

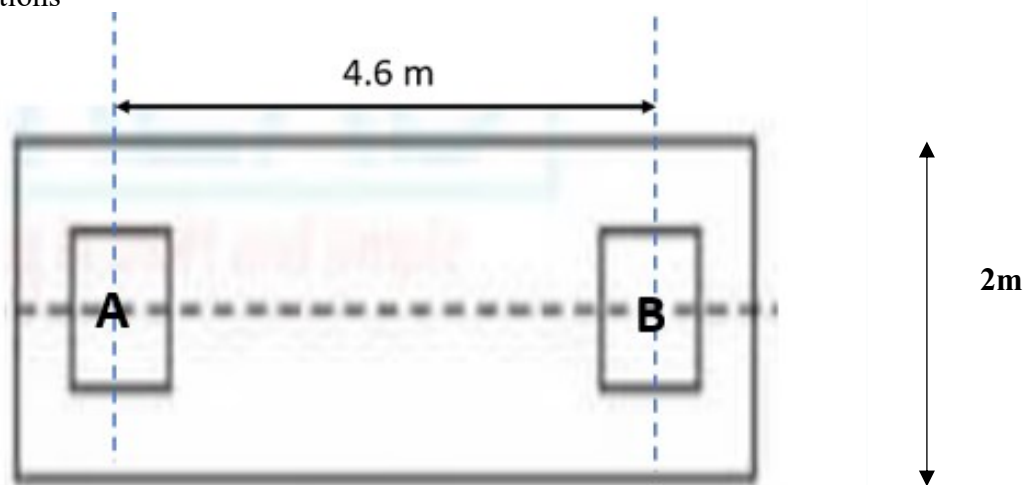
Sub :	Design of RC and steel structural elements				Sub Code:	18CV72/17CV 72	Branch:
Date:	15.11.21	Duration:	90 min's	Max Marks:	50	Sem / Sec:	ALL

Answer any one Questions- Use of IS 456 -2000 is permitted

MARKS
[50]

- 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/m². Use M20 concrete and Fe 415 steel. The design must include all necessary checks and draw the reinforcement details.

Solutions



- **Footing base dimensions**

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

$$\Delta P = (700 + 1200) \times 15/100 = 285 \text{ kN}$$

$$P_1 + P_2 = 700 + 1200 = 1900 \text{ kN}$$

Allowable soil pressure or safe bearing capacity, $q_a = 130 \text{ kN/m}^2$

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

Width of footing, $B = 2\text{m}$ (Given in question)

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

- **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:

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

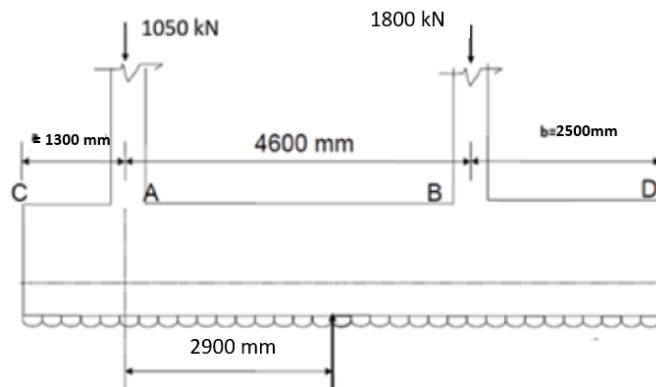
Let \bar{x} be the centroid of the column loads, where $s = 4.6 \text{ m}$

$$\Rightarrow \bar{x} = \frac{P_{u2} s}{P_{u1} + P_{u2}} = \frac{1800 \times 4.6}{1050 + 1800} = 2.9 \text{ m}$$

If the cantilever projection of footing beyond column A is 'a' then,
 $a + 2.9 = L/2 = a = 8.4/2 - 2.9 = 1.3 \text{ m}$

