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Internal Assessment Test 1 – Oct. 2022

Sub:	Design of RC and Steel Structure			Sub Code:	18CV72	Branch	Civil	
Date:	21/10/2022	Duration:	90 min's	Max Marks:	50	Sem / Sec:	7A	OBE

Answer ANY ONE FULL Questions Note: Use of IS 456:2000 is permitted, Assume missing data suitably.		MARKS	CO	RBT
1	Design a cantilever retaining wall to retain soil of earth of height 4.0 m above ground level. The soil is having safe bearing capacity and density are respectively 200kN/m^2 and 18kN/m^3. Design all components of retaining walls with all stability checks. Also draw detailed drawing. Use M 20 and Fe 415. Angle of Repose, $\Phi = 30^0$. Coefficient of friction between concrete and soil $\mu = 0.5$.	[50]	CO1	L2
2	A R.C.C. retaining wall with counterforts is required to support earth to a height of 7m above the ground level. The top surface of the backfill is horizontal. The trial pit taken at the site indicates that soil of bearing capacity 220kN/m^2 is available at a depth of 1.25m below the ground level. The weight of earth is 18kN/m^3 and angle of repose is 30°. The coefficient of friction between concrete and soil is 0.58. Use concrete M20 and steel grade Fe415. Design the retaining wall.	[50]	CO1	L2

1. **Design a cantilever retaining wall to retain soil of earth of height 4.0 m above ground level. The soil is having safe bearing capacity and density are respectively 200kN/m^2 and18kN/m^3. Design all components of retaining walls with all stability checks. Also draw detailed drawing. Use M 20 and Fe 415. Angle of Repose, $\Phi = 30^0$, Coefficient of friction between concrete and soil $\mu = 0.5$.**

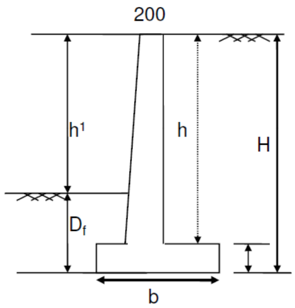
Solution

Height of earth fill, h'= 4.0 m, Safe bearing Capacity, SBC = 200 kN/m^2, Density of soil, γ = 18 kN/m^3, co-efficient of friction between concrete and soil, $\mu = 0.5$, angle of repose $\varphi = 30°$

We need to fix the total height of retaining wall, $H = h' + D_f$

- **Depth of foundation, D_f – (Height of wall below Ground level)**

Using Rankine's formula: find depth of foundation



$$D_f = \frac{SBC}{\gamma}\left[\frac{1-\sin\phi}{1+\sin\phi}\right]^2 = \frac{SBC}{\gamma} k_a{}^2$$

Active earth pressure coefficient $k_a = \frac{1-\sin 30^0}{1+\sin 30^0} = \frac{1}{3}$

Passive earth pressure coefficient $k_p = \frac{1}{k_a} = 3$

$= \frac{200}{18}\left[\frac{1-\sin 30^0}{1+\sin 30^0}\right]^2 = 1.23 \text{ m} \approx \ 1.2 \text{ m}$

Therefore total height of retaining wall H = 4.0 + 1.2 = 5.2 m

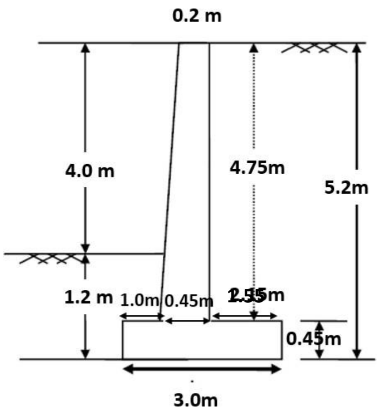
- **Proportioning of wall**

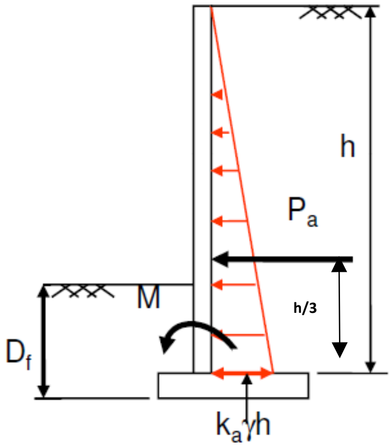
a. **Thickness of base slab** = (1/10 to 1/14) H = 1 /10 × 5.2 to 1 /14 × 5.2 = 0.52m to 0.37m, say 0.45 m - 450 mm

b. **Width of base slab** = b = (0.5 to 0.6) H = 0.5 × 5.2 or 0.6 × 5.2 = 2.6 m to 3.12 m say 3.0 m

c. **Toe projection** = (1/3 to ¼) b = 1 /3 × 3.0 or 1/4 × 3.0 = 1.0 m to 0.75 m say **1.0** m(your wish !!)

d. Provide **450 mm** **thickness for the stem at the base (overall depth D)** and 200 mm at the **top**





- **Design of stem**

To find Maximum bending moment at the junction

Height of stem, h = 5.2 – 0.45 = 4.75 m

Active earth pressure acting on stem slab, $P_a = \frac{1}{2}(k_a \times \gamma \times h)\, h =$

$P_a = 1/2 \times 1/3 \times 18 \times 4.75 \times 4.75 = 67.68$kN

Total Bending moment at any height, $M = P_a \times \frac{h}{3}$

$$M = 14.25 \times \frac{4.75}{3} = 107.16 \text{ kN-m}$$

$M_u = 1.5 \times M = 160.74$ kN-m

Taking 1m length of wall,

We have overall depth at base or thickness of stem slab as, D = 450 mm

Check for effective depth "d"

$$M_{u,lim} = 0.36 \frac{x_{u,\,max}}{d}\left(1 - 0.42 \frac{x_{u,\,max}}{d}\right) b d^2 f_{ck}$$

Put Mu,lim = 160.74×10^6, b = 1000mm , fck = 20N/mm^2

xu,max /d = 0.48 , Fe 415 , IS 456 2000

$160.74 \times 10^6 = 0.36 \times 0.48 \times (1 - 0.42 \times 0.48) \times 1000 \times d^2 \times 20$

d = 241.3 mm ≈ 242 mm

effective cover = clear cover + bar diameter/2 (assuming 12 mm φ bars)

= 40 + 12/2 = 46 ≈ 50 mm

d = Overall depth – effective cover = 450 – 50 = 400 mm >> 242 mm, hence safe

- **Area of steel for stem slab**

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

d = 400 mm, b =1000 mm, Mu =160.74 x 10^6 Nmm, fy = 415 N/mm^2 , fck = 20 N/mm^2

Ast = 1184.86 mm^2

Ast, min = $0.0012 \times b \times D = 0.0012 \times 1000 \times 450 = 540\ mm^2$

Ast > Ast,min, hence Ok.

- **Main steel**

Provide 16 mm φ bars as main steel

Spacing required, s = $\dfrac{1000 \times \frac{\pi}{4} \times 16^2}{1184.86}$ = 169.69 mm ≈ 170 mm or 160 mm (Your wish!!)

Main steel #16 mm φ @ 170 mm c/c < 300 mm or 3 times effective depth "d" (Check!!!) IS 456 2000

- **Distribution steel or Ast,min**

= 0.12% Gross Area = 0.0012 ✕ 450 ✕ 1000 = 540 mm^2

Use 10 mm φ bars, spacing required

Spacing required, s = $\dfrac{1000 \times \frac{\pi}{4} \times 10^2}{540}$ = 145.4 mm ≈ 140 mm or 150 mm (Your wish !!)

Distribution bars #10 mm φ @ 150 mm c/c < 450 mm and 5 times effective depth "d" ok (check!!) IS 456 2000

- **Development length L_d**

$L_d = 47\ \Phi_{bar} = 47 \times 16 = 752$ mm =0.752 m

- **Curtailment of bars**

Curtail 50% steel from top, Ast = $\frac{50}{100} \times 1184.86 = 590\ mm^2$

$\left(\frac{h_1}{h}\right)^2 = \frac{1}{2}$, $\left(\frac{h_1}{5.2}\right)^2 = \frac{1}{2}$, $\frac{h_1^2}{5.2^2} = \frac{1}{2}$, $h_1 = 3.67\ m$, is the curtailment length or cutting length

Actual point of cut off or cutting position = 3.67 –L_d =3.67 - 0.752 = 2.91 m is the cutting length from top.

Spacing required, s = $\dfrac{1000 \times \frac{\pi}{4} \times 16^2}{590}$ = 340.7 mm ≈ 340 mm As IS 456 spacing < 300 mm or 3 times effective depth "d" (Check!!!)

$s = 340$ mm > 300 mm

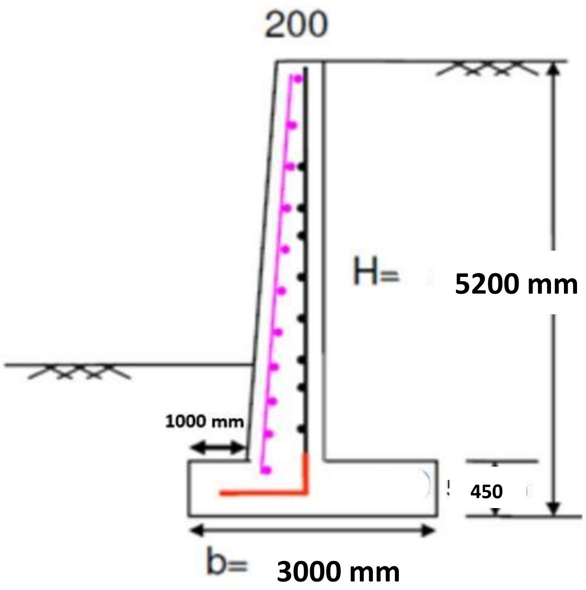
Instead of this you have to provide every alternate bars at 300 mm c/c.

Spacing of bars 16 mm φ @ 300 mm c/c. Hence it is ok.

Secondary steel for stem at front (Temperature steel)

0.12% Gross Area = 0.12 × 450 × 1000/100 = 540 mm^2

#10 @ 150 mm c/c< 450 mm and 5d Hence it is ok.



