Retaining walls.

- Retaining walls are used to retain earth or other loose materials.
- These walls are commonly constructed in the following cases:

i) In the construction of building basements.
ii) As wing wall or abutment in the bridge construction
iii) In the construction of embankments.

- The material which is retained by retaining wall is called as backfill.
- The sloping backfill is called as inclined surcharge.
- The term surcharge means backfill above the level of top fill of the wall
- The backfill exerts a push or lateral pressure on the retaining wall which tries to overturn, bend and slide retaining wall.

Type of retaining walls.

- The following are the common types of retaining wall

i) Gravity retaining wall
ii) Cantilever retaining wall
iii) Counter fort retaining wall
iv) Buttress retaining wall

Gravity retaining wall.

- A gravity retaining wall is that retaining. w

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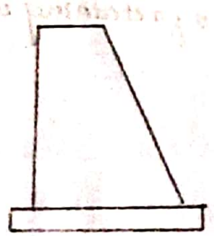
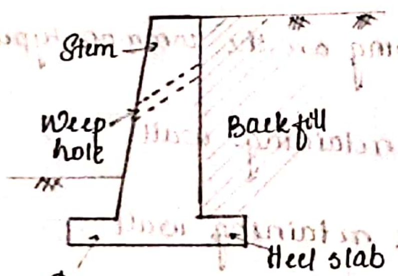
in which the weight of the retaining wall provides stability against pressure exerted by backfill.

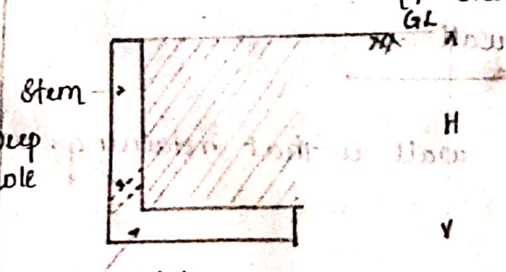
- Gravity retaining walls are made up of massive stone masonry and plain concrete.
- The principle of design of gravity retaining wall is that tension is not developed anywhere in the ~~sect~~ section.
- Therefore the wall is designed on the basis of middle third rule.



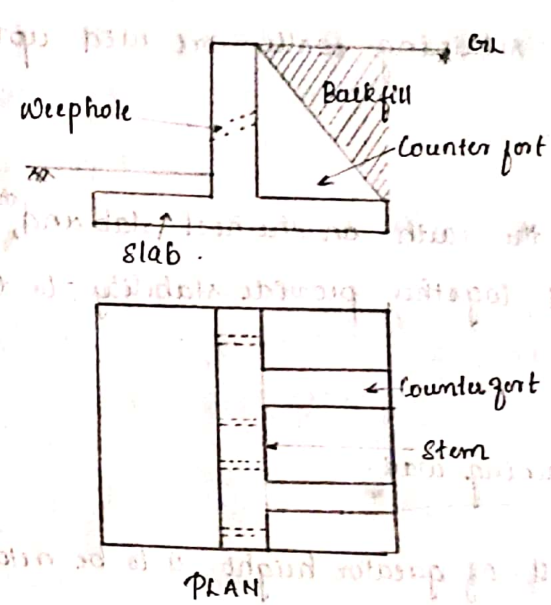
a) Gravity retaining wall.



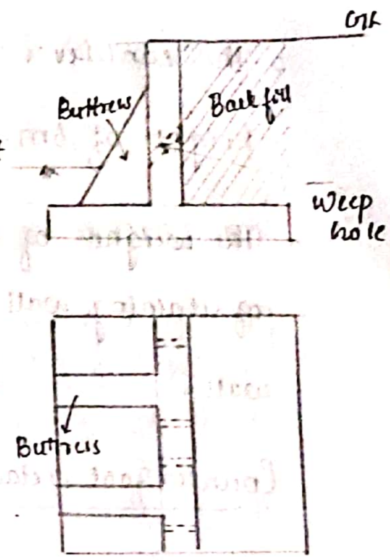
(b) Cantilever retaining wall (T - shape)



(c) Cantilever retaining wall (L-shape)



(d) Counterfort retaining wall



(e) Buttress retaining wall

Cantilever retaining wall:

- It is the most common type of retaining wall which consist of a vertical wall is called as stem, heel slab, toe slab.
- As all the 3 component of this wall act as cantilever, wall is called as cantilever retaining wall.
- The stem, heel and toe all resist the earth pressure by bending.
- These walls can be L or inverted T shaped.

- The cantilever retaining walls are used upto a height of 6m.
- The weight of the earth on the heel slab and the weight of retaining wall together provide stability to the wall.

Counterfort retaining wall.

- When the backfill of greater height is to be retained and the required height of cantilever retaining wall exceeds 6m, then it becomes uneconomical to provide cantilever retaining wall.
- In such cases, counterfort retaining wall is to be provided.
- In these walls, counterforts are provided at some suitable interval along the length of the wall, on the back fill side.
- These counterforts are concealed in the backfill and tie the vertical stem and heel slab together.
- In a counterfort retaining wall, the stem and heel donot act as a cantilever slab but as a continous slab because of the counterfort supports.
- This results in reduction in maximum bending moment and shear force.
- The weight of the retaining wall and wt of earth retaining on heel slab together impart stability to the retaining wall.

Buttress retaining wall.

- A buttress retaining wall is similar to the counterfort retaining wall but the difference that in buttress retaining wall, counterforts called as buttresses are provided on the opposite side of the back fill.
- These buttresses tie the stem and the toe slab together.
- These buttresses are designed as compression members and hence economical but still not preffered.
- It is because counterforts are concealed but buttresses are visible and they occupy the space in front of

the wall which could have been used for some other purposes.

Earth pressure on retaining wall.

- The main force that acts on retaining wall is the lateral force developed earth pressure or of pressure due to retained material.

- This force tends destabilize the retaining wall by overturning, bending and sliding the wall.

- The determination of this earth pressure is done by using principles of soil mechanics.

- The magnitude of the lateral earth pressure varies linearly with depth.

$p\phi = K\gamma H$

where,

$p\phi$ = lateral earth pressure which can be active or passive.

- Active earth pressure P_a is exerted on the wall when the wall has a tendency to move away from the backfill by passive earth pressure P_p is exerted on the wall when the wall has a tendency to move towards the backfill.

γ = unit weight of retained material

H = depth of retained material below the earth surface.

K = coefficient of earth pressure which is determined by using either by coulomb's theory or Rankine's theory of earth pressure.

- It is written as K_a for coefficient of active earth pressure and K_p for coefficient of passive earth pressure.

a) For horizontal backfill of cohesionless soil $K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$

where ϕ is the angle of internal friction or angle of repose.

b) For sloping backfill, $K_a = \left[\frac{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}\right] \times \cos\theta$

where θ is the angle of inclination of backfill with respect to the horizontal.

