RETAINING WALLS

In general, retaining walls can be divided into two major categories (a) conventional retaining walls (b) Mechanically stabilized earth walls.

(a) Conventional retaining walls can generally be classified as
1. Gravity retaining walls
2. Semi gravity retaining walls
3. Cantilever retaining walls
4. Counterfort retaining walls

- Gravity retaining walls are constructed with plain concrete or stone masonry. They depend on their own weight and any soil resting on the masonry for stability.
- Semi gravity retaining walls. The size of the section of a gravity retaining wall may be reduced if a small amount of reinforcement is provided near the backface. Such walls are knows as semi-gravity retaining walls.
- Cantilever retaining walls are made of reinforced cement concrete. The wall consists of a thin stem and a base slab cast monolithically. This type of wall is economical upto 8 m.
- Counterfort retaining walls are similar to cantilever retaining walls. At regular intervals, however, they have thin vertical concrete slabs known as conterforts that tie the wall and base slab together. The purpose of conterforts is to reduce the shear and bending moments.

(b) Mechanically stabilized retaining walls have their backfills stabilized by inclusion of reinforcing elements such as metal strips, bars, welded wire mats, geo textiles and geogrids. These walls are relatively flexible and can sustain large horizontal and vertical displacement without much damage.

Design basics of a conventional retaining wall

To properly design a retaining wall, the basic soil parameters unit weight, angle of friction and cohesion for the soil retained behind the wall and for the soil below the base slab must be known. Additionally the angle of wall friction must also be known.

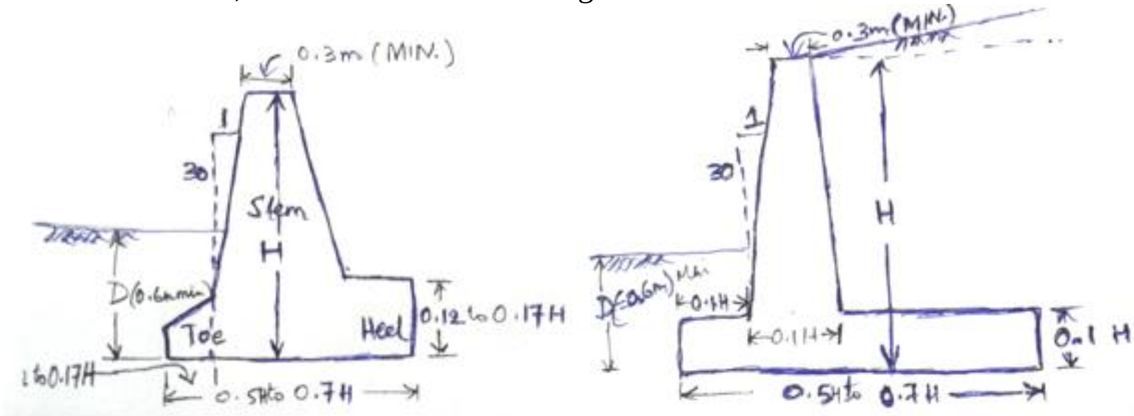
Knowing the properties of the soil behind the wall enables the determination of internal pressure distribution that has to be designed for.

There are two phases in the design of a conventional retaining wall.
1. First, with the lateral earth pressure known, the structure as a whole is checked for stability that includes checking for possible overturning, sliding and bearing capacity failures.
2. Second, each component of the structure is checked for adequate strength and the steel reinforcement of each component is determined.