- **Check for shear for stem slab**

Max. Shear Force at Stem Junction, P_a = 67.68kN (Lateral earth pressure)
Ultimate Shear Force = Vu =1.5 x [redacted] kN
Nominal shear stress = τ_v = Vu/bd = 101.52 x 10^3 / (1000 x 400) =0.25 N/mm^2
To find τc, calculate $p_t = \frac{100\,A_{st}}{b \times d} = \frac{100 \times 1184.}{1000 \times 400} = 0.295$ %

Use IS:456-2000, Page 73, Table 19, p_t = 0.295 %, M 20

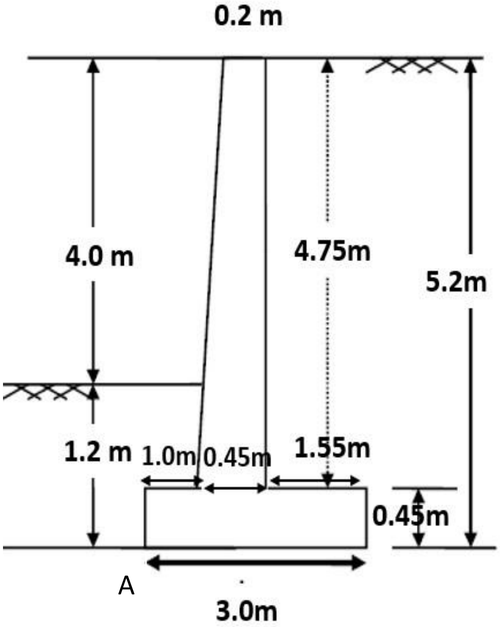
τc = 0.379N/mm^2≈ 0.38 N/mm^2

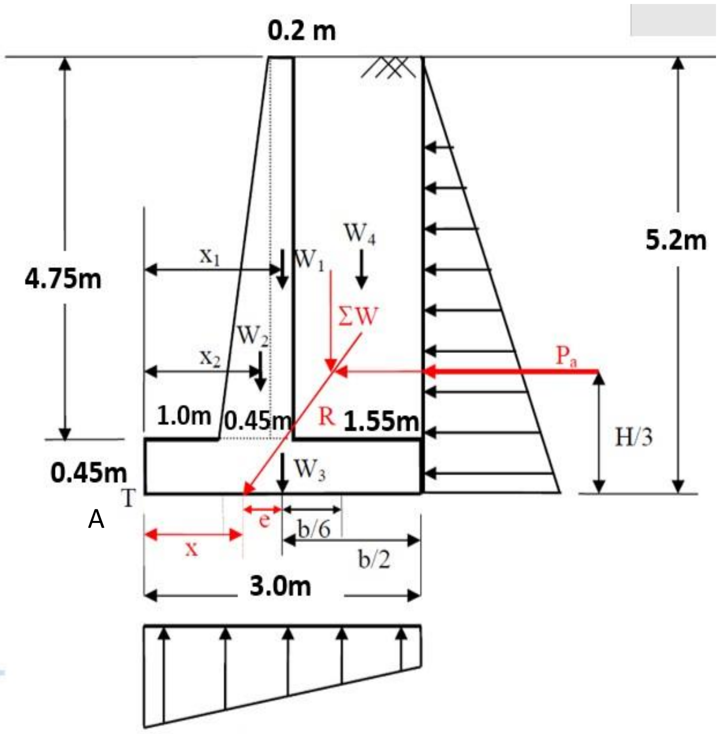
Compare τ_v and τ_c , [redacted]

$\tau_v < \tau_c$ Hence safe in shear. No need of shear reinforcement.

Stability analysis – 1. To find factor of safety against overturning

Calculations of **Resisting Moment** **ΣM_R** – **Self weight of wall and weight of earth fill retained by heel slab**





Pressure below the Retaining Wall

Load	Magnitude, kN	Distance from A, m	Bending moment about A kN-m
Stem W1	0.2 x4.75 x1x25 =23.75	(1.0+0.25+0.2/2) =1.35	32.06
Stem W2	½ x 0.25 x 4.75 x1x25 =14.84	1.0 + 2/3 x 0.25 = 1.16	17.21
Base slab W3	3.0 x0.45 x1 x25 = 33.75	3.0/2 = 1.5	50.63
Back fill, W4	1.55x4.75x1x18 = 132.525	1.0+0.45+1.55/2 =2.225	294.857
Total	**ΣW=204.86 kN**		**ΣM_R = 394.76 kN-m**

Calculations of Overturning Moment **M_O – Lateral earth pressure about the base slab**

Load	Magnitude, kN	Distance from A, m	Bending moment about A kN-m
Hori. earth pressure =P_H	P_H = ½ x 1/3x18x5.2^2 =81.12 kN	H/3 =5.2/3	**M_O = 140.61**

Stability checks:

1. **Check for overturning**:

As per IS: 456:2000, *(Factor of Safety) overturning* should satisfy condition that $\Sigma M_R / M_O >$ 1.55

ΣM_R = 394.757 kNm, M_O = 140.61 kNm

(F.S) overturning = $\Sigma M_R / M_O$ = 2.80 >1.55 Hence it is safe

2. **Check for Sliding**:

ΣW= 204.86kN

P_H =81.12 kN (Horizontal earth pressure)

As per IS: 456:2000, *(F.S) sliding* should satisfy condition that

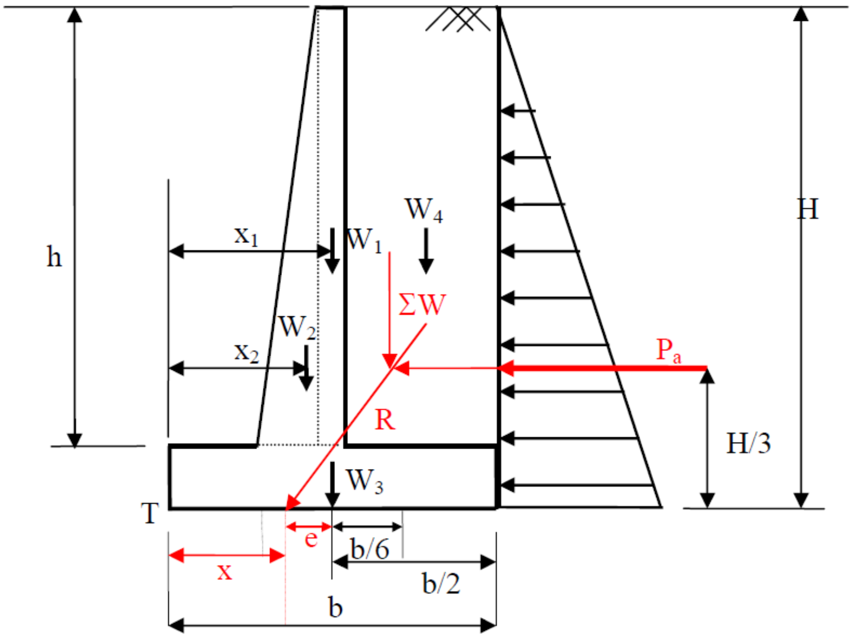
$\mu\sum W/P_H \geq 1.55$

$$\frac{\mu \sum W}{P_H} = \frac{0.5 \times 204.86}{81.04} = 1.26$$

(F.S) sliding = 1.26 ≤1.55 Hence it is not safe against sliding. Shear key is needed.

3. **Check for subsidence**: (Max. pressure at the toe should not exceed the safe bearing capacity of the soil under working condition)

Let the resultant cut the base at distance 'x' from toe T,
$x = \sum M / \Sigma W$, where $\sum M$ = Net moments about toe = $\Sigma M_R - M_O$ = 394.757 – 140.61 = 254.15 kNm



$$x = \frac{254.15}{204.86} = 1.24 \text{ m} \quad , b = 3\text{m}$$

- **Eccentricity e = b/2 – x** = 3/2 – 1.24 = 1.5 – 1.24 **= 0.26m < b/6** , 0.26 <0.5
- **e = 0.26m**
- (Eccentricity of force should not exceed one sixth of base)

  Here e < b/6. Hence it is safe.

Pressure below the base slab

$$\text{Max. pressure} = P_{max} = \frac{\sum W}{b}\left[1 + \frac{6e}{b}\right]$$

103.79kN/m^2 < SBC, safe bearing capacity

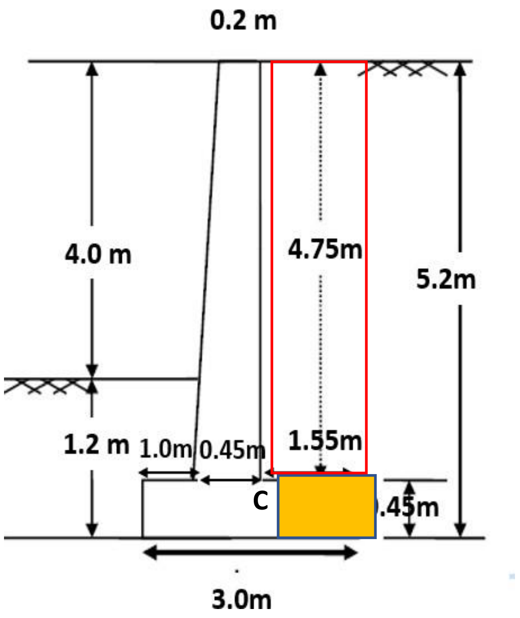
$$\text{Min. pressure} = P_{min} = \frac{\sum W}{b}\left[1 - \frac{6e}{b}\right]$$

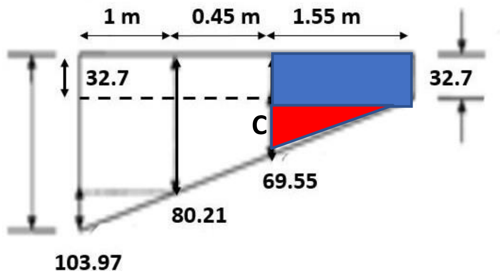
32.7kN/m^2 > zero, So there is no tension or separation developed at base slab, Hence it is safe

Both values of pressure are lesser than SBC (200 kN/m^2) . Hence it is safe.

