$, remains constant. The decrease in the lateral earth pressure thus causes increase in the diameter of Mohr's circle, causing it to approach the Coulomb's failure envelope.

The decrease in the lateral earth pressure due to movement of wall away from the backfill and consequent expansion and stress release continues until Mohr's circle touches the Coulomb's failure envelope of the backfill material. When Mohr's circle touches the failure envelope, as shown by Mohr's circle (III) in Fig.1(b), the backfill material is on the verge of failure (limiting equilibrium) and no further decrease in the lateral earth pressure can take place. The minimum lateral earth pressure exerted on the retaining wall, when the wall moves away from the backfill, and the backfill material is in the limiting equilibrium, is known as active earth pressure.

When the wall moves away from the backfill, the backfill is said to be in the active state and the minimum lateral earth pressure exerted by the backfill in the active state in its limiting equilibrium condition is known as active earth pressure. Active earth pressure occurs when Mohr's circle of stresses at any point in the backfill touches the Coulomb's failure envelope.

Active earth pressure is denoted by the symbol pa, and its units are kN/m^2, t/m^2, or kgf/cm^2. All retaining walls, which are free to move or rotate, are by default subjected to active earth pressure and are designed to resist the same.

2. Passive Earth Pressure:

All retaining walls are usually not placed on the ground surface on the front side but are laid at some depth. Hence, the retaining wall has soil to some depth on its front side. When the wall moves away from the backfill due to active earth pressure, it actually moves towards the soil on the front side.

The movement of the wall is resisted by the front soil and exerts a lateral pressure on the wall, in a direction opposite to that of active earth pressure, as shown in Fig.2. Also, the movement of the wall towards the front soil causes compression of the soil, which, in turn, increases the lateral pressure from the front soil.

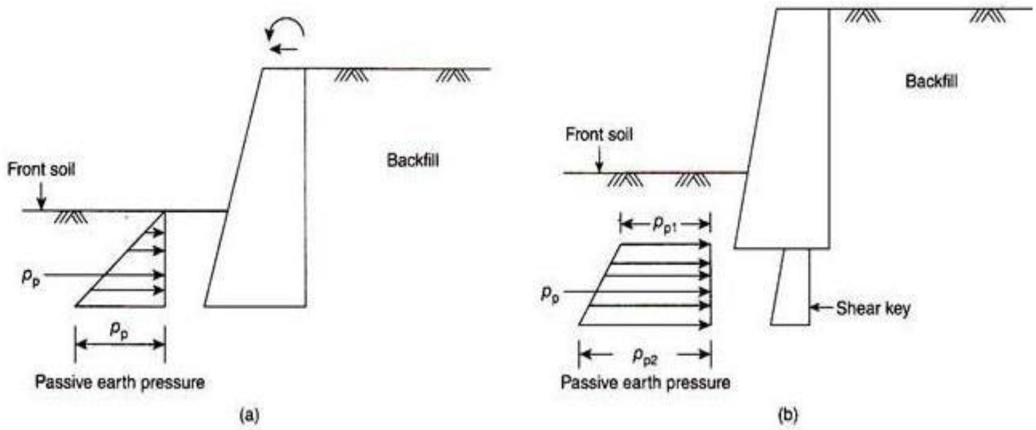


Fig 2 Practical cases of passive earth pressure a) In front of a retaining wall and b) on shear key below a retaining wall

[Fig 2 https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925]

Thus, the more is the movement of the wall toward the front soil, the more is the horizontal strain in the front soil, in the form of compression, and the more is the lateral earth pressure from the front soil opposite to that of active earth pressure. This is shown in Fig.3, by Mohr's circle (II), in which $\sigma_h = \sigma_3 = OA_2$ is the increased lateral earth pressure while the vertical stress, equal to $\sigma_v = \sigma_1 = OB$, remains constant. The increase in the lateral earth pressure causes decrease in the diameter of Mohr's circle as shown by Mohr's circles (II) and (III), and Mohr's circle reduces to a point, as represented by points A_4 and B, which become concurrent.

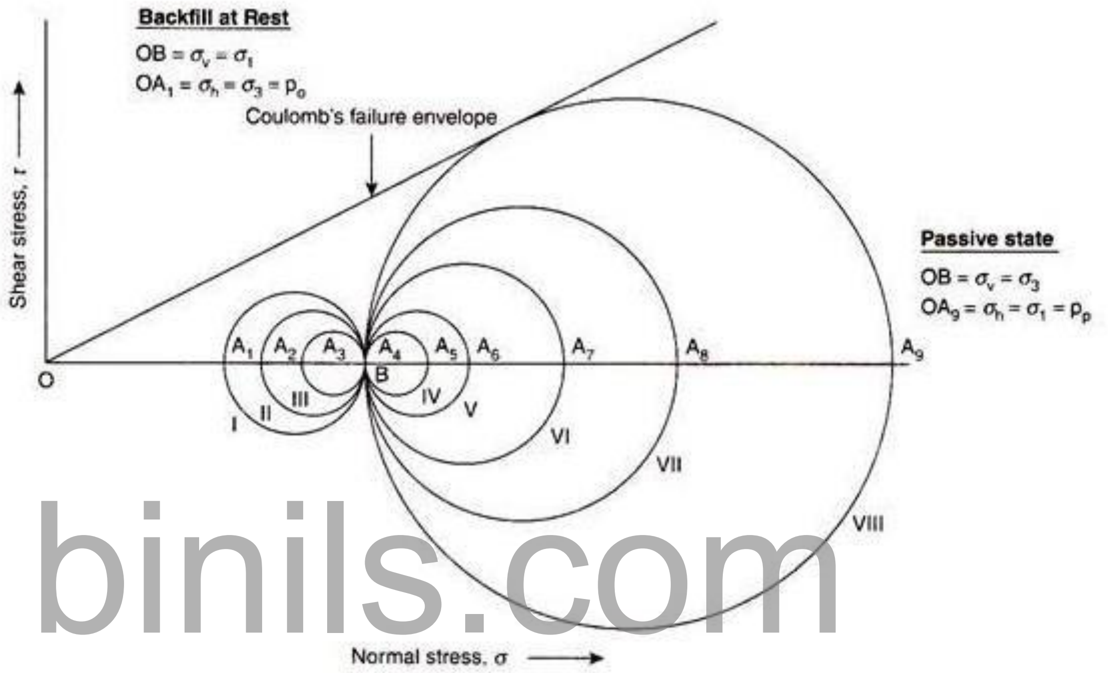


Fig 3 Mohrs circle showing the gradual decrease and then increase in lateral earth pressure in the passive case

[Fig 3 https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925]

Further increase of the lateral earth pressure from the front soil makes it higher than the vertical stress. At this stage, the lateral earth pressure becomes the major principal stress and the vertical stress becomes the minor principal stress. This is shown by Mohr's circles (IV), (V), (VI), etc., causing again an increase in the diameter of Mohr's circle.

The increase in the diameter of Mohr's circle leads it to approach the Coulomb's failure envelope. The increase in the lateral earth pressure due to the movement of wall towards the front soil and the consequent compression continues until Mohr's circle touches the Coulomb's failure envelope of the front soil.

When Mohr's circle touches the failure envelope, as shown by Mohr's circle (VIII) in Figure 3, the front soil is on the verge of failure (limiting equilibrium) and no further increase in the lateral earth pressure can take place. The maximum lateral earth pressure exerted on the retaining wall, when the wall moves towards the front soil, while it reaches its limiting equilibrium, is known as passive earth pressure.

When the wall moves towards the front soil, the front soil is said to be in the passive state and the maximum lateral earth pressure exerted by the front soil in the passive state in its limiting equilibrium condition is known as passive earth pressure. Passive earth pressure occurs when Mohr's circle of stresses at any point in the front soil touches the Coulomb's failure envelope.