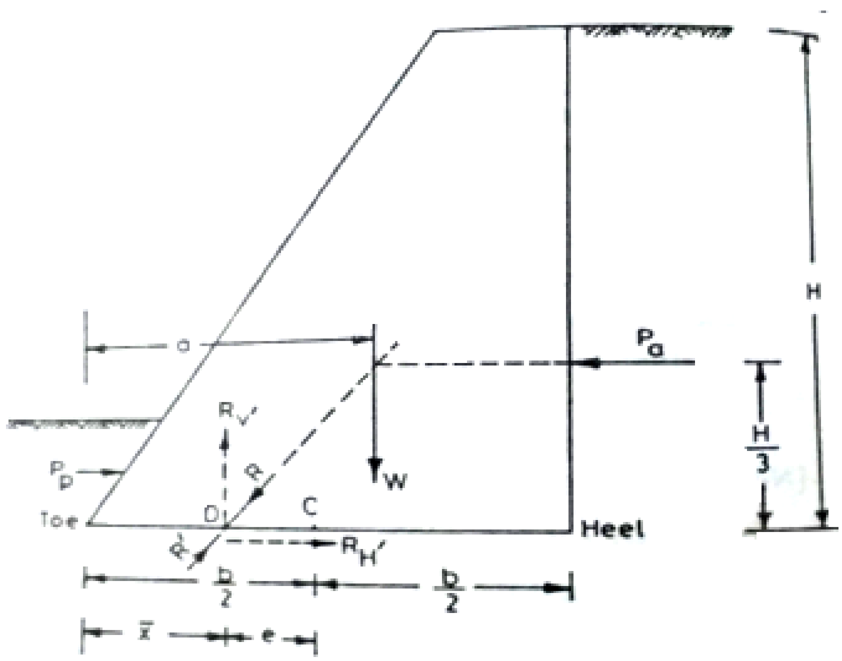
Gravity and Cantilever walls

Proportioning of Retaining walls

When designing retaining walls, some of the dimensions must be assumed (called proportioning), which allows to check trial sections for stability. If the stability checks yield undesirable results, the sections must be changed and rechecked.



Gravity Retaining wall – Stability Check



- Retaining walls with a smooth backface and no surcharge is constructed.
- The active pressure P_a acts horizontally.
- The front face of the wall is subjected to passive pressure P_p below the soil surface.
- The weight W of the wall and the active pressure P_a have their resultant R which strikes the base at D.
- There is an equal and opposite reaction R′ at the base between the wall and the foundation.

For convenience R′ is resolved into vertical component ($R_v{}'$) and horizontal component ($R_H{}'$).

For equilibrium, $R_v{}' = W$ and $R_H{}' = P_a$

The eccentricity 'e' of the force R_v' relative to the center C is determined using moment equation. Taking moments about the toe,

$$R_v' \; x \; \bar{x} = W \; x \; a - P_a \; x \; \left(\frac{H}{3}\right)$$

or

$$\bar{x} = \frac{W \; x \; a - P_a \; x \; \left(\frac{H}{3}\right)}{R_v'}$$

where $\bar{x}$ = distance of point D from the toe
thus, $e = \frac{b}{2} - \bar{x}$, where b = width of the base

For a safe design, the following requirements must be satisfied.

1. No sliding

The wall must be safe against sliding

$\mu R_v > R_H$

Factor of safety $F_s = \frac{\mu R_v}{R_H}$

μ = coefficient of friction between the base of the wall and soil ($= \tan \delta$)
Minimum factor of safety recommended = 1.5

2. No over turning

The wall must be safe against overturning about the toe. The factor of safety against overturning is

$$F_0 = \frac{\sum M_R}{\sum M_0}$$

ΣM_R = sum of resisting moments about the toe = W x a
ΣM_0 = sum of overturning moments about the toe = $P_a \; x \; \frac{H}{3}$

$$\therefore F_0 = \frac{\text{W x a}}{P_a \; x \; \frac{H}{3}}$$

F_o is usually kept between 1.5 and 2.0

3. No bearing capacity failure

The pressure caused by R_v at the toe of the wall must not exceed the allowable bearing capacity of the soil.
The pressure distribution is assumed to be linear.
The maximum pressure is given by

$$p_{max} = \frac{R_v}{b}\left[1 + \frac{6e}{b}\right]$$

The factor of safety against bearing failure is

$$F_b = \frac{q_a}{p_{max}}$$

q_a = allowable bearing pressure
F_b = 3 is usually specified, provided the settlement is also within allowable limit.

4. No Tension

There should be no tension at the base of the wall. When $> \frac{b}{6}$, tension depends at heel. Tension is not desirable. Tensile strength of soil is very small and the tensile crack would develop. The effective base area is reduced.

In such a case, the maximum stress is given by

$$p_{max} = \frac{4}{3}\left(\frac{R_v}{b-2e}\right)$$

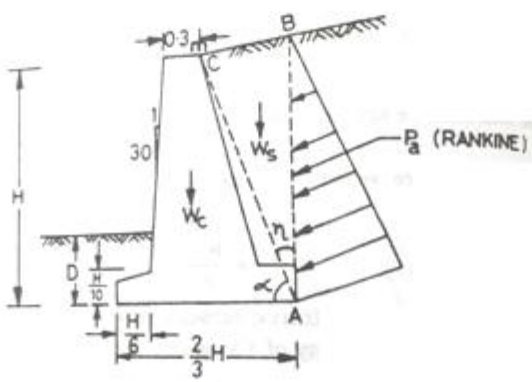
The earth pressure can be computed using the Rankine's theory or Coulomb's theory.