Design of Heel Slab

Calculations of Moment about heel slab C





Load	**Magnitude, kN**	**Distance from C, m**	**BM, M_C, kN-m**
Backfill or earthfill	1.55 x 4.75 x 1 x18 = 132.52	1.55/2 = 0.775	102.703
Heel slab	0.45 x1.55 x 25x1 = 17.43	1.55/2 = 0.775	13.51
Upward Pressure distribution, (below heel slab) rectangle	-32.7 x 1.55 = -50.68	1.55/2 = 0.775	-39.28

Upward Pressure distribution, Triangle 	- ½ x (69.55 - 32.7) x 1.55 = -28.55	$\frac{1}{3}$ x1.55 =0.516	-14.73
Total Load at junction C	70.72	**Total BM at Junction C**	**ΣM_C = 62.18**

ΣM_C = 62.18 kNm

Mu = 1.5 x 62.18 = 93.27 kNm

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

Mu = 62.18 x 10^6 Nmm, b = 1000 mm, d = 400 mm, f_{ck} =20 N/mm^2 f_y = 415 N/mm^2

Ast = 669.04 mm^2

Use 12 mm φ bars (it is base slab) (You can choose 12 mm also)

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 12^2}{669.04}$ = 170

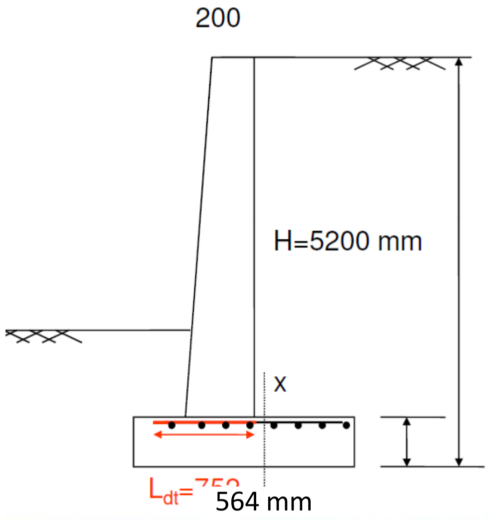
Main steel #12 mm Φ @170 mm c/c < 300 mm and 3d ok. Hence it is safe.

Development length

L_d = 47 φ_{bar} = 47 x 12 = 564 mm

Distribution steel

#10 mm Φ @ 140 mm c/c < 450 mm and 5d ok



Check for shear at junction (Tension)

Critical section for shear is at the face as it is subjected to tension.

Maximum shear =V= 70.72 kN, V_U, max = 70.72 x 1.5 =106.08 kN

$\tau_v = \frac{V_U}{b \times d} = \frac{106.08 \times 10^3}{1000 \times 400} = 0.26$

$pt = \frac{100 \times 669.04}{1000 \times 400} = 0.167\%$

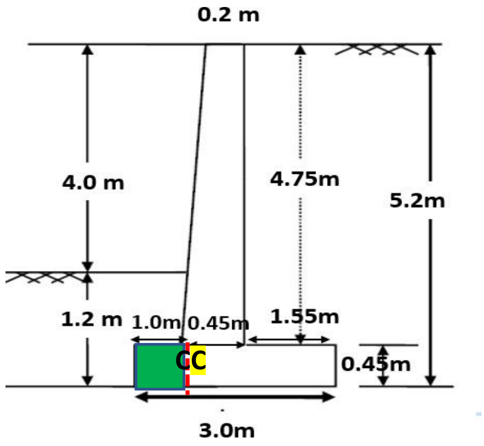
Use IS:456-2000, Page 73, Table 19, pt = %

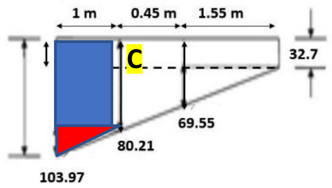
$\tau_c = 0.297$ N/mm^2

Compare . τ_v and τ_{uc} , $\tau_v < \tau_c$ Hence it is safe in shear.

Design of toe

To find the maximum bending moment





Load	Magnitude, kN	Distance from C, m	BM, M_C, kN-m
Self wt Toe slab	1.0 x 0.45 x 25 = 11.25	1.0/2	5.625
Upward Pressure distribution, rectangle	-80.21 x 1.0 = -80.21	1.0/2	-40.10
Upward Pressure distribution, Triangle	-½ x (103.97-80.21) x 1.0 = -11.88	2/3 x 1 = 0.66	-7.92
Total Load at junction		**Total BM at junction**	**ΣM_C = -42.4**

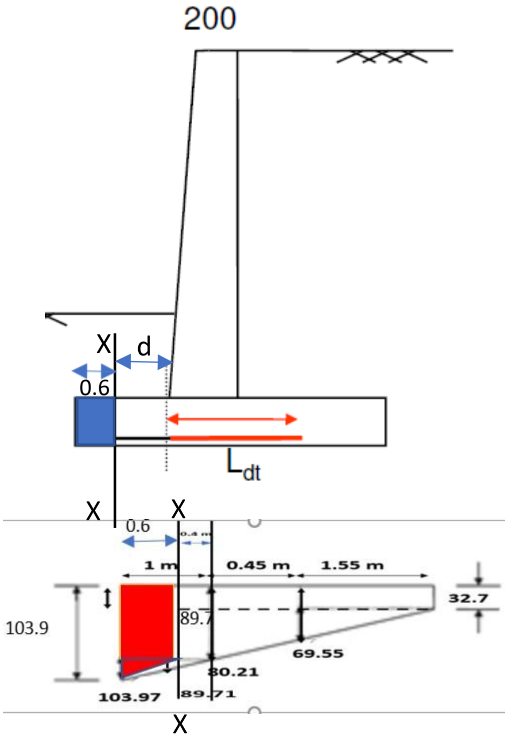
Mu = 1.5 x 42.4= 63.75kN-m,

Ast = 452.02 mm^2 , since Area of steel is less, we have used 10 mm Φ bars

Provide Main steel #10 @ 150 mm c/c < 300 mm and 3d ok

Development length:

L_d = 47 φbar = 47 x 10 = 470 mm



Check for shear: at d from junction, d =400 mm

Net shear force at the section XX

V= -(103.9 +89.7) /2 x 0.6 + 0.45 x 0.6 x 25 = -51.33 kN

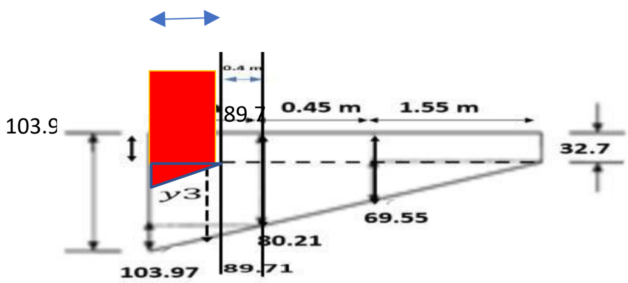
$V_{U,max}$ = 51.33 x 1.5 = 76.9 kN

ζv = 76.9x1000/ (1000x 400) =0.19 MPa,

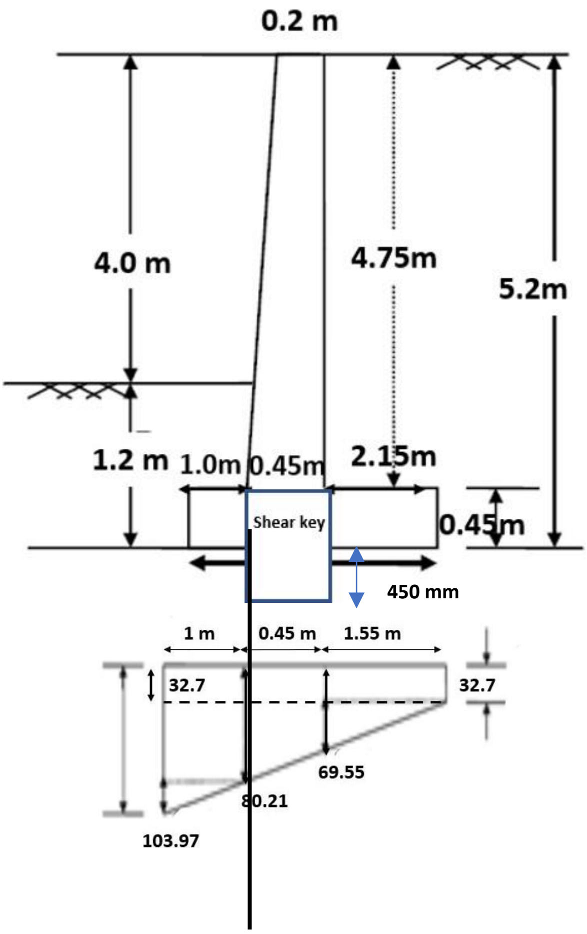
find pt = 100 x Ast /(1000 x 400) =0.113

From IS:456-2000, ζc = 0.28 MPa

ζv < ζc, Hence safe in shear.



- **Design of Shear key**



Assume width and height of shear key as 450 mm. Let Pp be Total passive force developed in the front of shear key.