Another practical example of passive earth pressure is the case of shear key provided below the base of a retaining wall. A shear key shown in Fig.3 is provided to improve the stability of the wall against sliding. When the retaining wall moves away from the backfill due to active pressure, the shear key also moves in the same direction but toward the soil below the base of the wall on the front side.

This generates passive earth pressure on the shear key. It is denoted by the symbol p_P, and its units are kN/m^2, t/m^2, or kgf/cm^2. Passive earth pressure is actually a stabilizing force improving the stability of the retaining wall, unlike active earth pressure.

3. Earth Pressure at Rest:

Figure 4 shows a basement retaining wall in which the wall is rigidly fixed to the basement slab. The basement retaining wall is therefore fixed in position and cannot move away from the backfill when subjected to lateral earth pressure. The lateral earth pressure exerted by the backfill on a retaining wall which is fixed in position and cannot move is known as earth pressure at rest.

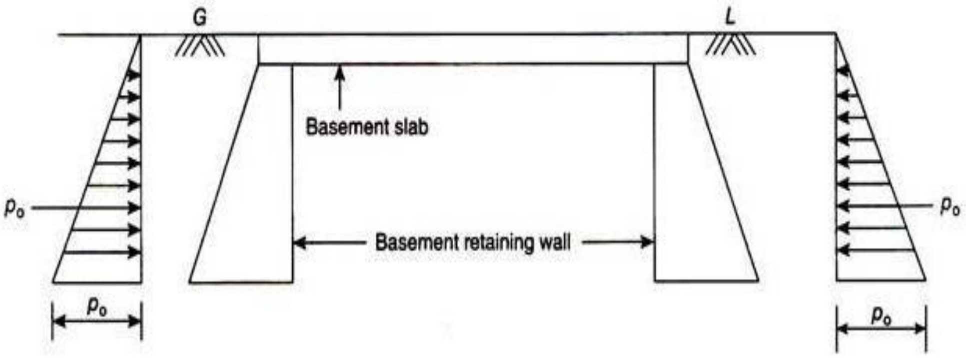


Fig 4 Earth pressure at rest on basement retaining walls

[Fig 4 https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925]

It is denoted by the symbol p_0, and its units are kN/m^2, t/m^2, or kgf/cm^2. As the wall does not move, the earth pressure exerted does not cause any lateral strain, and hence, there is no expansion of the backfill and no stress release. Earth pressure at rest is therefore always more than active earth pressure for the same depth of soil.

The abutment of a bridge is rigidly attached to the deck slab of the bridge and is also similarly fixed in position and hence subjected to earth pressure at rest.

Thus, lateral earth pressure exerted on a retaining wall depends on the direction and extent of the movement of the wall. Figure 5 shows the variation in lateral earth pressure on the y-axis as a function of the wall movement. When the wall moves away from the backfill, lateral pressure decreases with the increase in the movement of the wall; the minimum lateral earth pressure exerted on the wall is known as active earth pressure.

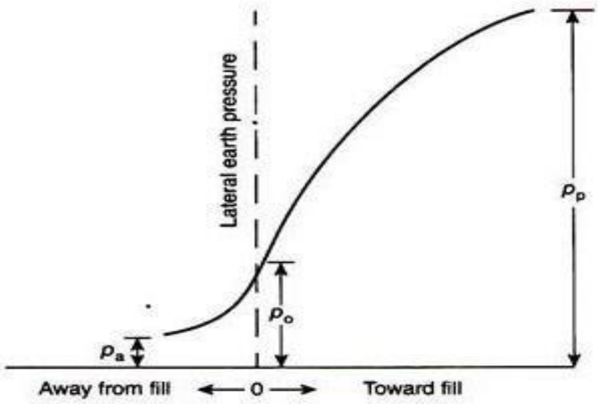


Fig 5 Variation in lateral earth pressure with the movement of the wall relative to the soil

[Fig 5 https://www.soilmanagementindia.com/lateral-earth-pressure/lateral-earth-pressure-types-and-derivation-soil/13925]

When the wall moves toward the soil, the lateral earth pressure generated increases with the increase in the movement of the wall; the maximum lateral earth pressure generated on the wall is known as passive earth pressure. The lateral earth pressure exerted on the wall when the wall is fixed in position is known as earth pressure at rest.

Derivation of Expression for Earth Pressure at Rest:

When a material is subjected to three-dimensional (3D) stresses, σ_x, σ_y and σ_z, along the three coordinate axes, x, y, and z, respectively, the strain along the x-axis can be computed from the principles of mechanics of materials as –

$$e_x = 1/E[\sigma_x - \mu(\sigma_y + \sigma_z)] \qquad (1)$$

where e_x is the horizontal strain (in the X-direction), E is the modulus of elasticity of soil, and μ is the Poisson's ratio. In the case of earth pressure at rest –

$$e_x = 0 \text{---------}(2)$$

$$\sigma_x = \sigma_y = P_0 \text{-------} (3)$$

Substituting these values in Eq. (1), we have –

$$e_x = 1/E\,[(p_0 - \mu(p_0 + \sigma_z)] = 0$$

$$\text{or } p_0 - \mu(p_0 + \sigma_z) = 0 \Rightarrow p_0 - \mu p_0 - \mu\sigma_z = 0 \Rightarrow p_0 - (1 + \mu) = \mu\sigma_z$$

$$p_0 = [\mu/(1 - \mu)]\sigma_z \ldots(4)$$

$$p_0 = K_0\sigma_z \text{ -------} (5)$$

where K_0 is the coefficient of the earth pressure at rest and σ_z is the vertical stress due to the self-weight of the soil at depth z, where the earth pressure at rest is to be computed

$$K_0 = \mu/(1 - \mu) \text{-------} (6)$$

Equation (6) is valid for elastic materials but not for soils, since soils are not elastic. Table 1 gives typical values of K_0 for different types of backfills, as obtained from actual measurement of earth pressure at rest.

Table 1 Coefficient of earth Pressure at rest for different soil

S.No	Type of soil	K_0
1	Loose Sand	0.5-0.6
2	Dense Sand	0.3-0.5
3	Undrained Clay	0.8-1.1
4	Over consolidated Clay	1.0-3.0

Coefficients of earth pressure - Earth Pressure Coefficient:

On a small unit at depth Z in the back there are two kinds of pressure.

i) Vertical Earth pressure:

The pressure applied in the vertical direction due to the back fill lying above it.

ii) Horizontal Earth pressure:

The pressure applied in the horizontal direction due to backfill is called the horizontal pressure or lateral earth pressure

Coefficient of active earth pressure at rest:

When the retaining wall is at rest then the ratio between the lateral earth pressure and the vertical pressure is called the co-efficient of the earth pressure at rest,

Ko = lateral pressure / vertical pressure

Co-efficient of active earth pressure:

When the retaining wall is moving away from the backfill the ratio between lateral earth pressure and vertical earth pressure is called coefficient of active earth pressure.

Ka = lateral pressure / vertical pressure

(or)

It is the ratio of horizontal and vertical principal effective stress when a retaining wall moves away from the retained soil

$$k_a = \frac{1 - sin\varphi}{1 + sin\varphi} = \tan^2(45 - \frac{\varphi}{2})$$

Coefficient of passive earth pressure:

When the retaining wall is moving towards the backfill, then the ratio between the lateral earth pressure and the vertical earth pressure is called the coefficient of passive earth pressure.

Kp = lateral pressure / vertical pressure

(or)

It is the ratio of horizontal and vertical principal effective stress when a retaining wall is forced against a soil mass.

$$k_p = \frac{1 + sin\varphi}{1 - sin\varphi} = \tan^2(45 + \frac{\varphi}{2})$$

5.2 RANKINE'S THEORY:

Rankine's theory of lateral earth pressure is applied to uniform cohesion less soils only. Later,it was extended to include cohesive soils, by Resal and by Bell. The theory has also been extended to stratified, partially immersed and submerged soils.

