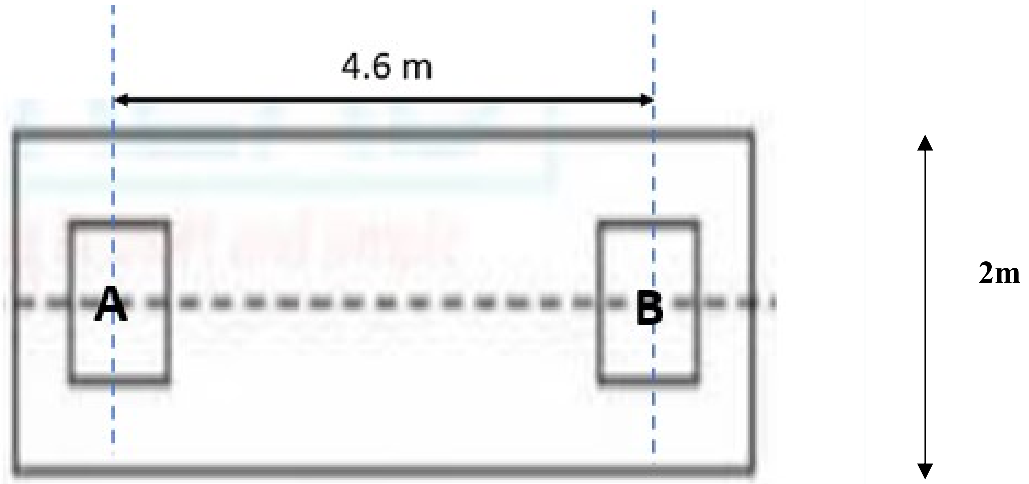
Sub :	Design of RC and steel structural elements					Sub Code:	18CV72/17CV 72	Branch:
Date:	10.02.22	Duration:	90 min's	Max Marks:	50	Sem / Sec:	ALL	

<u>Answer any one Questions- Use of IS 456 -2000 is permitted</u> MARKS

1 (a) Design a combined rectangular slab type footing for two columns A and B to carry loads of 500 kN and 700 kN. The cross section of column A is 300 x 300 mm and 400x 400 mm. The width of the footing is restricted to 1.8 m. The centre to centre spacing between the columns is 3.4 m. The safe bearing capacity of the soil is 150 kN/m^2. Use M20 concrete and Fe 415 steel. The design must include all necessary checks and draw the reinforcement details. [50]

Solutions



- **Footing base dimensions**

Assuming Δ P, the self-weight of the combined footing plus backfill to constitute 10 or 15 percent of the total column loads,

Δ P = (700 + 1200) x 15/100 = 285 kN

$P_1 + P_2$ = 700 +1200 = 1900 kN

Allowable soil pressure or safe bearing capacity, q_a = 130 kN/m^2

$Area\ of\ the\ footing, A_{req} = \frac{P_1 + P_2 + \Delta P}{q_a}$ = 16.8 m^2

Width of footing, B = 2m (Given in question)

Total Length of footing, L = $\frac{A_{req}}{B} = \frac{16.8}{2}$ = 8.4 m

- **Locate the point of application of the column loads**

In order to obtain a uniform soil pressure distribution, the line of action or point of application of the resultant column load must pass through the centroid of the footing.

Assuming a load factor of 1.5, the factored column loads are:

- $P_{u1} = 700 \times 1.5 = 1050$ kN; $P_{u2} = 1200 \times 1.5 = 1800$ kN $\Rightarrow P_{u1} + P_{u2} = 2850$ kN

  Let $\bar{x}$ be the centroid of the column loads, where s = 4.6 m

  $$\Rightarrow \bar{x} = \frac{P_{u2}\ s}{P_{u1+} + P_{u2}} =$$

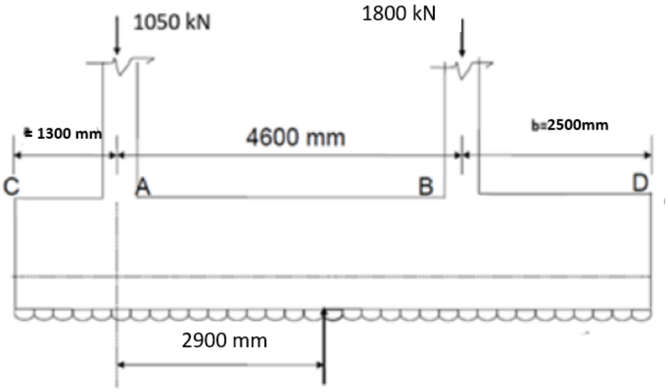
  $$= \frac{1800 \times 4.6}{1050+1800} = 2.9 \text{ m}$$

  If the cantilever projection of footing beyond column A is 'a' then,

  a + 2.9 = L /2 = a = 8.4/2 – 2.9 = 1.3 m

  Similarly, if the cantilever projection of footing beyond Column B is 'b' then,

  b = 8.4 - 1.3 - 4.6 = 2.5 m

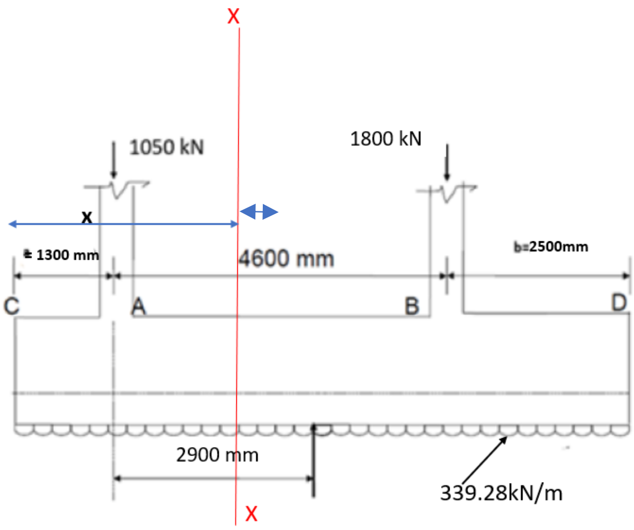


- **Uniformly distributed load acting in upward direction (soil pressure)**

Treating the footing as a wide beam (B = 2000 mm) in the longitudinal direction, the uniformly distributed load (acting upward) is given by q_{uB}

$$q_{uB} = \frac{P_{u1}+P_{u2}}{L} = \frac{1050+1800}{8.4} = 339.28 \text{ kN/m}$$

- **Shear force calculations**



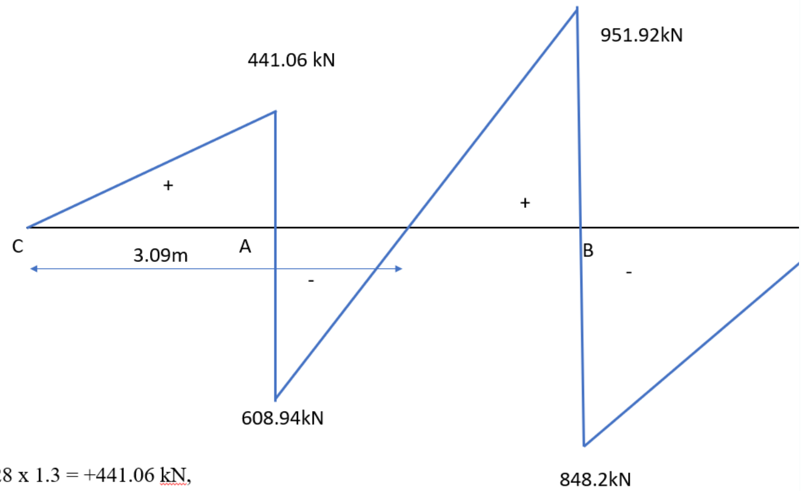
- Shear force at A, just before 1050 kN, left of section XX, V_{AC}= +339.28 x 1.3 = +441.06 kN
- Shear force at A, just after 1050 kN, left of section XX, V_{AB} = -1050+ 339.28 x 1.3 = -608.94 kN
- Shear force at B just after 1800kN, right of section XX , V_{BA} = +1800 – 339.28 x 2.5= + 951.92 kN
- Shear force at B just before 1800kN, right of section XX, V_{BD}=339.28 x 2.5= -848.2 kN