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

$$b = 8.4 - 1.3 - 4.6 = 2.5 \text{ m}$$

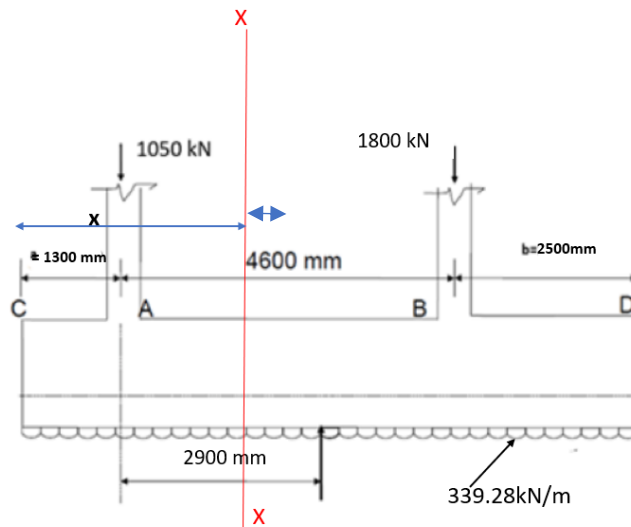


- **Uniformly distributed load acting in upward direction (soil pressure)**

Treating the footing as a wide beam ($B = 2000 \text{ mm}$) 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**



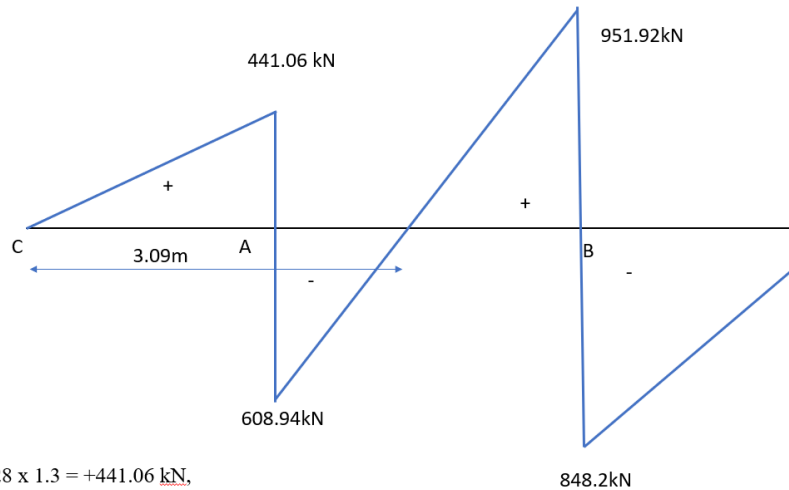
- Shear force at A, just before 1050 kN, left of section XX, $V_{AC} = +339.28 \times 1.3 = +441.06$ kN
- Shear force at A, just after 1050 kN, left of section XX, $V_{AB} = -1050 + 339.28 \times 1.3 = -608.94$ kN
- Shear force at B just after 1800kN, right of section XX, $V_{BA} = +1800 - 339.28 \times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

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

$$339.28 \times X - 1050 = 0, \text{ location of zero shear,}$$

$$339.28 \times X = 1050, 1050/339.28 = X, X = 3.09\text{m}$$

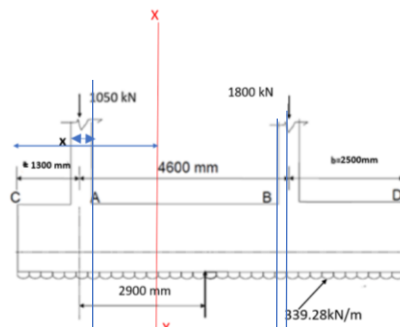
X = 3.09 m from C



- $V_{AC} = 339.28 \times 1.3 = +441.06 \text{ kN}$,
- $V_{AB} = -1050 + 339.28 \times 1.3 = -608.94 \text{ kN}$
- $V_{BA} = 1800 - 339.28 \times 2.5 = +951.92 \text{ kN}$
- $V_{BD} = 339.28 \times 2.5 = -848.2 \text{ kN}$