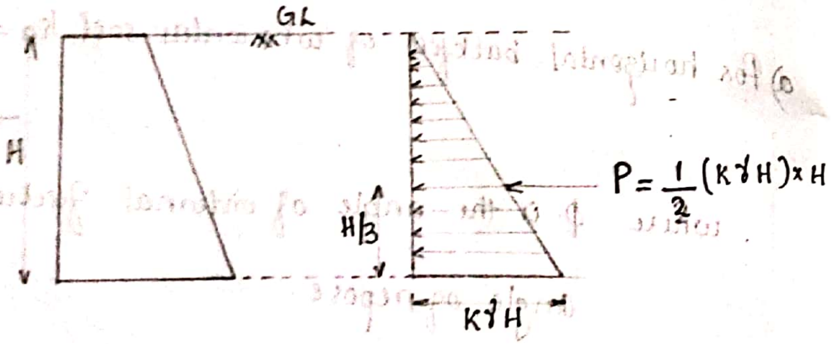
Passive earth pressure.

a) For horizontal back fill,

$$K_p = \frac{1 + \sin\phi}{1 - \sin\phi}$$

b) For sloping back fill

$$K_p = \left[\frac{\cos\theta + \sqrt{\cos^2\theta - \cos^2\phi}}{\cos\theta - \sqrt{\cos^2\theta - \cos^2\phi}}\right] \times \cos\theta$$

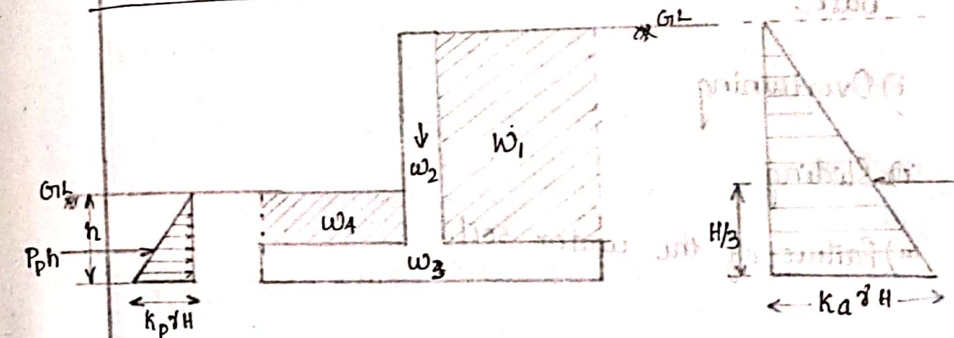


- Based on the above formula, the lateral earth pressure distribution can be plotted along the depth which gives maximum value as $K_a \gamma H$ as the bottom of the retaining wall.
- Horizontal pressure acting on the retaining wall $P_h = \frac{1}{2}(K_a \gamma H)H$

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

- The lateral force acts at a height $\frac{H}{3}$ from the base of the wall.

Forces on a cantilever retaining wall.



Consider a cantilever retaining wall, the various forces acting on the wall are as follows:

i) Lateral force P_{ah} due to active earth pressure acting at a height $\frac{H}{3}$ from the base.

ii) Weight of earth supported on heel slab (W_1)

iii) Weight of the stem

iv) Weight of the base slab W_3.

v) Weight of earth supported on toe slab W_4.

Stability of a cantilever retaining wall.

- A cantilever retaining wall may fail in the following base :

i) Overturning

ii) Sliding

iii) Failure of the under soil.

i) Overturning.

- A retaining wall is subjected to overturning moments under the action of lateral force developed due to lateral earth pressure, which tries to overturn the wall about the toe end.

- The overturning moment M_o is given as M_o -

$$M_o = P_{ah} \times \frac{H}{3}$$

ie,

$$M_o = \frac{1}{2} k_a \gamma H \times H \times \frac{H}{3}$$

$$M_o = \frac{1}{6} k_a \gamma H^3$$

- The resisting moment M_R is provided by the weight of backfill, surcharge and self weight of the retaining wall if ΣW is the resultant vertical load made up of selfweight of retaining and the weight of backfill on the base slab, then resisting moment is given as

$$M_R = \Sigma W \times \bar{x}$$

where $\bar{x}$ is the position of the resultant vertical load (ΣW from the toe end)

- As per code IS 456-2000 Cl: 20.1, the stability of the retaining wall against overturning should be ensured so that the resisting moment is not less than

1.4 times the maximum overturning moment due to characteristic imposed load (the lateral earth pressure in the case of retaining wall)

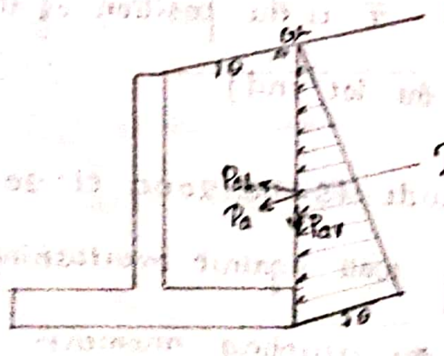
- If the dead load provides the resisting moment then as per code, only 0.9 times the characteristic dead load should be taken into consideration.

- Therefore $f_{s_1} = \frac{0.9 M_R}{M_o}$

f_{s_1} = factor of safety, $f_{s_1} \geq 1.4$

$$\therefore f_{s_1} = \frac{0.9(\Sigma W\bar{x})}{\left(\frac{K_a \gamma H^3}{6}\right)} \geq 1.4$$

iii) Sliding.



ii) Sliding.

- The lateral earth pressure tries to slide the retaining wall away from the back fill.

- This is opposed by the frictional force developed b/w the base slab and soil.

- If μ is the coefficient of friction between the concrete and soil, then the frictional force resisting the sliding is given as $F_R = \mu \Sigma W$

- The lateral force causing the sliding is P_{ah}

$$P_{ah} = \frac{K_a \gamma H^2}{2} = F_S$$

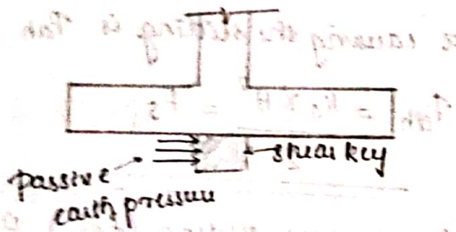
- Factor of safety against sliding (f_{s_2}) is given as

$$f_{s_2} = \frac{F_R}{F_S} = \frac{\mu \Sigma W}{\frac{K_a \gamma H^2}{2}} = \frac{\mu \Sigma W}{P_{ah}}$$

- As per IS456-2000, the minimum factor of safety of 1.4 is to be ensured against sliding and only 0.9 times the characteristic ~~dead to~~ dead load is to be considered for resisting force.

$$\therefore F_s = \frac{0.9\,\mu\,\Sigma W}{P_{ah}} \geq 1.4$$

- If the factor of safety against sliding comes out be less than 1.4, then a shear key may be provided.
- This shear key increases the resistance against sliding as the passive head pressure developed on the shear key provides additional resistance against sliding.



