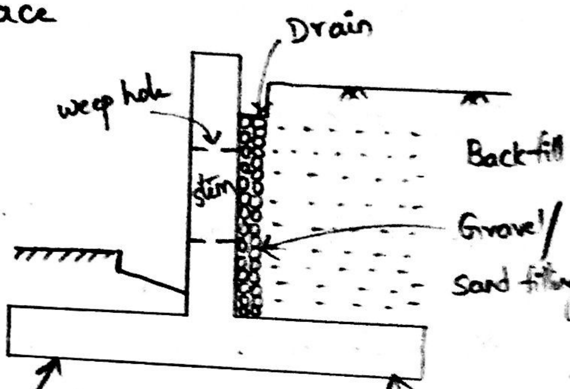
UNIT- II

DESIGN OF RETAINING WALLS

* A retaining wall is used for maintaining the ground surfaces at different elevations on either side of it.

* The material retained (or) supported by a retaining wall is called "Back fill" which may have its top surface horizontal (or) Inclined.



Retaining wall

* The position of backfill lying above the horizontal plane of the top surface of wall is called the "surcharge".

* In bridges, the wing walls and abutments are designed as retaining walls.

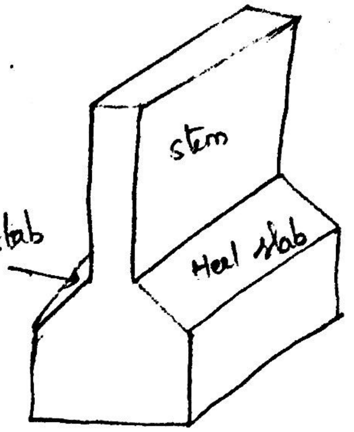
* In construction of buildings having basements, retaining walls are mandatory.

* To release unnecessary water pressure building up during rainy seasons, weep holes are provided in the retaining walls.

Types of Retaining walls :-

1. Gravity retaining wall
2. Cantilever retaining wall - (a) T-shaped, (b) L-shaped.
3. Counter-fort retaining wall
4. Buttressed retaining wall.

Cantilever retaining wall :-

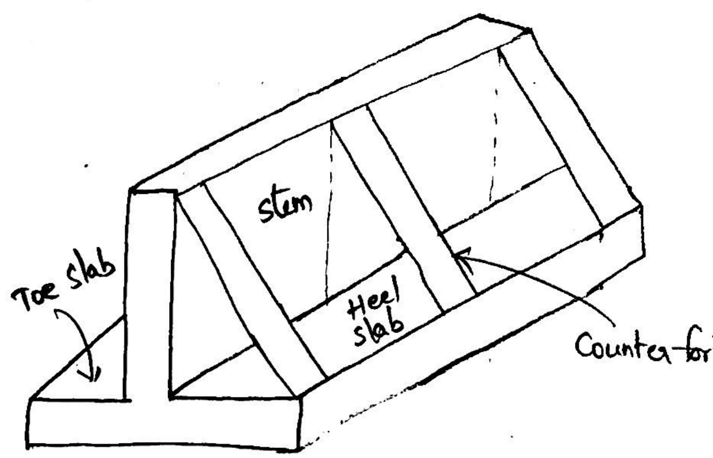


* In cantilever retaining wall system Heel slab, Toe slab and stem are acting as a cantilever due to pressure from backfill and soil.

* The cantilever retaining wall resist the horizontal as well as vertical pressures.

* Cantilever retaining walls are commonly T-shaped (or) L-shaped.

Counterfort retaining wall :-



* Counterforts are retained at particular intervals to strengthen the wall.

* In this case, stem and bed slab act as a continuous slab since stem is strengthened with counterfort.

Active earth pressure (Rankine's Theory) :-

* The retaining walls are constructed of masonary (or) Concrete, and hence the back of the wall is never smooth. Due to this frictional forces develop.

* To determine the active earth pressure, following cases of Cohession less backfill are considered.

1. Dry (or) moist backfill with no surcharge

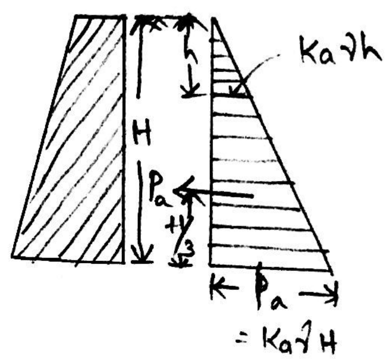
2. Submerged backfill

3. Backfill with uniform Surcharge

4. Backfill with sloping surfaces.

1. Dry (or) moist backfill with no surcharge :-

* Figure shows a retaining wall of height 'H', having dry (or) moist backfill with no surcharge



* According to Rankine's theory, the intensity of active earth pressure, $p_a = K_a \gamma H$.

where K_a = Coefficient of active earth pressure

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

ϕ = Angle of internal friction for the backfill

γ = Unit weight of backfill soil

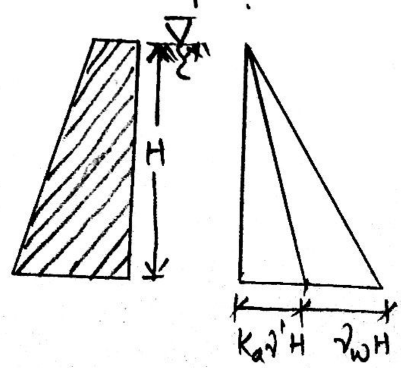
* The total active earth pressure 'P_a' per unit length of the wall is given by $P_a = \frac{1}{2} K_a \gamma H^2$ acting at $H/3$ above the base.

2. Submerged backfill :-

* In this case lateral pressure is made up of two components.

(i) Lateral pressure due to submerged weight 'γ'' of soil and

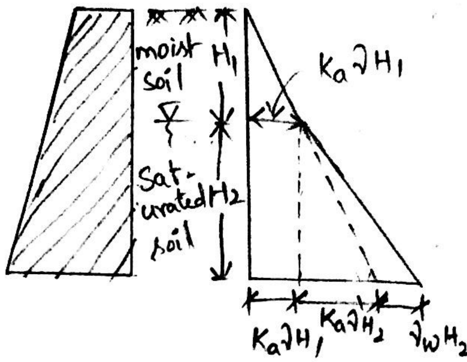
(ii) lateral pressure due to water



* The pressure at the base of the retaining wall

$p_a = K_a \gamma' H + \gamma_w H$

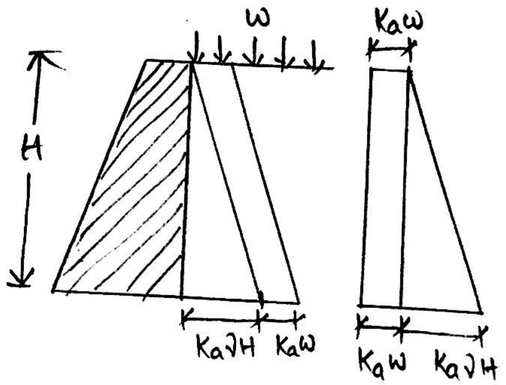
* If the back fill is Partly submerged, i.e the back-fill is moist to a depth 'H₁' below the ground level and then submerged, the lateral pressure intensity at the base of the wall is given by



$$p_a = K_a \gamma H_1 + K_a \gamma' H_2 + \gamma_w H_2$$

3. Back-fill with uniform surcharge :-

* If the backfill is horizantal and Carries Surcharge of uniform Intensity 'w' per unit area, The pressure intensity at the base of the wall is given by

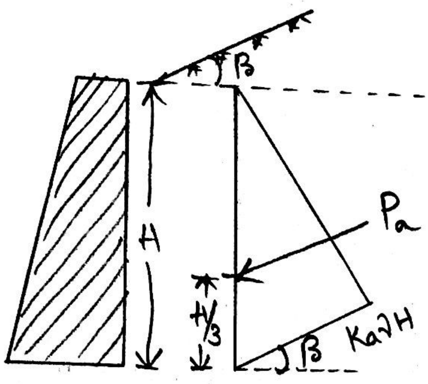


$$p_a = K_a \gamma H + K_a w$$

4. Back-fill with sloping surface :-

* Let the sloping surface behind the wall be inclined at an angle 'β' with the horizantal, β is called the surcharge angle.

* The intensity of lateral earth pressure at the base of the wall is given by



$$p_a = K_a \gamma H$$

$$K_a = \cos\beta \, \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

* The total active pressure $P_a = \frac{1}{2} K_a \gamma H^2$

Stability requirements :-

* In retailing walls, stability is checked against following cases.

(1) Over turning about Toe

(2) sliding

(1) Over-turning about Toe :-

* The earth pressure on the stem causes overturning moment about the toe.

* The weight of backfill earth, surcharge, self weight of retaining wall cause stabilizing moment about the toe.

* Hence, factor of safety against over turning is given by

$$F_1 = \frac{M_s}{M_o}$$

where M_s and M_o = stabilizing and overturning moments about toe

* A minimum factor of safety of 1.4 to be used.

* As per IS-456 : 2000 (Pg 33), only 0.9 times the characteristic dead load shall be taken into consideration

$$\therefore F_1 = 0.9 \frac{M_s}{M_o}$$

(2) sliding :-

* the horizontal pressure on stem tries to slide the retaining wall away from the back-fill.

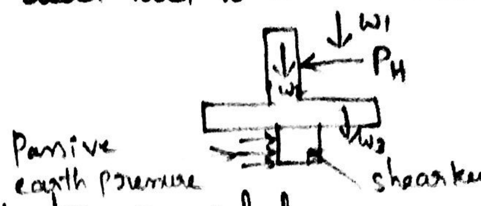
* this creates a frictional force b/n the soil and heel and toe slab.

* If 'μ' is the coefficient of friction and 'ΣW' is the total downward load, the maximum resisting force is, $F = \mu \Sigma W$

* If 'P_H' is the total horizontal pressure, then factor of safety against sliding is given by $F_s = \frac{\mu \Sigma W}{P_H}$

* As per IS : 456-2000, a minimum factor of safety of 1.4 is to be provided and only 0.9 times characteristic dead load to be considered.