Shear force diagram

- **Bending moment calculations**



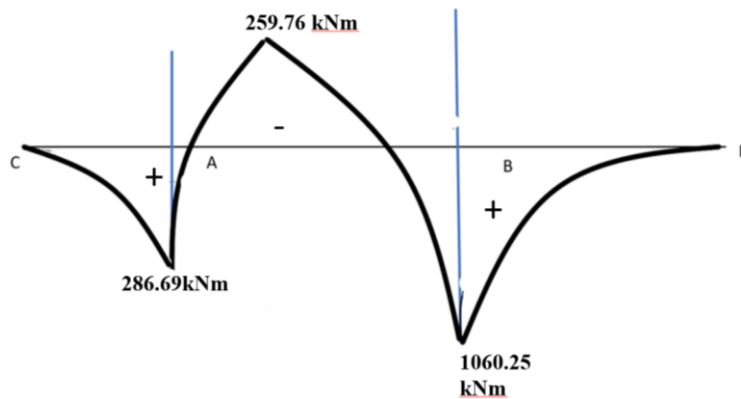
- BM at A, just before 1050kN, left of section XX, $M_{AC} = 339.28 \times 1.3 \times 1.3 / 2 = +286.69 \text{ kNm}$
- BM at just at the inner face of Column A (1050kN), left of section XX,

$$M_{AB} = -1050 \times 0.35/2 + 339.28 \times (1.3 + 0.35/2) \times (1.3 + 0.35/2)/2$$

$$= -1050 \times 0.35/2 + 339.28 \times (1.3 + 0.172) \times (1.3 + 0.172)/2 = +185.32 \text{ kNm}$$
- Negative Bending moment at $X = 3.09 \text{ m}$ (Location of zero shear)

$$M_u = -339.28 \times (3.09)^2/2 - 1050 \times (3.09 - 1.3) = -259.76 \text{ kNm}$$
- BM at B, just before 1800 kN, right of section XX = $+339.28 \times 2.5^2/2 = +1060.25 \text{ kNm}$
- BM at B, just after the inner face of Column B (1800 kN), right of section XX =

- $339.28 \times (2.5 + 0.4/2)^2/2 - 1800 \times 0.4/2 = + 876.67 \text{ kNm}$

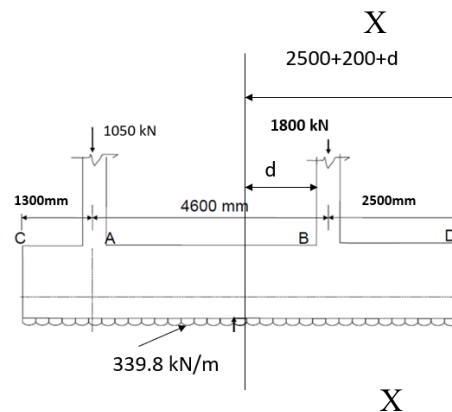


Bending moment diagram

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

One-way shear (longitudinal): V_{u1} 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, V_{u1} at section XX (just right of XX section) =

Column load (B) - Uniformly distributed upward load intensity \times (2500 + 200 + d)

$$= (1800 - 339.28 \times (2.5 + 0.200 + d)) = (882.54 - 339.28 \times d) \text{ kN} \dots$$

(1)

Take $\tau_c = 0.48 \text{ N/mm}^2$ (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

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

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

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

\Rightarrow 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**

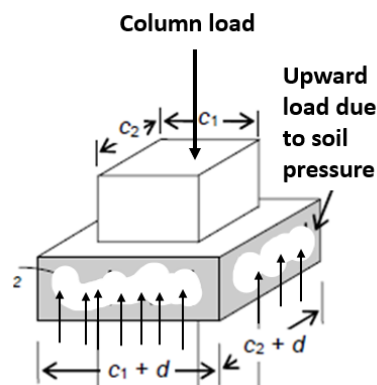
$$D = d + 75 + 20/2 = 680 + 75 + 20/2 = 765 \text{ mm}$$

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

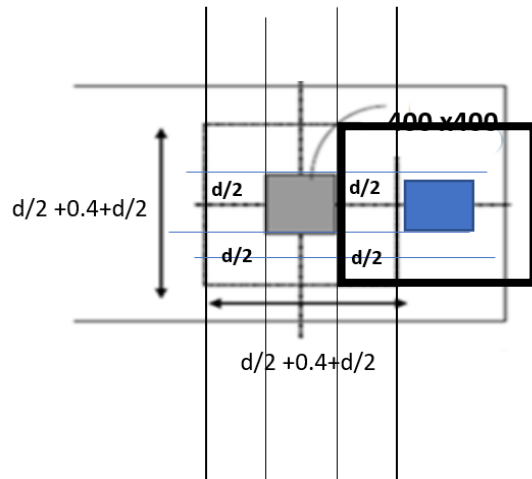
*Two-way shear or punching shear (we need to consider the upward soil pressure not upward soil intensity) * Since it is acting on an area.*