- **Location of zero shear , Left of section XX**

339.28 × X – 1050 = 0 , **location of zero shear,**

339.28 × X = 1050 , 1050/339.28 = X, X =3.09m

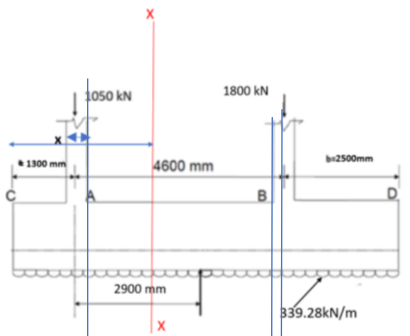
X = 3.09 m from C



- V_{AC}= 339.28 x 1.3 = +441.06 kN,
- V_{AB} = -1050+ 339.28 x 1.3= -608.94 kN
- V_{BA} = 1800 – 339.28 x 2.5= + 951.92 kN
- V_{BD}=339.28 x 2.5= -848.2 kN

Shear force diagram

- **Bending moment calculations**

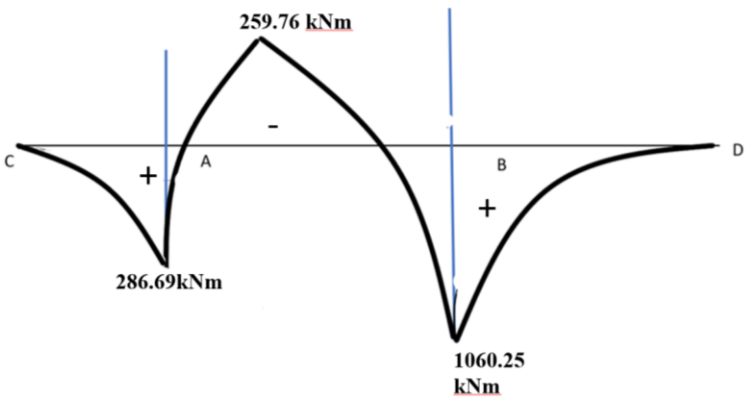


- BM at A, just before 1050kN, left of section XX, M_{AC}= 339.28 x 1.3 x1.3 /2 =+ 286.69 kNm
- BM at just at the inner face of Column A(1050kN), left of section XX,

  M_{AB} = -1050 x 0.35/2 + 339.28 x (1.3 + 0.35/2) x (1.3 + 0.35/2)/2

  = -1050 x 0.35/2 + 339.28 x (1.3 + 0.172) x (1.3 +0.172)/2 = + 185.32kNm
- Negative Bending moment at X = 3.09 m (Location of zero shear)
  Mu – = 339.28 × $(3.09)^2$/2 – 1050 × (3.09 -1.3) = - 259.76 kNm
- BM at B, just before 1800 kN , right of section XX = + 339.28 × 2.5^2/2 = +1060.25 kNm
- BM at B, just after the inner face of Column B (1800 kN) , right of section XX =

- 339.28 × **(2.5 +0.4/2)**^{2}/2 – 1800 × **0.4/2** = + 876.67 kNm

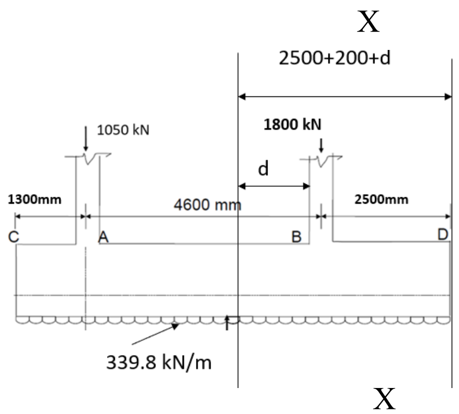


Bending moment diagram

- **Thickness of footing or effective depth of footing based on shear**

One-way shear (longitudinal): *Vu*1 calculate it at a distance "d" from the edge of the heavier column, where "d" is the effective depth of the footing.

The critical section (**always for column with heavier load**) for one-way shear is located at a distance *d* from the (**inner)**face of column B, and has a value



*Critical One-way shear force, Vu*1 at section XX (just right of XX section) =

Column load (B) - Uniformly distributed upward load intensity × (2500 + 200 + *d*)

= (1800 – 339.28 x (2.5 + 0. 200 + *d*)) = (882.54 – 339.28 x *d*) kN … (1)

Take τ_c = 0.48 N/mm^2 (for M 20 concrete, Assuming Percentage of steel as, pt = 0.50%) IS 456 2000, page 73, table 19

Design shear strength of concrete, $V_{uc} = \tau_c \times B \times d = 0.48 \times B \times d$

Equate V_{uc} *and* V_{u1}

B is width of footing = 2000 mm

$V_{uc} = 0.48 \times 2000 \times d = (960d)$ N…. (2)

Equating one-way shear force and design shear strength of concrete, (1) = (2)

$V_{u1} = V_{uc} \Rightarrow (882.54 \times 10^3 - 339.28 \times d) = 960d$, $882.54 \times 10^3 = 1299.8\ d$

⇒ Effective depth of footing, $d = 679.25$ **mm Rounded to 680 mm**

Use 20 mm φ bars with a clear cover of **75** mm, **Taking an overall depth or thickness of the footing**

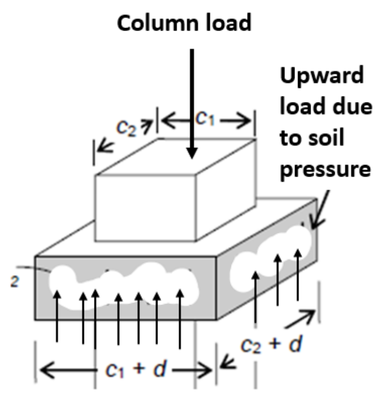
$\boldsymbol{D} = d + 75 + 20/2 = 680 + 75 + 20/2 = 765$ mm

- **Two-way shear force for columns *A* and *B* (Punching shear)**

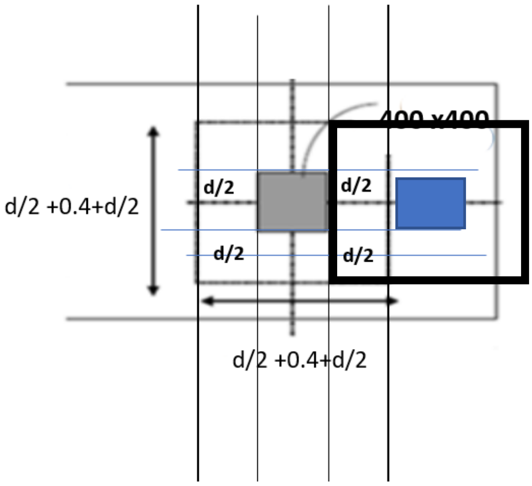
Two-way shear or punching shear ***(we need to consider the upward soil pressure not upward soil intensity) * Since it is acting on an area.***

Factored soil pressure or Upward soil pressure, $q_u = (339.28) / (B \times 1) = (339.8/2) = 169.64\text{kN/m}^2$

The critical section is located $d/2$ from the periphery of columns *A* and *B*.