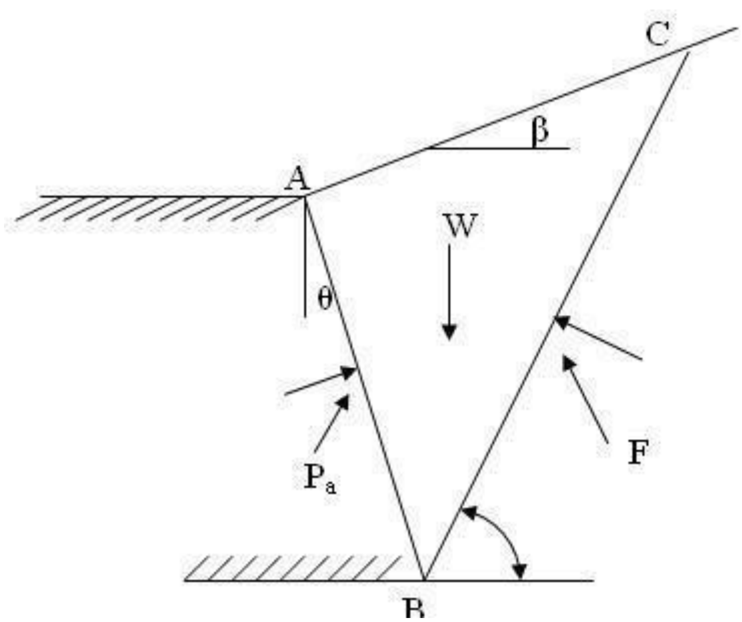
Following are theassumptions of the Rankine's theory:

- The soil mass semi-infinite, homogeneous, dry and cohesion less.
- The ground surface is a plane which may be horizontal and inclined
- The back of the wall is vertical and smooth. In other words, there are no shearing stresses between the wall and the soil and the stress relationship for any element adjacent to the wall is the same as for any other element fair away from the wall.
- The wall yields about the base and thus satisfies deformation condition for plastic equilibrium.

<u>ACTIVE EARTH PRESSURE:</u>

The following cases of cohesion less back fill will now be considered:

1. Dry or moist backfill with no surcharge.
2. Submerged backfill.
3. Backfill with uniform surcharge.
4. Backfill with sloping surface.
5. Inclined back and surcharge.



1.DRY OR MOIST BACKFILL WITH NO SURCHARGES:

Consider an element at a depth z below the ground surface. When the wall is at thepoint of moving outwards, the active state of plastic equilibrium is established.

Backfill is Cohesion less soil:

It is derived on the basis of the principal stress relationship on a failure plane.

$$\sigma_1 = 2Ctan\alpha + \sigma_3 \tan^2 \alpha$$

For active

$$\sigma_3 = \sigma_h$$

$$\sigma_1 = \sigma_v$$

Substitute in above equation

$$\sigma_v = 2Ctan\alpha + \sigma_h \tan^2 \alpha$$

Expression for active Pressure:

$$\sigma_v = \gamma Z$$

$$\sigma_h = P_a$$

According to principal stress relationship:

C=0

$$\sigma_v = \sigma_h \tan^2 \alpha - - - - - (1)$$

Substitute σ_v and σ_h value in eqn(1)

$$\gamma Z = P_a \tan^2 \alpha$$

$$P_a = \frac{\gamma Z}{\tan^2 \alpha} = \gamma Z cot^2\alpha$$

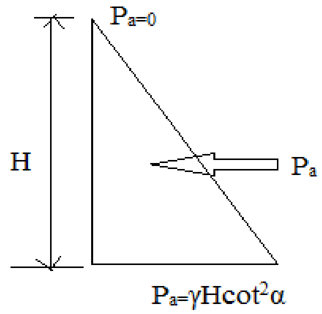
W.KT $K_a = \frac{1-sin\varphi}{1+sin\varphi} = Cot^2\alpha$

$$P_a = K_a\gamma Z - - - -(2)$$

Pressure Diagram:

At top ------- $Z=0, P_a=0$

At bottom-------- $Z=H, P_a=\gamma H\cot^2\alpha$



Consider for 1m run backfill

Total active earth pressure per m=Area of pressure diagram height

$$=\frac{1}{2}, K_a\gamma H.1.H$$

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

P_a act as a distance H/3 from base.

If the soil is dry, γ is the dry weight of the soil, if wet, γ is the moist weight.

Backfill is Cohesive soil:

For active Pressure:

$$\sigma_v=\gamma Z$$

$$\sigma_h=P_a$$

According to principal stress relationship:

$$\sigma_v=2Ctan\alpha+\sigma_h\tan^2\alpha------(1)$$

Substitute σ_v and σ_h value in eqn(1)

$$\gamma Z=2Ctan\alpha+P_a\tan^2\alpha$$

$$P_a\tan^2\alpha=\gamma Z-2Ctan\alpha$$

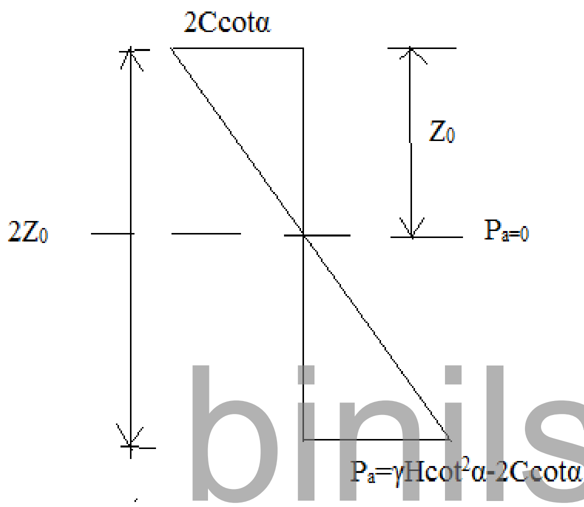
$$P_a = \frac{\gamma Z}{\tan^2 \alpha} - \frac{2Ctan\alpha}{\tan^2 \alpha}$$

$$P_a = \gamma Z\cot^2\alpha - 2C\cot\alpha ----(2)$$

Pressure Diagram:

At top ------- $Z=0, P_a=-2C\cot\alpha$

At bottom-------- $Z=H, P_a=\gamma H\cot^2\alpha-2C\cot\alpha$



If $P_a=0, Z=Z_0$

$$0 = \gamma Z_0 \cot^2 \alpha - 2C\cot\alpha$$

$$\gamma Z_0 \cot^2 \alpha = 2C\cot\alpha$$

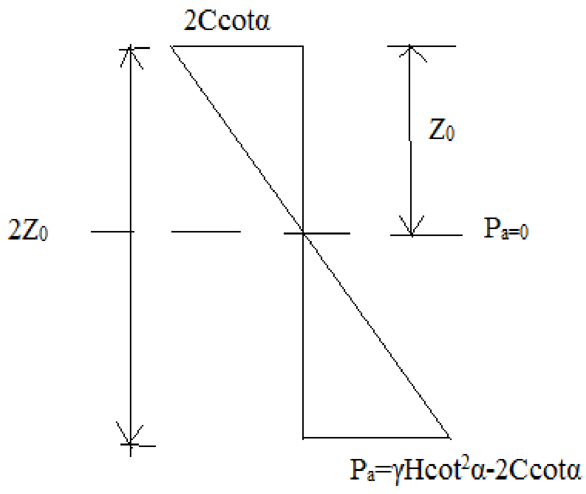
$$Z_0c = \frac{2C\cot\alpha}{\gamma \cot^2 \alpha} = \frac{2C}{\gamma \cot \alpha} = \frac{2C\tan\alpha}{\gamma}$$

Z_0 indicates the soil can withstand comfortably without slip.