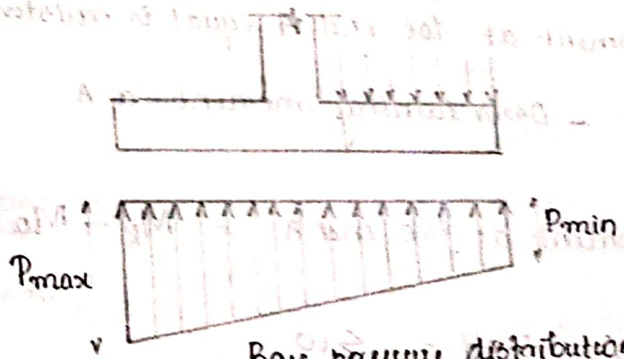
(ii) Failure of the under soil

- The base width of the retaining wall is designed in such a way that the maximum pressure on the under soil caused due to load distribution must not exceed the safe bearing capacity of soil.
- In addition to that it is to be ensured that no tension is developed anywhere on the section.

i.e., the resultant load must fall in the middle third zone (as per the middle third rule) so that negative pressure (tension) is not developed anywhere.

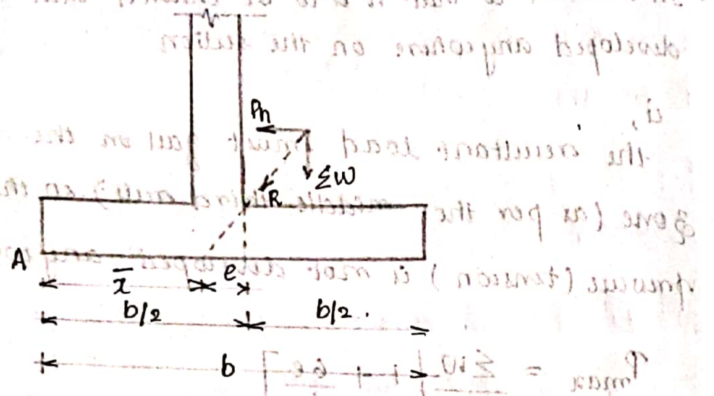
$$P_{max} = \frac{\Sigma W}{b}\left[1 + \frac{6e}{b}\right]$$

$$P_{min} = \frac{\Sigma W}{b}\left[1 - \frac{6e}{b}\right]$$



Base pressure distribution.

- The maximum pressure at the base i.e., P_{max} should be less than the safe bearing capacity of soil.
- The minimum pressure i.e., P_{min} should not be negative.



- Here e is the eccentricity of the resultant load and can be obtained as below:
- Total moment at toe end A equal to resisting moment above A − Overturning moment a A.

- Total moment at toe end A $= M_R - M_O$.
- Total vertical load $= \Sigma W$

$$\bar{x} = \frac{M_R - M_O}{\Sigma W}$$

- Eccentricity $e = \frac{b}{2} - \bar{x}$

Proportioning of the cantilever retaining wall.

(i) Depth of foundation.

- The minimum depth of foundation is determined on the basis of Rankine's formula.

$$h_{min} = \left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)^2 \frac{q_0}{\gamma}$$

where h_{min} = depth of foundation below the earth surface
q_0 = safe bearing capacity of soil
γ = unit weight of the soil.
ϕ = angle of internal friction / angle of repose.

ii) Height of the retaining wall (H).

- The height of the material to be retained (h) is given.
- The depth of foundation is added to the height of the material to be retained, to get the total height of the retaining wall H

$$H = h + h_{min}.$$

iii) Base width (b)

- The width of the base slab can be determined by considering the equilibrium of various forces at the base

- Based on exact analysis and experience, it is found that the base width (b) varies from 0.4H to 0.6H.

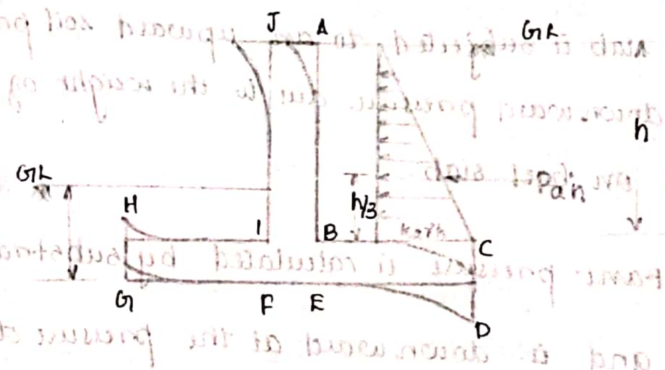
iv) **Thickness of base slab.**

- The thickness of base slab is assumed to be $\frac{H}{10}$ to $\frac{H}{15}$, where H is the total height of the retaining wall.
- The minimum thickness of base slab should not be less than 300mm.
- The thickness assumed should be checked from bending moment and shear force requirements.

v) **Thickness of stem.**

- The thickness of vertical stem or wall is governed by the bending moment criteria.
- As the stem behave like a cantilever, subjected to lateral pressure which is increasing with depth, it is economical to have a trapezoidal section of the stem with minimum thickness of 150mm at top.
- The thickness at the base of stem should not be less than 300mm.

Structural behaviour and design of cantilever retaining wall.



- A cantilever retaining wall subjected to a lateral force Pah.

i) **Stem**

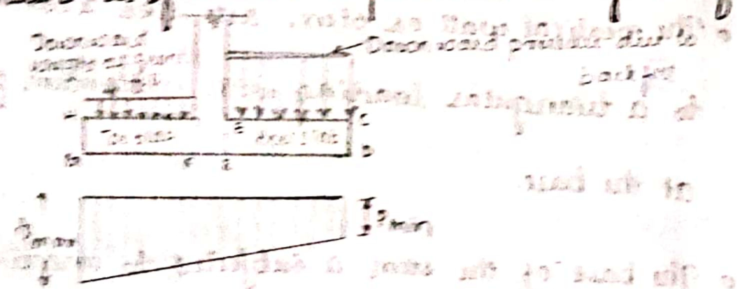
- The vertical wall or stem acts like a cantilever subjected to a triangular loading with maximum pressure developed at the base.
- The base of the stem is subjected to maximum bending moment (M_B)

$$M_B = \frac{1}{2} \times k_a \gamma h \times h \times \frac{h}{3}$$

$$M_B = \frac{k_a \gamma h^3}{6}$$

ii) Heel slab

- The heel slab is subjected to an upward soil pressure and a downward pressure due to the weight of backfill supported on heel slab.
- The resultant pressure is calculated by substracting these 2 and is downward as the pressure due to wt of backfill is more than the upward soil pressure.
- This causes tension on top face on.
- Hence main reinforcement is provided along this face.