Shear stresses in footing slab due to punching shear

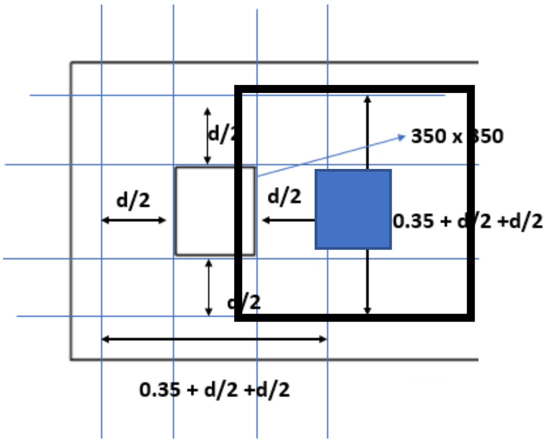


Punching shear or Two- way shear calculations for heavier Column B

V_{u2} = 1800 – 169.64 (0.4 + 0.680/2 + 0.680/2) X (0.4+0.680/2 + 0.680/2)

= 1602.13 kN @ B (Heavier column)

Punching shear or Two-way shear for Column A (350 mm x 350 mm)



Punching shear or Two-way shear @ A,

Two-way shear Vu2 = (Column load at A) 1050 – 169.64 x (0.35 +0.680/2 + 0.680/2) x (0.35 +0.680/2 + 0.680/2)

= 870 kN @ A (Lighter column)

- If no shear reinforcement is provided, Page 58, IS 456, Clause 31.6.3.1, calculated shear stress at critical section shall not exceed

  $k_s\,(0.25\sqrt{f_{ck}}\,)$

where

k_s = $(0.5 + \beta_c)$ but not greater than 1, β_c be ratio of short side to long side of the c capital; and

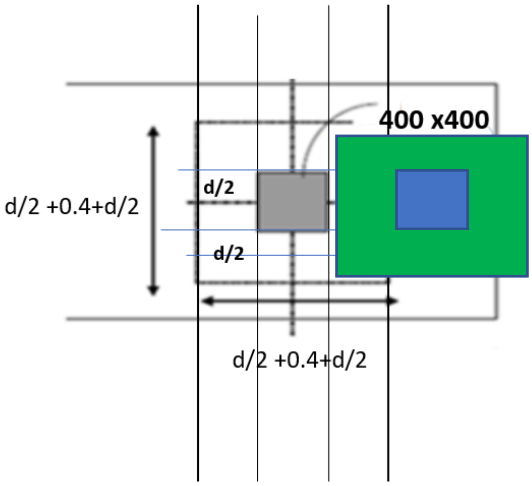
$\tau_c = 0.25\sqrt{f_{ck}}$ in limit state method of and $0.16\sqrt{f_{ck}}$ in working stress met design.

For square columns, $ks = (0.5 + \beta c)$, $\beta c = 350/350 = 400/400 = 1.0$, ks =(0.5 +1) but it should not be greater than 1, hence ks = 1

Permissible shear stress, $\tau_{c2} = k_s(0.25\sqrt{f_{ck}}) = 1.0 \times 0.25 \times \sqrt{20} =$ **1.118** N/mm^2

Permissible two-way shear force for column B *(heavier column)*

Permissible two-way shear force, V_{uc} = Permissible shear stress × **(Area of the footing slab enclosed by the perimeter of the critical section)**



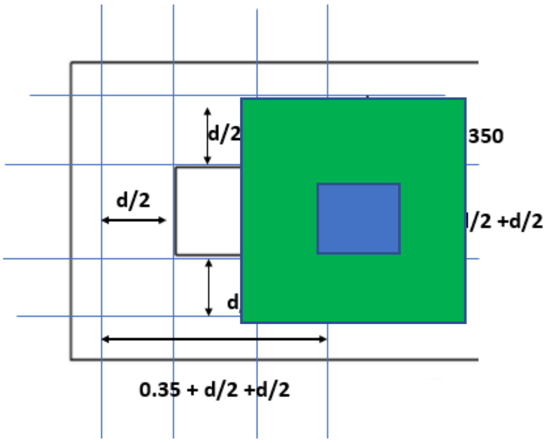
Perimeter of critical section (Green coloured area) = 4 X (400 + 680/2 + 680/2)

V_{uc} = 1.118 x [**4 X (400 + 680/2 + 680/2)**] × 680 = 3284.24 kN @ B

In the similar way lets calculate for Column A

Permissible two-way shear force for Column A

V_{uc} = 1.118 × [(350 + 680/2 + 680/2) × 4] × 680 = 3132.18kN @ A



Compare whether permissible two way shear force is greater than two shear way (Actual) force

V_{uc} = 3284.23 kN > V_{u2} = 1602.82 kN **@ B It is safe.**

V_{uc} = 3132.18**kN >** V_{u2} = 870.00kN **@ A . It is Safe.**

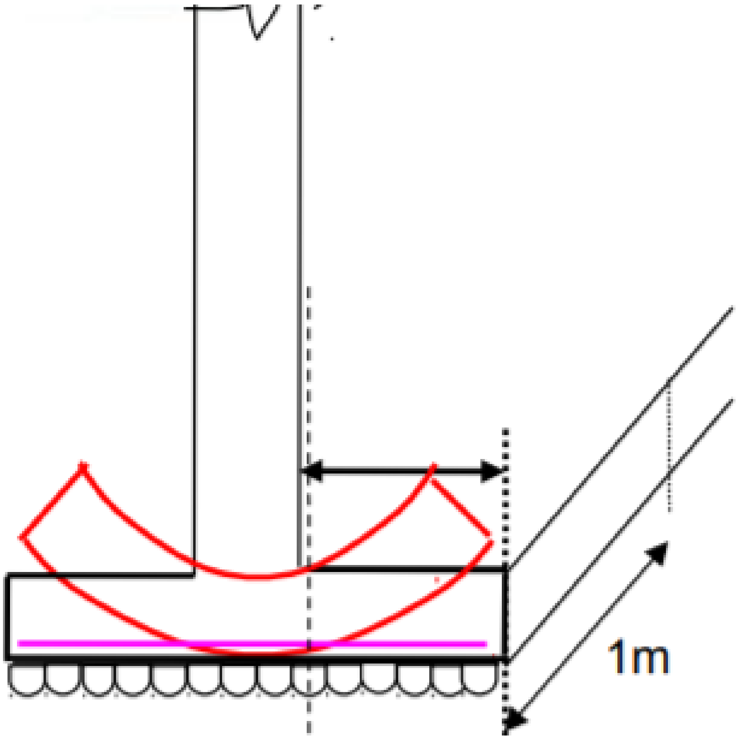
Hence safe against two way or punching shear, (if not provide shear reinforcement- stirrups or bent up bars)

- **Design of longitudinal flexural reinforcement**

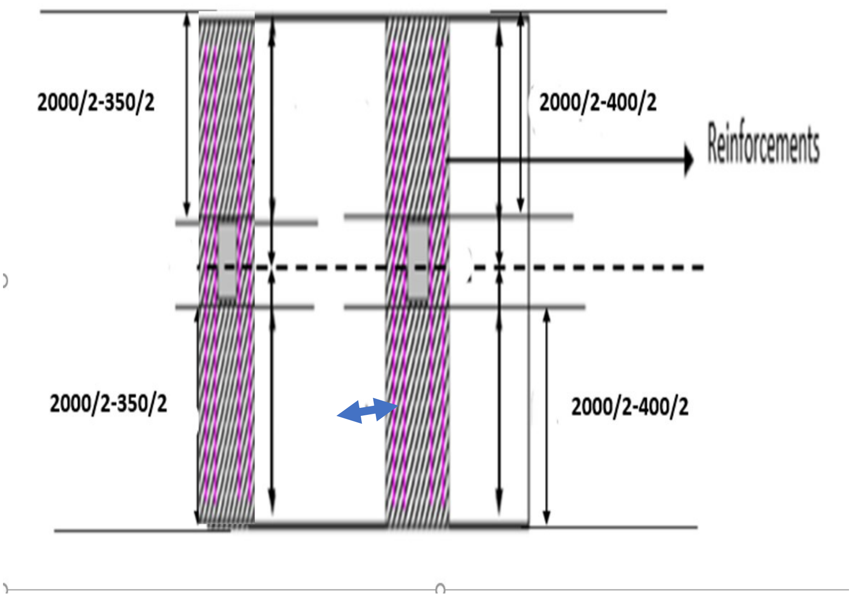
Maximum 'negative' moment: *Mu* = - 259.76 kNm	Maximum 'positiv
$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$$ *Mu* = 259.76 × 10^6 N mm B = b = 2000 mm, fck = 20 N/mm^2, fy = 415N/mm^2 d = 680 mm D = 765 mm Ast provided = 1075.67 mm^2 Check for (*Ast*)*min* = 0.0012 *BD* = 0.0012 × 2000 × 765 = 1836 mm^2 Ast provided < (*Ast*)*min, Hence provide (Ast)min*	$$M_u = 0.87\, f_y\, A_{st}\, d\left(1 - \frac{A_{st}\, f_y}{bd\, f_{ck}}\right)$$ *Mu* = 1060.25 × 1 B = b = 2000 mm d = 680 mm Ast provided = 4(Check for (*Ast*)*mi* 0.0012 × 2000 × 7 No of 20 mm dia b