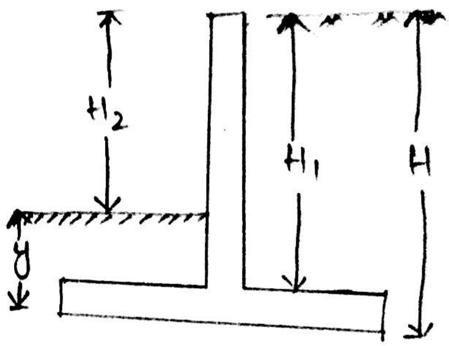
$$F_s = \frac{0.9 \mu \Sigma W}{P_H}$$



* If the value is lesser, the shear key may be provided.

Depth of foundation :-

* The depth of foundation 'y' should be such that good quality of soil to bear the induced pressure is available.



* The minimum depth of foundation by Rankine's formula is given as

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

where, q_0 = Safe bearing capacity of soil.

Design of Cantilever retaining wall :-

1. Base width (b) = 0.48 H to 0.56 H

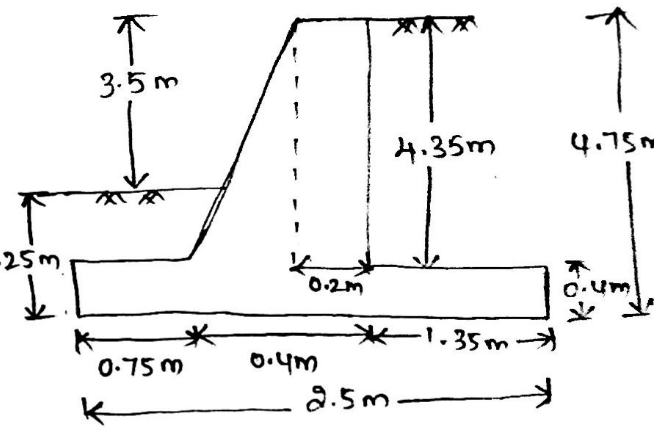
2. Toe projection = 0.3 b

3. Thickness of base slab = Thickness of stem = H/12

4. Top width of stem = 150 to 300 mm.

1) Design a Cantilever retaining wall to retain an earth embankment with a horizontal top 3.5m above ground level. Density of earth is equal to 18 KN/m³. Angle of internal friction $\phi = 30^\circ$. S.B.C of soil is 200 KN/m². Take Coefficient of friction b/n soil and concrete is equal to 0.5. Adopt M_{20} grade Concrete and Fe_{415} grade steel.

Sol:- Given data :-



$H_2 = 3.5m$

$\gamma = 18 \ KN/m^3, \ \phi = 30^\circ$

$q_0 = 200 \ KN/m^2, \ \mu = 0.5$

$f_{ck} = 20 \ N/mm^2, \ f_y = 415 \ N/mm^2$

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

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

Minimum depth of foundation, $y_{min} = \dfrac{q_0}{\gamma} K_a^2$

$$= \frac{200}{18} \times \left(\frac{1}{3}\right)^2 = 1.23m$$

Provide depth of foundation as 1.25m.

Base width (b) = 0.48H to 0.56 H

$= 0.48 \times 4.75$ to 0.56×4.75

$= 2.28$ m to 2.66 m

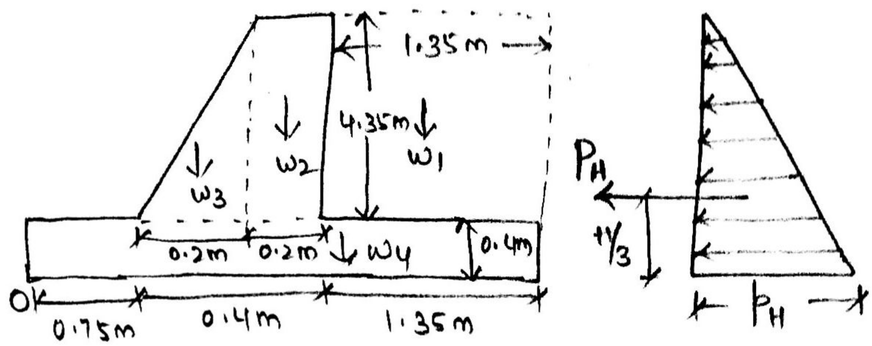
Provide base width of slab, b = 2.5m

Toe projection = $0.3b = 0.3 \times 2.5 = 0.75$ m

Thickness of base slab = Thickness of stem = $H/12 = \dfrac{4.75}{12} = 0.395m \simeq 0.4m$

Let the top width of stem be 0.2m

check for stability :-



Description	weight in 'KN'	x in 'm'	Ms in 'KNm'
weight of backfill (W_1)	$1.35 \times 4.35 \times 1 \times 18$ $= 105.7$ KN	$2.5 - \frac{1.35}{2}$ $= 1.825$m	192.9
Rectangular portion of stem (W_2)	$0.2 \times 4.35 \times 25 \times 1$ $= 21.75$ KN	$0.75 + 0.2 + \frac{0.2}{2}$ $= 1.05$m	22.83
Triangular portion of stem (W_3)	$\frac{1}{2} \times 0.2 \times 4.35 \times 1 \times 25$ $= 10.88$ KN	$0.75 + \frac{2}{3} \times 0.2$ $= 0.88$m	9.57
Base slab (W_4)	$0.4 \times 2.5 \times 1 \times 25$ $= 25$ KN	$2.5/2$ $= 1.25$m	31.25
	$\Sigma W = 163.33$ KN		$\Sigma M_s = 256.55$ KNm

Horizontal pressure, $P_H = \frac{1}{2} K_a \gamma H^2$

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 4.75^2 \times 1$

$= 67.68$ KN.

over turning moment, $M_o = P_H \times H/3$

$= 67.68 \times \frac{4.75}{3}$

$= 107.16$ KNm

As per IS: 456 - 2000, Factor of safety for overturning is

$$F_1 = 0.9 \frac{\Sigma M_s}{M_O} = 0.9 \times \frac{256.55}{107.16} = 2.15 > 1.4$$ - Hence safe.

Factor of safety for sliding,

$$F_2 = \frac{0.9 \mu \Sigma W}{P_H} = \frac{0.9 \times 0.5 \times 163.33}{67.68} = 1.08 < 1.4$$

Hence shear key is to be provided to resist sliding.

Pressure under base slab :-

Total moment at point 'O' = sliding moment - over turning moment

$= M_S - M_O$

$= 256.55 - 107.16 = 149.43$ KNm

Total vertical load $= \Sigma W = 163.33$ KN

$$\bar{x} = \frac{\text{Moment at 'o'}}{\text{Total vertical load}} = \frac{149.43}{163.33} = 0.915\,m$$

Eccentricity, $e = \frac{2.5}{2} - 0.915 = 0.335\,m$

Maximum pressure, $p_1 = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right)$

$$= \frac{163.33}{2.5}\left(1 + \frac{6 \times 0.335}{2.5}\right) = 117.85 \text{ KN/m}^2$$

Minimum pressure, $p_2 = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right)$

$$= \frac{163.33}{2.5}\left(1 - \frac{6 \times 0.335}{2.5}\right) = 12.8 \text{ KN/m}^2$$

p_1 and p_2 < safe bearing capacity = 200 KN/m

Hence the design is safe.

Design of stem :-

Stem acts as a cantilever of height 4.35m subjected to uniformly varying load of $K_a \gamma H_1$

$$= \frac{1}{3} \times 18 \times 4.35 \times 1 = 26.1 \text{ KN/m}$$

Maximum moment at the base of the cantilever $= \frac{1}{2} K_a \gamma H_1^2 \times H_1/3$

$$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 4.35^2 \times 4.35/3 = 82.31 \text{ KNm}$$

Ultimate moment $= 1.5 \times 82.31 = 123.47$ KNm

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

$$d = \sqrt{\frac{123.47 \times 10^6}{0.138 \times 20 \times 1000}} = 211.5 \text{ mm}$$

Provide effective depth of 350mm and overall depth of 400mm.

Area of steel required,

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

$$123.47 \times 10^6 = 0.87 \times 415 \times A_{st} \times 350 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 350}\right)$$

$$= 126367.5 A_{st} - 7.49 A_{st}^2$$

$$A_{st} = 1041.29 \text{ mm}^2$$

Using 12 mm dia bars, spacing $S = \frac{\pi/4 \times 12^2}{1041.29} \times 1000 = 108.65$ mm

Provide 12mm dia bars @ 100 mm c/c.

Distribution reinforcement :-

Average thickness of wall $= \frac{200 + 400}{2} = 300$ mm

$A_{st} = 0.12 \times$ Gross c/s area $= \frac{0.12}{100} \times 1000 \times 300 = 360 \text{ mm}^2$

Provide 180 mm^2 on each face and using 8 mm dia bars,

$$\text{spacing } s = \frac{\pi/4 \times 8^2}{180} \times 1000 = 279 \text{ mm}$$

Provide 8 mm dia bars @ 270 mm c/c on tension face. A mesh of 8 mm dia bars @ 270 mm is given on Compression face of wall.