The depth upto which the cohesive soil can withstand without any support is known as critical height (H_c)

$$H_c = 2Z_0$$

$$H_c = 2x\frac{2C}{\gamma}tan\alpha$$



$$H_c = \frac{4C}{\gamma}\sqrt{N\varphi} \qquad [\alpha = N\varphi]$$

Height of backfill below P_a=0,H-Z_0

Consider for 1m run backfill

Total active earth pressure per m=Area of pressure diagram x Height

$$= \frac{1}{2}, (\gamma H cot^2\alpha - 2C\,cot\alpha)x(H - Z_0)$$

2.SUBMERGED BACKFILL:

In this case, the sand fill behind the retaining wall is saturated with water. The lateral pressure is made up of two components:

For Active Pressure:

a)Backfill is fully submerged

Lateral pressure due to two component

Due to submerged unit weight of soil

Due to pore water

Consider a cohesionless soil with unit weight γ' and height Z

$$P_a = K_a\gamma' Z --- for\ submerged\ soil$$

$$P_a = \gamma_w Z --- pore\ water$$

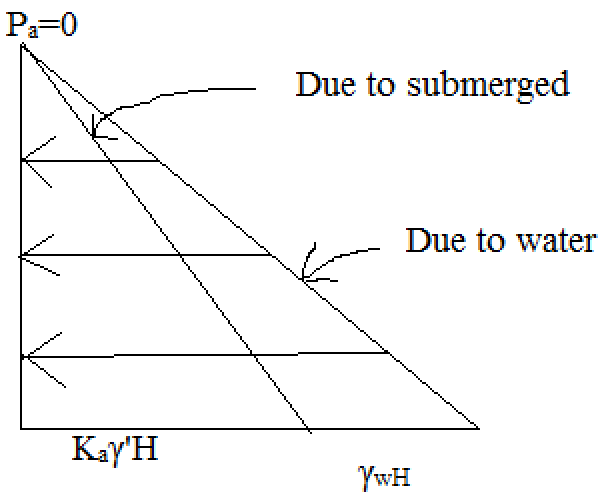
Active earth pressure,

$$P_a = K_a\gamma' Z + \gamma_w Z$$

Pressure diagram,

At top ------- Z=0,P_a=0

At bottom ------- $Z = H, P_a = K_a\gamma' H + \gamma_w H$



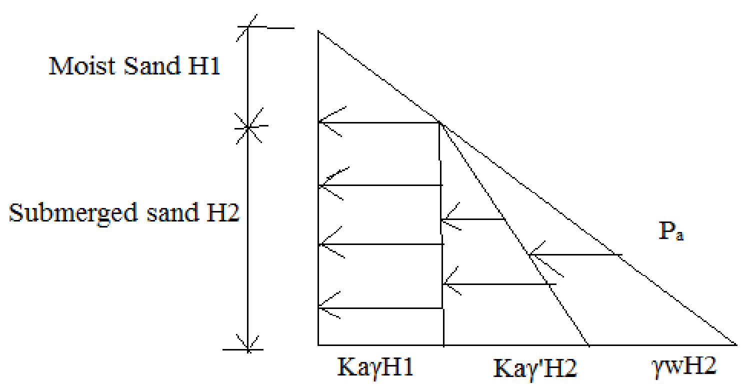
Total Pressure $P_a = \frac{1}{2}K_a\gamma' H^2 + \frac{1}{2}\gamma_w H^2$

Case b) If backfill is partially submerged

The pressure diagram

γ=unit weight of moist sand having depth H1

γ'=unit weight of moist sand having depth H2



$$P_a = K_a\gamma H_1 + K_a\gamma' H + \gamma_w H$$

Total Pressure per m

$$P_a = \frac{1}{2} K_a \gamma H_1^2 + K_a \gamma' H_2^2 + \frac{1}{2} \gamma_w H_H^2$$

$$Z = \frac{P_1 \left(\frac{H}{2}\right) + P_2 \left(\frac{H}{3}\right)}{P_a}$$

3.<u>BACKFILL WITH UNIFORM SURCHARGE:</u>

If the backfill is horizontal and carries a surcharge of uniform intensity q per unit area. The vertical pressure increment, at any depth z, will increase by q. the increase in the lateral pressure due to this will be K_aq.

For active pressure:

Consider a surcharge load (q) is acting on the top of backfill. It act as vertical stress[σ_v=q]

For surcharge load alone C=0

According to principal stress relationship,

$$\sigma_v = 2Ctan\alpha + \sigma_h \tan^2 \alpha -----(1)$$

$$\sigma_v = q$$

$$\sigma_h = P_a$$

Substitute σ_v and σ_h value in eqn(1)

$$q = P_a \tan^2 \alpha$$

$$P_a \tan^2 \alpha = \gamma Z - 2Ctan\alpha$$

$$P_a = \frac{q}{\tan^2 \alpha}$$

$$P_a = q\cot^2\alpha = qK_a ----(2)$$

Pressure Diagram:

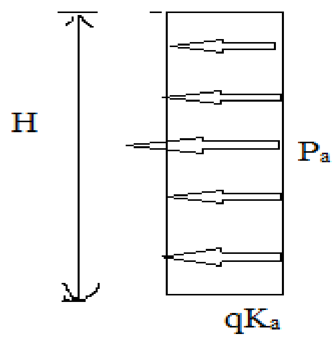
At top ------- Z=0,P_a=qK_a

At bottom-------- Z=H,P_a= qK_a

Total active pressure per m run

P_a=Area of pressure diagram X height

$= qK_a \, x1xH = q\cot^2 \alpha$



Total active pressure=Pressure due to surcharge + pressure due to backfill

$$= q\cot^2\alpha + \frac{1}{2}, K_a \gamma H^2$$

At the base of the wall, the pressure intensity is $P_a = 1/2K_a\gamma H + K_a q$

4.<u>BACKFILL WITH SLOPING SURFACE</u>:

Let the sloping surface behind the wall be inclined at the angle β with the horizontal; β is called the surcharge angle. In finding out the active earth pressure for this case byRankine's theory, an additional assumption that the vertical and lateral stresses are conjugateis made. It can be shown that if the stress on the given plane at a given point is parallel to the another plane, the stress on the latter plane at the same point must be parallel to the first plane. such planes are called the conjugate planes the stresses acting on them are called conjugate stresses.

Consider a soil element at point A at depth z with in a backfill with a sloping surface. The top plane of the element is parallel to the ground plane and the other plane conjugate to this is vertical. Let σ and p be the conjugate stresses, σ being vertical and p being the parallel to the sloping backfill. Being conjugate, both the vertical pressure and lateral pressure have the same angle of obliquity β, which is equal to the surcharge angle.

$$\sigma_1 - \sigma_3 / \sigma_1 + \sigma_3 = \sin \phi \quad (1)$$

Mohr circle corresponding to the principal stress intensity σ_1 and $\sigma_3$at A. OA_1 represents the resultant stress p and OA_2 represents the resultant stress σ. Draw OB perpendicular to $A_1 A_2$.

$$OB = OC \cos\beta \text{-------}(2)$$

$$BC = OC \sin\beta = \sin\beta\,(\sigma 1 + \sigma 3)/2 \text{ -----} (3)$$

$$A_1B = BA_2 = \sqrt{(A_1C^2 - BC^2)} = \sqrt{((\sigma_1 - \sigma_3)/2)^2 - (\sigma_1 + \sigma_3)/2)^2 \sin^2\beta)}$$

From (1) ,

$$A_1B = BA_2 = (\sigma1 + \sigma3)/2 \sqrt{(\sin^2\phi - \sin^2\beta)} \text{------}(4)$$

Now stress

$$\sigma = OB + BA_2$$

$$= (\sigma1 + \sigma3)/2 \cos\beta + (\sigma1 + \sigma3)/2 \sqrt{(\sin^2\phi - \sin^2\beta)} \text{ ------ } (5)$$

And stress p= $OB - A_1B$

$$= (\sigma1 + \sigma3)/2 \cos\beta - (\sigma1 + \sigma3)/2 \sqrt{(\sin^2\phi - \sin^2\beta)} \text{ ------ } (6)$$

Dividing (5) & (6) we get,

$$p/\sigma = K = \cos\beta - \sqrt{(\sin^2\phi - \sin^2\beta)}/\cos\beta + \sqrt{(\sin^2\phi - \sin^2\beta)}$$

$$K = \cos\beta - \sqrt{(\cos^2\beta - \cos^2\phi)}/\cos\beta + \sqrt{(\cos^2\beta - \cos^2\phi)}$$

The ratio K is called conjugate ratio.