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

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



Shear stresses in footing slab due to punching shear

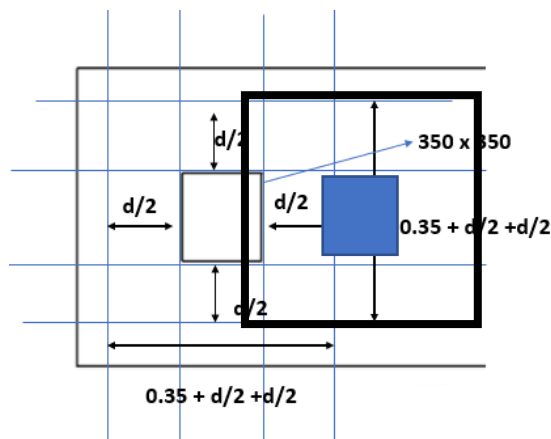


Punching shear or Two-way shear calculations for heavier Column B

$$V_{u2} = 1800 - 169.64 (0.4 + 0.680/2 + 0.680/2) \times (0.4 + 0.680/2 + 0.680/2)$$

$$= 1602.13 \text{ kN @ B (Heavier column)}$$

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



Punching shear or Two-way shear @ A,

$$\text{Two-way shear } V_{u2} = (\text{Column load at A}) 1050 - 169.64 \times (0.35 + 0.680/2 + 0.680/2) \times (0.35 + 0.680/2 + 0.680/2)$$

$$= 870 \text{ kN @ A (Lighter column)}$$

- 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 column capital; and

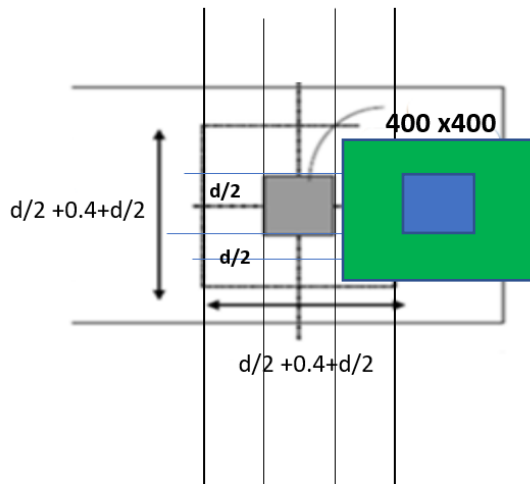
$\tau_c = 0.25 \sqrt{f_{ck}}$ in limit state method of design and $0.16 \sqrt{f_{ck}}$ in working stress method design.

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

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

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

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



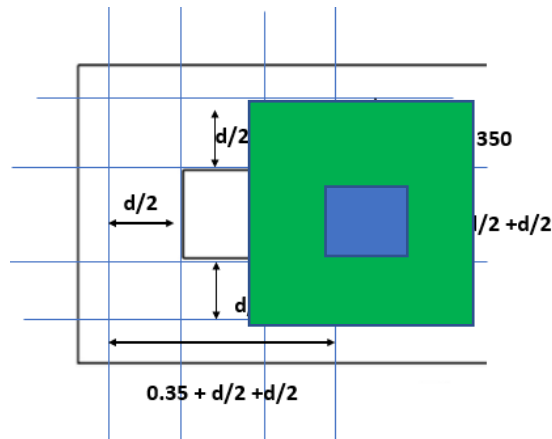
Perimeter of critical section (Green coloured area) = $4 \times (400 + 680/2 + 680/2)$

$V_{uc} = 1.118 \times [4 \times (400 + 680/2 + 680/2)] \times 680 = 3284.24 \text{ kN @ B}$