Curtailment of vertical bars:-

Bending moment is proportional to cube of depth of filling and thickness varies linearly from 200 mm to 400 mm at a depth of 4.35m one third of vertical bars may be curtail at a height of 1.5m from base and other one-third at a height of 3m from the base

check for shear:-

$$V = P_H = 67.68 \text{ KN}$$

$$V_u = 1.5 \times 67.68 = 101.52 \text{ KN}$$

$$\text{Nominal shear stress, } \tau_v = \frac{V_u}{bd} = \frac{101.52 \times 10^3}{1000 \times 350} = 0.29 \text{ N/mm}^2$$

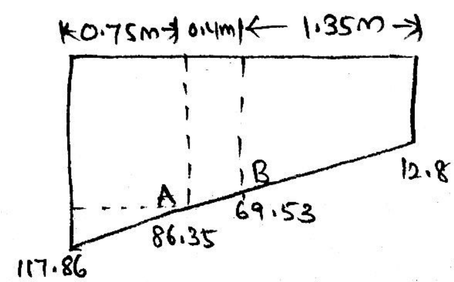
$$p_t = \frac{100 A_{st}}{bd} = \frac{100 \times \pi/4 \times 12^2}{350 \times 1000} = 0.32$$

$$\tau_c = 0.393 \text{ N/mm}^2$$

$$\tau_v < \tau_c$$

$\therefore$ shear reinforcement is not required

Design of toe slab:-



Pressure in toe slab at A,

$$= 12.8 + \frac{1.75}{2.5}(117.86 - 12.8) = 86.35 \text{ KN/m}^2$$

Pressure at B $= 12.8 + \frac{1.35}{2.5}(117.86 - 12.8) = 69.53\ KN/m^2$

Moment in the toe slab from pressure diagram,

$$M = 86.35 \times 0.75 \times \frac{0.75}{2} + \frac{1}{2}\left(0.75 \times (117.86 - 86.35) \times \frac{2}{3} \times 0.75\right)$$

$$= 24.28 + 5.9 = 30.18\ KNm$$

$$M_u = 1.5 \times 30.18 = 45.29\ KNm$$

Effective depth $d = \sqrt{\frac{45.29 \times 10^6}{0.138 \times 20 \times 1000}} = 128.1\ mm$

provide depth maximum of stem and toe slab

$\therefore d = 350mm$

Area of steel required, $45.29 \times 10^6 = 126367.5 A_{st} - 7.49 A_{st}^2$

$A_{st} = 366.35\ mm^2$

$A_{st\,min} = \frac{0.12}{100} \times 1000 \times 400 = 480\ mm^2$

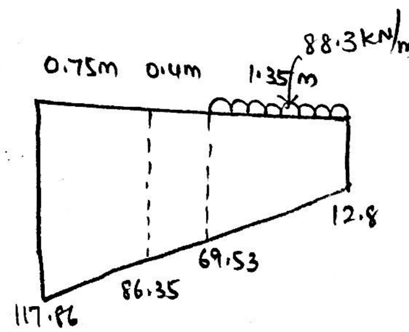
$\therefore$ Provide $A_{st} = 480\ mm^2$

Provide 12mm dia bars, spacing $= \frac{\pi/4 \times 12^2 \times \cancel{1000}}{480} = 235\ mm$

$\therefore$ Provide 12 mm dia bars @ 200mm c/c in both directions.

Design of Heel slab:-

Heel slab width is 1.35m, pressure varies from $12.8\ KN/m^2$ on outer edge to $69.53\ KN/m^2$ at the face of the column.



weight of backfill $= \gamma H_1 = 18 \times 4.35 \times 1$

$= 78.3\ KN/m$

Self weight $= 0.4 \times 1 \times 25 = 10\ KN/m$

Total downward load $= 78.3 + 10 = 88.3$ kN/m

Maximum bending moment $= \dfrac{88.3 \times 1.35^2}{2} - \left(12.8 \times 1.35 \times \dfrac{1.35}{2} + \tfrac{1}{2} \times 56.73 \times 1.35 \times \dfrac{1.35}{3}\right)$

$= 51.56$ kNm

Ultimate moment, $M_u = 1.5 \times 51.56 = 77.34$ kNm

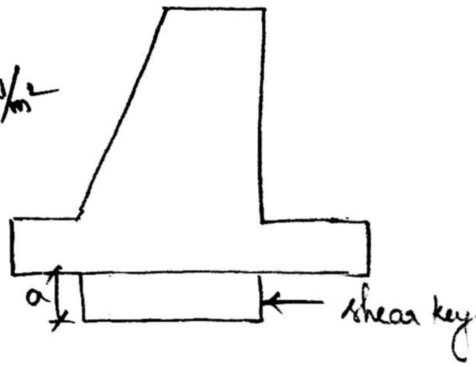
Area of steel required, $77.34 \times 10^6 = 126367.5\, A_{st} - 7.49\, A_{st}^2$

$\therefore A_{st} = 636\ mm^2$

Provide 12 mm dia bars, spacing $= \dfrac{\pi/4 \times 12^2}{636} \times 1000 = 177.84$ mm

$\therefore$ Provide 12 mm dia bars @ 170 mm c/c.

Design of shear key :-



Pressure at the face of shear key $= 86.35\ kN/m^2$

Coefficient of passive earth pressure,

$K_p = 1/K_a = 3$

If 'a' is the projection of the shear key,

resistance offered by passive earth pressure $= K_p \times a \times 86.35$

$= 259.05\, a$ kN

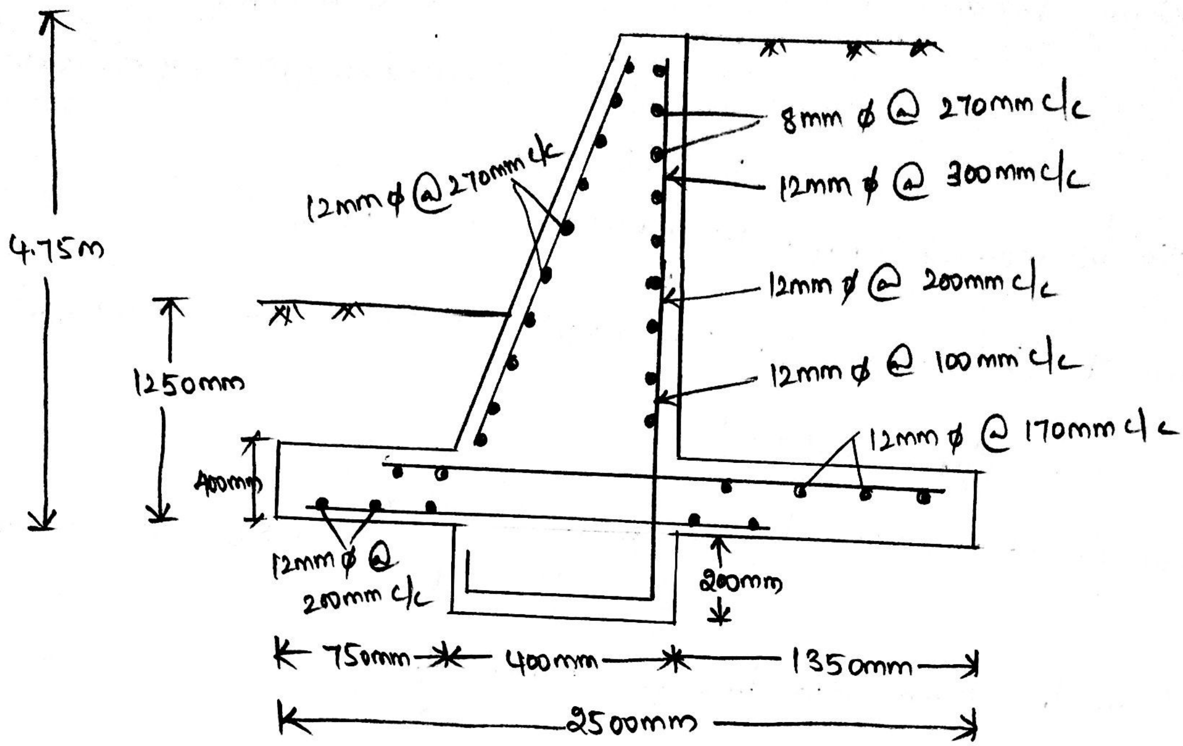
Factor of safety against sliding, $F_2 = \dfrac{0.9\, \mu \Sigma W + 259.05 a}{67.68}$

$= \dfrac{0.9 \times 0.5 \times 163.33 + 259.05 \times a}{67.68} = 1.4$

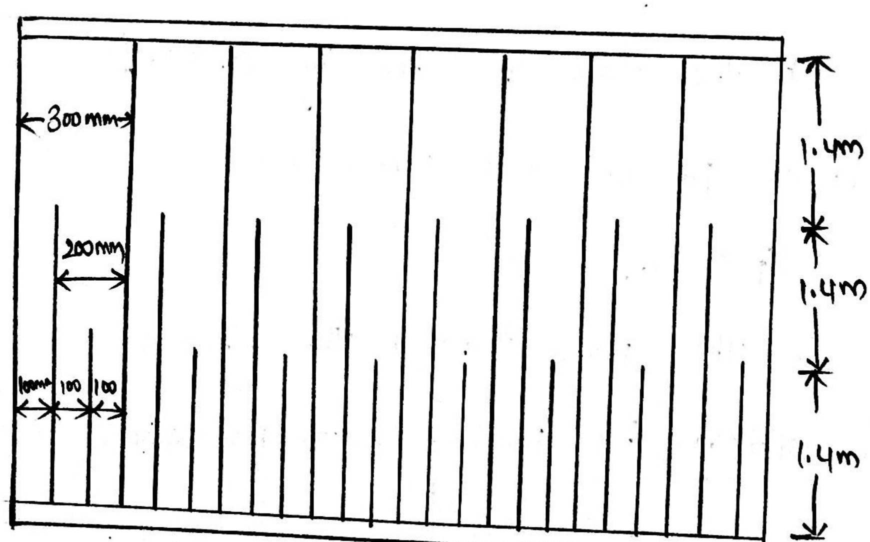
$\Rightarrow a = 0.085\ m = 85\ mm$

It is less than the permissible value of 200 mm

Hence provide 200 mm deep shear key.



Reinforcement Details

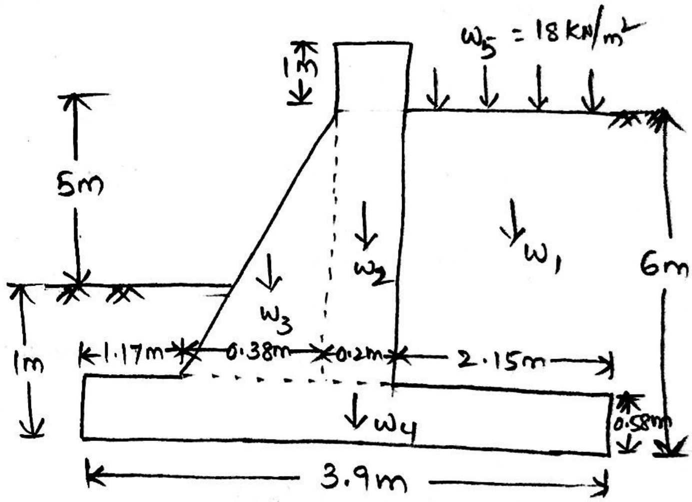


Stem reinforcement (longitudinal view)

(22)

2. Design a cantilever retaining wall for a road for the following requirements. (i) Height of wall from the bottom of the base and top of stem = 6m. (ii) Super imposed load due to road traffic = 18 KN/m². (iii) Unit weight of back fill = 18 KN/m³. (iv) Angle of internal friction for filling material = 30°. (v) Allowable bearing pressure on ground equal to 160 KN/m². (vi) Coefficient of friction b/n Concrete and ground = 0.4. Also provide a parapet wall of 1m height on top of stem. Use M_{20} and Fe 415 materials.