For the present case,

$$\sigma = (\gamma.z.b\cos\beta/b)$$

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

$$P_a = \gamma.z.\cos\beta\ (\cos\beta - \sqrt{(\cos^2\beta - \cos^2\phi)}/\cos\beta + \sqrt{(\cos^2\beta - \cos^2\phi)})$$

$$P_a = K_a \gamma Z$$

$$K_a = cos\beta\ [\frac{cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}{cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}}]$$

$$K_a = (1 - \sin\phi)/(1 + \sin\phi)$$

The total active pressure P_a for the wall of height H is given by$P_a = 1/2\ K_a\gamma.H^2$

If the backfill is submerged, the lateral pressure due to the submerged weight of the soil will act at β with horizontal, while the lateral pressure due to water will act normal to the wall.

5.<u>INCLINED BACK AND SURCHARGE</u>:

A retaining wall with an inclined back supporting a backfill with horizontal ground surface. The total active pressure P_1 is first calculated on a vertical plane BC passing through the heel. The total pressure P is the resultant of the horizontal pressure P_1 and the weight W of thewedge ABC:

Where,

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

$$P_1 = 1/2\ K_a\gamma.H^2$$

The active earth pressure is first calculated on a vertical plane passing through the heel and intersecting the surface of the backfill or it's extension in point C. the height H of vertical plane is represented by BC. The resultant of P is the vector sum of P_1 and W, where W is the weight of the soil contained in the triangle ABC.

For Passive Pressure:

Case i) Dry or moisture backfill with no surcharge:

Backfill is Cohesion less soil:

$\sigma_1 = \sigma_h, \sigma_3 = \sigma_v$

$$\sigma_v = \gamma Z$$

$$\sigma_h = P_p$$

According to principal stress relationship:

C=0

$$\sigma_h = 2Ctan\alpha + \sigma_v \tan^2\alpha - - - - - - (1)$$

Substitute σ_v and σ_h value in eqn(1)

$$P_p = 2Ctan\alpha + \gamma Z \tan^2\alpha$$

$$P_p = \gamma Z \tan^2\alpha$$

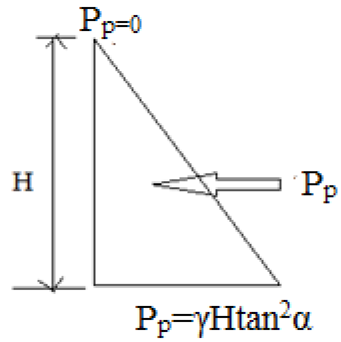
W.KT $K_p = \frac{1+sin\varphi}{1-sin\varphi} = tan^2\alpha$

$$P_p = K_p\gamma Z - - - -(2)$$

Pressure Diagram:

At top ------- Z=0,P_p=0

At bottom-------- Z=H,P_p=γH$\tan^2\alpha$



Consider for 1m run backfill

Total passive pressure per m=Area of pressure diagram height

$$= \frac{1}{2}, K_p \gamma H. 1. H$$

$$P_p = \frac{1}{2}, K_p \gamma H^2$$

P_p act as a distance H/3 from base.

Backfill is Cohesive soil:

$\sigma_1 = \sigma_h, \sigma_3 = \sigma_v$

$$\sigma_v = \gamma Z$$

$$\sigma_h = P_p$$

According to principal stress relationship:

$$\sigma_h = 2Ctan\alpha + \sigma_v \tan^2 \alpha ------(1)$$

Substitute σ_v and σ_h value in eqn(1)

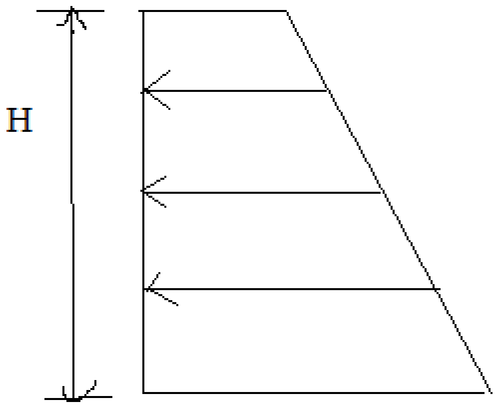
$$P_p = 2Ctan\alpha + \gamma Z \tan^2 \alpha$$

$$P_p = 2\mathrm{C}\tan\alpha + \gamma Z \tan^2\alpha ----(2)$$

Pressure Diagram:

At top ------- Z=0, P_p=2Ctanα

At bottom-------- Z=H, P_p= 2Ctanα+ γHtan²α



Consider for 1m run backfill

Total passive earth pressure per m=Area of pressure diagram x Height

$$= \frac{1}{2}(2C\ tan\alpha + \gamma Htan^2\alpha + 2C\ tan\alpha)xHx1$$

$$= \frac{1}{2}(4C\ tan\alpha + \gamma Htan^2\alpha)xHx1$$

Case iii) Surcharge Load:

For passive pressure:

Consider a surcharge load (q) is acting on the top of backfill. It act as vertical stress[σ_v=q]

For surcharge load alone C,

$\sigma_1 = \sigma_h, \sigma_3 = \sigma_v$

According to principal stress relationship,

$$\sigma_v = 2Ctan\alpha + \sigma_h \tan^2\alpha - - - - - - (1)$$

$$\sigma_v = q$$

$$\sigma_h = P_p$$

Substitute σ_v and σ_h value in eqn(1)

$$P_p = q \tan^2\alpha$$

$$P_a = q\tan^2\alpha = qK_p - - - - (2)$$

Pressure Diagram:

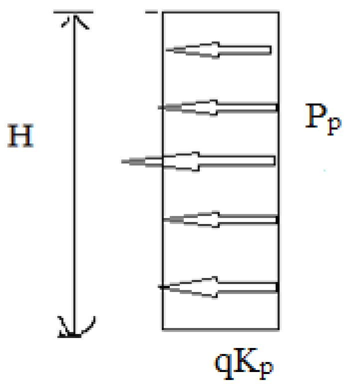
At top ------- $Z=0, P_p=qK_p$

At bottom-------- $Z=H, P_a= qK_p$

Total active pressure per m run

P_p=Area of pressure diagram X height

$= qK_p$ x1xH=qtan^2 α



Case iv) Effect of inclined surcharge or sloping backfill (or) Expression for earth pressure in case of sub charge angle.

$$\sigma_3 = \sigma_v$$

$$\sigma_1 = \sigma_h$$

For passive pressure:

$$P_p = K_p \gamma Z$$

$$K_p = cos\beta \left[\frac{cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}}{cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}\right]$$

Case v) Effect of inclined surcharge or sloping backfill (or) Expression for earth pressure in case of sub charge angle.

$$\sigma_3 = \sigma_h$$

$$\sigma_1 = \sigma_v$$

For active pressure:

$$P_a = K_a \gamma Z$$

$$K_a = cos\beta \left[\frac{cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}{cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}}\right]$$

For passive pressure:

$$P_p = K_p \gamma Z$$

$$K_p = cos\beta\,[\frac{cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}}{cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}]$$

Problems:

1. A gravity retaining wall retains 10 m of a backfill, unit weight of soil =18 kN/m^{3,} angle of shearing resistance =30° with a horizontal surface. Assume the wall interface to be vertical, determine (i) the magnitude and point of application of the total active pressure (ii) if the water table is at a height of 5m, and how far do the magnitude and the point of the application of active pressure changed. Take submerged unit weight = 10kN/m^3.