In the similar way let's calculate for Column A

Permissible two-way shear force for Column A

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



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

$$V_{uc} = 3284.23 \text{ kN} > V_{u2} = 1602.82 \text{ kN} \quad @ \text{ B It is safe.}$$

$$V_{uc} = 3132.18 \text{ kN} > V_{u2} = 870.00 \text{ kN} \quad @ \text{ A . It is Safe.}$$

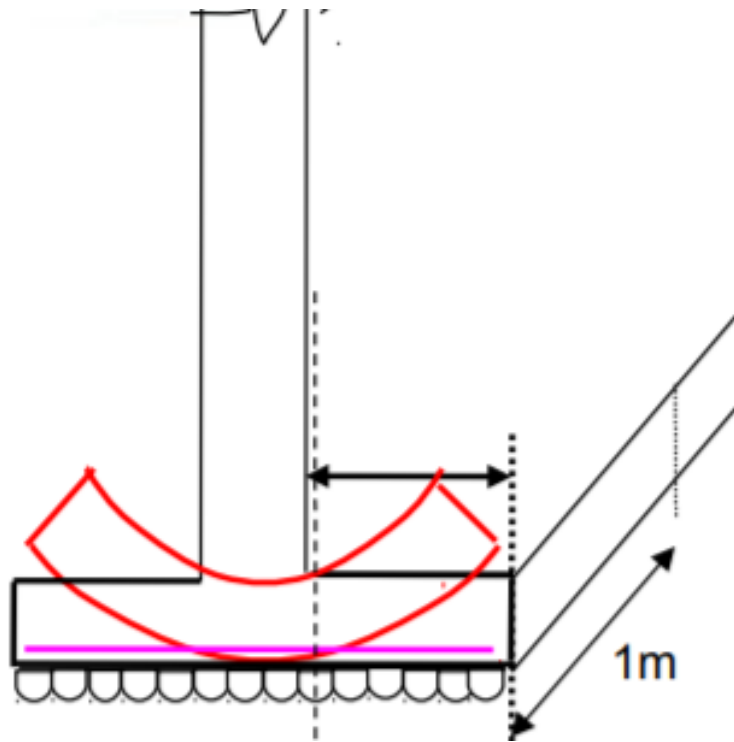
Hence safe against two way or punching shear, (if not provide shear reinforcement- stirrups or bent up bars)

- **Design of longitudinal flexural reinforcement**

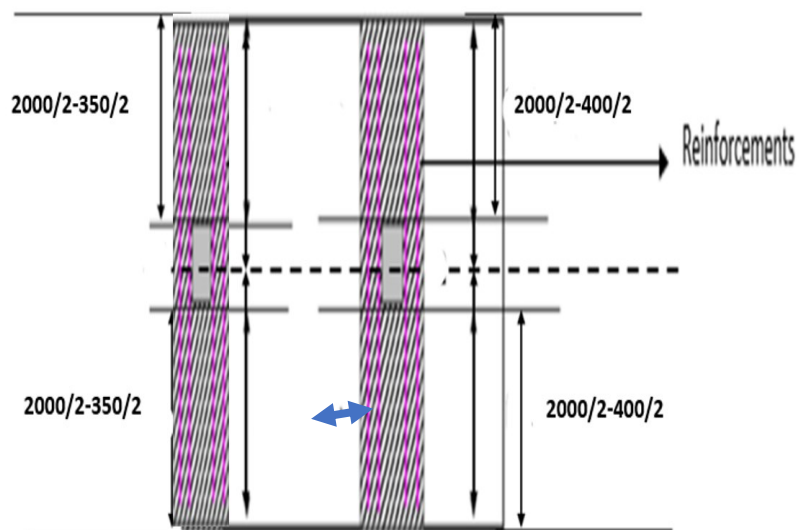
<u>Maximum 'negative' moment: $M_u =$</u> - 259.76 kNm	<u>Maximum 'positiv</u>
$M_u = 0.87 f_y A_{st} d \left(1 - \frac{A_{st} f_y}{bd f_{ck}} \right)$ <p> $M_u = 259.76 \times 10^6 \text{ N mm}$ $B = b = 2000 \text{ mm}, f_{ck} = 20 \text{ N/mm}^2,$ $f_y = 415 \text{ N/mm}^2$ $d = 680 \text{ mm}$ $D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