Sol:- Given data :-



$H = 6 + 1 = 7m$

Super imposed load = 18 KN/m²

Unit weight of soil, $\gamma = 18$ KN/m³

$\phi = 30°$, S.B.C = $q_0 = 160$ KN/m²

$\mu = 0.4$

$f_{ck} = 20$ N/mm²

$f_y = 415$ N/mm²

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

$= \dfrac{1 - \sin 30°}{1 + \sin 30°} = 1/3$

Minimum depth of foundation, $y_{min} = \dfrac{q_0}{\gamma} K_a^2$

$= \dfrac{160}{18} \times (1/3)^2 = 0.98m$

Provide depth of foundation of 1m

Base width (b) = 0.48 H to 0.56 H = 3.36 to 3.92m

Provide base width of slab = 3.9m

Toe projection : $0.3b = 0.3 \times 3.9 = 1.17m$

Thickness of slab = Thickness of stem = $H/12 = 7/12 = 0.58m$

Let the top width of stem = 0.2m

Check for stability :-

Description	weight in 'KN'	x in 'm'	M_s in 'KNm'
weight of back fill (W_1)	$2.15 \times 5.42 \times 18 \times 1$ = 209.75 KN	$1.17 + 0.58 + \frac{2.15}{2}$ = 2.825m	592.54
Rectangular portion of stem (W_2)	$0.2 \times 6.42 \times 25 \times 1$ = 32.1 KN	$1.17 + 0.38 + \frac{0.2}{2}$ = 1.65m	52.965
Triangular portion of stem (W_3)	$\frac{1}{2} \times 0.38 \times 5.42 \times 25 \times 1$ = 25.745 KN	$1.17 + \frac{2}{3} \times 0.38$ = 1.42m	36.55
Base slab (W_4)	$3.9 \times 0.58 \times 1 \times 25$ = 56.55	$\frac{3.9}{2} = 1.95m$	110.27
Super imposed load due to traffic (W_5)	$2.15 \times 1 \times 18$ = 38.7 KN	$1.17 + 0.58 + \frac{2.15}{2}$ = 2.825m	109.33
	$\Sigma W = 362.84$ KN		$\Sigma M_s = 901.65$ KNm

Horizontal pressure, $P_H = \frac{1}{2} K_a \gamma H^2$

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 7^2 \times 1$

$= 147$ KN

Overturning moment, $M_o = P_H \times H/3 = 147 \times 7/3 = 343$ KNm

The factor of safety for over turning,

$F_1 = 0.9 \frac{\Sigma M_s}{M_o} = \frac{0.9 \times 901.65}{343} = 2.365 > 1.4$ — Hence safe.

Factor of safety for sliding,

$$F_2 = \frac{0.9 \mu \Sigma W}{P_H} = \frac{0.9 \times 0.4 \times 362.845}{147} = 0.88 < 1.4$$

Hence shear key is to be provided to resist sliding.

Pressure under base slab :-

Total moment at point 'O' = sliding moment − over turning moment

$= 901.65 - 343 = 558.65$ KNm

Total vertical load, $\Sigma W = 362.845$ KN.

$$\bar{x} = \frac{558.65}{362.845} = 1.54\,m$$

Eccentricity, $e = \frac{3.9}{2} - 1.54 = 0.41\,m$

Maximum pressure, $p_1 = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right)$

$= \frac{362.84}{3.9}\left(1 + \frac{6 \times 0.41}{3.9}\right) = 151.72\ KN/m^2$

Minimum pressure, $p_2 = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right)$

$= \frac{362.84}{3.9}\left(1 - \frac{6 \times 0.41}{3.9}\right) = 34.5\ KN/m^2$

p_1 and p_2 < Safe bearing capacity ($160\ KN/m^2$)

Hence the design is safe.

Design of stem :-

stem act as a cantilever of height 6.48 m subjected to uniformly varying load of $K_a \gamma H = \frac{1}{3} \times 18 \times 6.481 \times 1 = 38.88$ KN/m

Maximum moment at the base of cantilever $= \frac{1}{2} K_a \gamma H^2 \times H/3$

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 6.48^2 \times \frac{6.48}{3} = 272.09$ KNm

Ultimate moment $= 1.5 \times 272.09 = 408.14$ KNm

Effective depth, $d = \sqrt{\frac{408.14 \times 10^6}{0.138 \times 20 \times 1000}} = 384.55$ mm

Provide effective depth of 500 mm and overall depth of 550mm.

Area of steel required,

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

$$408.14 \times 10^6 = 0.87 \times 415 A_{st} \times 500 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 500}\right)$$

$$= 180525 A_{st} - 7.49 A_{st}^2$$

$$A_{st} = 3302.54 \text{ mm}^2$$

Use 16mm diameter bars, spacing $S = \frac{\pi/4 \times 16^2}{3302.54} \times 1000$

$= 60.88$ mm

Provide 16mm dia bars @ 60 mm c/c.

Distribution reinforcement:-

Average thickness of wall $= \frac{0.2 + 0.58}{2} = 0.39$ m

$A_{st} = 0.12\%$ of gross c/s area

$= \frac{0.12}{100} \times 390 \times 1000 = 468 \text{ mm}^2$

Provide 234 mm² on each face and using 8mm dia bars

spacing $= \frac{\pi/4 \times 8^2}{234} \times 1000 = 214.8$ mm

Provide 8mm dia bars @ 200mm c/c on tension face. A mesh of 8mm ø bars @ 200 mm is given on compression face of wall.

Curtailement of vertical bars:-

Bending moment is proportional to cube of depth of filling and thickness varies linearly from 200mm to 580mm at a depth of 7m. one-third of vertical bars may be curtail at a height of 2m from base and other one-third at a height of 4m from the base.

Check for shear:-

$V = P_H = 147\ KN$

$V_u = 1.5 \times 147 = 220.5\ KN$

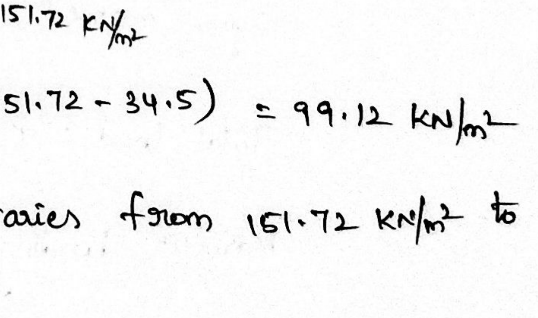
Nominal shear stress, $\tau_v = \frac{V_u}{bd} = \frac{220.5 \times 10^3}{1000 \times 500} = 0.441\ N/mm^2$

$p_t = \frac{100\ A_{st}}{bd} = \frac{100 \times \pi/4 \times 16^2}{500 \times 60} = 0.67$

$\tau_c = 0.534\ N/mm^2$

$\tau_v < \tau_c$ - Hence Section is Safe in shear.

Design of toe slab:-



Pressure at point A

$= 34.5 + \frac{2.73}{3.9}(151.72 - 34.5)$

$= 116.55\ KN/m^2$

Pressure at point B $= 34.5 + \frac{2.15}{3.9}(151.72 - 34.5) = 99.12\ KN/m^2$

Pressure diagram under base slab varies from $151.72\ KN/m^2$ to $34.5\ KN/m^2$ as shown in figure.

Dividing it into UDL of $34.5\ KN/m^2$ and a UVL

$M = 116.55 \times 1.17 \times \frac{1.17}{2} + \frac{1}{2}(151.72 - 116.55) \times 1.17 \times \frac{2}{3} \times 1.17$

$= 79.77 + 16.04 = 95.81$ KNm

$M_u = 1.5 \times 95.81 = 143.71$ KNm

Effective depth, $d = \sqrt{\frac{143.71 \times 10^6}{0.138 \times 20 \times 1000}} = 228.19$ mm

Area of steel required,

$143.71 \times 10^6 = 180525\, A_{st} - 7.49\, A_{st}^2$

$A_{st} = 824.25\ mm^2$

Use 12 mm dia bars, spacing $= \frac{\pi/4 \times 12^2}{824.25} \times 1000 = 137.2$ mm

Provide 12 mm dia bars @ 130 mm c/c.

Design of Heel slab :-

Heel slab width is 2.15m, Pressure varies from 34.5 KN/m^2 at outer edge to 99.12 KN/m^2 at the face of the column.

Weight of back fill $= \gamma H_1 = 18 \times 5.42 = 97.56$ KN/m

Self weight $= 0.58 \times 1 \times 25 = 14.5$ KN/m

Super imposed load $= 18 \times 1 = 18$ KN/m

Total load $= 97.56 + 14.5 + 18 = 130.06$ KN/m

Maximum bending moment $= \frac{130.06 \times 2.15^2}{2} - \left(34.5 \times 2.15 \times \frac{2.15}{2} + \frac{1}{2} \times 64.62 \times 2.15 \times \frac{1}{3} \times 2.15\right)$

$= 171$ KNm

Ultimate moment $= 1.5 \times 171 = 256.6$ KNm

Area of steel required,

$$256.6 \times 10^6 = 180525 \, A_{st} - 7.49 \, A_{st}^2$$

$$A_{st} = 1516.87 \text{ mm}^2$$

Provide 16 mm dia bars, spacing $= \dfrac{\pi/4 \times 16^2}{1516.87} \times 1000$

$= 132.05$ mm

Provide 12 mm dia bars @ 120 mm c/c.