Pp = Pressure at junction of toe × width of shear key ×coefficient of passive pressure × 1m

= **80.21** × 0.45 × 3×1 = 108.28kN

Factor of safety against sliding

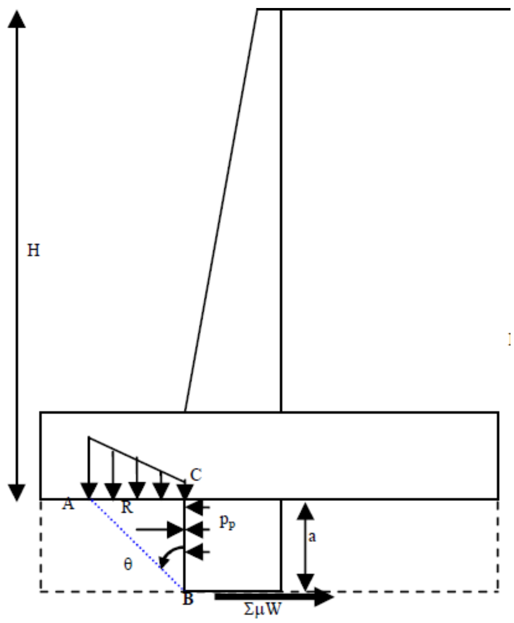
$$FOS= \frac{Pp + \mu \sum W}{P_A} = \frac{108.28+0.6 \times 204.86}{\mathbf{81.12}} = 2.85 \geq 1.55$$

Hence safe

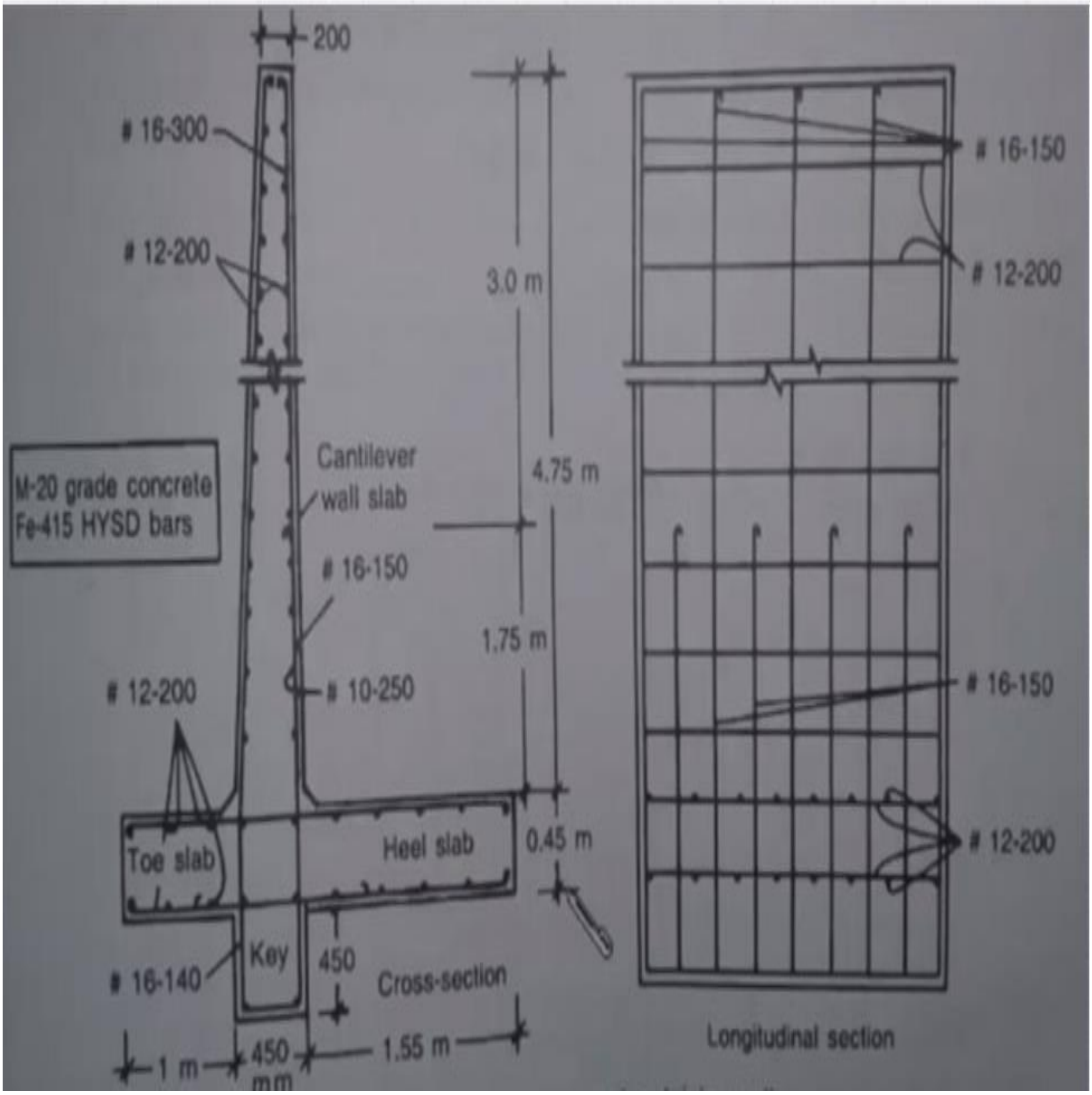
Provide 0.3 % of cross-sectional area for shear key as reinforcement

$= \frac{0.3}{100} \times 450 \times 1000 = 1350 mm^2$

Provide 16mm dia @140mmc/c



$\theta = 45 + \phi/2$



Reinforcement detailing of Retaining wall with shear key

2. A R.C.C. retaining wall with counterforts is required to support earth to a heightof 7m above the ground level. The top surface of the backfill is horizontal. The trial pit taken at the site indicates that soil of bearing capacity 220kN/m^2 is available at a depth of 1.25m below the ground level. The weight of earth is 18kN/m^3 and angle of repose is 30°. The coefficient of friction between concrete and soil is 0.58. Use concrete M20 and steel grade Fe415. Design the retaining wall.

Given Data:
fck= 20 N/mm^2, fy = 415N/mm^2, H = 7 m above G.L, Depth of footing below G.L. = 1.25 m, γ = 18 kN/m^3, μ = 0.58, SBC= 220 kN/m^2

Coefficient of active pressure = k_a $1-\sin 30^0$ 2 1

$= [_{1+\sin 30^0}] \;=\; _3$

Coefficient of passive pressure = $k_p = \frac{1}{ka} = 3$

Taking depth of foundation as 1.25 m

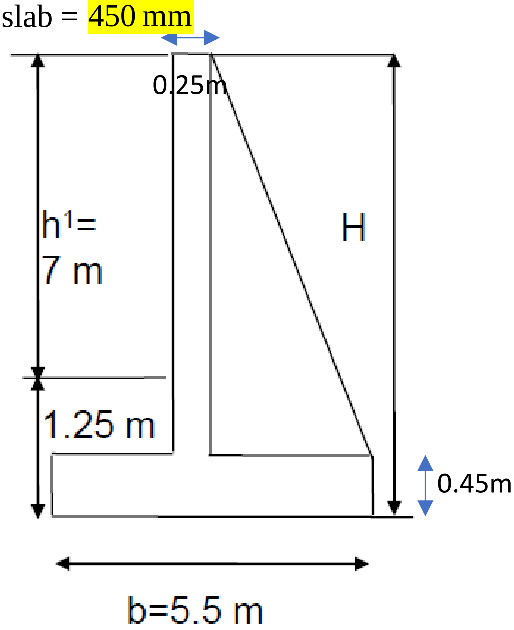
The height of the wall above the base or Total height of retaining wall, H in metres

= H = 7 + 1.25 = 8.25 m.

- **Proportioning of Wall Components – Stem, Heel, Toe and Counterforts**

1. Base width of retaining wall, b = 0.6 H to 0.7 H = 0.6 x 8.25 or 0.7 x 8.25

   = (4.95 m to 5.78 m), Say b = 5.5 m
2. Width of Toe or Toe projection = b/4 = 5.5/4 =1.375 say 1.2 m or 1.3m
3. Assume thickness of vertical wall or stem = 250 mm (We are assuming constant thickness for stem slab)
4. Assume thickness of base slab = 450 mm



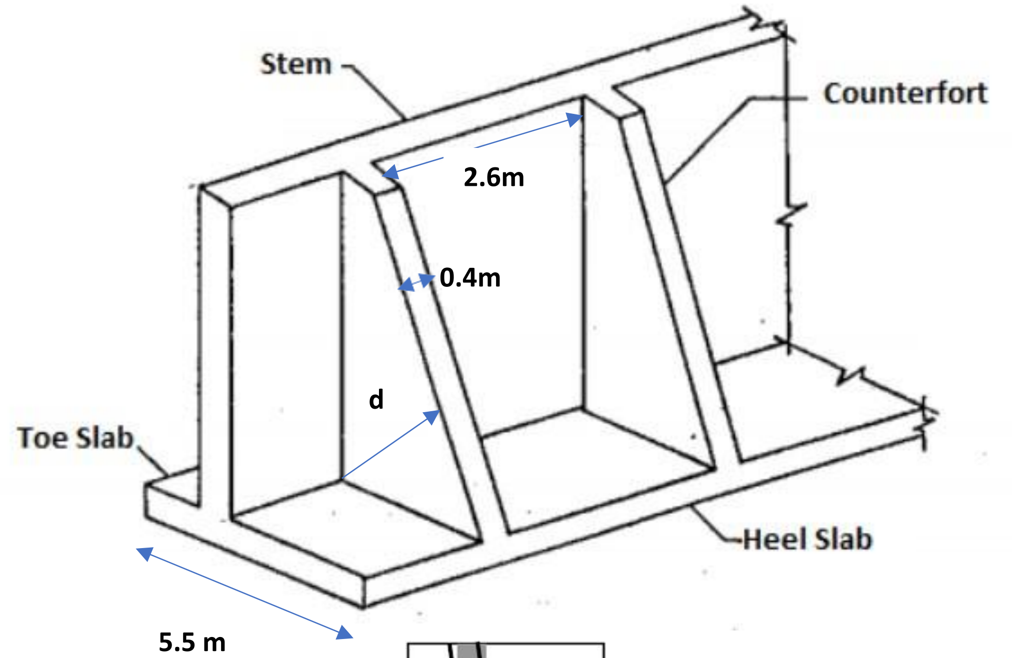
5. **Spacing of counterforts**

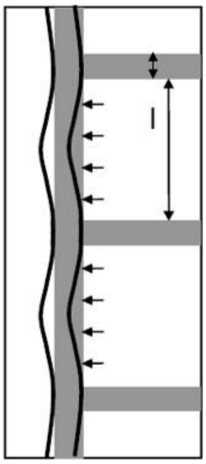
Clear spacing of counterforts, l = $3.5\left(\frac{H}{\gamma}\right)^{0.25} = 3.5\left(\frac{8.25}{18}\right)^{0.25} = 2.88$ m