<p>But we have assumed $p_t = 0.5$</p> <p>$p_t = 100 A_{st, req} / (B \times d)$</p> <p>Choose 20 mm diameter bars, calculate no of bars</p> <p>$= 1836 / (\pi/4 \times 20^2) = 6$</p> <p>Provide 6 # 20 mm diameter bars at top</p> <ul style="list-style-type: none"> • Development length $L_d = 47 \times \text{dia of bar}$ $= 47 \times 20 = 940 \text{ mm}$ 	<p>Provide 15 # 20mm</p> <ul style="list-style-type: none"> • Development length $= 47 \times 20 = 940 \text{ mm}$
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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 = $1050/2.0 = 525 \text{ kN/m}$
- Cantilever Projection of beam beyond column face = $(2000 - 350)/2 = 825 \text{ mm} = 0.825 \text{ m}$
- Maximum transverse moment at column face A :
 $M_u = 525 \times 0.825^2/2 = 178.66 \text{ kNm}$

-
- Assume width of transverse beam, b =
 $\text{width of column} + 2 \times 0.75d$

$$b = 350 + 2 \times 0.75 \times 577 = 1215.5 \text{ mm}$$

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

$$b = 1215.5 \text{ mm}, d = 577 \text{ mm}$$

$$M_u = 178.6 \times 10^6 \text{ N mm}$$

$$A_{st} = 880.23 \text{ mm}^2$$

- Factored Column load A per width of footing = 900 kN/m
- Cantilever Projection of beam beyond column face = $800 \text{ mm} = 0.8 \text{ m}$
- Moment at column face A :
 $900 \times 0.80^2/2$

- Width of transverse beam, b =
 $\text{width of column} + 2 \times 0.75d$

- $400 + 2 \times 0.75 \times 577 = 1265.5 \text{ mm}$

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

$$M_u = 384 \times 10^6 \text{ N mm}$$

$$b = 1265.5 \text{ mm}$$

$$d = 577 \text{ mm}$$

$$A_{st} = 1956 \text{ mm}^2$$

<p>Page 48, CL No 5.2.1</p> <ul style="list-style-type: none"> Minimum $A_{st} = 0.0012 bD =$ $A_{st \text{ min}} = .0012 \times 1215.5 \times 765 = 1115.83 \text{ mm}^2$ Use 12mm dia bars (Your wish!!) Number of 12 mm ϕ bars required = $A_{st} / \text{area of one bar} = 1115.83 / (\pi/4 \times 12^2) = 10$ <p>Provide 10 nos 12 mmϕ bars</p> <p>Check for development length = $47 \times 12 = 564 \text{ mm}$</p>	<ul style="list-style-type: none"> Provide (A_{st}) 1265.5×765 <p>Use 12 mm dia bars Number of 12 mm ϕ $= 1956.2 / (\pi/4 \times 12^2)$</p> <p>Provide 18 nos 12 mmϕ bars</p> <ul style="list-style-type: none"> Required dev $47.0 \times 12 = 564 \text{ mm}$ beyond the cc
<p>Transfer of force at column base -Column A</p>	<p>Transfer of force at column base -Column B</p>
<ul style="list-style-type: none"> Limiting bearing stress at column base <p>IS 456 Page 65 , CL34.4</p> <p>34.4 Transfer of Load at the Base of Column</p> <p>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;</p> <p>where</p> <p>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</p> <p>A_2 = loaded area at the column base.</p> <p>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 $0.45 f_{ck}$.</p> <p>Permissible bearing stress = $0.45 f_{ck} \sqrt{\frac{A_1}{A_2}}$</p>	<ul style="list-style-type: none"> Limiting bearing stress <p>Permissible bearing stress</p> <p>$[A_1 = 2000^2, A_2 = 400^2]$</p> <p>$\sqrt{\frac{A_1}{A_2}} = 5 > 2, \sqrt{\frac{A_1}{A_2}} = 2$</p> <p>$= 0.45 \times 20 \times 2 = 18 \text{ MPa}$</p> <p>Permissible bearing stress $18 \times 400^2 = 2880 \text{ kN}$</p> <p>$2880 \text{ kN} > 1800 \text{ kN}$, Hence, the bearing capacity is not sufficient.</p>