Design of shear key :-

Pressure at the face of shear key $= 116.55 \text{ kN/m}^2$

Coefficient of passive earth pressure, $K_p = 1/K_a = 3$

If 'a' is the projection of the shear key, resistance offered by passive earth pressure $= K_p \times a \times 116.55$

$= 349.65 \, a$ kN

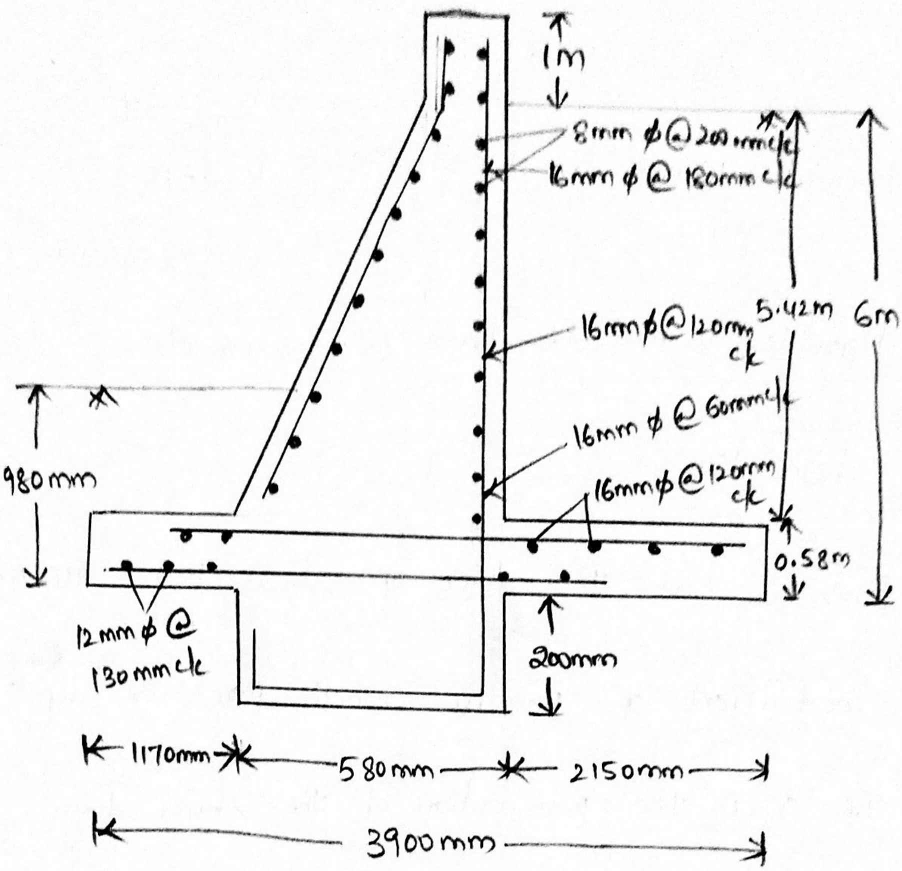
Factor of safety against sliding,

$$F_2 = \frac{0.9 \mu \Sigma W + 349.65 a}{147} = 1.4$$

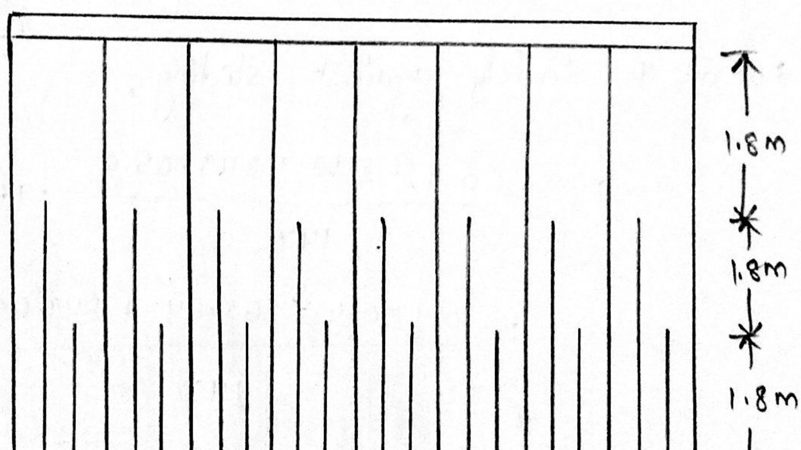
$$= \frac{0.9 \times 0.4 \times 362.84 + 349.65 a}{147} = 1.4$$

$\Rightarrow a = 121.8$ mm

Hence provide 200 mm deep shear key



Reinforcement Details



Stem reinforcement (Longitudinal view)

Design of Cantilever retaining wall with sloping backfill :-

$$\alpha = 1 - \frac{q_0}{2.7 \gamma H}$$

$$b = \frac{H \sqrt{K_a \cos\beta}}{(1-\alpha)(1+3\alpha)}$$

3) Design a T-shaped cantilever retaining wall to retain embankment 3m height above ground level. The embankment is surcharged at an angle of 16° to the horizantal. The unit weight of earth is 18 KN/m³ and its angle of repose is 30°. The safe bearing capacity of soil is 100 KN/m². The Coefficient of friction b/n concrete and soil is taken as 0.5. Use M_{20} and Fe 415 materials.

Sol:-

Given data :-

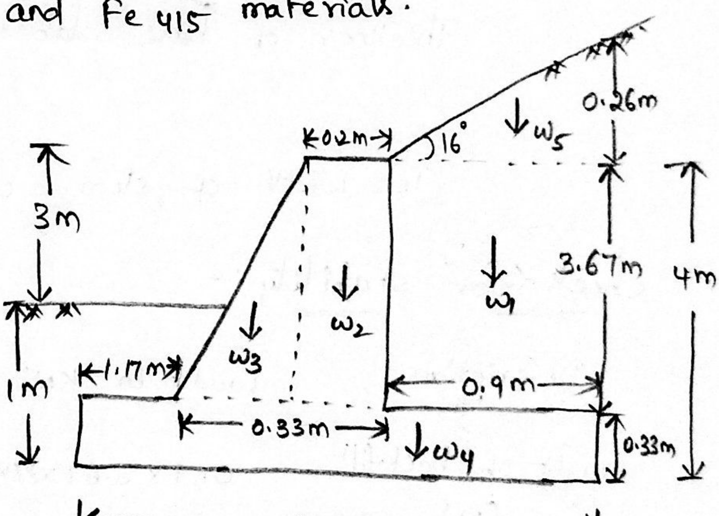
Height $H_2 = 3m$

$\beta = 16°$, $\phi = 30°$

$\gamma = 18\ KN/m^3$, $q_0 = 100\ KN/m^2$

$\mu = 0.5$, $f_{ck} = 20\ N/mm^2$

$f_y = 415\ N/mm^2$



Coefficient of active earth pressure,

$$K_a = \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi}}$$

$$= \cos 16° \times \frac{\cos 16° - \sqrt{\cos^2 16° - \cos^2 30°}}{\cos 16° - \sqrt{\cos^2 16° - \cos^2 30°}}$$

$= 0.38$

Minimum depth of foundation, $y_{min} = \frac{q_0}{\gamma} \times Ka^2$

$= \frac{100}{18} \times 0.38^2 = 0.8\ m$

Provide depth of foundation as 1m.

Total height, $H = 3+1 = 4m$

$\alpha = 1 - \frac{q_0}{2.7\gamma H} = 1 - \frac{100}{2.7 \times 18 \times 4} = 0.49$

Base width, $b = \frac{H\sqrt{Ka \cos\beta}}{(1-\alpha)(1+3\alpha)} = \frac{4\sqrt{0.38 \cos 16}}{(1-0.49)(1+3 \times 0.49)} = 1.9m$

Provide minimum base width $= 0.6H = 2.4m$

Toe projection $= \alpha b = 0.49 \times 2.4 = 1.17m$

Thickness of base slab = thickness of stem $= H/12 = \frac{4}{12}$

$= 0.33m$

Top width of stem $= 0.2m$

Check for stability :-

Description	weight in 'KN'	x in 'm'	M_s in 'KNm'
Weight of back-fill (w_1)	$0.9 \times 3.67 \times 18 \times 1 = 59.45$	$0.17 + 0.33 + \frac{0.9}{2} = 1.95$	115.92
Rectangular portion of stem (w_2)	$0.2 \times 3.67 \times 1 \times 25 = 18.35$	$1.17 + 0.13 + 0.2/2 = 1.4$	25.64
Triangular portion of stem (w_3)	$\frac{1}{2} \times 0.13 \times 367 \times 1 \times 25 = 5.96$	$1.17 + 2/3 \times 0.13 = 1.25$	7.45
Base slab (w_4)	$0.33 \times 2.4 \times 1 \times 25 = 19.8$	$2.4/2 = 1.2m$	23.76
weight of backfill in sloped portion (w_5)	$\frac{1}{2} \times 0.9 \times 0.26 \times 18 = 2.106$	$1.17 + 0.33 + 2/3 \times 0.9 = 2.1$	4.42

$\Sigma W = 106\ KN$

$\Sigma M_s = 177.19\ KNm$

Horizantal Pressure, $P_H = \frac{1}{2} K_a \gamma H^2$

$= \frac{1}{2} \times 0.38 \times 18 \times 4^2 = 54.72$ kN

Over turning moment, $M_o = P_H \times H/3$

$= 54.72 \times 4/3 = 72.96$ kNm

Factor of Safety for overturning, $F_1 = \frac{0.9 \Sigma M_s}{M_o}$

$= \frac{0.9 \times 177.19}{72.96} = 2.17 > 1.4$ - Hence safe.

Factor of safety for sliding, $F_2 = \frac{0.9 \mu \Sigma W}{P_H}$

$= \frac{0.9 \times 0.5 \times 106}{54.72} = 0.87 < 1.4$

Hence shear key is to be provided to resist sliding.

Pressure under base slab :-

Total moment at point 'o' $= M_s - M_o$

$= 117.19 - 72.96 = 104.23$ kNm

Total vertical load $= \Sigma W = 106$ kN

$\bar{x} = \frac{\text{Moment}}{\text{load}} = \frac{104.23}{106} = 0.983$

eccentricity, $e = \frac{2.4}{2} - 0.983 = 0.22$ m

Maximum pressure, $P_1 = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right)$

$= \frac{106}{2.4}\left(1 + \frac{6 \times 0.22}{2.4}\right) = 68.45$ KN/m^2

Minimum pressure, $P_2 = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right)$

$= \frac{106}{2.4}\left(1 - \frac{6 \times 0.21}{2.4}\right) = 19.87$ KN/m^2

P_1 and P_2 < safe bearing capacity (q_o)

Hence safe.

Design of stem :-

Stem act as a cantilever of height 3.67m subjected to U.V.L

of $K_a \gamma H = 0.38 \times 18 \times 3.67 \times 1 = 25.1$ KN/m.

Maximum moment at the base of cantilever

$$= \frac{1}{2} K_a \gamma H^2 \times \frac{H}{3}$$

$$= \frac{1}{2} \times 0.38 \times 18 \times 3.67^2 \times \frac{3.67}{3} = 56.35 \text{ KNm}$$

Ultimate moment $= 1.5 \times 56.35 = 84.53$ KNm

Effective depth, $d = \sqrt{\dfrac{84.53 \times 10^6}{0.138 \times 20 \times 1000}} = 175$ mm

Provide effective depth of 250mm and overall depth of 300mm.