But we have assumed pt = 0.5 pt = 100 Ast, req/ (B × d) Choose 20 mm diameter bars, calculate no of bars = 1836 /($\pi/4 \times 20^2$) = 6 **Provide 6 # 20 mm diameter bars at top** • **Development length L_d = 47 x dia of bar** **= 47 x 20 = 940 mm**	**Provide 15 # 20m** • **Development** **= 47 x 20 = 94**

Design of column strips as transverse beams



Transverse bending of footing



Transverse beam under column A	*Transverse beam unc*

- Factored Column load A per width of footing = 1050/2.0 = 525 kN/m
- Cantilever Projection of beam beyond column face = (2000 – 350)/2 = 825 mm =0.825 m
- Maximum transverse moment at column face A :

$Mu = 525 \times 0.825^2/2 = 178.66$ kNm

- Assume width of transverse beam, b =

width of column + 2 x 0.75*d*

b = 350 + 2 x 0.75 × 577 = 1215.5 mm

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

b = 1215.5 mm, d =577 mm

Mu = 178.6 x 10^6 N mm

Ast = 880.23 mm^2

- Factored Colu
width of footi
900 kN/m
- Cantilever Pr
column face =
800 mm = 0.8
- Moment at cc
$900 \times 0.80^2/2$
- Width of tran
width of colu
- 400 + 2 ×0.7
1265.5mm

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

Mu = 384 x 1

b = 1265.5m

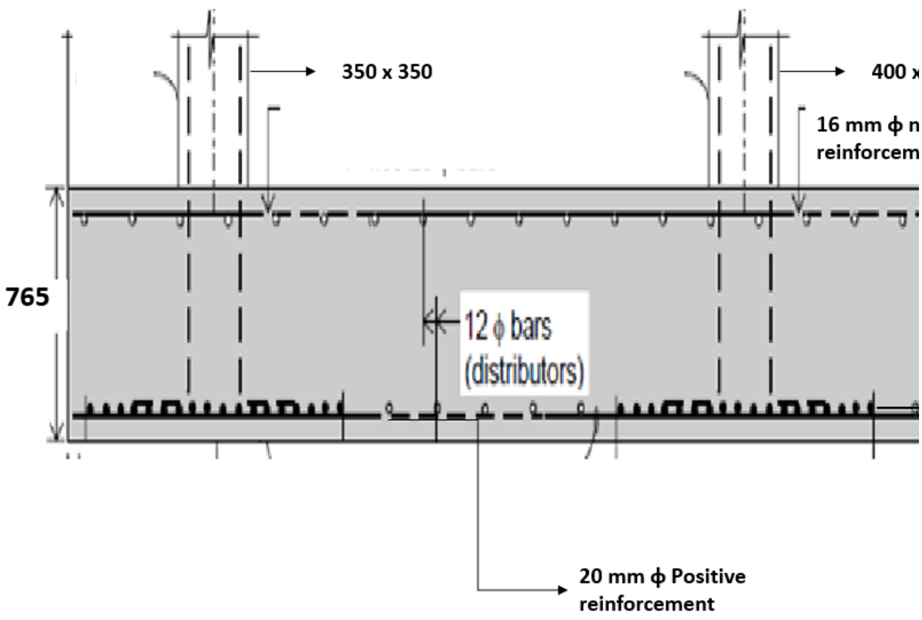
d = 577 mm

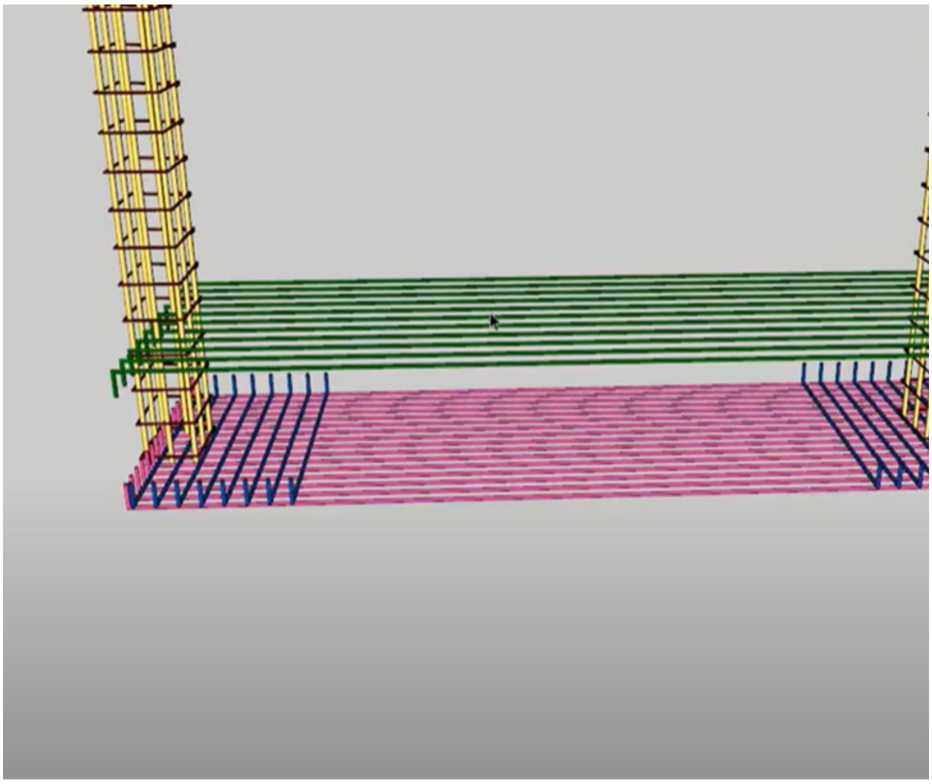
Ast = 1956.

<table>
<tr>
<td>
Page 48, CL No 5.2.1
• Minimum Ast = 0.0012 bD =
$Ast\ min$ = .0012 ×1215.5 ×765 = 1115.83 mm^2
• Use 12mm dia bars (Your wish!!)
• Number of 12 mm φ bars required = Ast/ area of one bar = 1115.83/ (π/4 x 12^2) =10
Provide 10 nos 12 mmφ bars
Check for development length = 47 × 12 = 564 mm
</td>
<td>
• Provide (Ast)
1265.5×765
Use 12 mm dia bars
Number of 12 mm φ
=1956.2 /(π/4 x 12^2)
Provide 18 nos 12 n
• Required dev
47.0 × 12 = 5
beyond the cc
</td>
</tr>
<tr>
<td>Transfer of force at column base -Column A</td>
<td>Transfer of force
Column B</td>
</tr>
<tr>
<td>
• Limiting bearing stress at
IS 456 Page 65 , CL34.4
34.4 Transfer of Load at the Base of Column
The compressive stress in concrete at the base of a column or pedestal shall be considered as being transferred by bearing to the top of the supporting pedestal or footing. The bearing pressure on the loaded area shall not exceed the permissible bearing stress in direct compression multiplied by a value equal to $\sqrt{\frac{A_1}{A_2}}$ but not greater than 2;
where
A_1 = supporting area for bearing of footing, which in sloped or stepped footing may be taken as the area of the lower base of the largest frustum of a pyramid or cone contained wholly within the footing and having for its upper base, the area actually loaded and having side slope of one vertical to two horizontal; and
A_2 = loaded area at the column base.
For working stress method of design the permissible bearing stress on full area of concrete shall be taken as $0.25 f_{ck}$; for limit state method of design the permissible bearing stress shall be 0.45 f_{ck}.
Permissible bearing stress = $0.45 fck \sqrt{\frac{A1}{A2}}$
</td>
<td>
• Limiting bear
Permissible bearing s
[$A1 = 2000^2$, $A2$ = 4
$\sqrt{\frac{A1}{A2}} = 5 > 2$, $\sqrt{\frac{A1}{A2}}$ =
= 0.45 × 20 × 2 = 18
Permissible bearing
400^2 = 2880 kN
2880kN > 1800kN ,
</td>
</tr>
</table>