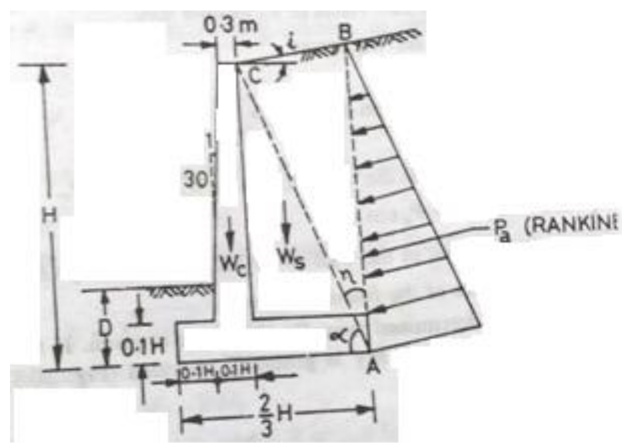
For using Rankine's theory, a vertical line AB us drawn through the heel A. It is assumed that the Rankine's active conditions exist along the vertical line AB. Rankine's active earth pressure equations may then be used to calculate the lateral pressure on the face AB.

The assumption for the development of Rankine's active pressure along the soil face AB is theoretically correct if the shear zone bounded by the line AC is not obstructed by the stem of the wall.

AC makes an angle η with the vertical given by

$$\eta = 45 + \frac{i}{2} - \frac{\Phi'}{2} - \sin^{-1}\left(\frac{\sin i}{\sin \Phi'}\right)$$





While checking for stability, weight of the soil (W_s) above the heel in the zone ABC should also be considered in addition to P_a on AB and weight of the wall W_c in case of Rankine's theory.

If Coulomb's theory is used W_s need not be considered only P_a (Coulomb) and wall weight W_c are to be considered.

Fig

Check for overturning

$$P_p = \frac{1}{2} K_p \gamma_2 D^2 + 2C_2 \sqrt{K_p} D$$

γ_2= unit weight of soil in front of the heel and under base slab

$$K_p = \tan^2\left(45 + \frac{\Phi_2}{2}\right)$$

C_2 , Φ_2 = shear parameters of soil under bas slab.

The factor of safety against overturning about the toe ('C')

$$F_0 = \frac{\sum M_R}{\sum M_0}$$

$\sum M_0 = P_h \left(\frac{H'}{3}\right) \ \ where\ P_h = P_a \cos\alpha$

P_v also contribute to the Resisting moment

$$P_v = P_a \sin\alpha$$

M_v = moment of P_v about C

= P_v B = $P_a \sin\alpha\ B$

Where B = width of the base slab

$$F_0 = \frac{M_1 + M_2 + M_3 + M_4 + M_5 + M_6 + M_v}{P_a \cos\alpha \left(\frac{H'}{3}\right)}$$

F_0 should be between 2 and 3.

Calculation of ΣM_R

Section	Area	Weight/unit length of the wall	Moment arm measured form C	Moment about C
1	A_1	$W_1 = \gamma_1$ x A_1	x_1	M_1
2	A_2	$W_1 = \gamma_2$ x A_2	x_2	M_2
3	A_3	$W_1 = \gamma_3$ x A_3	x_3	M_3
4	A_4	$W_1 = \gamma_4$ x A_4	x_4	M_4
5	A_5	$W_1 = \gamma_5$ x A_5	x_5	M_5
6	A_6	$W_1 = \gamma_6$ x A_6	x_6	M_6
		P_v	B	M_v
		ΣV		ΣM_R

Check for sliding along base

$$F_s = \frac{\sum F_R'}{\sum F_d}$$

$\Sigma F_R'$ = sum of horizontal resisting forces

ΣF_d= sum of horizontal driving forces

Shear strength of the soil immediately below the base slab

$$s = \sigma \tan\delta + C_a$$

δ = angle of friction between the soil & base slab

C_a = adhesion between the soil and base slab

The maximum resisting force that can be derived from the soil per unit length of the wall along the bottom of the base slab is

$$R' = s(area\ of\ cross\ section) = s(B\ x\ 1)$$
$$= B\ \sigma \tan\delta + B\ C_a$$

But Bσ = sum of vertical forces = ΣV

$\therefore R' = (\Sigma V)\tan\delta + B\ C_a$

Passive force P_p is also horizontal resisting force

$$P_p = \frac{1}{2}K_p\gamma_2 D^2 + 2C_2\sqrt{K_p}D$$

$$\therefore \sum F_R = (\Sigma V)\tan\delta + B\ C_a + P_p$$

The only horizontal force that will cause the wall to slide (driving force) is horizontal component of P_a

$$\therefore \sum F_d = P_a \cos i$$

$$F_s = \frac{(\Sigma V)\tan\delta + B\ C_a + P_p}{P_a \cos i}$$

Min Fs = 1.5 ; P_p may be ignored for calculation.