Area of steel required,

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

$$84.53 \times 10^6 = 0.87 \times 415 A_{st} \times 250 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 250}\right)$$

$$= 90262.5 A_{st} - 7.49 A_{st}^2$$

$$A_{st} = 1023.4 \text{ mm}^2$$

Using 16 mm dia bars, spacing $= \dfrac{\pi/4 \times 16^2}{1023.4} \times 1000 = 196.5$ mm

Provide 16 mm dia bars @ 190 mm c/c.

Distribution reinforcement :-

Average thickness of wall $= \dfrac{0.2 + 0.33}{2} = 0.26$m

$A_{st\,min} = 0.12\%$ Gross c/s area

$$= \frac{0.12}{100} \times 1000 \times 260 = 312 \text{ mm}^2$$

Provide 160 mm² on each face. and using 8mm dia bar,

$$\text{spacing} = \frac{\pi/4 \times 8^2}{160} \times 1000 = 314\text{ mm}$$

Provide 8mm dia bars @ 300mm c/c on tension face. A mesh of 8mm dia bars @ 300 mm c/c is given on compression face of wall.

Curtailement of vertical bars:-

Bending moment is proportional to cube of depth of filling and thickness varies linearly from 200 mm to 330mm at a depth of 3.67m. one-third of vertical bars may be curtail at a height of 1.2m from base and other one-third at a height of 2.4 m from base.

Check for shear:-

$$V = P_H = 54.72\text{ KN}$$

$$V_u = 1.5 \times 54.72 = 82.08\text{ kN}$$

$$\tau_v = \frac{V_u}{bd} = \frac{82.08 \times 10^3}{1000 \times 250} = 0.32\text{ N/mm}^2$$

$$P_t = \frac{100 A_{st}}{bd} = \frac{100 \times \pi/4 \times 16^2}{190 \times 250} = 0.42\%$$

$\tau_c = 0.44\text{ N/mm}^2$ for $P_t = 0.42$

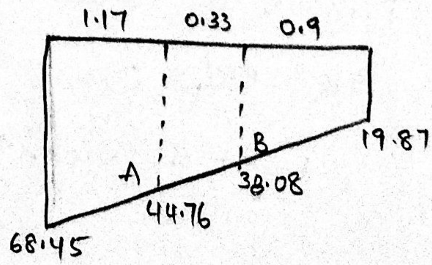
$\tau_v < \tau_c$

Hence shear reinforcement is not required.

Design of Toe slab:-

$$\text{Pressure at A} = 19.87 + \frac{1.23}{2.4}(68.45 - 19.87)$$

$$= 44.76\text{ KN/m}^2$$



Pressure at B = $19.87 + \frac{0.9}{2.4}(68.45 - 19.87) = 38.08 \ KN/m^2$

Pressure diagram under base slab varies from $68.45 \ KN/m^2$ to $19.87 \ KN/m^2$ as shown in the figure.

Dividing it into a UDL of 19.87 KN/m and a linearly varying load.

$$M = 44.76 \times 1.17 \times \frac{1.17}{2} + \frac{1}{2}(68.45 - 44.76) \times \frac{2}{3} \times 1.17$$

$$= 39.87 \ KNm$$

$$M_u = 1.5 \times 39.87 = 59.81 \ KNm$$

Effective depth, $d = \sqrt{\frac{59.81 \times 10^6}{0.138 \times 20 \times 1000}} = 146 \ mm$

∴ Provide effective depth of 250mm.

$$59.81 \times 10^6 = 90262.5 \ A_{st} - 7.49 \ A_{st}^2$$

$$A_{st} = 695.56 \ mm^2$$

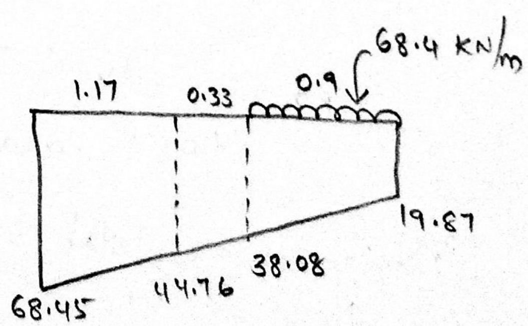
$$A_{st \ min} = \frac{0.12}{100} \times 1000 \times 300 = 360 mm^2$$

∴ Provide $A_{st} = 696 \ mm^2$

Using 12mm dia bars, spacing = 163mm

Provide 12 mm dia bars @ 150mm c/c.

Design of heel slab:-



Heel slab width is 0.9m and the Pressure is varies from $38.08 \ KN/m^2$ to $19.87 \ KN/m^2$.

weight of back-fill = $(18 \times 3.67 \times 1) + (\frac{1}{2} \times 0.26 \times 1 \times 18)$

$= 68.4 \ KN/m$.

Self weight $= 0.33 \times 1 \times 25 = 8.25$ KN/m

Total load $= 68.4 + 8.25 = 76.65$ KN/m

Maximum bending moment $= \dfrac{76.65 \times 0.9^2}{2} - \left(\overset{19.87}{\cancel{20.97}} \times 0.9 \times \dfrac{0.9}{2} + \dfrac{1}{2} \times 18.21 \times \dfrac{0.9^2}{3} \right)$

$= 20.53$ KNm

$M_u = 1.5 \times 20.53 = 30.8$ KNm

$30.8 \times 10^6 = \cancel{0.87 \times}\ 90262.5\, A_{st} - 7.49\, A_{st}^2$

$A_{st} = 345.6\ mm^2$

Minimum $A_{st} = 360\ mm^2$

Using 12 mm dia bars, spacing $= \dfrac{\pi/4 \times 12^2}{360} \times 1000 = 314.2$ mm

Provide 12mm dia bars @ 300 mm c/c.

Design of shear key :-

Pressure at the face of shear key $= 44.76\ KN/m^2$

Coefficient of passive earth pressure $= K_p = 1/K_a = 2.63$

If 'a' is the projection of shear key, resistance offered by

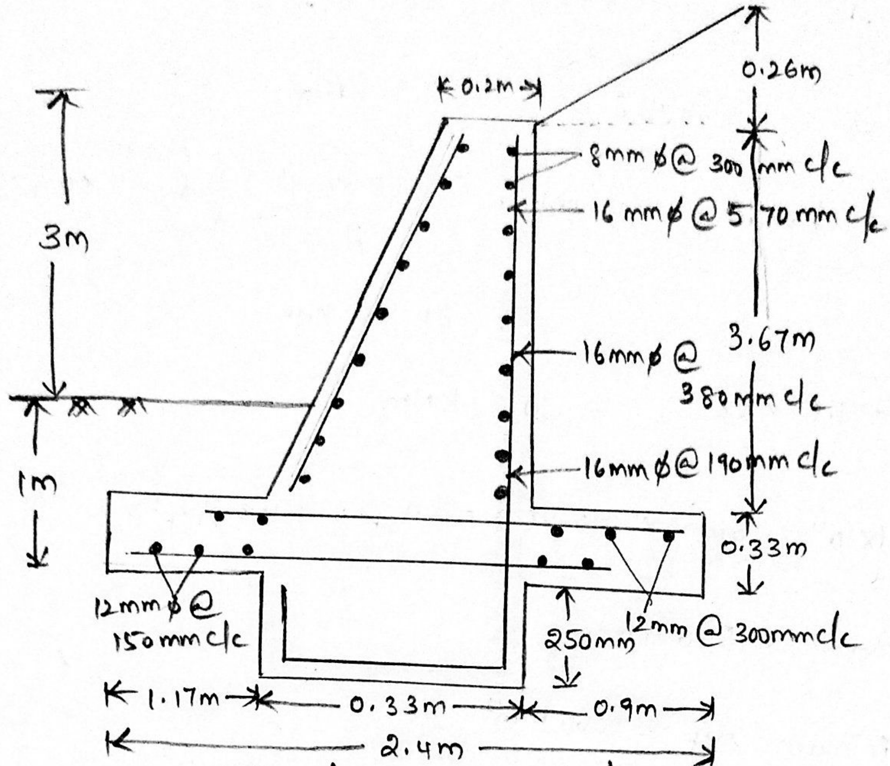
Passive earth pressure $= K_p \times a \times 44.74$

$= 117.73\, a$

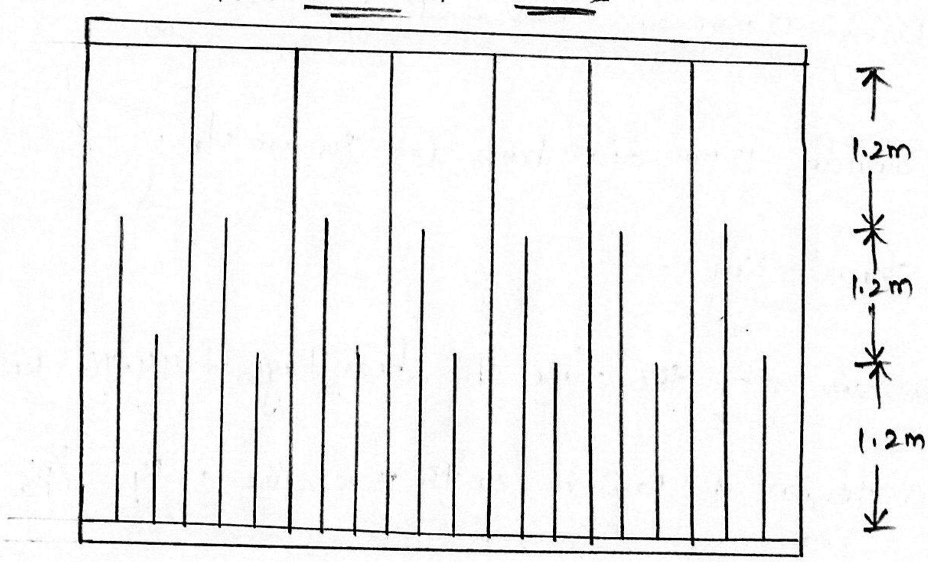
Factor of safety against sliding, $F_2 = \dfrac{0.9\, \mu \Sigma W + 117.73 a}{54.72} = 1.4$

$a = 245$ mm

Hence provide 250 mm deep shear key.