$A1 = 2000^2$ (2000mm is footing width) A2 = 350 x 350 = $= \sqrt{\frac{A1}{A2}} < 2$ = 5.71> 2 , Take $\sqrt{\frac{A1}{A2}} = 2$ Permissible bearing stress = 0.45 X 20 X 2 =18 N/mm^2 Permissible bearing resistance or force F_{br} = Permissible bearing stress × column area $= 18 \times 350^2 = 2205 \times 10^3$ N = 2205 kN *2205> 1050 kN, Hence safe.*	

Reinforcement detailing





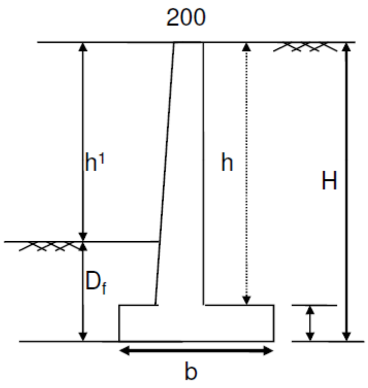
OR

2 (a) Design a cantilever retaining wall to retain the earth embankment of height 4m above the ground level. The density of soil is 18 kN/m^3 and angle of repose is 30^0. The safe bearing capacity of the soil is 200 kN/m^2. Coefficient of friction between the soil and concrete is 0.5. Use M20 concrete and Fe 415 steel. The design must include all necessary checks and draw the reinforcement details. [50]

Height of earth fill, h'= 4m, Safe bearing Capacity = 200 kN/m^2,
Density of soil, γ = 18 kN/m^3, co-efficient of friction between concrete and soil, μ = 0.6, angle of repose φ = 30°
We need to fix the height of retaining wall, $H = h' + D_f$

- **Depth of foundation, D_f**

Using Rankine's formula: find depth of foundation



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

$$k_a = \left[\frac{1-\sin 30^0}{1+\sin 30^0}\right]^2 = \frac{1}{3}$$

$$= \frac{200}{18} \times \frac{1}{3}$$

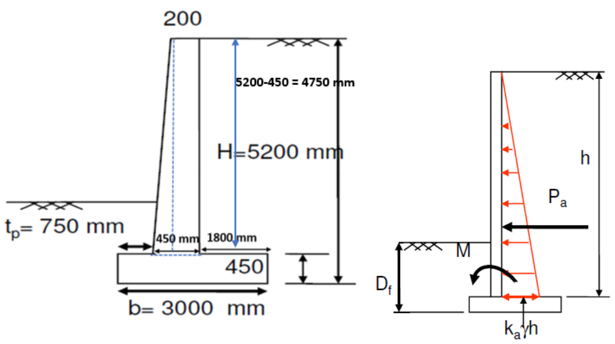
=1.23m say 1.2m, therefore H = 4 +1.2 = 5.2 m

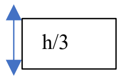
- **Proportioning of stem wall and base slab**

a. Thickness of base slab = **(1/10 to 1/14) H** = 1 /10 × 5.2 to 1 /14 × 5.2 = 0.52m to 0.43m, say 0.45 m or 450 mm
b. Width of base slab = b = **(0.5 to 0.6) H** = 0.5 × 5.2 to 0.6 ×5.2 = 2.6m to 3.12 m say 3m
c. Toe projection or width of toe slab = pt = (**1/3 to ¼) b** = 1 /3 × 3 to 1/4 × 3 = 1.0 m to 0.75 m say 0.75 m
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 ,Pa $= \frac{1}{2}(k_a \times \gamma \times h)\, h$

Pa = $\frac{1}{2} \times \frac{1}{3} \times 18 \times 4.75 \times 4.75 = 67.68$ kN

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

M = $67.68 \times \frac{4.75}{3}$ =107.17 kN-m

Mu = 1.5 × M = 160.6 kN-m

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

Check for effective depth

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

Put Mu,lim = 160.6 x 10^6, b = 1000mm , fck = 20N/mm^2

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

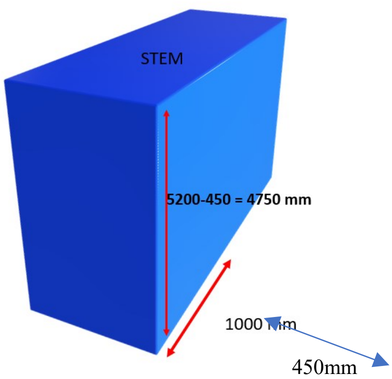
160.6 x 10^6 = 0.36 x 0.48 x (1 - 0.42 x 0.48) x 1000 x d^2 x 20

Find 'd' , d = 242 mm

Required, d = 242 mm

Assumed overall depth at base of the stem, D = 450 mm, effective, d = 450 - 50 = 400 mm >> 242 mm

Taking 1m length of stem wall,



b = 1000 mm, d = 450 - effective cover

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

= 40 + 12/2 = 46 ≈ 50 mm

d = 450 – 50 = 400 mm >> 242 mm , hence safe

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

d = 400 mm, b =1000 mm, Mu =160.6 x 10^6 Nmm, fy = 415 N/mm^2 , fck = 20 N/mm^2

Ast = 1184.6 mm^2

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

Ast > Ast,min, hence Ok.

Provide 12 mm φ bars as main steel

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 12^2}{1184.6}$ = 96 mm ≈ 100 mm or 95 mm (Your wish!!)

Main steel #12 mm φ @ 100 mm c/c < 300 mm or 3 times effective depth "d" (Check!!!) IS 456 2000

Distribution steel

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

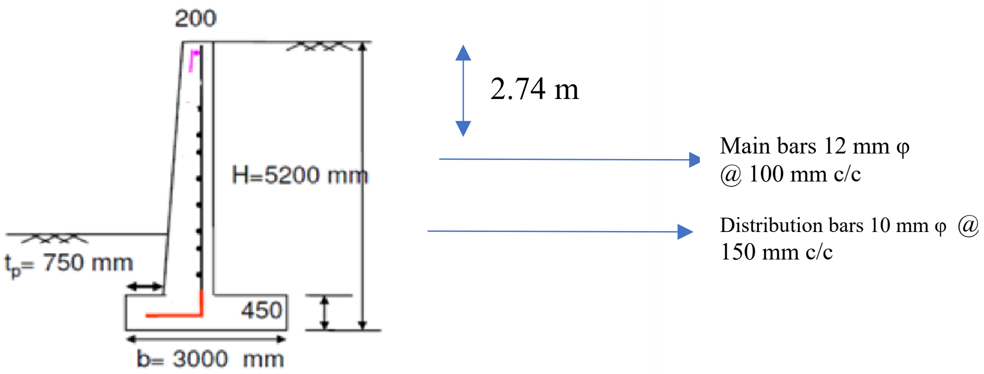
Use 10 mm φ bars , spacing required

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 10^2}{540}$ = 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

$L_d = 47\ \Phi_{bar} = 47 \times 12 = 564$ mm =0.564 m



Curtailment of bars

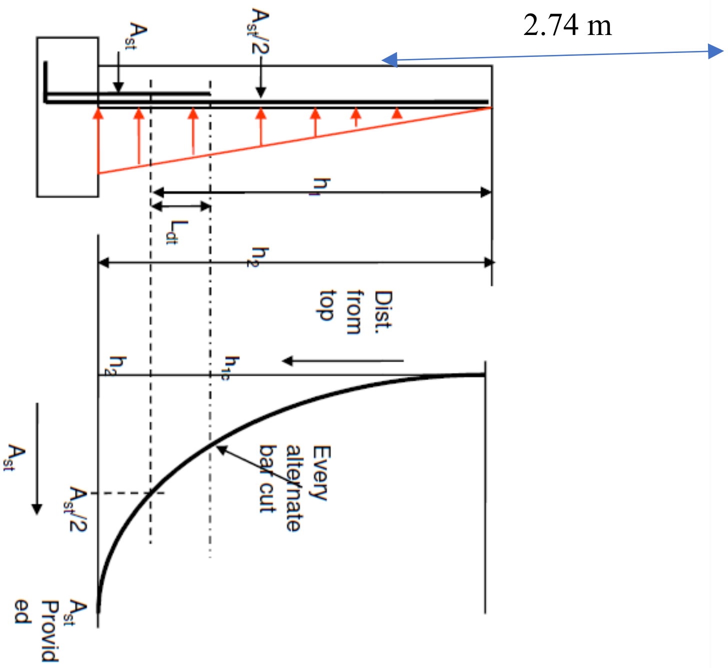
Curtail 50% steel from top, Ast = $\frac{50}{100} \times 1184.6 = 592.3\ mm^2$