If $F_s < 1.5$, resistance to sliding can be increased by providing a base key.

Fig.

If key is included,

$$P_p = \frac{1}{2}K_p\gamma_2 {D_1}^2 + 2C_2 D_1\sqrt{K_p}$$

$$K_p = \tan^2\left(45 + \frac{\Phi_2}{2}\right)$$

Usually the base key is constructed below the stem and some main steel is run into the key.

Check for bearing capacity failure

Qtoe and qheel are the maximum and minimum pressures occurring at the ends of the toe and heel sections.

The sum of vertical forces acting on the base slab is ΣV (as determined from table for moments) and the horizontal force is $P_a \cos\alpha$

Let R be the Resultant force.

$$R = \Sigma V + P_a \cos\alpha$$

The net moment of the forces about point C is

$$M_{net} = \Sigma M_R - \Sigma M_0$$

$$CE = \bar{x} = \frac{M_{net}}{\Sigma V}$$

$$\therefore e = \frac{B}{2} - \bar{x}$$

$$q = \frac{\Sigma V}{A} \pm \frac{M_{net}\cdot y}{I}$$

M_{net} = moment = $(\Sigma V)\ e$

I = Moment of inertia of base section / unit length = $\frac{1}{12}(1)(B^2)$

For max and min pressures

$$y = \frac{B}{2}$$

$$\therefore q_{max} = q_{toe} = \frac{\Sigma V}{B}\left(1 + \frac{6e}{B}\right)$$

$$q_{min} = q_{heel} = \frac{\Sigma V}{B}\left(1 - \frac{6e}{B}\right)$$

Types of Backfill material for Retaining walls

1. Coarse grained soil without admixture of fine soil particles, very permeable (clean sand or gravel).
2. Coarse grained soil of low permeability due to admixture of particles of silt size.
3. Residual soil with stones, fine silty sand and granular materials with conspicuous clay continent.
4. Very soft clay, organic silts or silty clays.
5. Medium or stiff clay, deposited in chunks and protected in such a way that negligible amount of water enters the spaces between the chunks during floods or heavy rains. If this condition of protection cannot be satisfied, the clay should not be used as backfill material with increasing stiffness of clay, the danger to the wall due to infiltration of water increases rapidly.

Drainage from the Backfill of Retaining walls

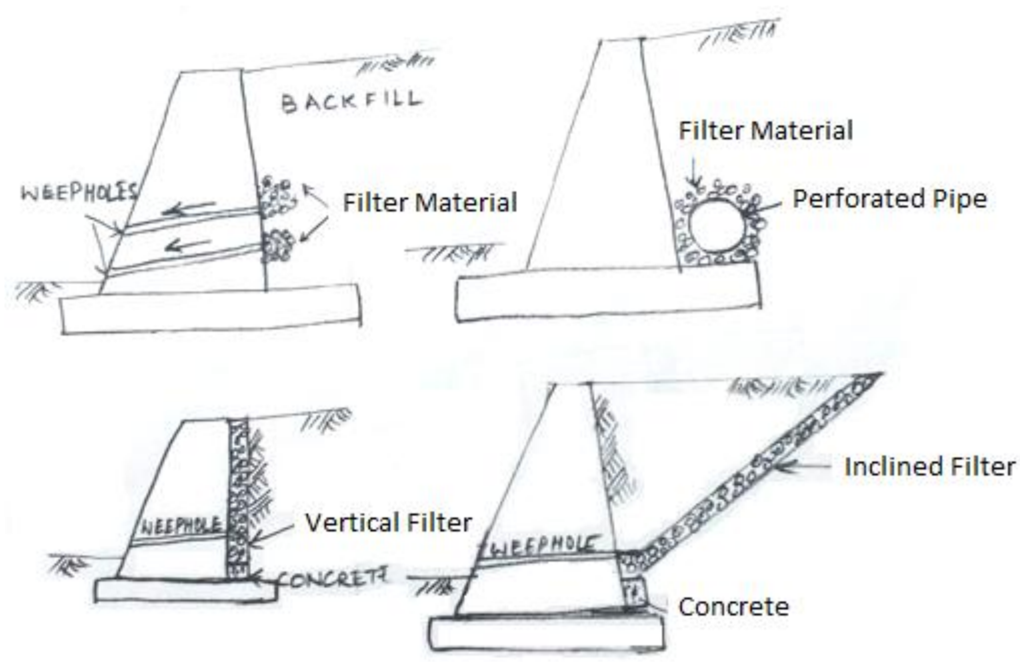
Wherever there is a choice clean granular backfill material should be preferred because the computed active earth pressures are more reliable in such cases. Also, there is less likelyhood of hydrostatic pressure buildup under adequate provision of drainage.