Toe slab

- The toe slab is also subjected to an upward soil pressure and downward pressure due to the wt of front-fill supported on the toe slab.
- The weight of the front fill is very small and hence neglected so the resultant pressure on the toe slab is upward which causes tension on the bottom face of the toe slab.
- Hence main reinforcement is put along this face.

1 Design a cantilever retaining wall to retain horizontal earthen embankment of height 4m above the ground level. The earthen backfill is having a density of 18 kN/m³ and angle of internal friction as 30°. The safe bearing capacity of soil is 180 kN/m³. The coefficient of friction between soil and concrete is assumed to be 0.45. Use M20 concrete and Fe 415 Steel.

Given,

$f_{ck} = 20 N/mm^2$

$f_y = 415 N/mm^2$

$\gamma = 18 kN/m^3$

$\phi = 30°$

Safe bearing capacity $q_0 = 180\,kN/m^2$

$\mu = 0.45$

Ht of earthen embankment = 4m

Coefficient of active earth pressure $= K_a = \frac{1 - \sin\phi}{1 + \sin\phi}$

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

Minimum depth of foundation $(h_{min}) = \frac{q_0}{\gamma}\left(\frac{1 - \sin\phi}{1 + \sin\phi}\right)^2$

$= \frac{180}{18}\left(\frac{1 - \sin 30}{1 + \sin 30}\right)^2 = 1.11m \approx 1.2m$

∴ Providing total depth of foundation as 1.2m

Total height of retaining wall = depth of found^n + ht of embankment

$= 1.2 + 4 = 5.2m$

Preliminary dimensions of the retaining wall

i) Base width (b)

It varies from 0.4 H to 0.6 H

Assuming $b = 2.8m$

$0.4H = 0.4 \times 5.2 = 2.08m$
$0.6H = 0.6 \times 5.2 = 3.12m$ } $b = 2.8m$ is in b/w them.

→ Length of toe slab = 0.3b − 0.4b

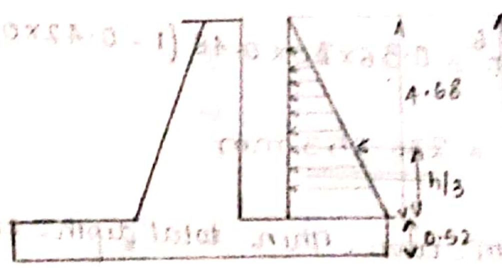
$L = 850mm$

$0.3b = 0.3 \times 2800 = 840mm$
$0.4b = 0.4 \times 2800 = 1120mm$ } 850 is in b/w these values.

ii) Thickness of base slab.

Thickness of base slab is assumed to be $\frac{H}{10} = \frac{5.2 \times 10^3}{10} = 520mm$

Thickness of vertical wall or stem $= 5.2 - 0.52 = 4.68m$.



Pressure at the base of the stem $= K_a \gamma h$

$= \frac{1}{3} \times 18 \times 4.68$

$= 28.08\,kN/m^2$

Moment at the base of the stem $= \frac{1}{2} \times K_a \gamma h \times h \times \frac{h}{3}$

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 4.68^2 \times \frac{4.68}{3}$

$= 102.5$ kNm.

Ultimate moment at the base of the stem $= 1.5 \times 102.5$

$= 153.75$ kNm.

Minimum depth required for a balanced section.

Here consider $M_u = M_{ulim}$

~~$M_u = 153.75 \times 10$~~

$M_{ulim} = 0.36 f_{ck} \frac{x_{umax}}{d} \left(1 - 0.42 \frac{x_{umax}}{d}\right) bd^2$

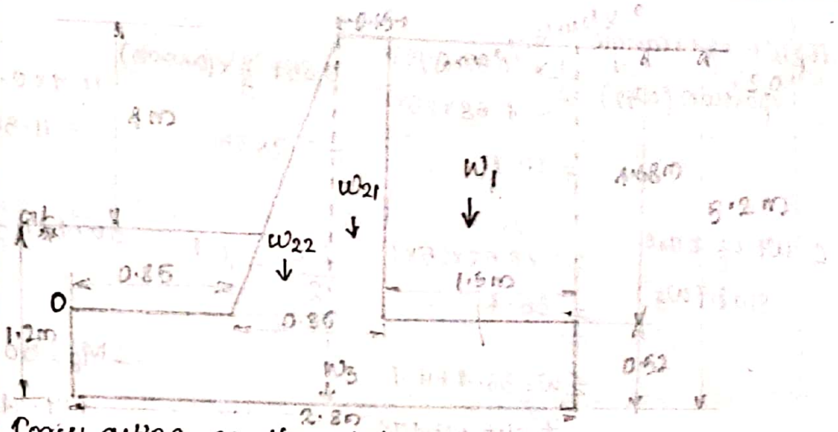
$153.75 \times 10^6 = 0.36 \times 20 \times 0.48 \, (1 - 0.42 \times 0.48) \times 1000 d^2$

$d = 236.053$ mm

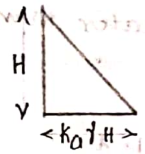
Assuming 60mm cover. Then total depth required

$D = 236.05 + 60 = 296.05$ mm

Hence taking $D = 350$mm at base of the stem and reducing it to 150mm at top



Forces acting on the retaining wall.

	Type of force	Magnitude of force (kN)	Position of force from toe end - O (m)	Bending moment at toe end - O (kNm)
1	Overturning force $P_{ah} = \left(\frac{1}{2} K_a \gamma H\right) H$ 	$\frac{1}{2} \times \left(\frac{1}{3} \times 18 \times 5.2\right) \times 5.2$ $= 81.12$	$\frac{H}{3} = \frac{5.2}{3} = 1.733$	$81.12 \times 1.733 = 140.61$ $\Sigma M_0 = 140.61$
2	Resisting forces a) Wt of backfill (W_1)	$1.6 \times 4.68 \times 18 \times 1$ $= 134.784$	$2.8 - \frac{1.6}{2} = 2$	$134.784 \times 2 = 269.56$ ~~ΣM~~
	b) Wt of stem i) Wt of rectangular portion of stem (W_{21})	(unit wt of concrete) $0.15 \times 4.68 \times 25 \times 1$ $= 17.55$	$0.85 + 0.35 - \frac{0.15}{2}$ $= 1.125$	17.55×1.125 $= 19.743$

ii)	Wt of triangle portion (W_{22})	$\frac{1}{2}\times(0.35-0.15)\times 4.68\times25\times1$ = 11.7	$0.85+\frac{2}{3}\times(0.35-0.15)$ = 0.9833	11.7×0.9833 = 11.505
c	Wt of base slab (W_3)	$2.8\times0.52\times25\times1$ = 36.4	$\frac{2.8}{2}$ = 1.4	$36.4\times1.4 = 50.96$
		$\Sigma W = 36.4+11.7+17.5+134.78$ = 200.384		$\Sigma M_R = 50.96+11.505+19.743+269.56$ = 351.768

Stability checks.

i) Overturning

$F_{S1} = \frac{0.9 M_R}{M_0}$ should be greater than 1.4

$= \frac{0.9\times351.768}{140.61} = 2.25 > 1.4$

Hence Ok.

ii) Sliding.