$\left(\frac{h_1}{h}\right)^2 = \frac{1}{2}$, $\left(\frac{h_1}{4.75}\right)^2 = \frac{1}{2}$, $\frac{h_1{}^2}{4.75^2} = \frac{1}{2}$, $h_1 = 3.36\ m$, is the curtailment length

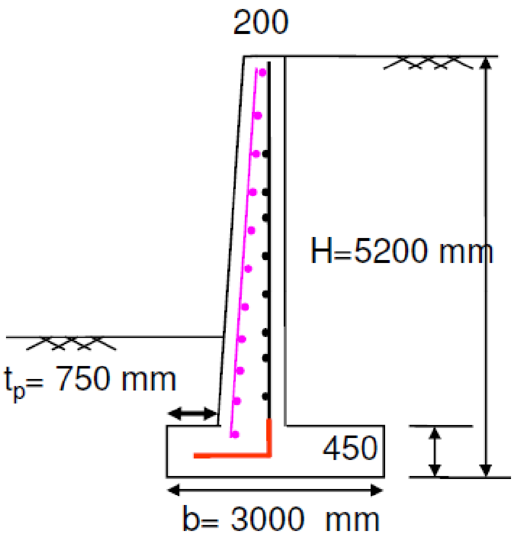
Actual point of cut off or cutting position = 3.36 - L_d =3.36 - 0.564 = 2.74m from top.

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 12^2}{592.3}$ = 190.9 mm ≈ 190 mm

Spacing of bars 12 mm φ @ 190 mm c/c < 300 mm and 3 x effective depth . Hence it is ok

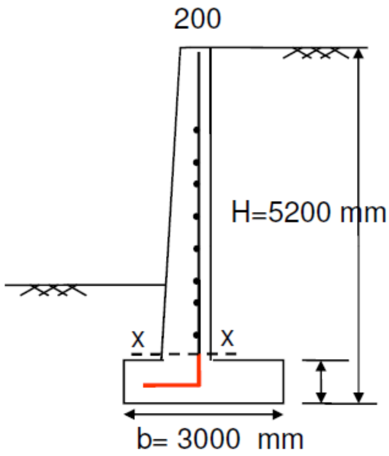


Secondary steel for stem at front (Temperature steel)
0.12% Gross Area = 0.12 × 450 × 1000/100 = 540 mm²
This can be copied from distribution steel calculations
#10 @ 150 mm c/c< 450 mm and 5d ok



Check for shear

Max. Shear Force at Junction XX= P_a = 67.68 kN (Lateral earth pressure)
Ultimate Shear Force = Vu =1.5 x 67.68 = 101.52 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 pt $= \frac{100\,A_{st}}{b \times d} = \frac{100 \times 1184.6}{1000 \times 400} = 0.29$ %

Use IS:456-2000, Page 73, Table 19, pt = 0.29 %, M 20

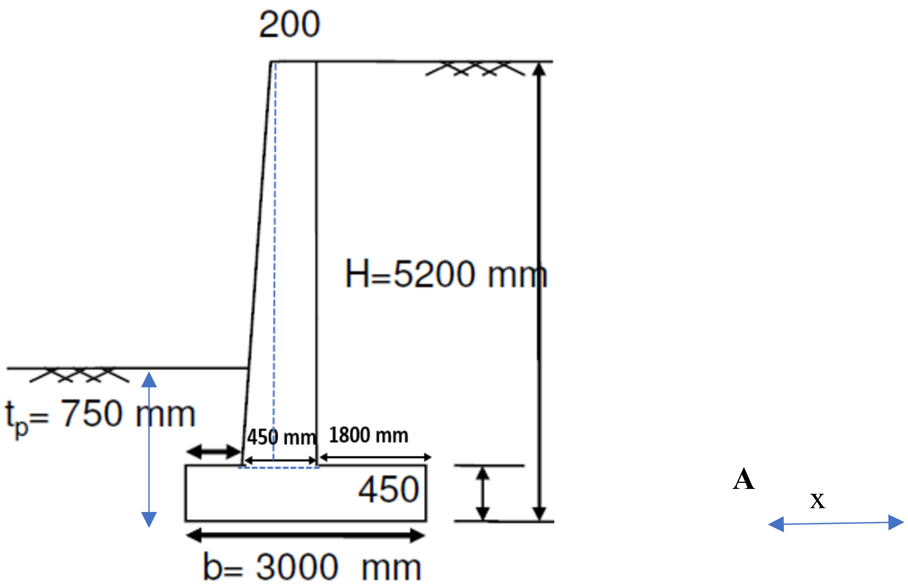
$\tau c = 0.38$ N/mm^2

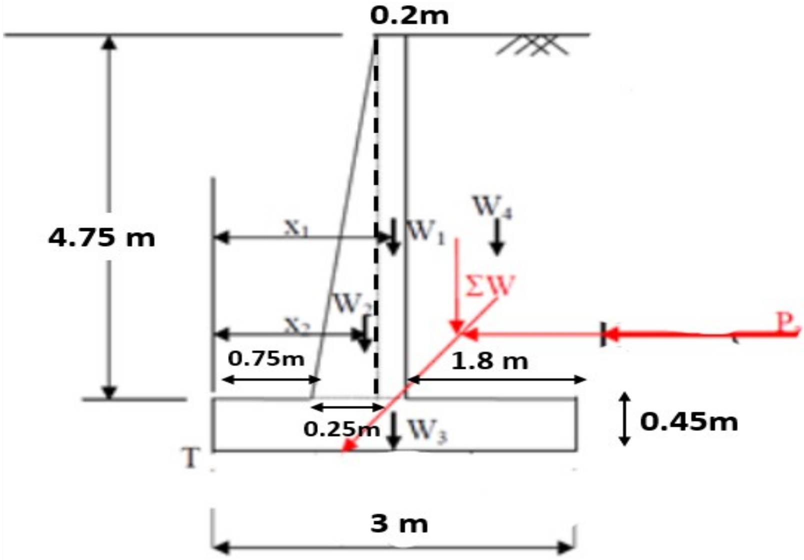
Compare τ_v and τ_c , 0.25 < 0.38

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

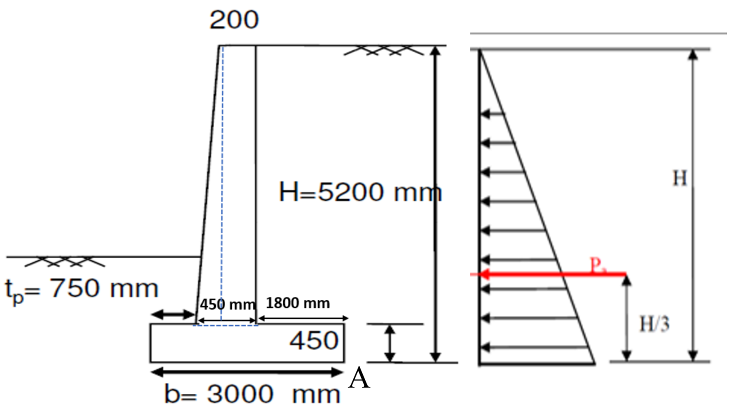
Stability analysis – 1. To find factor of safety against overturning

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





Load	**Magnitude, kN**	**Distance from A m**
Stem W1	0.2 x 4.75x1x25 = 23.75	(0.75+0.25+0.2/2) =1.1
Stem W2	½ x 0.25x4.75x1x25 = 14.84	0.75 + 2/3 x 0.25 = 0.916
Base slab W3	3.0x0.45x1x25 = 33.75	3/2
Back fill, W4	1.8x4.75x1x18 = 153.9	0.75+0.45+1.8/2 =2.1
Total	**ΣW= 226.24 kN**	



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

Load	Magnitude, kN	Distance from A, m	
Hori. earth pressure $=P_H$	$P_H = ½ \times 1/3 \times 18 \times 5.2^2$ =81.12 kN	H/3 =5.2/3	