Reinforcement Details



Stem Reinforcement (Longitudinal view)

Counterfort Retaining Wall :-

* If the height of the backfill to be retained is more than 4.5m to 5m, it is preferable to provide Counter fort retaining walls than cantilever retaining walls since they are uneconomical.

* The moment due to vertical deflection is small and it is usually taken care by minimum reinforcement which is provided on both sides (or) faces of stem.

* Stem is designed as a continuous slab for bending in horizontal direction.

Design requirements for Counterfort retaining wall :-

* Base width = 0.5 H to 0.6 H

* Toe projection = $1/4^{th}$ to $1/5^{th}$ b

* Width of Counterfort = 0.03 H to 0.06 H

* Thickness of stem = thickness of base slab = $H/25$

1) Design a Counterfort retaining wall if the height of wall above the ground level is 5.5m. The safe bearing capacity of soil is 180 kN/m^2. Angle of internal friction = 30° and unit weight of back-fill = 18 kN/m^3. Keep spacing of counterfort as 3m. Coefficient of friction b/n soil and Concrete = 0.5. Use M_{20} and Fe_{415} materials.

Sol:- Given data:-

$H_2 = 5.5\,m$

$q_0 = SBC = 180\ kN/m^2$

$\phi = 30°, \ \gamma = 18\ kN/m^3$

$\mu = 0.5,\ f_{ck} = 20\ N/mm^2,\ f_y = 415\ N/mm^2$

Coefficient of active earth pressure, $K_a = \dfrac{1-\sin\phi}{1+\sin\phi} = \dfrac{1}{3}$

Minimum depth of foundation, $y_{min} = K_a^2 \times q_0/\gamma$

$= (1/3)^2 \times \dfrac{180}{18} = 1.11\,m$

Provide depth of foundation 1.3 m.

Total height of retaining wall, $H = 5.5 + 1.3 = 6.8\,m$

Base width, $b = 0.5H$ to $0.6H$

$= 0.5 \times 6.8$ to $0.6 \times 6.8 = 3.4$ to $4.08\,m$

Let the base width be 4m.

Toe projection = $1/4^{th}$ to $1/5^{th}$ b = 1m to 0.8m

Let the toe projection be 0.8 m.

Width of Counterfort = $0.03H$ to $0.06H$ = 0.204 to 0.408

Let the width of Counterfort be 0.3m.

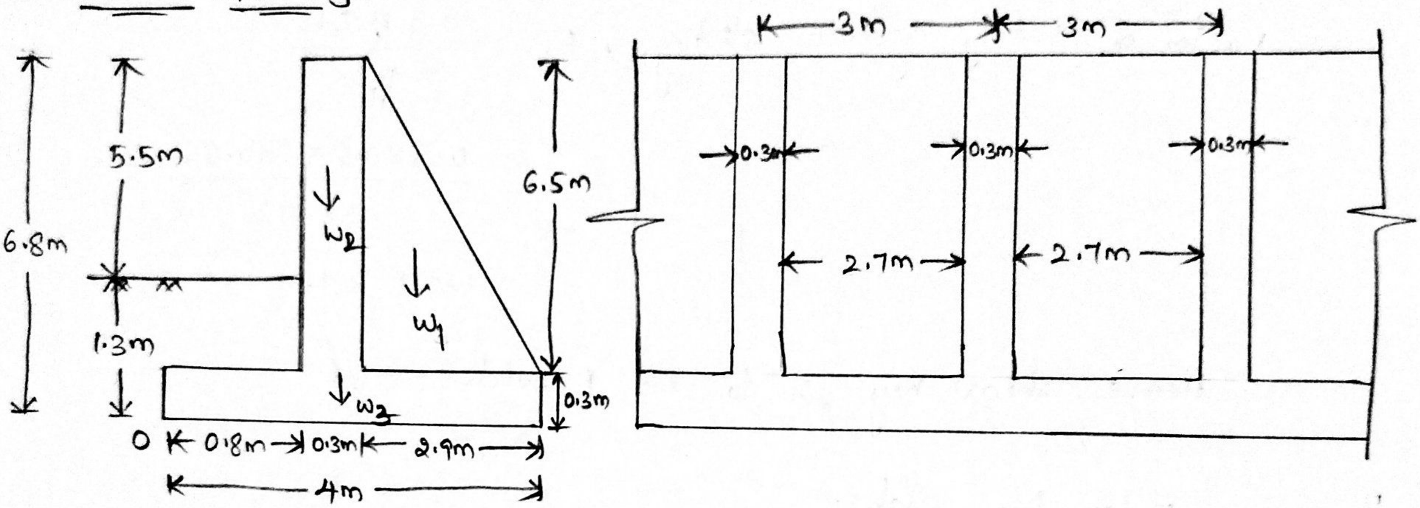
Thickness of stem = Thickness of base slab = $\frac{H}{25}$

$= \frac{6.8}{25} = 0.272\,m$

Let the effective depth d = 270 mm and

overall depth D = 300 mm

Check for stability :-



Description	weight in 'kN'	x in m	M_s in kNm
weight of backfill (w_1)	$2.7 \times 6.5 \times 18 \times 1$ $= 315.9$	$4 - \frac{2.9}{2} = 2.55$	805.5
weight of stem (w_2)	$0.3 \times 6.5 \times 2.5 \times 1$ $= 48.75$	$0.8 + 0.3/2$ $= 0.95\,m$	46.31
Base slab	$0.3 \times 4 \times 1 \times 25$ ~~$2.7 \times 6.5 \times 18 \times 1$~~ $=$ ~~315.9~~ 30	$\frac{4}{2}$ ~~$- \frac{2.9}{2}$~~ $= 2.$~~55~~	~~805.5~~ 60

$\Sigma W = 394.65$ kN

$\Sigma M_s = 911.81$ kNm

Horizontal pressure, $P_H = \frac{1}{2} K_a \gamma H^2$

$= \frac{1}{2} \times \frac{1}{3} \times 18 \times 6.8^2$

$= 138.72$ kN

over turning moment, $M_o = P_H \times H/3 = 138.72 \times \frac{6.8}{3}$

$= 314.43$ kNm

Factor of Safety against over turning $= \frac{0.9 M_s}{M_o}$

$$F_1 = \frac{0.9 \times 911.81}{314.43} = 2.6 > 1.4$$

Hence the structure is safe against overturning.

Factor of safety against sliding, $F_2 = \frac{0.9 \mu \Sigma W}{P_H}$

$$= \frac{0.9 \times 0.5 \times 394.65}{138.72}$$

$$= 1.28 < 1.4$$

Hence shear key is to be provided.

Pressure under base slab:-

Total moment at 'O' $= M_s - M_o = 911.81 - 314.43$

$= 597.38$ KN/m

$$\bar{x} = \frac{59.38}{394.65} = 1.51 m$$

Eccentricity, $e = b/2 - \bar{x} = \frac{4}{2} - 1.51 = 0.49 m$

Maximum pressure, $P_1 = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right)$

$$= \frac{394.65}{4}\left(1 + \frac{6 \times 0.49}{4}\right) = 171.18 \text{ KN/m}^2$$

Minimum pressure, $P_2 = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right)$

$$= \frac{394.65}{4}\left(1 - \frac{6 \times 0.49}{4}\right) = 26.14 \text{ KN/m}^2$$

Design of stem :-

spacing of counterfort

stem acts as a horizantal slab of span 3m. Maximum horizontal

Pressure on stem $= K_a \gamma H_1 = \frac{1}{3} \times 18 \times 6.5 \times 1 = 39$ KN/m

Maximum moment $= \frac{wl^2}{12} = \frac{39 \times 3^2}{12} = 29.25$ KNm

ultimate moment $= 43.9$ KNm

$Mulim = 0.138\, f_{ck}\, bd^2 = 0.138 \times 20 \times 1000 \times 270^2$

$= 201$ KNm

$Mu < Mulim$

Area of steel required,

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

$43.9 \times 10^6 = 0.87 \times 415\, A_{st} \times 270 \left(1 - \frac{415\, A_{st}}{20 \times 1000 \times 270}\right)$

$= 97483.5\, A_{st} - 7.49\, A_{st}^2$

$A_{st} = 467\ mm^2$

using 12 mm diameter bars, spacing, $S = \frac{\pi/4 \times 12^2}{467} \times 1000 = 242$ mm

Provide 12 mm dia bars @ 200 mm c/c.

Distribution reinforcement :-

$A_{st\,min} = 0.12\%$ gross c/s area

$= \frac{0.12}{100} \times 1000 \times 300 = 360\ mm^2$

using 12 mm dia bars, spacing $S = \frac{\pi/4 \times 12^2}{360} \times 1000 = 314.2$ mm

Provide 12 mm dia bars @ 300 mm c/c.

Check for shear :-

Maximum shear force at the face of Counter fort $= \frac{wl}{2}$

$= \frac{39 \times 3}{2} = 58.5$ KN

Factored shear force $= 1.5 \times 58.5 = 87.75$ KN

Nominal shear stress, $\tau_v = \frac{V_u}{bd} = \frac{87.75 \times 10^3}{1000 \times 270} = 0.325\ N/mm^2$

$p_t = \frac{100\ A_{st}}{bd} = \frac{100 \times \pi/4 \times 12^2}{200 \times 270} = 0.209$

$\tau_c = 0.28 + \frac{0.36 - 0.28}{0.25 - 0.15}(0.209 - 0.15) = 0.327\ N/mm^2$

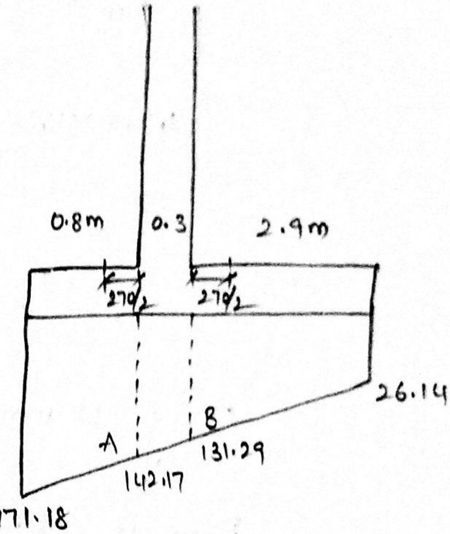
$\tau_v < \tau_c$ - Hence safe in shear.