As a result of rainfall or other wet conditions, the backfill material may become saturated. Saturation will increase the pressure on the wall and may create an unstable condition. Adequate drainage must therefore be provided by means of weepholes and/or perforated drainage pipes.

Weepholes are provided in the walls. They are about 0.1 m diameter and spaced at 1.5 m to 3.0 m in the horizontal direction. To prevent the backfill material from getting washed into the weepholes and clogging the weephole, filter material is placed around the weephole. Geotextiles are being increasingly used for this purpose.

Perforated pipes are also frequently used for the drainage of the backfill. These pipes are laid near the base. The water is collected from the backfill and discharged at a suitable place at the ends.

Filter material may be placed around the weepholes or perforated pipes. It may also be provided as a vertical filter immediately behind the retaining wall, or as an inclined filter. Inclined filter is more effective than the vertical filter.



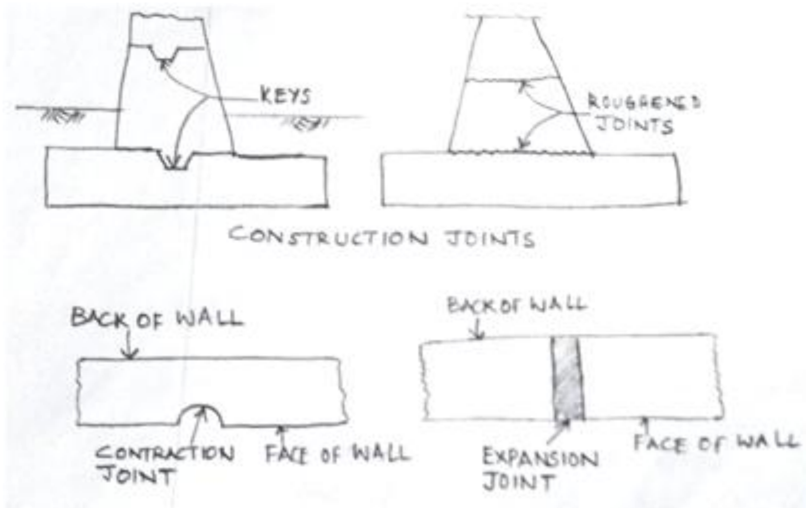
Joints in Retaining wall construction

A retaining wall may be constructed with one or more of the following joints.

1. **Construction joints:** These are vertical and horizontal joints that are placed between two successive pours of concrete. To increase the shear at the joints, keys may be used. If keys are not used, the surface of the first pour is cleaned and roughened before the next pour of concrete.

2. **Contraction joints:** Contraction joints are vertical joints (grooves) placed in the face of a wall, from the top of the base slab to the top of the wall, that allow concrete to shrink without noticeable harm. The grooves may be about 6 to 8 mm wide and 12 to 16 mm deep.

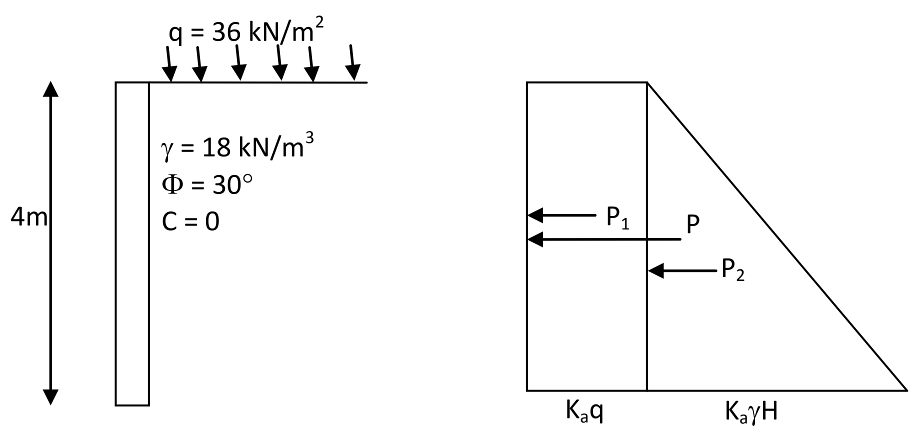
3. **Expansion joints:** Expansion joints allow for the expansion of concrete caused by temperature changes. Vertical expansion joints from the base to the top of the wall may also be used. These joints may be filled with flexible joint filters. In most cases, horizontal reinforcing steel bars running across the stem are continuous through all joints.