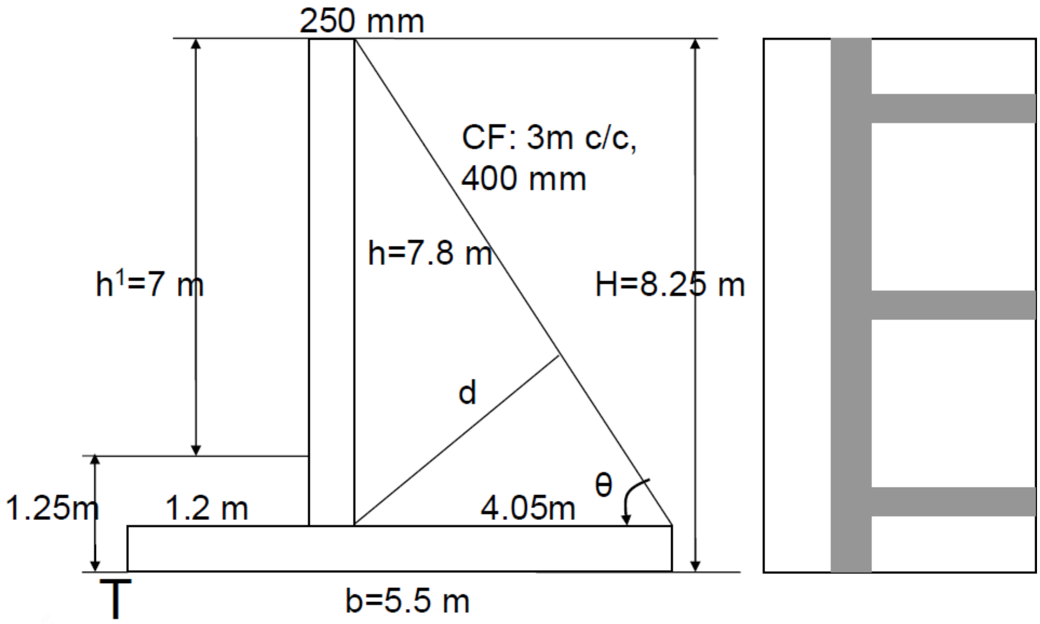
Assume width of counterfort = 400 mm,

c/c spacing of counterforts = 2.88 + 0.40 = 3.28 m ≈ 3.00 m or 3.5m

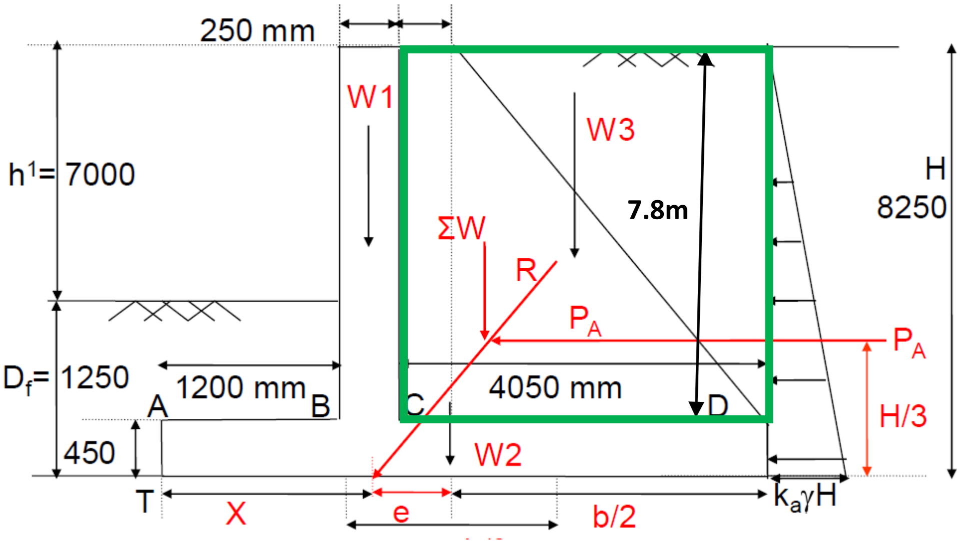
So, clear spacing of counterforts becomes = 3.00 - 0.4 = 2.6 m







b. Check Stability of Wall



Calculations of Restoring moment – Weight of retaining wall and weight of earth fill retained on heel slab

Sr. No	Description of loads	Loads in kN	Dist. Of C G from T in m	Moment about T in kN-m
1	Weight of stem W1	25x0.25x1x7.8 = 48.75	1.2 + 0.25/2 =1.325	64.6
2	Weight of base slab W2	25x5.5x1x0.45 = 61.88	5.5/2 =2.75	170.2
3	Weight of earth over heel slab W3	18x4.05x1x7.8 = 568.62	1.20+0.25+4.05/2 = 3.475	1975.95
Total		ΣW = 679.25		ΣM =2210.69

Calculations of overturning moment – Active earth pressure

Sr. No	Description of loads	Loads in kN	Dist. Of CG from T in m	Moment about T in kN-m
1	Horizontal earth pressure on stem slab	$\frac{1}{2}(k_a \times \gamma \times H)\,H =$ $=\frac{1}{2} \times \frac{1}{3} \times 18 \times 8.25 \times 8.25 =$ **204.19**	8.25/3	561.52kN.m

- **Check for overturning**

Factor of safety against overturning

FOS = $\frac{2210.71}{561.52}$ = 3.94 > **1.55**, Hence it is safe against overturning.

- **Check for sliding FOS =** $\mu\sum W/P_H \geq 1.55$

Total horizontal force tending to slide the wall =Ph=**204.19kN**

Resisting force =μ ΣW = 0.58 x 679.25 = 393.97kN

Factor of safety against sliding = $\frac{\Sigma W\ \mu}{P_h} = \frac{393.97}{204.19}$ = 1.93>1.55 Hence it is safe against sliding.

- **Check for pressure distribution at base**

Let **X** be the distance of Resultant R from toe(T), $=\frac{\text{Net Moment}}{\Sigma W} = \frac{2210.71 - 561.52}{679.25}$

=2.43m

Eccentricity= **e = b/2 - X** = 5.5/2 - 2.43 = 0.32 < b/6 (0.91m)

Whole base is under compression.

Maximum pressure at toe

$$\text{Max. pressure} = P_{max} = \frac{\sum W}{b}\left[1 + \frac{6e}{b}\right]$$

$=166.61 kN/m^2 < SBC = 220 kN/m^2$

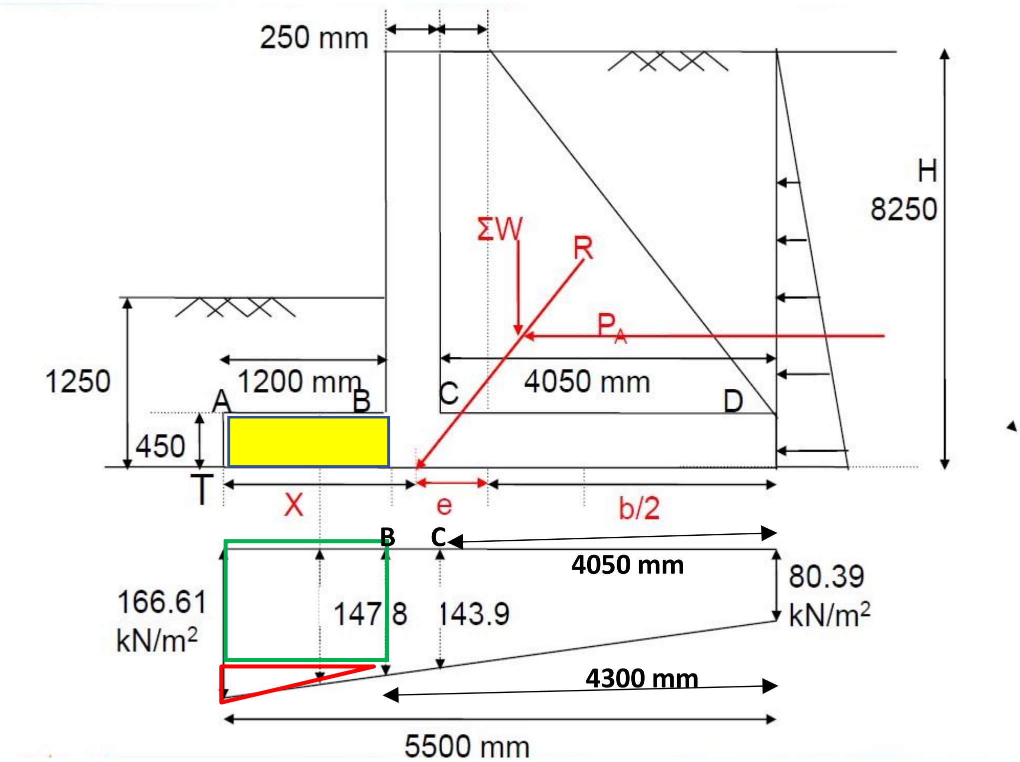
Minimum pressure at heel

$$\text{Min. pressure} = P_{min} = \frac{\sum W}{b}\left[1 - \frac{6e}{b}\right]$$

$= 80.39 kN/m^2 < SBC = 220 kN/m^2$

By interpolation, Intensity of pressure at junction of stem with toe i.e. under B = p_B= 80.39 + (166.61 -80.39) x 4.3/5.5 = **147.8kN**/m^2

By interpolation, Intensity of pressure at junction of stem with heel i.e. under C =p_C= 80.39 + (166.61 -80.39) x 4.05/5.5 = **143.9 kN**/m^2



b) Design of Toe slab

Sr. No	Description of loads	Loads in kN	Dist. Of C G. from B in m	Moment about B in kN-m
1	Weight of Toe slab	25x 1.2 x 0.45 =	1.2/2	8.1
2	Weight due to upward soil pressure	-147.8 x 1.2 = -177.3	1.2/2	-106.4
3	Weight due to upward pressure	- ½ x (166.61- 147.8) x 1.2	2/3 x 1.2	-9.02
Total				ΣM = - 107.3
Factored Moment Mu				= -160.9 kNm

- **To find steel**

b = 1000mm, d = 400 mm, fck = 20N/mm², fy = 415N/mm², Mu = 160.9 x 10⁶ Nmm

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

Ast = 1188.22mm²

Take 16 mm diameter bars as main bars, Spacing s $= \frac{1000 \times \frac{\pi}{4} \times 16^2}{1188.22} = 170$ mm < 300 mm and 3 d.