$F_{S2} = \frac{0.9 F_R}{F_S} \geq 1.4$

$F_R = \mu\Sigma W$

$= 0.45\times200.384 = 90.1728\ kN$

$F_S = P_{ah} = 81.12\ kN$

$F_{S2} = \frac{0.9\times90.1728}{81.12} = 1 < 1.4$

Hence shear key is to be ~~pro~~ provided to increase the resistance against sliding.

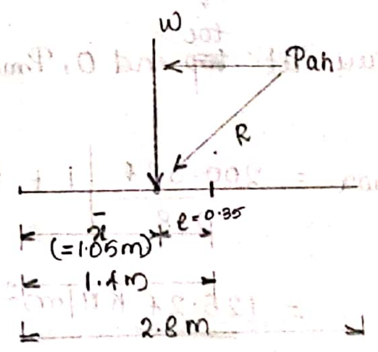
iii) Base pressure.

Resultant moment at toe end O $= M_R - M_0$

$= 351.768 - 140.61$

$= 211.158\ kNm$.

The resultant vertical load $= \Sigma W = 200.384\ kN$.



- The resultant vertical load (ΣW) acts at a distance from the toe end O.

$$\bar{x} = \frac{\text{Resultant moment}}{\Sigma W} = \frac{M_R - M_O}{\Sigma W}$$

$$= \frac{211.158}{200.384} = \underline{1.053\ m}.$$

Eccentricity $e = \frac{b}{2} - \bar{x}$

$$= \frac{2.8}{2} - 1.05 = \underline{0.346\ m} \approx \underline{0.35\ m}$$

e lies in the middle third zone

i.e, $\frac{b}{6}$ from center. ($\frac{2.8}{6} = 0.466$)

Hence it is safe. {If $e > \frac{b}{6}$, it creates tension}

Maximum pressure at toe end O, $P_{max} = \frac{\Sigma W}{b}\left[1 + \frac{6e}{b}\right]$

$$P_{max} = \frac{200.384}{2.8}\left[1 + \frac{6 \times 0.35}{2.8}\right]$$

$= \underline{125.24\ kN/m^2} < q_0\ (180\ kN/m^2)$ safe bearing capacity of soil.

Hence it is safe.

Minimum pressure at heel end O, $P_{min} = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right)$

$$P_{min} = \frac{200.384}{2.8}\left(1 - \frac{6 \times 0.35}{2.8}\right)$$

$= \underline{17.89\ kN/m^2}$, which is positive

Hence ok, as no tension develops anywhere in the base slab.

Design of stem.

- The depth required for stem is already checked while assuming the preliminary dimension.

$D = 350\ mm$.

$d = 350 - 60 = 290\ mm$.

- Maximum moment at the base of stem = $\underline{153.75\ kNm}$
- Area of steel (A_{st}) in stem

$$M_u = M_{ulim} = 0.87 f_y A_{st} d\left(1 - \frac{f_y A_{st}}{b d f_{ck}}\right)$$

$$153.75 \times 10^6 = 0.87 \times 415 \times A_{st} \times 290\left(1 - \frac{415 \times A_{st}}{290 \times 20 \times 1000}\right)$$

(as we consid for per meter length)

$$= 104704.5\ A_{st} - 7.49\ A_{st}^2$$

$A_{st} = \underline{1667.33}\ mm^2$

Using 16mm diameter bars spacing required

$$\text{Spacing} = \frac{A_\phi \times 1000}{A_{st}} = \frac{\frac{\pi}{4} \times (16)^2 \times 1000}{1667.33}$$

$= 120.5\underline{89}\ mm \approx \underline{110}$

Hence provide 16mm diameter bars ~~120~~ 110 mm c/c.

Distribution steel.

Distribution steel is provided @ 0.12 % of total cross sectio-nal area ($\frac{150+350}{2}$ is the average thickness of the stem)

$Ast_{min} = \frac{0.12}{100} \times \frac{Ast}{bd}$ $\quad$ $\% Ast = \frac{100 Ast}{bd}$

$= \frac{0.12}{100}$

$Ast_{min} = \frac{0.12 \times bd}{100}$

$= \frac{0.12 \times 1000 \times ~~250~~ \left(\frac{150+350}{2}\right)}{100}$

$= 390 mm^2$

Using 8mm diameter bars spacing required

$Spacing = \frac{A\phi \times 1000}{Ast}$

~~= 167.55mm~~ $= \frac{\frac{\pi}{4}\times(8)^2 \times 1000}{390}$

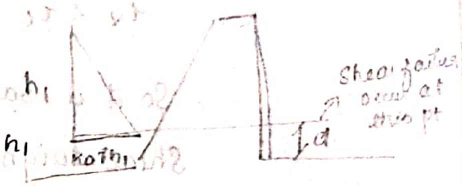
$= 128.885 mm$

Hence provide 8mm diameter bars 120mm c/c on the inner face of the stem as distribution steel.

Similarly provide 8mm ϕ bars at 120mm c/c at the outer face (front face) of the stem as temperature & shrinkage reinforcement since the space is exposed to weather).

Check for shear.

The critical section for shear is at a distance d from base of stem



i,

$1.68 - 0.29 = 1.39 m = h_1$

Shear force at this section $= \frac{1}{2} \times k_a \gamma h_1 \times h_1$

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 1.39 \times 1.39$

$= 5.7.8163 KN$

Factored shear force $V_u = 57.81 \times 1.5 = 86.72 KN$

Nominal shear stress $\tau_v = \frac{V_u}{bd} = \frac{86.72 \times 10^3}{1000 \times 290}$

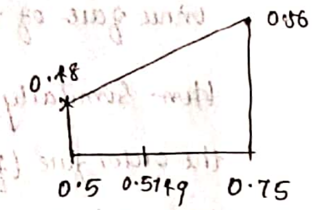
$= 0.299 N/mm^2$

For $P_t = \frac{100 Ast}{bd} = \frac{100 \times 1667.33}{1000 \times 290} = 0.5749 N/mm^2$

τ_c =

0.50 – 0.48

0.75 – 0.56.