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 = 413.55 kNm, M_O = 140.05 kNm

(F.S) overturning = $\Sigma M_R / M_O$ = 2.94 >1.55 Hence it is safe

2. **Check for Sliding**:

ΣW= 226.24kN

P_H =81.12 kN

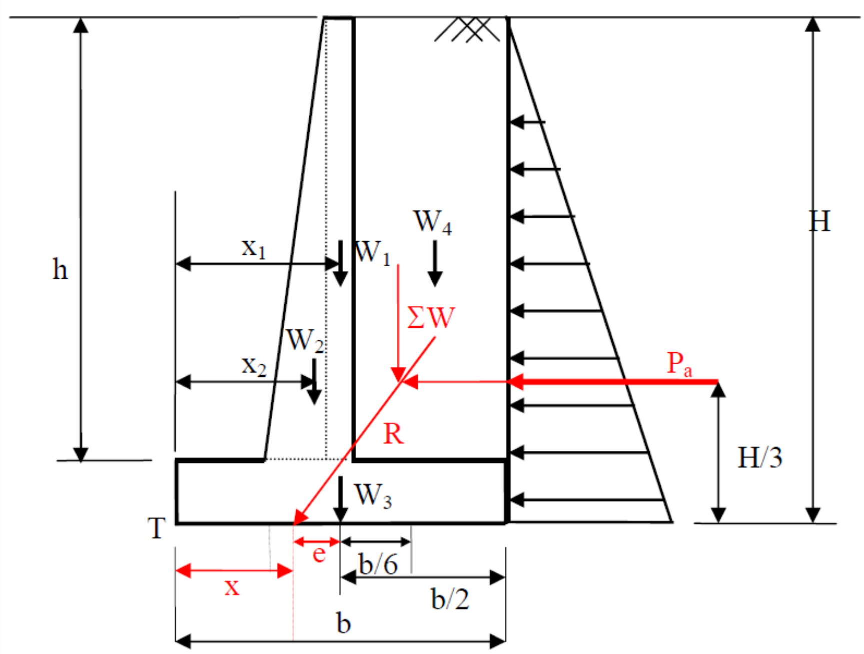
As per IS: 456:2000, *(F.S) sliding* should satisfy condition that $\mu \sum W / P_H \geq 1.55$

$$\frac{\mu \sum W}{P_H} = \frac{0.6 \times 226.24}{81.12} = 1.67$$

(F.S) sliding = 1.67 ≥1.55 Hence it is safe

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 = $\Sigma M_R - M_O$ = 413.55 -140.05 =273.5 kNm



$$x = \frac{273.5}{226.24} = 1.2 \text{ m} \quad , b = 3\text{m}$$

- **Eccentricity** **e = b/2 – x** = 3/2 – 1.2 = 1.5 – 1.2 **= 0.3m < b/6** (0.3 < 0.5)(Eccentricity of force should not exceed one sixth of base)

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

Pressure below the base slab , SBC =200 kN/m²

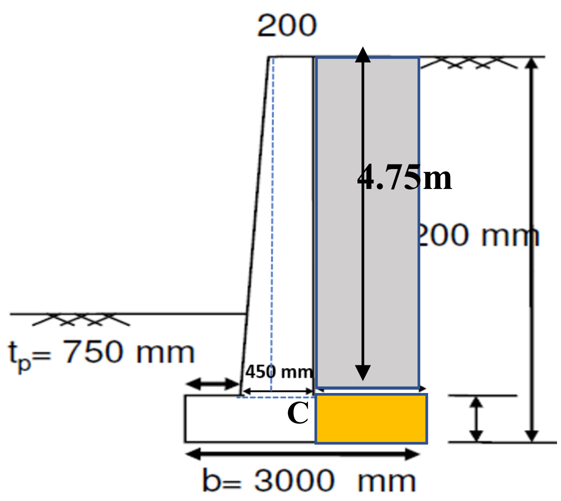
$$\text{Max. pressure} = P_{max} = \frac{\sum W}{b}\left[1 + \frac{6e}{b}\right]$$

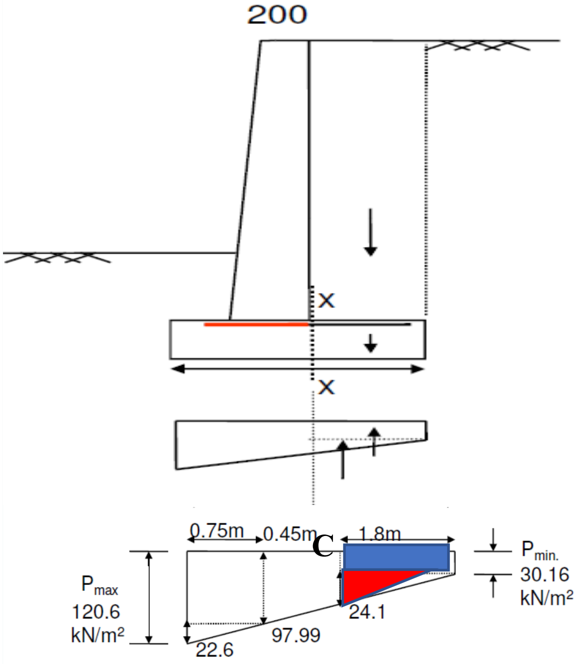
120.66 kN/m^2 < SBC, safe

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

30.16 kN/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.





Pressure below the Retaining Wall

Using similar triangles,

$$\frac{120.6-30.16}{3} = \frac{30.16+y}{1.8}$$, y =24.1 kN/m^2 (Sample calculations)

Design of Heel Slab

Calculations of Moment about heel slab C

Load	Magnitude, kN	Distance from C, m	B
Wt of Backfill or Earth fill	1.8 x 4.75 x 1x18 =153.9	1.8/2 = 0.9	=-
Heel slab	0.45x1.8x25 = 20.25	1.8/2 =0.9	=
Pressure distribution,(below heel slab) rectangle	- 30.16 x 1.8 = -	1.8/2 = 0.9	-

Pressure distribution, Triangle 	$\frac{1}{2}$ x -24.1 x1.8= -21.69	$\frac{1}{3}$ x1.8 = 0.6	-1
Total Load at junction C	98.17	**Total BM at Junction C**	**Σ**

$\Sigma M_C = 94.87$
Mu = 1.5 x 94.87 =142.3 kNm = 142.3 x 10^6 N mm

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

b = 1000 mm, d = 400 mm, f_{ck} =20 N/mm^2 f_y = 415 N/mm^2

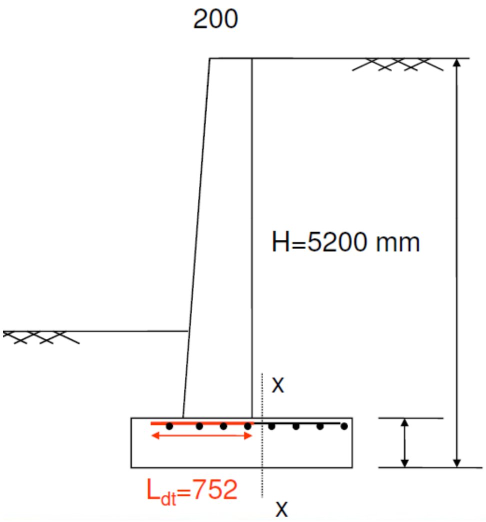
Ast = 1041.5 mm^2
Use 16 mm φ bars (it is base slab) (You can choose 12 mm also)

Spacing required, s = $\frac{1000 \times \frac{\pi}{4} \times 16^2}{1041.5}$ = 193 mm ≈ 190 mm

Main steel #16 mm Φ @ 190mm c/c < 300 mm and 3d ok. Hence it is safe.