Main bars - Provide **16 mm** Φ dia @ 170 mm c/c .

Distribution steel =

0.12 % x b x D = 0.12 x 1000 x 450/100 = 540 mm²

Let's provide 12 **mm** Φ diameter bars, Spacing s $= \frac{1000 \times \frac{\pi}{4} \times 12^2}{540} = 210$ mm < 450mm and 5 d

Distribution bars - Provide **12 mm** Φ dia @ 210 mm c/c .

- **Development length = 47 x diameter of main bar = 47 x 16 = 750 mm**

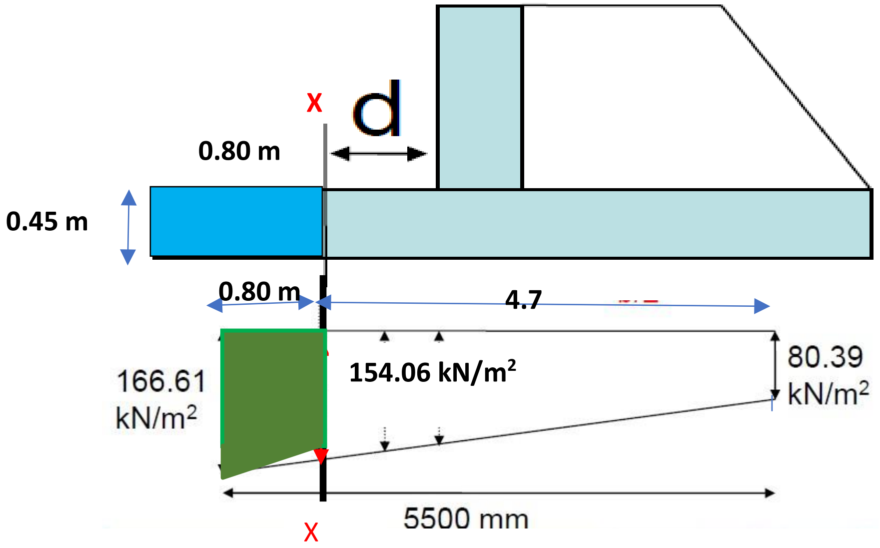
- **Check for Shear**

Locate a critical section XX at a distance 'd' from junction of toe slab

Critical section for shear: At distance d (= 400 mm) from the **junction** of the toe

$\frac{166.61-80.39}{5.5} = \frac{y}{4.7}$, y = 73.67

Pressure at section XX = 73.67 + 80.39 = 154.06 kN/m^2



Net vertical shear = - (**166.61 + 154.06) x 0.80/2** + (**25 x 0.45 x 0.80**) = 119.28kN

Net ultimate shear = Vu.$_{max}$ = 1.5 x 119.28 = 178.9 kN
ζv = 178.9 x 1000/1000 x 400 = 0.447 MPa
pt = 100 x 1188.22/ (1000 x 400) = 0.29 %
ζc = 0.39 N/mm^2 ζc < ζv

Hence it is not safe in shear. Provide stirrups or increase percentage of steel, pt = 0.5%.

Shear reinforcement shall be provided to carry a shear equal to $V_u - \tau_c bd$ The strength of shear reinforcement V_{us} shall be calculated as below:

a) For vertical stirrups:

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

Shear carried by steel, Vus = $V_u - \tau_c bd$ = 178.9 x 1000 – 0.39 x 1000 x 400 = 22.9kN

Using #8 mm 2-legged stirrups, Asv = 2 x π x 8^2 / 4 = 100.53 mm^2

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

Spacing of vertical stirrups, s_v = 633.99 mm < 0.75 x 400 < 300 mm

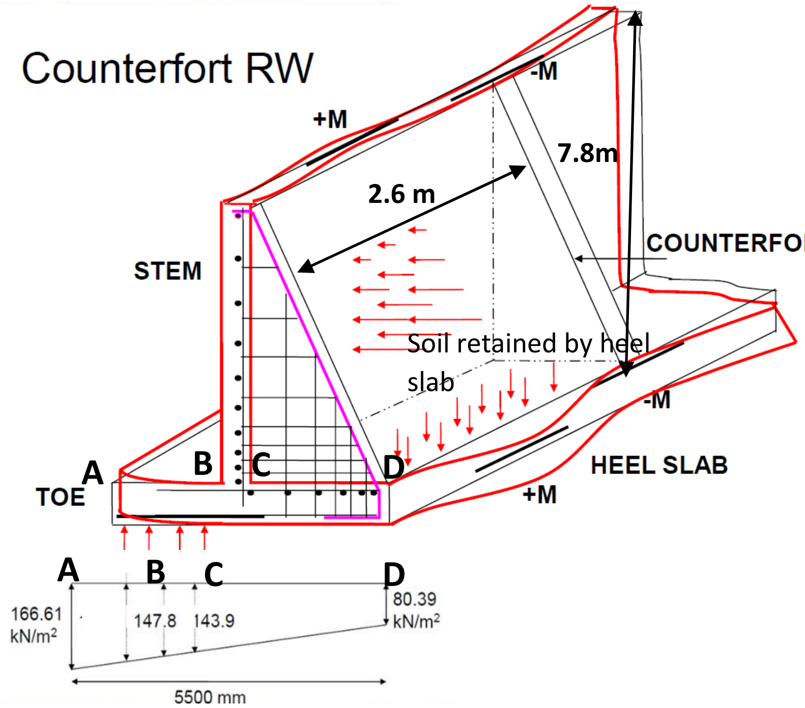
Provide #8 mm 2-legged stirrups at 290 mm c/c.

- **Design of Heel Slab**

Heel slab is a continuous slab. Consider 1 m wide strip near the outer edge **D**

The forces acting near the edge **D** are

1. Downward wt. of soil retained on heel slab =18 x 7.8 x l = 140.4 kN/m
2. Downward self wt. of heel slab = 25 x 0.45 x 1= 11.25 kN/m



Upward soil pressure at D = -80.39 kN/m^2

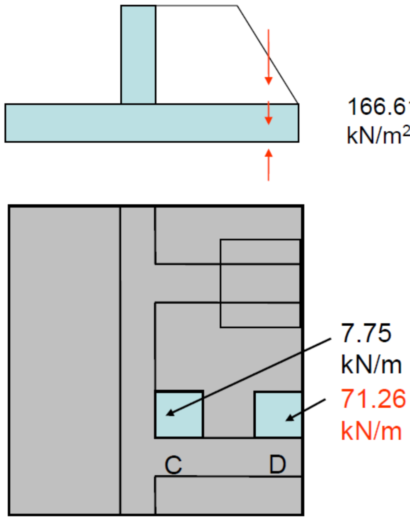
Upward wt due to soil pressure = - 80.39 x 1= - 80.39 kN/m

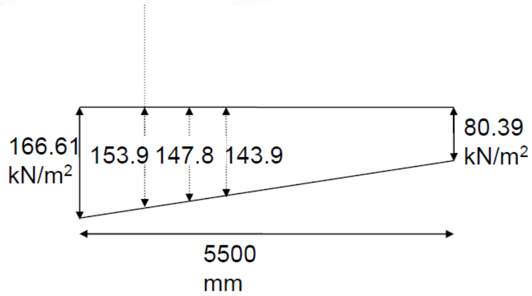
Net force at D , p = 140.4 + 11.25 - 80.39 = 71.26 kN/m

Also Net force at C, p = 140.4 + 11.25 - 143.9 = 7.75 kN/m

Factored Negative Bending Moment for heel at junction of counterfort (D)

Mu = 1.5 x pl^2/12 = 1.5 x 71.26 x 2.6^2/12 = 60.2 kN-m (At the junction of Counter Fort)





Forces on heel slab

- **To find steel**

b = 1000mm, d = 400 mm, fck = 20N/mm², fy = 415N/mm², Mu = 60.2 x 10⁶ Nmm

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

Find Ast = 426 mm^2 , Ast$_{min}$ = 0.12 x 1000 x 450/100 = 540 mm^2

426 mm^2 < 540 mm^2 Provide Ast = 540 mm^2

Provide # 12 mm @ 210 mm c/c < 300 mm

Check for shear (Heel slab)

Shear Force at D = 71.26 x 2.6/2 =

Factored shear = V_u= 1.5 x Shear Force = 139 kN

pt = 100 x 540 / (1000 x 400) = 0.13 and M20 concrete, ζ_c = 0.28 N/mm^2

ζ_v = V_{umax}/bd = 139 x 1000 /(1000 x 400) = 0.35 N/mm^2

$\zeta_c < \zeta_v$, Unsafe, hence shear steel is needed.