Design of toe slab:-

Pressure at point 'A'

$= 26.14 + \frac{3.2}{4}(171.18 - 26.14)$

$= 142.17\ KN/m^2$



Pressure at point 'B'

$= 26.14 + \frac{2.9}{4}(171.18 - 26.14)$

$= 131.29\ KN/m^2$

Cantilever moment $= 142.17 \times 0.8 \times \frac{0.8}{2} + \frac{1}{2}(171.18 - 142.17) \times 0.8 \times \frac{2}{3} \times 0.8$

$= 51.68\ KNm$

$M_u = 1.5 \times 51.68 = 77.53\ KNm$

Area of steel required,

$77.53 \times 10^6 = 97483.5\ A_{st} - 7.49\ A_{st}^2$

$A_{st} = 851\ mm^2$

Using 16 mm dia bars, spacing $s = \frac{\pi/4 \times 16^2}{851} \times 1000 = 236\ mm$

Provide 16 mm dia bars @ 220 mm c/c.

Check for shear :-

Critical section is at a distance, $d = 270mm$ from the face of stem. Pressure at $d/2$ distance from left of stem

$$= 26.14 + \frac{3.335}{4}(171.18 - 26.14) = 147.06 \ KN/m^2$$

Pressure at $d/2$ distance from right of stem

$$= 26.14 + \frac{2.765}{4}(171.18 - 26.14) = 126.4 \ KN/m^2$$

Shear force, $V = \frac{1}{2}(171.18 + 26.14)(0.8 - 0.27)$

$$= 52.3 \ KN.$$

Factored shear force $= 52.3 \times 1.5 = 78.43 \ KN$

$$\tau_v = \frac{V_u}{bd} = \frac{78.43 \times 10^3}{1000 \times 270} = 0.29 \ N/mm^2$$

$$p_t = \frac{100 \times \pi/4 \times 16^2}{220 \times 270} = 0.34$$

$\tau_c = 0.4 \ N/mm^2$ for $p_t = 0.34\%$

$\tau_v < \tau_c$ - Hence Safe in shear.

Design of heel slab:-

Load from back fill : $6.5 \times 18 = 117 \ KN/m^2$

Self weight of base slab $= 0.3 \times 1 \times 25 = 7.5 \ KN/m^2$

Total down ward load $= 117 + 7.5 = 124.5 \ KN/m^2$

Maximum downward pressure intensity is at edge,

$$P_{max} = \overset{124.5}{\cancel{126.14}} - 26.14 = \overset{98.36}{\cancel{100.26}} \ KN/m^2$$

$$\text{Moment} = \frac{wl^2}{12} = \frac{\overset{98.36}{\cancel{100.26}} \times 3^2}{12} = \overset{73.77}{\cancel{75.19}} \ KNm$$

$1235.8 \times 10^6 = 108315\, A_{st} - 2.83\, A_{st}^2$

$A_{st} = 1339\ mm^2$

Minimum reinforcement required $\Rightarrow \dfrac{A_{st}}{bd} = \dfrac{0.85}{f_y}$

$$A_{st_{min}} = \frac{0.85\, bd}{f_y}$$

$$= \frac{0.85 \times 2648 \times 300}{415} = 1627\ mm^2$$

Provide 4 no's of 25 mm dia bars

$$A_{st_{prov}} = \pi/4 \times 25^2 \times 4 = 1963\ mm^2$$

Horizantal ties:-

Consider bottom 1m height, Maximum pressure = 39 KN/m²

) Maximum horizontal pressure on stem (39 kN/m)

Total lateral pressure to be transfered from stem to slab = $39 \times 2.7 = 105.3$ KN.

Factored force = $105.3 \times 1.5 = 158$ KN.

Area of tensile steel required = $\dfrac{153 \times 10^3}{0.87 \times 415} = 437.5\ mm^2$

Provide 10 mm dia bars, spacing = $\dfrac{\pi/4 \times 10^2}{437.5} \times 1000 = 179.5$ mm

∴ Provide 10 mm dia bars @ 170 mm c/c.

Vertical stirrups:-

Near the end of heel downward force = $\overset{98.36}{\cancel{101.28}}$ KN/m²

Factored force = $1.5 \times \overset{98.36}{\cancel{100.26}} = \overset{147.54}{\cancel{150.39}}$ KN/m²

steel required to resist tensile force = $\dfrac{\overset{147.54}{\cancel{150.39}} \times 10^3}{0.87 \times 415} = \overset{408.6}{\cancel{416}}\ mm^2$

Using 10 mm dia bars, spacing = $\dfrac{\pi/4 \times 10^2}{\underset{408.6}{\cancel{416}}} \times 1000 = \underset{192.2}{\cancel{188.8}}$ mm

Provide 10 mm dia bars @ 180 mm c/c

$M_u = 1.5 \times \overset{73.77}{\cancel{75.19}} = \overset{110.65}{\cancel{112.78}}$ KNm

Area of steel required,

$\overset{110.65}{\cancel{112.78}} \times 10^6 = 97483.5\, A_{st} - 7.49\, A_{st}^2$

$A_{st} = \overset{1256.3}{\cancel{1283.48}}$ mm²

Using 16 mm dia bars, Spacing, $S = \dfrac{\pi/4 \times 16^2}{12\cancel{83.98}} \times 1000$

$= \cancel{156.67}$ mm
160

Provide 16 mm dia bars @ 150 mm c/c.

Distribution reinforcement ~~of~~ 12 mm dia bars @ 225 mm c/c is

Provided at right angles to the main bars.

Design of Counterfort :-

Reinforcements are required for beam action and for against

Seperating force in horizantal and vertical directions.

(i) Inclination of Counterfort with horizantal, $\theta = \tan^{-1}\left(\frac{6.5}{2.9}\right)$

$\theta = \cancel{65.75°} \simeq \cancel{66°}$ $\theta = 66.95° \simeq 66°$
$\cancel{73.28°}$

Depth of Counterfort for beam action at Junction $= 2.9 \sin\theta$

$= 2.65$ m

Maximum moment on Counterfort $= K_a \dfrac{\gamma H_1^3}{6} \times l$

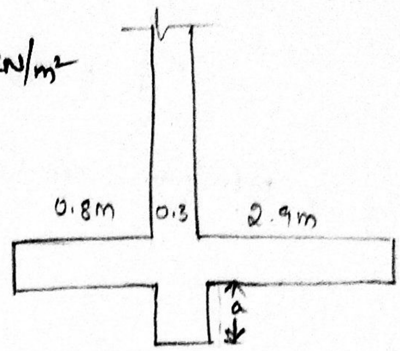
$= \frac{1}{3} \times 18 \times \dfrac{6.5^3}{6} \times 3 = 823.87$ KNm

$M_u = 1.5 \times 823.87 = 1235.8$ KNm

Area of steel required, $1235.8 \times 10^6 = \cancel{108315}\ 0.87 \times 415 \times A_{st} \times 300 \left(1 - \dfrac{415\, A_{st}}{2650 \times 300 \times 20}\right)$

Design of shear key :-

Pressure at the face of shear key $= 142.17\ kN/m^2$



Coefficient of passive earth pressure, $K_p = 3$

If 'a' is the projection of shear key,

resistance offered by passive earth pressure

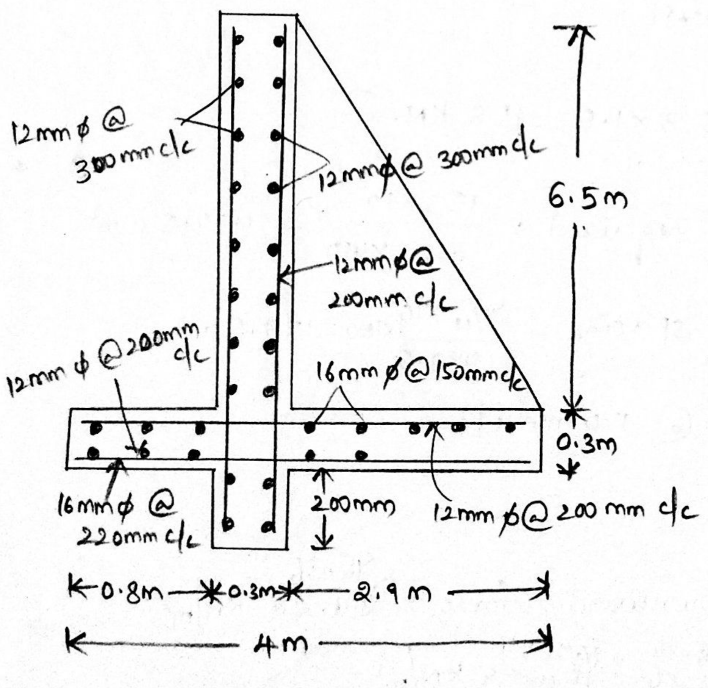
$= K_p \times a \times 142.17$

$= 426.51\ a$

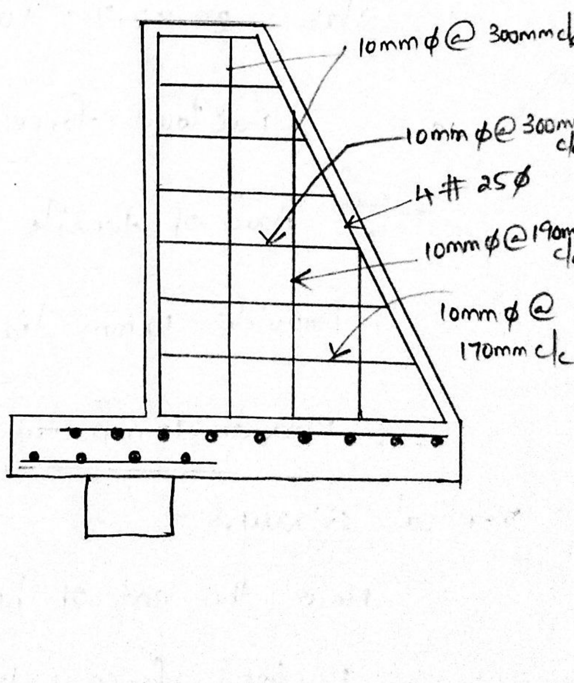
Factor of safety against sliding, $F_2 = \dfrac{0.9\mu\Sigma W + 426.51\,a}{138.72} = 1.4$

$a = 38.95\ mm$

Hence provide 200 mm deep shear key.



Section near Counterfort



Section through Counterfort