Development length
$L_d = 47\ \varphi_{bar} = 47 \times 16 = 752$ mm

Distribution steel
Same, #10 dia @ 140 < 450 mm and 5d ok



Check for shear at junction (Tension)
The critical section for shear in the heel slab should be taken at the face of the support and not *d* away from it, because there is no compression introduced by the support reaction, and the probable inclined crack may extend ahead of the rear face of the stem

Critical section for shear is at the face as it is subjected to tension.
Maximum shear =V=98.17 kN, $V_{U, max}$= 98.17 x 1.5 =147.255 kN

$\tau_v = \frac{V_U}{b \times d} = \frac{147.255 \times 10^3}{1000 \times 400} = 0.368$ N/mm^2

$\tau_v = 0.368$ N/mm^2

$pt = \frac{100 \times 1041.5}{1000 \times 400} = 0.260$ % , Find τ_c

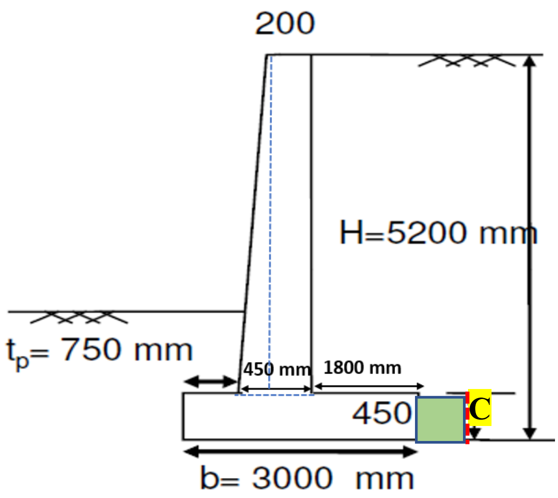
Use IS:456-2000, Page 73, Table 19, pt =0.26 %

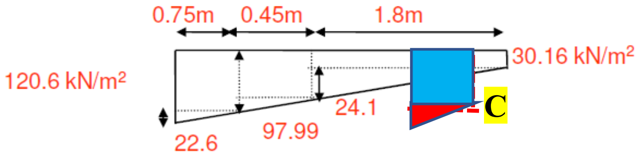
$\tau_c = 0.365$ N/mm^2

Here τ_v and τ_c are almost close. There is no providing shear reinforcement. May be Ok.

Design of toe slab

To find the maximum bending moment

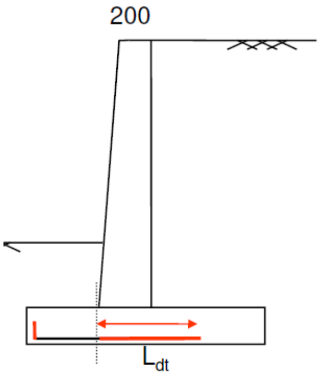




Pressure below the Retaining Wall

Load	Magnitude, kN	Distance from C,
Self wt of Toe slab	0.75 x 0.45 x 25 = 8.44	0.75/2
Pressure distribution, rectangle	-97.99 x 0.75 = -73.49	0.75/2
Pressure distribution, Triangle	½ x -22.6 x 0.75 = -8.474	2/3 x 0.75=
		Total BM at Junction C

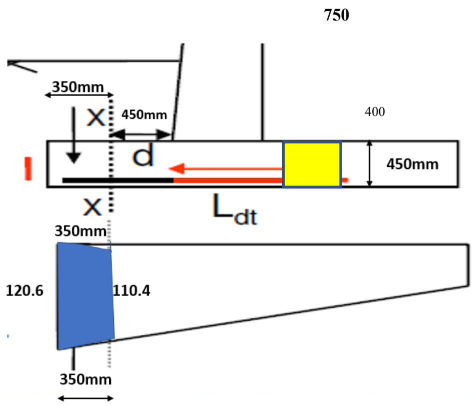
$M_u = 1.5 \times 28.63 = 43$ kN-m,
$A_{st} = 302.48\ mm^2$
Provide #10mm @ 140mm < 300 mm and 3d ok (If you have time you can do calculation for spacing otherwise you proceed like this!)
Development length:
$L_d = 47\ \varphi bar = 47 \times 10 = 470$ mm

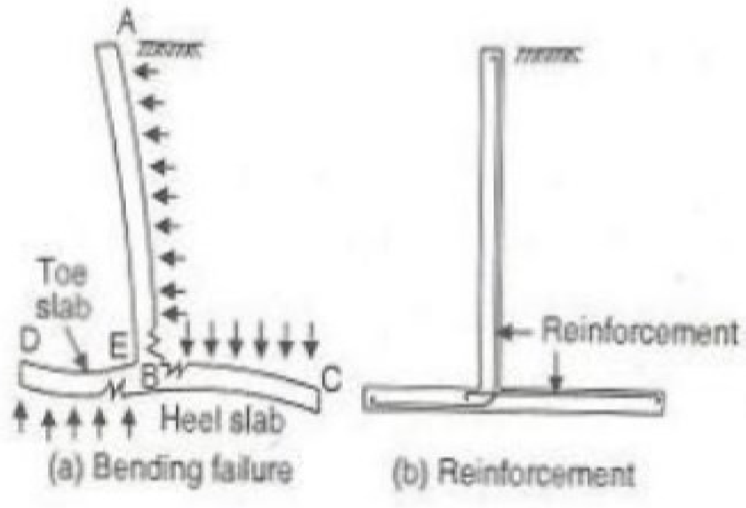


Check for shear: at "d" from junction of the toe slab (at XX as wall is in compression), d = 400 mm

Net shear force at the section XX

Net shear force for toe slab V= -(120.6 + 110.04) /2 x 0.35 + 0.45 x 0.35 x 25 = -36.42kN
$V_{U,max} = 36.42 \times 1.5 = 54.63$ kN
$\zeta v = (54.63 \times 1000) / (1000 \times 400) = 0.13\ N/mm^2$
$pt = (100 \times 302.48) / (1000 \times 400) = 0.075$
From IS:456-2000, Page 73, $pt \leq 0.15\%$, $\zeta c = 0.28\ N/mm^2$
$\zeta v < \zeta c$, Hence safe in shear.

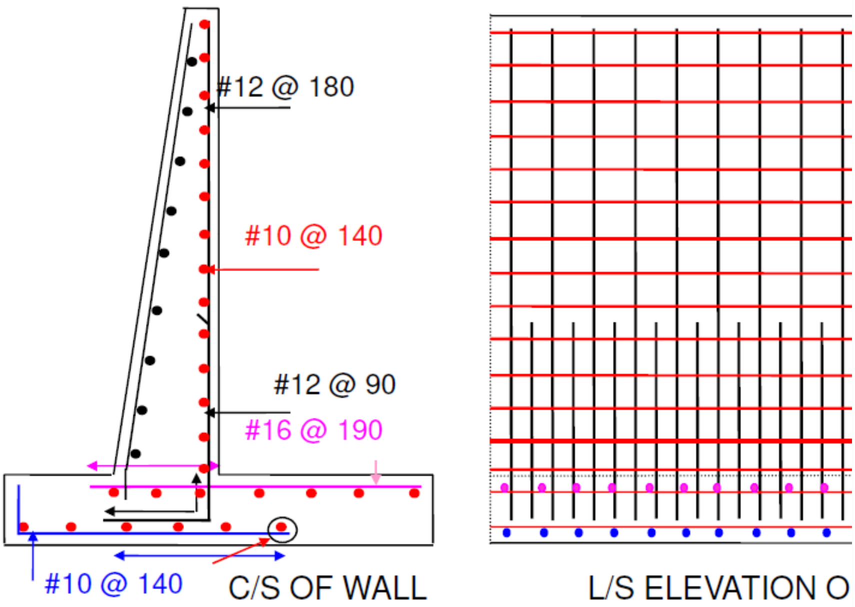




Construction joint
A key 200 mm wide x 50 mm deep with nominal steel #10 @ 250, 600 mm length in two rows.

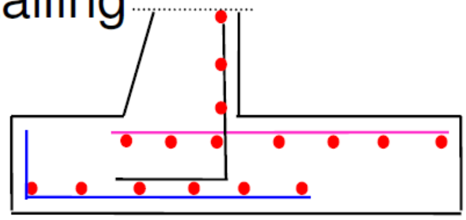
Drainage
100 mm dia. pipes as weep holes at 3m c/c at bottom. Also provide 200 mm gravel blanket at the back of the stem for back drain.

Drawing and detailing



Drawing and detailing

BASE SLAB DETAILS



PLAN OF BASE SLAB