Introduction

A **retaining wall** or **retaining structure** is used for maintaining the ground surfaces at different elevations on either side of it. The material retained or supported by the structure is called **backfill** which may have its top surface **horizontal** or **inclined**. The position of the backfill lying above a horizontal plane at the elevation of the top of a wall is called the **surcharge**, and its inclination to the horizontal is called **surcharge angle** β.

Problems

1	Compute the intensities of active and passive earth pressure at depth of 8 m in dry cohesionless soil with Φ = 30° and γ = 18 kN/m^3. What will be intensities of active and passive earth pressure if the water level rises to the ground level. γ_{sat} = 22 kN/m^3
Sol.	(a) Dry soil $$k_a = \frac{1-\sin\Phi}{1+\sin\Phi} = \frac{1-\sin 30}{1+\sin 30} = 0.33$$ $$p_a = k_a\,\gamma H = 0.333\ x\ 18\ x\ 8 = 48\ \text{kN/m}^2$$ $$k_p = \frac{1}{k_a} = 3$$ $$p_p = k_p\,\gamma H = 3\ x\ 18\ x\ 8 = 432\ \text{kN/m}^2$$ (b) Submerged backfill $$\gamma' = \gamma_{sat} - \gamma_w = 22 - 9.81 = 12.19\ \text{kN/m}^3$$ $$p_a = k_a\,\gamma' H + \gamma_w H = 0.333\ x\ 12.19\ x\ 8 + 9.81\ x\ 8 = 110.95\ \text{kN/m}^2$$ $$p_p = k_p\,\gamma' H + \gamma_w H = 3\ x\ 12.19\ x\ 8 + 8\ x\ 9.81 = 371.04\ \text{kN/m}^2$$
2	A retaining wall 4 m high has a smooth vertical back. The backfill has a horizontal surface in level with the top of the wall. There is uniformly distributed surcharge load of 36 kN/m^2 over the backfill. The unit weight of the backfill is 18 kN/m^3, Φ = 30° and c = 0. Determine the magnitude and point of application of active pressure per meter length of the wall.
Sol.	
	$$k_a = \frac{1-\sin\Phi}{1+\sin\Phi} = \frac{1-\sin 30}{1+\sin 30} = 0.333$$ The lateral pressure due to surcharge = k_a.q p_1 = 0.333 x 36 = 12 kN/m^2 The lateral pressure intensity due to backfill at depth z = 4 m is $p_2 = k_a\ \gamma$ H = 0.333 x 18 x 4 = 24 kN/m^2

The total pressure intensity at the base (p_a) = $p_1 + p_2 = 12 + 24 = 36$ kN/m^2

$P_1 = 12 \times 4 = 48$ kN/m

$P_2 = 1/2 \times 24 \times 4 = 48$ kN/m

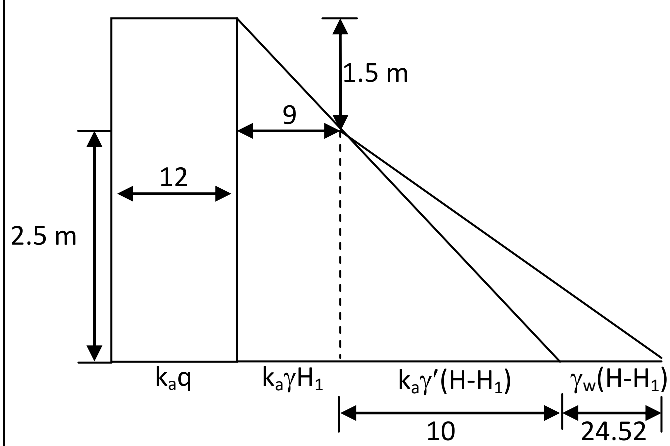
P_1 acts at a height 2 m above the base of wall

P_2 acts at a height H/3 = 4/3 m = 1.33 m above the base.

Taking moments about the base

$$\bar{z} = \frac{48\ x\ 2 + 48\ x\ 1.33}{96} = 1.67\ m$$

(b) if the water table rises behind the wall to 1.5 m below the top, determine the total active pressure and its point of application. Take $\gamma' = 12$ kN/m^3.