H= 10m, $\emptyset = 30°$, $\gamma = 18kN/m^2$, γ_{sub} =10kN/m^2

$$K_a = \frac{1-\sin\varphi}{1+\sin\varphi} = \frac{1-\sin 30}{1+\sin 30} = \frac{1}{3}$$

$$\text{Active Pressure at base of wall} = \frac{1}{3}x18x10 = 60KN/m^2$$

$$Total\ active\ thrust\ , P_a = \frac{1}{2}K_a\gamma H^2$$

$$P_a = \frac{1}{2}x\frac{1}{3}x18x10 = \frac{60KN}{m^2}$$

ii)Water – table at 5m from surface:

$P_a = K_a\,\gamma\,H_1 + K_a\,\gamma_{sub}\,H_2 + \gamma_w H$

$$= \frac{1}{3}x18x5 + \frac{1}{2}x10x5 + 9.81x5$$

= 104.05 kN/m^2 at base

P_1 = 1/2 x 1/3 x 18 x 5 x 5 = 75 kN,Y_1 =5/3 + 5 = 6.67m

P_2 =5 x 1/3 x18 x 5 = 150 kN,Y_2 = 5/2 = 2.5m

P_3 = ½ x 1/3 x 10 x 5 x 5 =41.67 kN,Y_3 = H / 3 =5/3 = 1.67m

P_4 = 1/3 X 9.81 X 5 X 5 =122.63 kNY_4 = H/3 = 1.67 m

Total thrust, $p_a = p_1 + p_2 + p_3 + p_4$

= 75 +150+41.67 +122.63

P_a = 389.3 kN per meter length of wall.

Taking moments about base

$$P \times \bar{Y} = p_1 y_1 + p_2 y_2 + p_3 y_3 + p_4 y_4$$

$$389.3\ \bar{y} = (75 \times 6.67) + (150 \times 2.5) + (41.67 \times 1.67) + (122.63 \times 1.67)$$

$$\therefore \bar{y} = 2.95 \text{ m}$$

∴ Total thrust of 389.3 kN per meter length of wall will act at 2.95 m from base of wall.

2.A retaining wall is 4 m high. Its back is vertical and it has got sandy backfill up to its top. The top of the fill is horizontal and carries a uniform surcharge of 85 kN/m^2. Determine the active pressure on the wall per meter length of wall. Water table is 1mbelow the top of the fill. Dry density of soil = 18.5 kN/m^3. Moisture content of soil above water table =12%. Angle of internal friction of soil = 30°, specific gravity of soil particles = 2.65. Porosity of backfill = 30°. The wall friction may be neglected.

$$e = \frac{n}{1-n}$$

$$= 0.43$$

$$\gamma = (1 + w) \times \gamma_d = (1+0.12) \times 18.5 = 20.7 \text{ kN/m}^3$$

$$K_a = \frac{1 - sin\varphi}{1 + sin\varphi}$$

$$= \frac{1 - 05}{1 + 0.5} = 0.3$$

$$\gamma_{sub} = \gamma_{sat} - \gamma_w = 20.7 - 9.81 = 11.52 KN/m^3$$

(i) Due to soil above W.T

$P_1 = ½\ K_a \gamma H^2 + K_a \gamma H$

=1/2x0.33x18.5x4^2+0.33x18.5x4

= 21.85 kN/m

(ii) Due to submerged soil

P_2= ½ x 0.333 x 11.52x 3^2 = 17.3 kN/m.

Due to water pressure,P_3 = ½ x 9.81 x 3^2 = 45 kN/m.

∴ Total active pressure = P1 + P_2 + P_3 + P_4

P_a = 21.58 + 17.3 + 45 + 113.2

=197.08 kN/meter length of wall.

3. A retaining wall is 9 m high retain on a cohesion less soil with an angle internal friction 33^0.The surface is level with the level with the top of wall. The unit weight of the top 3m of the fill is $21 KN/m^3$ and that the rest is $27 KN/m^3$.Find the magnitude and point of application of resultant active thrust.

$$K_a = \frac{1 - sin\varphi}{1 + sin\varphi 1}$$

$$= \frac{-sin33^o}{1 + sin33^0} = 0.295$$

P_a @3m= $K_a \gamma H$=0.295x21x3=18.6KN/m³

P_a @3m= $K_a \gamma H$=0.295x(18.6+27x6) = 66.4KN/m³

Total active thrust,

P_a=1/2x3x18.6+18.6x6+1/2x6x47.8=283KN

$$\bar{z} = \frac{27.9(6 + 1) + 111.6x3 + 143.4x2}{283} = 2.89m$$

4.A wall of6m height sand having a unit weight of $20 KN/m^3$ and angle of internal friction of 30^0.If the surface of the backfill slope upwards @15^0 to the horizontal. Find the active thrust per unit length of the wall using Rankine's theory. Solve the problem both analytically.

$$P_a = \frac{K_a \gamma H^2}{2}$$

$$K_a = cos\beta \left[\frac{cos\beta - \sqrt{\cos^2\beta - \cos^2\varphi}}{cos\beta + \sqrt{\cos^2\beta - \cos^2\varphi}}\right]$$

Cos15=0.96, $\cos^2 15$=0.93, $\cos^2 30$=0.75

$$K_a = cos15 \left[\frac{cos15 - \sqrt{\cos^2 15 - \cos^2 30}}{cos15 + \sqrt{\cos^2 15 - \cos^2 30}}\right]$$

$$K_a = 0.96 \left[\frac{0.96 - \sqrt{0.93 - 0.75}}{0.96 + \sqrt{0.93 - 0.75}}\right]$$

$$= 0.96x \frac{0.5}{1.418} = 0.37$$

$$P_a = \frac{0.37x20x6^2}{2} = 131.2KN/m$$

5.3 CULMANN'S GRAPHICAL METHOD FOR ACTIVE PRESSURE:

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

PROCEDURE:

1. Draw the ground line φ line and the ψ line as usual
2. Take a slip plane BC1.calculate the weight of the wedge $ABC_1$and plot it as BE_1 to some scale on the φ line.
3. Through E_1, draw E_1, F_1 parallel to the line ψ, to cut the slip plane BC_2 in F_1.
4. Similarly take another slip plane BC_2, calculate the weight of wedge ABC_2 and plot it as BE_2 on the line. Draw E_2F_2 parallel to the line cut the slip plane BC_2 in F_2
5. Take number of such slip planes $BC_{3,}$ BC4. Plot the weight of the corresponding wedge s on the ψ line and obtain point's F3, F4.
6. Draw a smooth curve through points B, F_1, F_2, F_3, F_4 etc. This curve is known as the Culmann's line.
7. Draw a tangent to the Culmann's line parallel to the φ line. The maximum value of the earth pressure is represented by the intercept EF, on the adopted scale. EF being drawn through the points of tangency parallel to the line ψ line. BFC represents the critical slip plane.
8. To locate the points of application of the resultant pressure, draw a line parallel to the critical slip plane BC, through the center of gravity of the sliding wedge ABC and obtain its intersection on the back AB.