$0.75 - 0.5 = 0.56 - 0.48$

$0.5149 - 0.5 = x$.

$\tau_c = 0.5039\ N/mm^2$ $(0.48 + x)$

$\tau_v < \tau_c$

So it is safe in shear.

Shear design is not necessary.

Curtailment of tension reinforcement (as it is a cantilever)

- As the stem of retaining wall behaves like a cantilever [g]oeson reducing towards the top of the wall and becomes zero at the top.
- Therefore tension reinforcement can be curtailed along the height of the stem.

Development length l_d for 16mm diameter bars

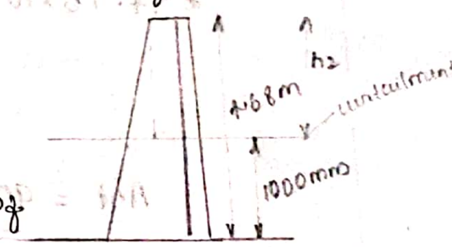
$$l_d = \frac{0.87 \phi f_y}{4 \tau_{bd}}.$$

$\tau_{bd} = 1.2 \times 1.6 = 1.92\ N/mm^2$ (1.6 is used as Fe415 is used)

$$l_d = \frac{0.87 \times 16 \times 415}{4 \times 1.92} = 752.18\ mm.$$

∴ No bar can be curtailed up to a distance of 752mm from base of the stem.

Curtailing bar at a distance 1000mm from base of the stem.



$h_2 = 4680 - 1000$

$= 3680mm$ from top of the stem

Total depth at the section $= 150 + \frac{200 \times 3680}{4680}$ } → by interpolation

$= 307.26\ mm$

Effective depth at the section $= 307.26 - 60$

$= 247.265\ mm$

The moment due to earth pressure at 3.68m from top $= \frac{K_a \gamma h_2^3}{6}$

$= \frac{\frac{1}{3} \times 18 \times 3.68^3}{6} = 49.836$ KNm.

$M_u = 1.5 \times 49.836 = 74.75$ KNm

Area of steel required for an ultimate BM of 74.75KNm,

$M_u = M_{ulim} = 0.87 f_y A_{st} \left(1 - \frac{f_y A_{st}}{b d f_{ck}}\right) d$

$74.75 \times 10^6 = 0.87 \times 415 A_{st} \left(1 - \frac{415 A_{st}}{1000 \times 20 \times 247.265}\right) 247.265$

$= 89275.028 A_{st} - 7.49 A_{st}^2$

$A_{st} = 906.216$ mm^2

Using 16mm diameter bars spacing required

$\text{Spacing} = \frac{A_\phi \times 1000}{A_{st}}$

$= \frac{\frac{\pi}{4} \times (16)^2 \times 1000}{906.216} = 221.869$ mm

Hence, half of the bars can be curtailed but as per IS code 12ϕ or d distance, which ever is more is to be provide beyond the point of curtailment.

$12\phi = 12 \times 16 = 192$ mm.

$d = 247.265$ mm

Hence curtailment the bars at 1.3m from the base or 3.38m from top of stem.

Thus providing 16mm diameter bars @ 220mm c/c after a distance of 1.3m from base of stem.

Similarly one more curtailment can be done at 1.5m from top of stem.

Moment at the section $= \frac{K_a \gamma h^3}{6}$

$= \frac{\frac{1}{3} \times 18 \times 1.5^3}{6} = 3.375$ KNm.

$M_u = 1.5 \times 3.375 = 5.0625$ KNm

Depth at the section.

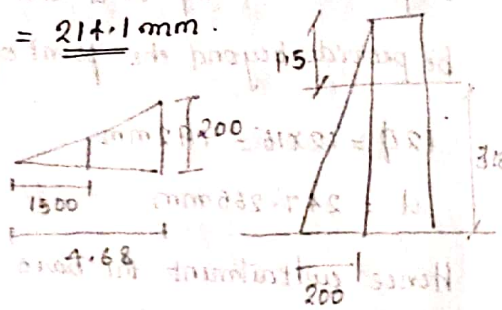
~~$M_u = M_{ulim} = 0.36 f_{ck} b d^2 \left(1 - \frac{x_{umax} \times 0.42}{d}\right) \frac{x_{umax}}{d}$~~

~~5.0625~~

$D = \frac{150 + 1500 \times 200}{4680} = 214.1$

by interpolation

$D = 150 + \frac{1600 \times 200}{4680} = \underline{214.1\,mm}$.



$d = 214.1 - 60$

$= \underline{154.1\,mm}$

Ast required,

$Mu = 0.87 fy\, Ast\, d \left[1 - \frac{fy\, Ast}{bd\, fck} \right]$

$5.0625 \times 10^6 = 0.87 \times 415 \times 154.1 \times Ast \left[1 - \frac{415\, Ast}{1000 \times 154.1 \times 20} \right]$

$= 55637.805\, Ast - 7.49\, Ast^2$

$Ast = \underline{92.133\ mm^2}$

$Ast_{min} = \frac{0.12}{100} \times 1000 \times 154.1 = \underline{184.92\ mm^2}$

$Ast_{min} > Ast$

So provide $Ast_{min} = 184.92\,mm^2$

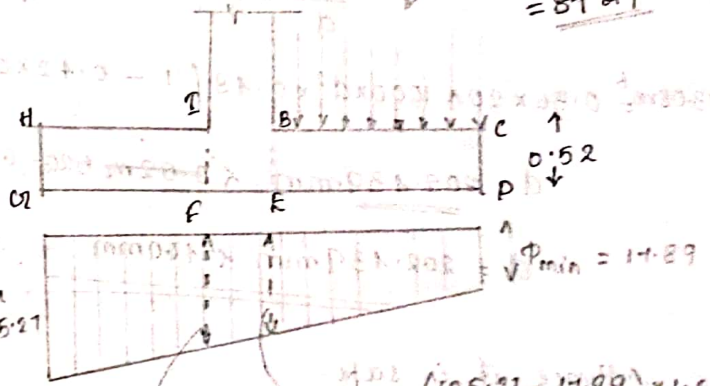
Spacing for Ast_{min},

$Spacing = \frac{A\phi \times 1000}{Ast} = \frac{\frac{\pi}{4} \times (16)^2 \times 1000}{184.92}$

$= \underline{1087.29\,mm}$.

Hence curtailing another half of the bars at 1.5m from top and providing 16mm diameter bars @ 440mm c/c.

Design of heel slab.



$= 84.24\,kN/m$

$17.89 + \left(\frac{125.21 - 17.89}{2.8} \right) \times 1.6 = \underline{79.25\,kN}$

$17.89 + \left(\frac{125.21 - 17.89}{2.8} \right) \times 1.95 = 92.6725\,kN/m^2$

Wt of earth supported on heel $= 18 \times 4.68 \times 1 = \underline{84.24\ kN/m}$

Self weight of heel slab $= 0.52 \times 1 \times 25 = \underline{13\ kN/m}$.