$p_1 = k_a.\ q = 0.333 \times 36 = 12$ kN/m^2

$p_2 = k_a\ \gamma\ H_1 = 0.333 \times 18 \times 1.5 = 9$ kN/m^2

p3 = $k_a\ \gamma'\ (H - H_1) = 0.333 \times 12 \times 2.5 = 10$ kN/m^2

p4 = $\gamma_w\ (H - H_1) = 9.81 \times 2.5 = 24.52$ kN/m^2

Thrust per unit length of the wall

P1 = 12 x 4 = 48 kN/m acting at 2 m from the base

P2 = ½ x 9 x 1.5 = 6.75 kN/m acting at 2.5 + 1.5/3 = 3 m from the base.

P3 = 9 x 2.5 = 22.5 kN/m @1.25 m from the base

P4 = ½ x 10 x 2.5 = 12.5 kN/m @ 0.833 from the base

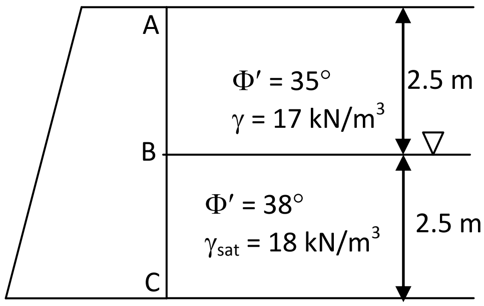
P5 = ½ x 24.52 x 2.5 = 30.65 kN/m @ 0.833 m from the base

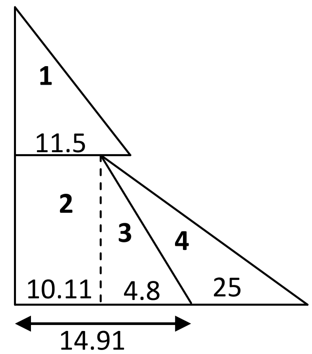
Total thrust (P) = P1 + P2 + P3 + P4 + P5 = 120.4 kN/m

Taking moments about the base

$$\bar{z} = \frac{48\ x\ 2 + 6.75\ x\ 3 + 22.5\ x\ 1.25 + 12.5\ x\ 0.833 + 30.65\ x\ 0.8}{120.4} = 1.5\ m$$

3 Determine the active pressure on the retaining wall shown. Take γ_w = 10 kN/m^3





$$k_{a1} = \frac{1 - \sin \Phi}{1 + \sin \Phi} = \frac{1 - \sin 35}{1 + \sin 35} = 0.271$$

$$k_{a2} = \frac{1 - \sin \Phi}{1 + \sin \Phi} = \frac{1 - \sin 38}{1 + \sin 38} = 0.238$$

At B,

$$\sigma_z' = 17\ x\ 2.5 = 42.5 \text{ kN/m}^2$$

p_a = 0.271 x 42.5 = 11.52 kN/m^2

just below the interface

p_a =0.238 x 42.5 = 10.11 kN/m^2

At C,

$$\sigma_z = 2.5\ x\ 17 + 18\ x\ 2.5 = 87.5 \text{ kN/m}^2$$

$$u = 2.5\ x\ 9.82 = 25 \text{ kN/m}^2$$

$$\sigma_z' = 62.5 \text{ kN/m}^2$$

$$\therefore p_a = 0.238\ x\ 62.5 = 14.9 \text{ kN/m}^2$$

P1 = 14.4 kN/m   P2 = 25.3 kN/m

P3 = 6 kN/m   P4 = 31.3 kN/m

P = 77 kN/m

$$\bar{z} = 1.44\ m$$

Coulomb's wedge theory

Coulomb (1776) presented a theory for active and passive earth pressure against retaining walls.

Assumptions:

1. Coulomb assumed that the failure surface is a plane which passes through the heel of the wall.
2. The backfill is dry, cohesionless, homogeneous, isotropic and ideally plastic material.
3. The wall surface is rough. The resultant earth pressure on the wall is inclined at an angle δ to the normal to the wall, where δ is the angle of the friction between wall and backfill.
4. The sliding wedge itself acts as a rigid body.

The magnitude of earth pressure is obtained by considering the equation of the sliding wedge as whole.

Fig…

Let AB be the back face of retaining wall supporting granular soil, the surface of which is constantly sloping at an angle α with the horizontal.

BC is a trail failure surface.

In the stability consideration of the probable failure wedge ABC, the following forces are involved (per unit length of the wall).