Using #8 mm 2-legged stirrups,

Shear reinforcement shall be provided to carry a shear equal to $V_u - \tau_c bd$ The strength of shear reinforcement V_{us} shall be calculated as below:

a) For vertical stirrups:

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

Shear carried by steel, Vus = $V_u - \tau_c bd$ = 139 x 1000 – 0.28 x 1000 x 400 =27 kN

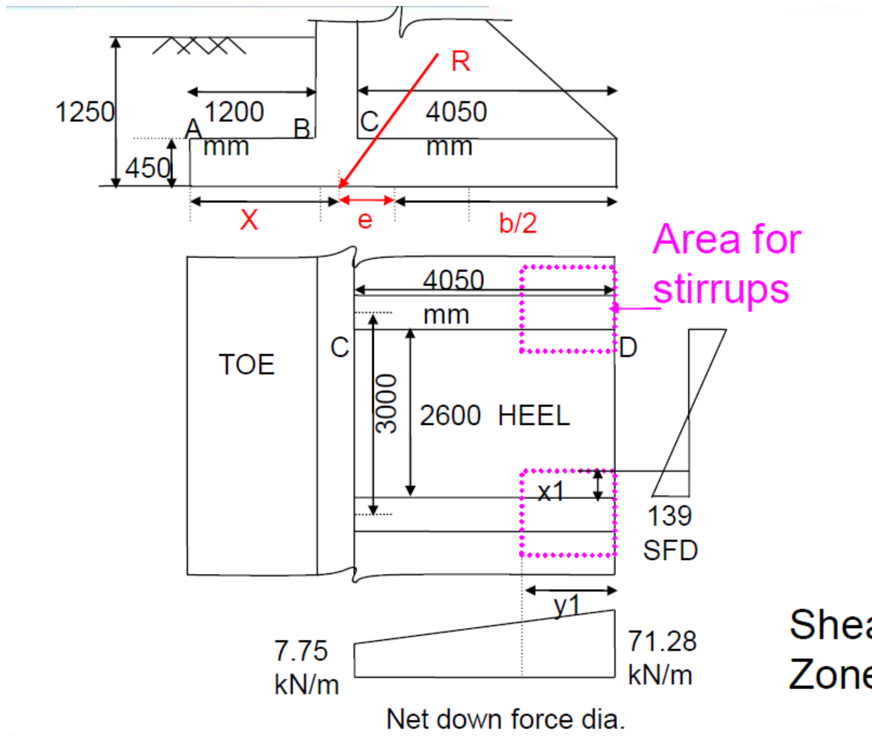
Using #8 mm 2-legged stirrups, Asv = 2 x π x 8^2 / 4 = 100.53 mm^2

$$V_{us} = \frac{0.87 f_y A_{sv} d}{s_v}$$

Spacing s_v = 538 mm < 0.75 x 400 = 300 mm

Provide #8 mm 2-legged stirrups at 290 mm c/c.

Provide for 1m x 1m area as shown in figure



Shear analysis and Zone of shear steel

- **Area of steel for +ve moment (Heel slab)**

Maximum +ve ultimate moment at mid span of heel slab = 1.5 x 71.26 x 2.6^2/16

= + 45.15 kN-m

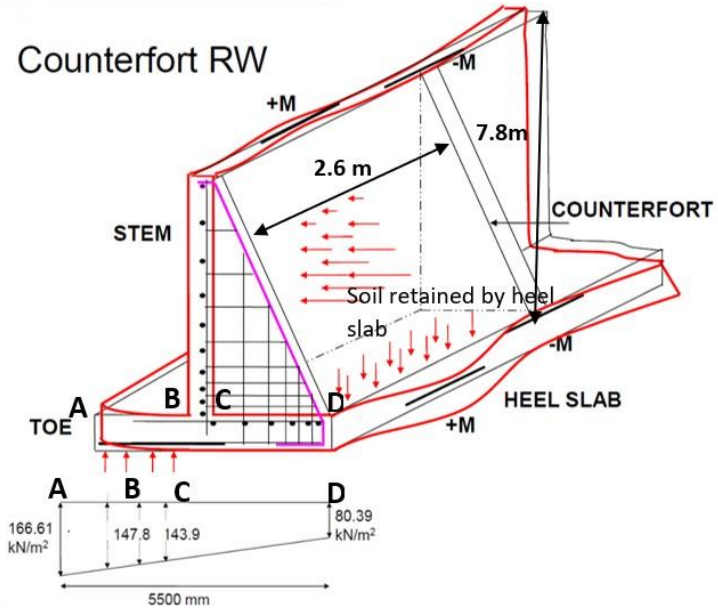
Since 45.15 kNm < 60. 2 kNm, Provide minimum steel.

Ast, min = 540 mm^2

Provide Main bars # 12 mm bars at 200 mm c/c < 300 mm

Also provide distribution steel # 12 mm at 200 mm c/c < 300 mm

- **Design of Stem (Vertical Slab)**



Consider stem slab as continuous slab spanning between the counterforts and subjected to earth pressure.

The intensity of earth pressure = p_a= ka x γ x h = $\frac{1}{3}$ x 18 x 7.8= 46.8 kN/m^2

For 1m , it will be 46.8 kN/m^2 x 1 m = 46.8 kN

Maximum -ve ultimate moment near ends of counterforts,

Mu= 1.5 x $p_a l^2/12$ = 1.5 x 46.8 x 2.6^2/12 = 39.54 kN.m.

Find the required effective depth or thickness of the stem slab

$$M_{u,lim} = 0.36 \frac{x_{u,\,max}}{d}\left(1 - 0.42 \frac{x_{u,\,max}}{d}\right) bd^2 f_{ck}$$

Mu, lim =39.54 x 10^6 N mm, xu, max/d = 0.48, b = 1000, fck =20N/mm^2

After calculations find 'd', d = 119.70 mm ≈ 120 mm

However, provide total depth or thickness, D = 250 mm. Hence safe.

- To find steel:
- Effective depth, d = 250 – 50 = 200 mm , (effective cover = 50 mm)
- **b = 1000mm, d = 200 mm, fck = 20N/mm^2, fy = 415N/mm^2, Mu = 39.54x 10^6 Nmm**
- Ast = 582.1 mm^2 , Ast min = 0.0012 x 1000 x 250 =300 mm^2
- Ast provided > Ast,min . Hence safe
- Provide #12 mm @ 210 mm c/c

As the earth pressure decreases towards the top, the spacing of the bars is increased .

Max. Ultimate shear = Vu_{max} = 1.5 x 46.8 x 2.6/2 = 91.26 kN

For pt = 100 x Ast / (1000 x 200) = 0.29 % and M20 concrete ζc = 0.38 N/mm^2

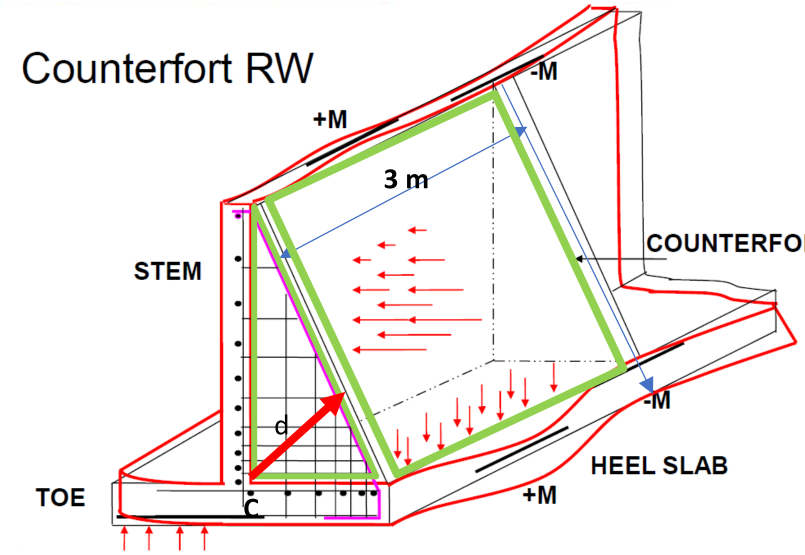
ζv = V_{umax}/bd = 91.26 x1000/(1000 x 200) = 0.45 N/mm^2

ζv > ζc , It is not safe in shear. Either increase the pt = 0.5%, so that ζc = 0.48 N/mm^2 or Provide shear reinforcement in the form of stirrups.

- **Design of Counterfort**

The total horizontal earth pressure acting on the counterfort = $\frac{1}{2}$ x k_a x γ x h^2 x c/c distance between counterfort

= $\frac{1}{2}$ x $\frac{1}{3}$ x18 x 7.8^2 x 3 = 547.56kN



B.M. at the base at C= **547.56 x 7.8/3** = **1423.65kN.m**.

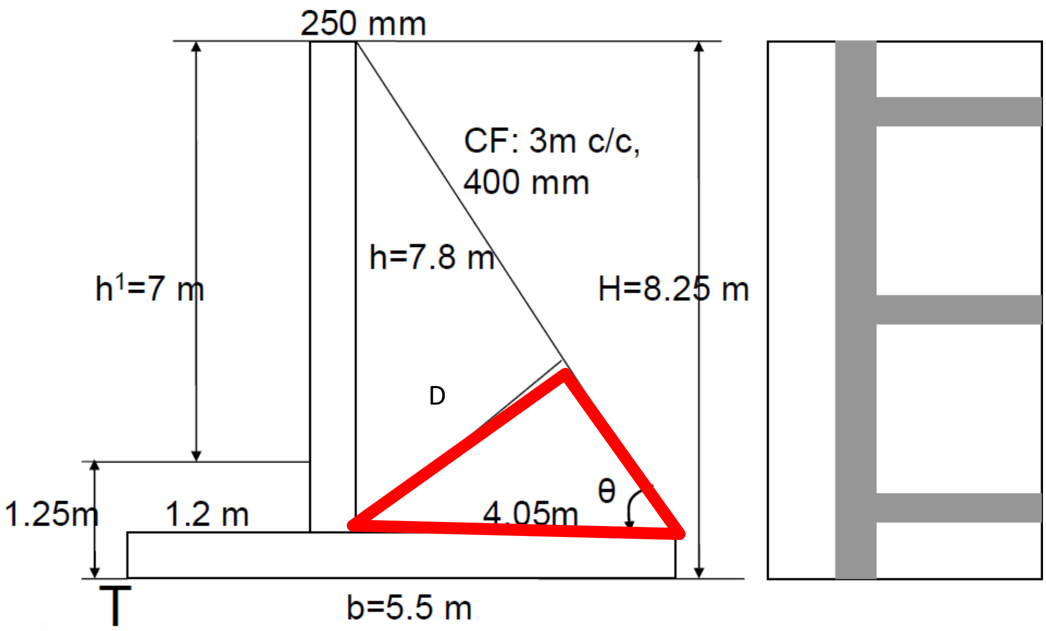
Ultimate moment = Mu= 1.5 x 1423.65 = 2135.48 kN.m.