Total load $= 84.24 + 13 = \underline{97.24\,kN/m}$

Maximum BM at B $= 97.24 \times 1.6 \times \frac{1.6}{2} - \frac{17.89 \times 1.6 \times 1.6}{2}$

$- \frac{1}{2}(79.25 - 17.89) \times \frac{1.6 \times 1.6}{3}$

$= \underline{75.38\ kNm}$

$M_u = 1.5 \times 75.38 = 113.0816$ kNm.

Depth required.

Here consider $M_u = M_{u\,lim}$

$$M_u = 0.36 f_{ck}\, b d^2 \frac{x_{umax}}{d}\left(1 - 0.42\frac{x_{umax}}{d}\right)$$

$$113.08\times10^6 = 0.36\times20\times1000\times d^2\times0.48\,(1 - 0.42\times0.48)$$

$d = 202.439$ mm. < ~~0.52m~~ $520 - 60 = 460$ mm.

$d = 202.439\text{ mm} < 460\text{ mm}$

Hence it is safe.

Area of steel for heel slab.

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

$$113.0816\times10^6 = 0.87\times415\times A_{st}\times460\left[1 - \frac{415 A_{st}}{1000\times460\times20}\right]$$

$= 166083\, A_{st} - 7.4917\, A_{st}^2$

$A_{st} = 703.178\text{ mm}^2$

∴ Spacing of 12mm bars

$$\text{Spacing} = \frac{A_\phi \times 1000}{A_{st}} = \frac{\frac{\pi}{4}\times(12)^2\times1000}{703.178}$$

$= 160.83$ mm

∴ Provide 12mm diameter bars at the rate 160mm c/c.

at the top face of the heel slab. i.e, BC.

Distribution steel is provided at the rate 0.12 % of cross sectional area ~~at t~~ in the other direction.

$$A_{st\,min} = \frac{0.12 \times bD}{100}$$

$$= \frac{0.12\times1000\times520}{100} = 624\text{ mm}^2$$

Using 10mm diameter bars,

$$\text{Spacing} = \frac{A_\phi\times1000}{A_{st}} = \frac{\frac{\pi}{4}\times(10)^2\times1000}{624}$$

$= 125.86\text{ mm} \simeq 125\text{ mm}$

Hence provide 10mm ϕ bars @ 125mm c/c in the other direction.

Design of toe slab.

- The weight of front fill above the slab is neglected and the maximum moment is calculated at the face of the stem.

Maximum moment at I = $(0.52 \times 1 \times 25) \times 0.85 \times \frac{0.85}{2} - 92.67 \times 0.85 \times \frac{0.85}{2} - \left[\frac{1}{2} \times (125.27 - 92.67) \times 0.85 \times \frac{2}{3} \times 0.85\right]$

= -36.63 kNm. (−ve show opp direct^n)

Factored moment = 36.63×1.5 = 54.945 kNm.

Area of steel for toe slab,

$M_{u\,lim} = 0.87 f_y A_{st} d \left(1 - \frac{f_y A_{st}}{b d f_{ck}}\right)$

$54.945 \times 10^6 = 0.87 \times 415 \times A_{st} \times 460 \left(1 - \frac{415 A_{st}}{1000 \times 460 \times 20}\right)$

$= 166083 A_{st} - 7.4917 A_{st}^2$

$A_{st} = 335.918\ mm^2 < A_{st\,min}\ (624\ mm^2)$

$A_{st\,min} = 624\ mm^2$

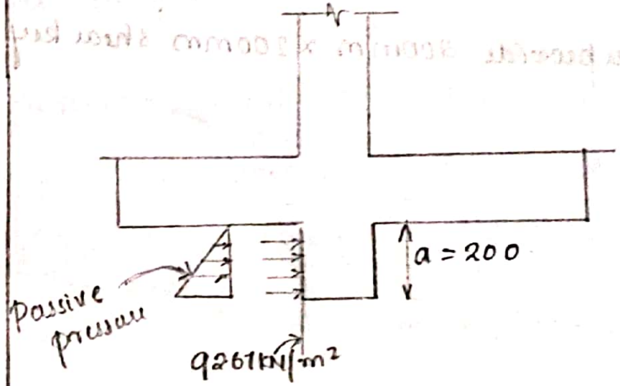
Hence providing minimum area of steel of $624\ mm^2$.

∴ Provide 10mm diameter @ 125mm c/c in both direction

Design of shear key.

- As the wall is not safe in sliding, shear key is to be provided.

- Pressure at face of shear key = $92.67\ kN/m^2$.



Coefficient of passive earth pressure (k_p) = $\frac{1 + \sin\phi}{1 - \sin\phi}$ = 3

Let the depth of key = a

Resistance offered by shear key = $k_p \times P_p \times a = 3 \times 92.67 \times a$