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

Using sine law for the force triangle

$$\frac{W}{\sin(90 + \theta + \delta - \beta + \Phi')} = \frac{P_a}{\sin(\beta - \Phi')} ------ (Eq. 1)$$

$$P_a = \frac{\sin(\beta - \Phi')}{\sin(90 + \theta + \delta - \beta + \Phi')} . W ------ (Eq. 2)$$

The above equation can be written in the form

$$P_a = \frac{1}{2}\,\gamma\, H^2 \left[\frac{\cos(\theta - \beta)\,\cos(\theta - \alpha)\,\sin(\beta - \Phi')}{\cos^2\theta\,\sin(\beta - \alpha)\,\sin(90 + \theta + \delta - \beta + \Phi')}\right] ----(Eq. 3)$$

Where γ = unit weight of soil (backfill)
The values of γ, H, θ, α, Φ' and δ are constants, and β is the only variable.

To determine the critical value of β for maximum P_a,

$$\frac{dP_a}{d\beta} = 0 ----(Eq. 4)$$

After solving the above equation (4) and substituting for β in eq. (3), Coulomb's active earth pressure is obtained as

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

Where K_a = Coulomb's active earth pressure coefficient

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

Note that when $\alpha = 0°$, $\theta = 0°$ and $\delta = 0°$,

Coulomb's active earth pressure coefficient becomes equal to $\frac{1 - \sin\Phi'}{1 + \sin\Phi'}$, which is same as Rankine's earth pressure coefficient.

Graphical solutions for Coulomb's Active Earth Pressure

1. Rebhann's construction for Active pressure

Rebhann (1871) gave a graphical method for the determination of the total active pressure according to Coulmob's theory. It is based on Poncelet's solution, and is therefore known as Poncelet's method.

Fig.

1. The line BD is drawn at angle Φ' to the horizontal.
2. The line BL is drawn at angle $\Psi = 90 - \theta - \delta$ with the line BD. BL is known as the earth pressure line.
3. A semi – circle BMD is drawn on BD as diameter.
4. The line AH is drawn parallel to BL, intersecting the line at BD at H.
5. A perpendicular HM is drawn at H, intersecting the semi-circle at M.
6. With B as center and BM as radius, an arc MF is drawn intersecting BD at F.
7. The line FE is drawn parallel to BL, intersecting the ground surface at E.
8. With F as center and FE as radius, an arc is drawn to intersect BD at N.
9. The line BE represents the critical failure plane.

The total active earth pressure is given by
$P_a = \gamma$ (area of triangle NEF)
$P_a = \gamma$ (1/2 x NF x x)
Where x is the perpendicular distance between E and BD.

Case 1

When ground line and Φ-line are nearly parallel ($\alpha \approx \Phi$)

Fig.

1. Choose any arbitrary point D′ on the Φ-line.
2. Draw D′A′ parallel to ground line.
3. Draw A′E′ parallel to Ψ-line.
4. Draw semi-circle on BD′ as diameter.
5. Erect a perpendicular from E′ to Φ-line to …. The semi-circle at M.
6. With B as center and BM as radius, draw an arc to cut the Φ-line at G′
7. Join A′G′
8. Draw AG parallel to A′G′
9. Through G, draw a line GC parallel to Ψ-line to cut the ground line at C.
10. With G as center and GC as radius, draw an arc to cut the Φ-line at N.

$P_a = \gamma$ (area of ΔGCN)

$P_a = \gamma$ (1/2 x GN x *x*)

Case 2 ($\alpha = \Phi$)

Fig..

1. From any point D′ on the Φ-line, draw D′C parallel to Ψ-line.
2. With D′ as center and D′C as radius, draw an arc to cut the Φ-line at N.

$$P_a = \frac{1}{2}\, ND'.x\,.\gamma$$

<u>Rebhann's method for passive earth pressure</u>

<u>Ψ = 90 - θ + δ</u> (for active Ψ = 90 - θ -δ)

Fig..

1. Draw Φ-line below the horizontal line and produce Φ-line backward to meet ground line at D.
2. Draw Ψ line making an angle Ψ with Φ-line.
3. Draw a semi-circle with BD as diameter,
4. Draw AE parallel to BL.
5. From E, draw a perpendicular EM to Φ-line BD.
6. With B as center and BM as radius, draw an arc to cut the Φ-line at G.
7. Draw GC parallel to Ψ-line (BL)
8. Join BC. BC is the rupture line.
9. With G as center and GC as radius, draw arc to cut Φ-line at N.

$$P_p = \gamma \text{ x area of } \Delta GCN$$