When the ground line is a plane, the weights of the wedges ABC_1, AC_1 =L3, etc. since the height of soil wedge is constant being equal to H_1. Hence the weights of these wedges are plotted as their base lengths L_1, L_2, L_3, etc. on the φ line.

$$P_a=1/2\gamma H_1(EF)$$

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

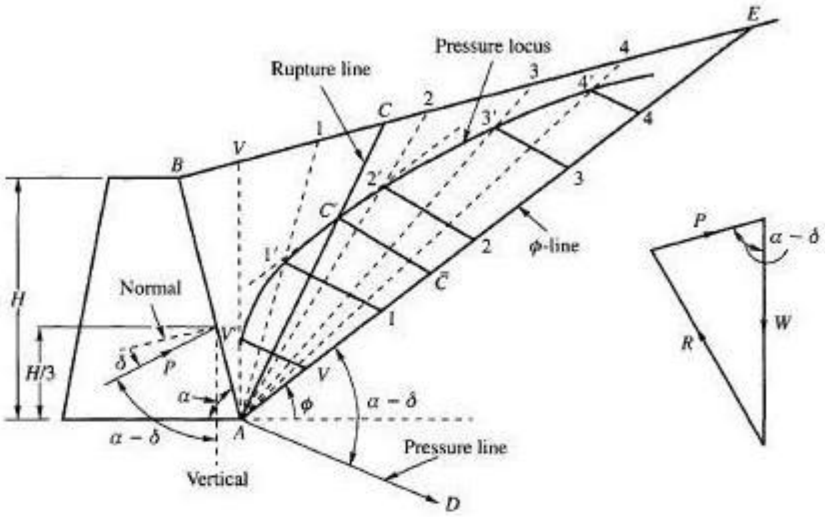


Fig 1 Active Earth Pressure by Graphical Method

[Fig1 https://civilengineeringx.com/foundation/active-pressure-by-culmanns-method-for-cohesionless-soils/]

EFFECT OF LINE LOAD:

Culmann's graphical method can also be used to take into account the running parallel to the retaining wall. A line load of intensity q per unit length, acting at a point C1, distant from the top of the wall. BEF1, F_n shows the Culmann's line and BC is the failure plane in absence of the line load. Let w1 be the weight of the wedge ABC1 which is plotted as BE1on the line ℂ and point F1 is obtained if there were no line load. However, when the line load is there the weight of the wedge ABC1 increases by q. thus BE^I represents and a point change in the Culmann's line the change being proportional to q. for all other failure wedges to the right, the weight q is added to the weight of the wedge and then plotted on the ℂ line. The modified Culmann's line is thus represents by BFF_1FF_n. when the slip plane is BC the pressure on the wall is represented by EF and when the slip plane is BC1, the pressure is represented by E, aF. if E^IF^I<EF slip occurs along BC^I and the pressure on the wall is increased

The Culmann's line BFF_2 is plotted by ignoring the line load. The modified Culmann's line BF^IF^I is then plotted by taking into account the line load, when the load q is added to the weight of each soil wedge considered. By drawing tangents to two Culmann's lines parallel to ℂ line, intercepts FE and F^IE^I are obtained. The intercept E^IF gives the greatest value of pressure due to backfill acted upon by q, whereas FE gives the maximumpressure in the absence of the line load. If the tangent at F is prolonged to meet the modified Culmann's line in F_2^I the intercept E_2^I F_2^I equals to FE. This means that if the line is placed beyond $C_{2,}$ there is no effect of the line load on the pressure for the other plotted. it will be seen that is maximum when the load is just at face of the wall, it remains constant with the position of q up to point c1 and then decreases gradually to zero at C_2.

For load positions beyond C_2 the pressure on the wall is not due to q. This method is very much used in locating the position of the railway line or the footing of building on the backfill at such a safe distance that the earth pressure on the (existing) wall does not increase.

5.4 COULOMB'S WEDGE THEORY

Instead of considering the equilibrium of an element within the mass of the material, Coulomb (1776) considered of equilibrium of whole of the material supported by a retaining wall when the wall is on the point of moving slightly away from the filling. The wedge theory of earth pressure is based on the concept of a sliding wedge which is torn off from the rest of the backfill on movement of the wall. In the case of active earth pressure, the sliding wedge moves downwards on a slip surface relative to the intact backfill and in the case of passive earth pressure, the sliding wedge moves upwards and inwards. The pressure on the wall is, in fact, a force of reaction which it has to exert to keep the sliding wedge in equilibrium. Factors such as well friction, irregular soil surfaces and different soil strata can easily take into account in this method.

Following are the basic Assumptions of the wedge theory:

- The backfill is dry, cohesion less, homogenous, isotopic and elastically undeform able but breakable.
- The slip surface is plane which passes through the heel of the wall.
- The sliding wedge itself acts as a rigid body and the value of earth pressure is obtained by considering the limiting equilibrium of the sliding wedge as a whole.
- The position and direction of the resultant earth pressure are known. The resultant pressure acts on the back of the wall at one-third the height of the wall from the base andis inclined at an angle δ (called the angle of wall friction) to the normal to the back. (The assumption means that the pressure distribution is hydrostatic, i.e., triangular). The back of wall is rough and a relative movement of the wall and the soil on the back takes place which develops frictional forces that influence the direction of the resultant pressure.

The forces acting on a wedge of soil are: its weight W, the reaction R along the plane of sliding and the active thrust Pa against the retaining wall. R will act at an angle $\emptyset$ to the normal of the plane of sliding. The pressure P is inclined at an angle of wall friction δ to the normal which is considered positive as marked in Fig. 2 Both R and P will be inclined in a direction so as to oppose the movement of the wedge. For the condition of the yield of the wall from the backfill the most dangerous or the critical slip surface is that for which the wall reaction is maximum, i.e., the wall must resist the maximum lateral pressure before it moves away from the fill.

Condition for maximum pressure from a sliding wedge. BD shows a plane inclined at an angle ϕ to the horizontal at which the soil is expected to stay in the absence of any lateral support. The line BD, therefore, is called the natural slope line, repose line or the $\emptyset$ – line. AD, inclined at β to the horizontal, is called the ground line or surcharge line. Plane BC, inclined at angle λ (to be determined) is the line or rupture plane or slip plane; the angle λ is called the critical slip angle. The reaction R inclined at an angle ϕ to the normal to the slip line; R is also inclined at an angle (λ-ϕ) to the vertical. The wall reaction P_a is inclined at an angle to the normal to the wall. The inclination of P_a to vertical is represented by angle ψ = 90˚- θ - δ (= constant for given value of θ and δ). The value of P_a depends upon the slip angle λ. P_a is zero when λ = ϕ. As λ increases beyond ϕ, P also increases and after reaching a maximum value it again reduces to zero when λ equals 90 $+\theta$. Thus, the critical slip plane lies between the line and back of the wall.

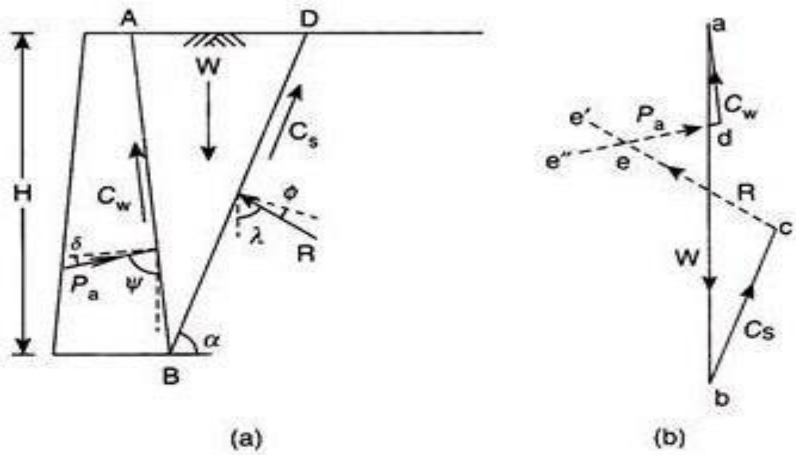


Fig1 Condition for maximum pressure from a sliding wedge

[Fig1https://www.soilmanagementindia.com/lateral-earth-pressure/coulombs-theory/coulombs-theory-for-earth-pressure-soil/13949]

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

$$\frac{P_a}{CE} = \frac{W}{BE}$$

$$P_a = W\frac{CE}{BE} - - - - - - (1)$$

From triangle CFE,

$$sin\varphi = \frac{x}{CE}$$

$$CE = \frac{x}{sin\varphi} - - - - (2)$$

Where A_1=cosec Φ

BE=BD-FD+FE

From triangle CFD,

$$\tan(\varphi - \beta) = \frac{x}{FD}$$

$$FD = x\cot(\varphi - \beta)$$

From triangle CFE,

$$\tan(\varphi) = \frac{x}{FE}$$

FD=x cot Φ

Hence, BE=n-x[cot(Φ-β)-cot Φ]

or

$BE=n-A_2X$ ------- (3)