$A_1 = 2000^2$ (2000mm is footing width)

$A_2 = 350 \times 350 =$

$$= \sqrt{\frac{A_1}{A_2}} < 2$$

$$= 5.71 > 2, \text{ Take } \sqrt{\frac{A_1}{A_2}} = 2$$

Permissible bearing stress = $0.45 \times 20 \times 2$

$$= 18 \text{ N/mm}^2$$

Permissible bearing resistance or force

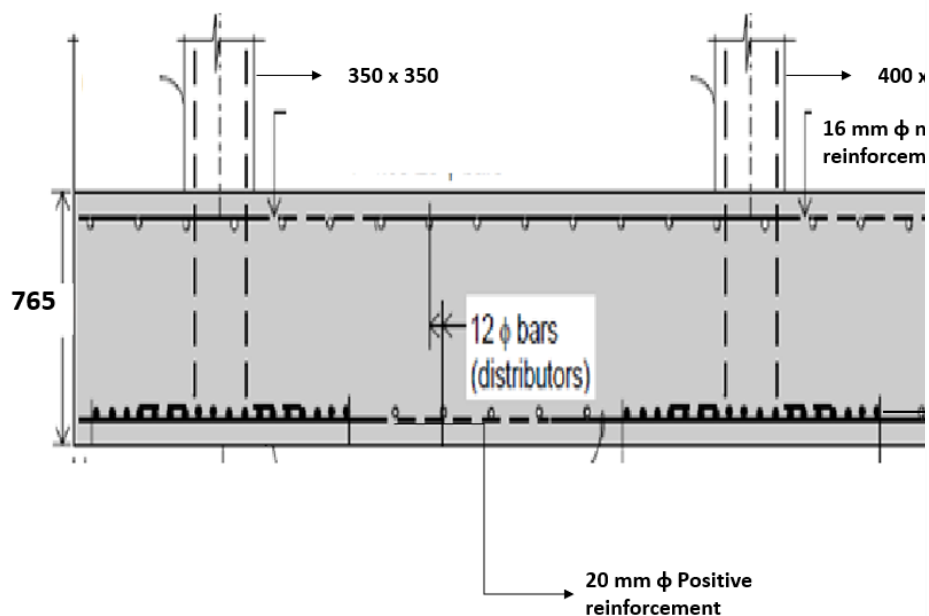
$F_{br} = \text{Permissible bearing stress} \times \text{column area}$

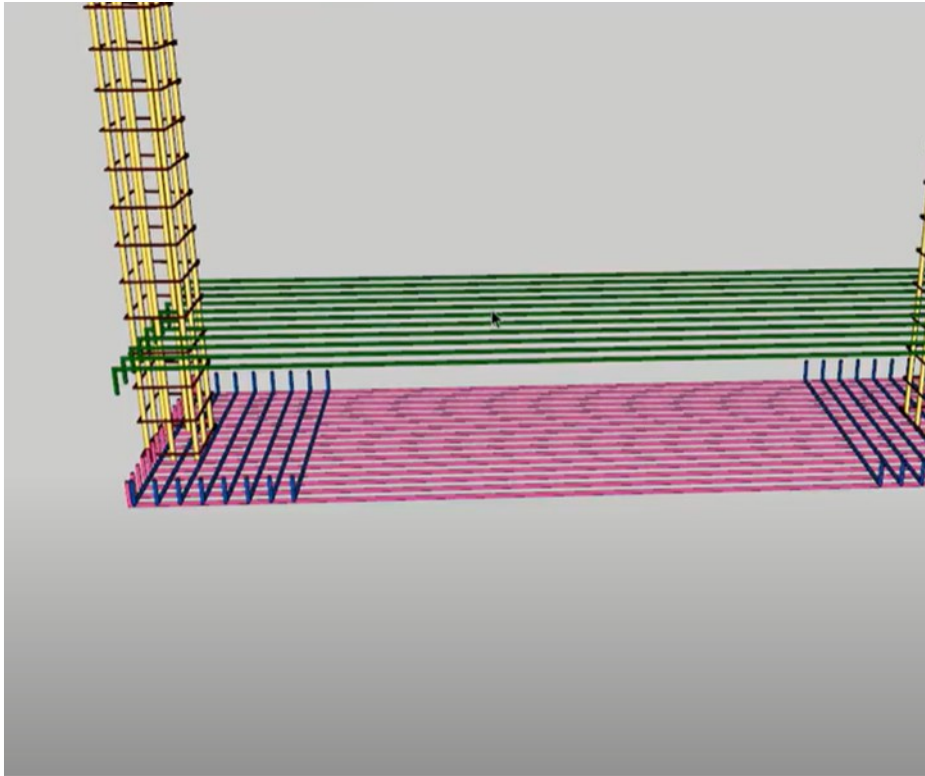
$$= 18 \times 350^2 = 2205 \times 10^3 \text{ N} = 2205 \text{ kN}$$