Counterfort acts as a T-beam, lets find the effective depth

$$M_{u,lim} = 0.36\,\frac{x_{u,\,max}}{d}\left(1 - 0.42\,\frac{x_{u,\,max}}{d}\right) b d^2 f_{ck}$$

Mu,lim = 2135.48 x 10^6, xu, max/d = 0.48, b = 400, fck =20N/mm^2

Find ‘d’ = 1390 mm



The effective depth is taken at right angle to the sloping face of counterfort

tan θ = 7.8/4.05 = 1.93,

θ = $\tan^{-1}$(1.93) = 62.5° ,

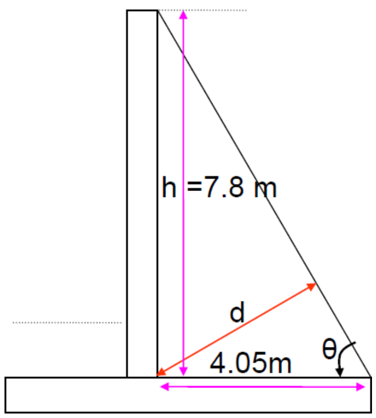
From the geometry

D /4.05 = sin 62.5 , D = 3.6 m = 3600 mm, d = 3600 – 50 = 3550 mm > 1390 mm . Hence depth of counterfort provided is safe.

- **To find steel**
  b = 400mm, d = 3550 mm, fck = 20N/mm^2, fy = 415N/mm^2, Mu = 2135.48 **x 10^6 Nmm**

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

Ast = 1708 mm^2



- **Check for minimum steel – IS 456 2000 CL 26.5.1.1**

26.5.1.1 *Tension reinforcement*

a) *Minimum reinforcement*—The minimum area of tension reinforcement shall be not less than that given by the following:

$$\frac{A_s}{bd} = \frac{0.85}{f_y}$$

where

A_s = minimum area of tension reinforcement,

b = breadth of beam or the breadth of the web of T-beam,

d = effective depth, and

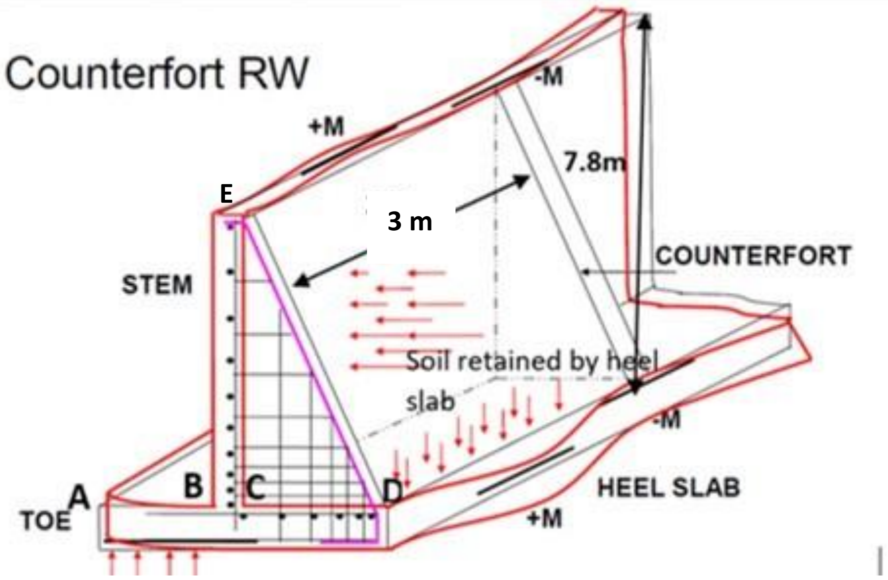
f_y = characteristic strength of reinforcement in N/mm².

- As per IS 456, Ast.min = 0.85 bd/fy = 0.85 x 400 x 3550/415 = 2908.4 mm^2 (T Beam section)
- Use 22 mm diameter bars, calculate no of bars = 2908.4 / (π x 22^2 /4) = 7.65 ≈ 8
- Provide 2 layers of bars ie 4 # 22 mm, 4 # 22 mm

- Development length = L_d = 47 x 22 = 1030 mm = **1.03 m**
- The half of the reinforcement can be curtailed is equal to √H = √7.8 = 2.79 m - **1.03 m** = 1.7 m from top, Bars are curtailed.

- **Design of Horizontal Ties or Horizontal stirrups (H S)**

The counter forts are subjected to tensile stresses along the outer face ED of the counter forts



The tension exerted on counterfort for 1 m height at base due to horizontal earth pressure, T

T = $k_a \times \gamma \times h \times$ c/c distance between counterfort $\times 1\ m$

= 1/3 x 18 x 7.8 x 3 x 1 = 140.4 kN

Area of steel required to resist the tension = Ast = $\frac{1.5 \times T}{0.87 \times f_y}$

1.5 x 140.4 x 10^3/ (0.87 x 415) = 583 mm^2

Using # 8 mm 2-legged stirrups, Ast = 100 mm^2 , spacing , s =

spacing = 1000 x 100/583 = 170 mm c/c.

Provide horizontal stirrups (H S) 2-legged # 8 mm at 170 mm c/c near bottom.

Since the horizontal pressure decreases with height, the spacing of stirrups can be increased from 170 mm c/c to 450 mm c/c towards the top.

- **Design of Vertical Ties or Vertical stirrups (V S)**

The maximum vertical tension exerted at the end of heel slab due to net downward force at D = 71.26 kN/m.

Total tension at D = 71.26 x c/c distance between counterforts = 71.26 x 3 = 213.78 kN

Area of steel required to resist the vertical tension = Ast = $\frac{1.5 \times T}{0.87 \times f_y}$=

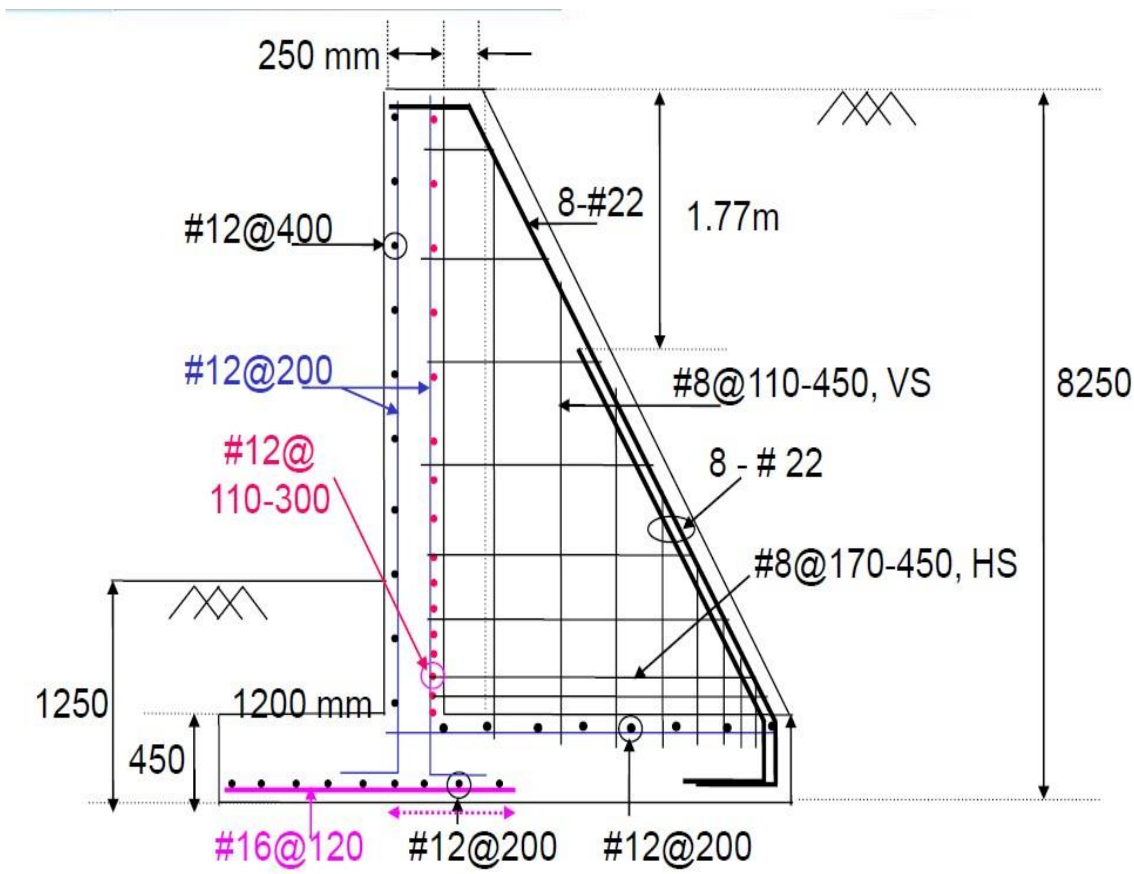
Required Ast = 1.5 x 213.78 x 10^3/ (0.87 x 415) = 888 mm^2

Using # 8 mm 2-legged stirrups, Ast = 100 mm^2

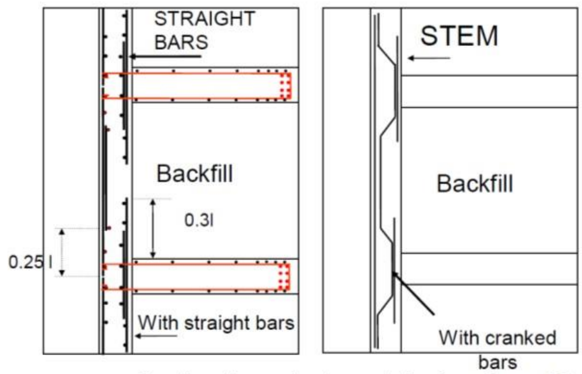
Spacing = 1000 x 100/888 = 110 mm c/c.

Provide vertical stirrups (V S) # 8 mm 2-legged stirrups at 110 mm c/c.

Increase the spacing of vertical stirrups from 110 mm c/c to 450 mm c/c towards the end C.



Cross section through counterforts



Section through stem at the junction of Base slab.