Where A_2=[cot(Φ-β)-cot Φ]

$$w = \gamma(\Delta ABC) = \gamma[\Delta ABD - \Delta BCD]$$

$$W = \frac{1}{2}\gamma(m-x)n - - - -(4)$$

Substituting equation 2,3&4 in 1

$$P_a = \frac{1}{2}\gamma(m-x)n\frac{A_1 x}{n - A_2 x} = (\frac{1}{2}\gamma n A_1)\,(\frac{mx - x^2}{n - A_2 x})$$

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

$$\frac{dP_a}{dx} = (\frac{1}{2}\gamma n A_1)\,\frac{(m-2x)(n-A_2x) - (-A_2)(mx - x^2)}{(n - A_2 x)^2} = 0$$

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

$mn-A_2mx-2nx+2A2X^2=-A_2mx+A_2x^2$

$mn - 2xn = -\ A_2\ x^2$

Rearranging,

$mn-xn= xn - A_2\ x^2=x(n- A_2\ x)=x\ BE$

We can Write,

$$\frac{mn}{2} - \frac{xn}{2} = X\frac{BE}{2}$$

$$\Delta ABD - \Delta BCD = \Delta BCE$$

$$\Delta ABC = \Delta BCE$$

Thus the criterion for maximum active pressure is that the slip plane is so chosen that

ΔABC and ΔBCE are equal in area.

5.5 Stability Analysis of Retaining Wall:

Introduction

Stabilization incorporates the various methods employed for modifying the properties of a soil to improve its engineering performance. Stabilization is used for a variety of engineering works, the most common application being in the construction of road & air- field pavements, where the main objective is to increase the strength or stability of soil & to reduce the construction cost by making best use of the locally available materials.

1.Mechanical stabilisation:

- Mechanical stabilization involves two operations :
  - (i) changing the composition of soil by addition or removal of certain constituents
  - (ii) Densification or compaction .the particle size distribution and composition are the important factors governing the engineering behaviour of a Soil. Significant changes in theproperties can be made by addition or removal of suitable soil fractions. For mechanical stabilizations where the primary purpose is to have a soil resistant to deformation and displacement under the loads, soil materials can be divided in two fractions: The granular fraction retained on a 75 microns

IS sieve and the fine soil fraction passing a 75 –microns sieve. The granular fraction imparts strength and hardness. The fine fraction provides cohesion or binding property, water – retention capacity and also acts as a filler for the voids of the coarse fraction.

2.Cement stabilization:

a).Soil cements and its influencing factors

- The soil stabilized with cement (Portland) is known as soil cement.
- The cementing action is believed to be the result of chemical reaction of cement with the siliceous soil during hydration. The binding action of individual particles through cement may be possible only in coarse-grained soils .in fine grained,

cohesive soils, only some of the particles can be expected to have cement bonds, and the rest will be bonded through natural pollution. The important factors affecting soil cement are: nature of soil, cement content, condition of mixing, compaction and curing and admixtures.

b)Construction methods

The normal construction sequence for soil – cement bases is as follow:

(i) shaping the sub-grade and scarifying the soil,

(ii) Pulverising the soil,

(iii) addition and mixing cements,

(iv) adding and mixing water,

(v) compacting,

(vi) finishing,

(vii) curing and

(viii) adding wearing surfacing.

There are three methods of carrying out these operations:

(i) mix-in place method,

(ii) travelling plant method and

(iii) stationary plant method.

3.Lime stabilization:

Hydrated (or slaked) lime is very effective in treating heavy, plastic clayey soils . Lime may be used alone, or in combination with cement, bitumen or fly ash. Sandy soils can also be stabilized with these combinations. Lime has been mainly used for stabilizing the road bases and sub- grades on addition of lime to soil , two main types of chemical reactions occurs:

i. Alteration in the nature of absorbed layer through Base Exchange phenomenon,

ii. Cementing or puzzolanic action. Lime reduces the plasticity index of highly plastic soils making them more friable and easy to be handled and pulverized. The plasticity index of soils of low plasticity generally increases. There is generally and increase in the optimum water content and a decrease in the maximum compacted density, but the strength and durability increase.

4.Bitumen stabilization:

Asphalts and tars are the bituminous materials which are used for stabilization of soil, generally for pavement construction. These materials are normally too viscous to be incorporated directly with soil .the fluidity of asphalts is increased by either heating, emulsifying or by cut-back process. Tars are heated or cut back. The bituminous materials when added to a soil impart cohesion or binding action and reduced water absorption. Thus either the binding action or the water proofing action or both the actions, may be utilized for stabilization. depending upon these actions and the nature of soils , bitumen stabilization is classified under the following four types :

(i) sand-bitumen ,

(ii) (ii)soil-bitumen

(iii) (iii) water-proofed mechanical stabilization and

(iv) (iv) oiled earth

5.Chemical stabilization:

There are a great many chemicals which are used for stabilization. Only the chemicals which are commonly used for stabilizing moisture in the soil and for cementation of particles will be described here,

1. Calcium chloride
2. Sodium chloride
3. Sodium silicate

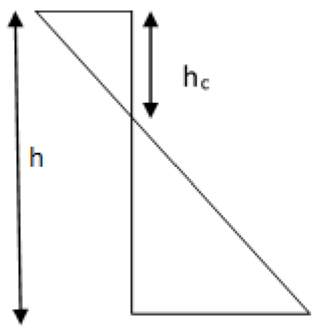
6.Stabilization by heating

Heating a fine grained soil to temperature of the order of 400-600®c causes irreversible changes in clay minerals. The soil becomes non – plastics, less water sensitive and non- expansive. Also the clay clods get converted into aggregates. Soil can be baked in kilns, orin –situ downwards draft slow moving furnaces. The artificial aggregates so produced can be used for mechanical stabilization.

7.Electrical stabilization:

The stability or shear strength of fine-grained soils can be increased by draining them withthe passage of direct current through them. The process is also known as electro-osmosis. Electrical drainage is accompanied by electro- chemical composition of the electrodes and the deposition of the metal salts in the soils pores. There may also be some changed in the structure of soil. The resulting cementing of soil due to all these reactions, is also knownas electro-chemical hardening and for these purpose the use of aluminium anodes is recommended.

Tension cracks :



In clay under undrained condition at climatic temperature variation and due to water drain in season‟s soil volume shrinkage, cause compression due to soil self weight, surcharge and live load, cause tensile stress and spilt or fracture in the clay mass especially.

Hence in figure the depth of the tension zone was given the symbol h_c. It is possible for cracks to develop over this depth and a value for h_c is obtained as

From active earth pressure theory, $h_c = 2C / \gamma$

Different modes of failure of retaining wall:

1. Failure against sliding
2. Failure against overturning
3. Failure against bearing capacity

Failure against sliding

- Ths soil in front of the wall provides active and passive pressure resistance as the wall tends to slide.
- Use of a key beneath the base provides additional sliding stability.

- The sliding resistance along the base $F_R = \mu R$, where R includes all the vertical forces, including the vertical components of P_a, acting at the base and μ the coefficient of wall friction.

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

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

Failure against overturning:

- For a wall to be stable the resultant thrust must be within the base. Most walls are so designing that the thrust is within the middle third of the wall base. It is to avoid lossof contact of base with soil.

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

- Overturning is usually considered with respect to toe and the factor of safety should beat least 1.5 for sandy soil and 2.0 for clayey soil. The resisting moments are normally due to vertical component of all the forces namely weight of wall, weight of soil overbase, vertical component active pressure and passive pressure.

Failure against Bearing Capacity:

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

- Vertical load causes uniform contact pressure at the base. Over turning moment causescompressive pressure at toe and tensile pressure at heel. The sum contact pressure is maximum at toe. Factor of safety against bearing capacity should be atleast 2.5 for sandy soil and 3.0 for clayey soil.