= 278.01a

Factor of safety against sliding along with shear key

$= \frac{0.9 \mu \Sigma W + 278.01a}{Pah} = 1.4$

$= \frac{0.9 \times 0.45 \times 200.43 + 278.01a}{81.12} = 1.4$

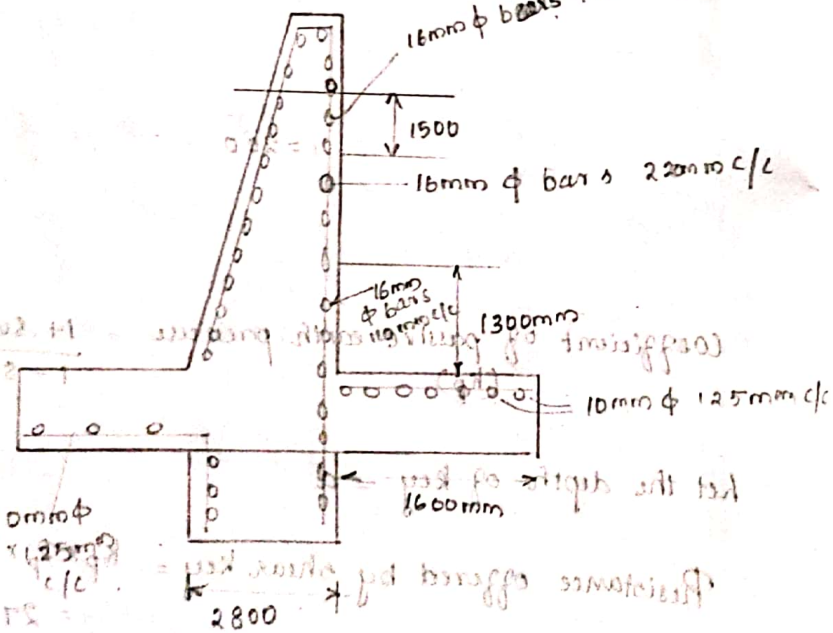
$1 + 3.4271a = 1.4$

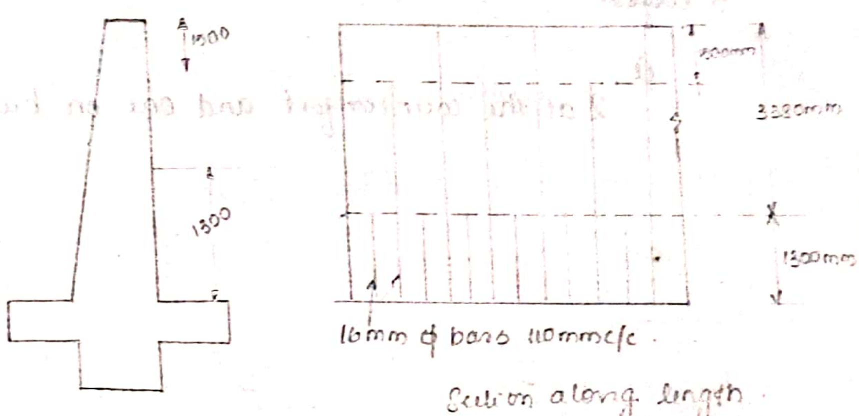
$3.4271a = 0.4$

$a = 0.116m$

$a = .116 m \approx 200mm$

However provide 200mm x 200mm shear key.



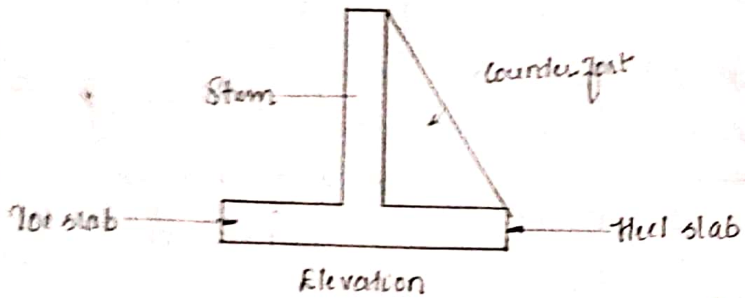


Counterfort retaining wall.

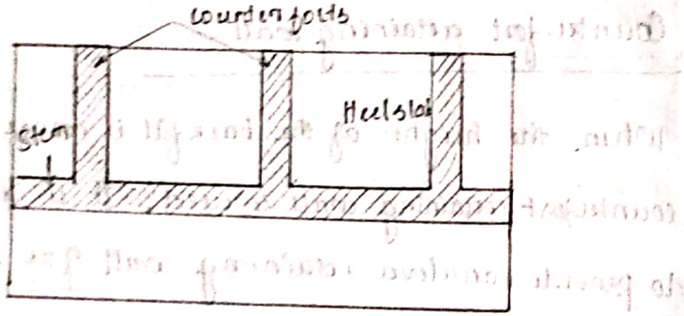
- When the height of the backfill is more than 6m then counterfort retaining wall is used, It is uneconomical to provide cantilever retaining wall for ~~largest~~ larger height as the section becomes very thick due to larger BM
- ~~As the sect~~ The structural behaviour of stem and heel in a counterfort retaining is entirely different from cantilever retaining wall ~~as the count~~.
- As the counterfort retaining wall they ~~have~~ behave like a continous slab supported on the

3 edges.

ii, 2 at the counterfort and one on base slab.



Elevation



Plan

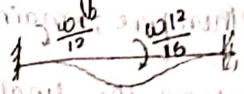
i) Design of stem:

- The stem of the counterfort retaining wall acts as a continuous slab supported on the counterforts which are spaced at 3 - 3.5m along the length of retaining wall.
- The stem is subjected to earth pressure which tries to deflect the wall away causing tension on the outer face and compression on inner face.
- Therefore, main reinforcement is put on the outer face along the length of the retaining wall.
- Due to the fixity provided by the counterfort support some negative BM develops at the supports which causes tension on the inner face near the counterforts.
- Hence, main reinforcement is also provided at inner face near the counter forts.
- The maximum BM occurs at the base of the stem.
- The load at the base of the stem, say ω per meter length is determined as below

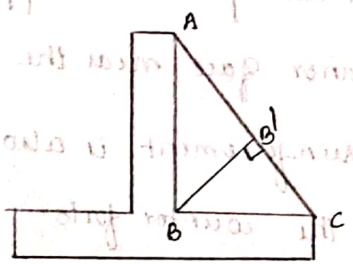
$$\omega = P_a \times 1 \times 1$$

$$\omega = k_a \gamma H \times 1 \times 1$$

- The maximum negative BM in the stem at the counterfort support may be assumed as $\frac{wl^2}{12}$ and a positive BM at the midspan may be taken as $\frac{wl^2}{16}$



ii) Design of counterfort.



BB' = depth of counterfort

- Counterforts are attached to the stem and heel slab.
- They act like a T beam of varying cross-section
- The earth pressure acting on the stem is transferred to the counterforts which tries to seperate the counterfort from stem.
- Therefore, horizontal ties are provided which connect a stem and counterfort together firmly.
- Similarly, the downward weight of backfill acting on the heel slab also tries to seperate out the heel slab and counterfort and hence ties are also provided to connect the heel slab and the counter fort.
- The counterfort act like a Tee beam of varying section supported on edges AB and BC and free at AC.
- As the outer face AC is in tension, main reinforcement is put parallel to the edge AC.
- The depth of the T beam is considered as the depth at the junction of stem and base (BB').
- The spacing of counter forts is kept about 3-3.5m and the thickness of counterfort may be taken as same as that of base slab.
- Counter forts are designed for the maximum BM.

$$M_{max} = \frac{K_a \gamma H^3}{6} \times l.$$

where, h = height of retaining wall above base.

l = spacing of counterforts.

iii) Design of heel slab.

- The heel slab behaves like the stem.
- It acts as a continous slab supporting on 3 edges and subjected to downward weight of backfill and upward soil pressure.
- The resultant load P acts in the downward direction, so the heel slab deflects downward causing tension at the bottom face in b/w the counterfort and at the top face near the counterforts.
- The maximum negative moment occurs at the counterfort and may be assumed as $\frac{Pl^2}{12}$.
- The maximum positive BM may be assumed as $\frac{Pl^2}{16}$.

iv) Design of toe slab

- The design of toe slab in a counterfort retaining wall is same as that in a cantilever retaining wall.
- It behaves like a cantilever bending upwards due to soil pressure.
- If counterforts are also provided on the toe slab, then it also acts a like a continous slab supported on counterforts.