$2205 > 1050 \text{ kN}$, Hence safe.

Reinforcement detailing

20





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° . 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.

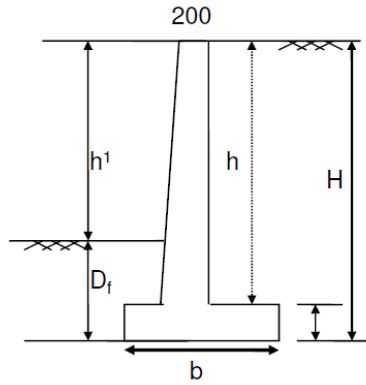
[50]

Height of earth fill, $h' = 4\text{m}$, Safe bearing Capacity = 200 kN/m^2 ,
Density of soil, $\gamma = 18 \text{ kN/m}^3$, co-efficient of friction between concrete
and soil, $\mu = 0.6$, angle of repose $\phi = 30^\circ$

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



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

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

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

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

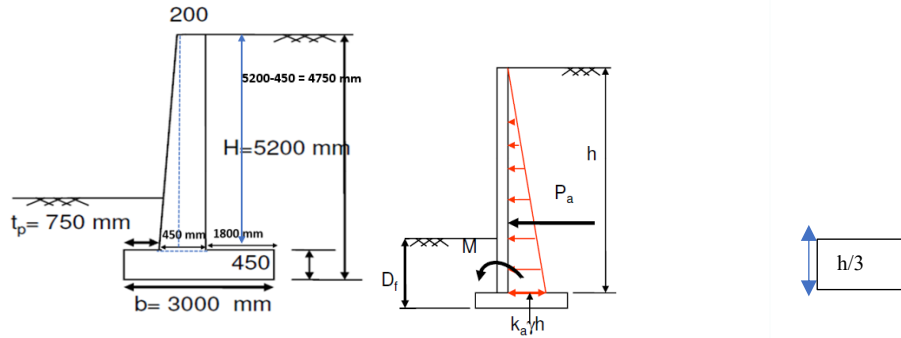
- **Proportioning of stem wall and base slab**

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

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

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

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

$$M_u = 1.5 \times M = 160.6 \text{ kN-m}$$

We have overall depth at base or thickness of stem slab as, $D = 450 \text{ mm}$

Check for effective depth

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

Put $M_{u, \text{lim}} = 160.6 \times 10^6$, $b = 1000 \text{ mm}$, $f_{ck} = 20 \text{ N/mm}^2$

$x_{u, \text{max}} / d = 0.48$, Fe 415, IS 456 2000

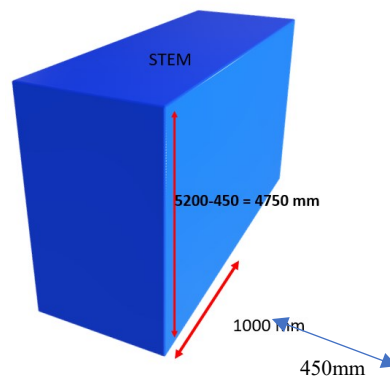
$$160.6 \times 10^6 = 0.36 \times 0.48 \times (1 - 0.42 \times 0.48) \times 1000 \times d^2 \times 20$$

Find 'd', $d = 242 \text{ mm}$

Required, $d = 242 \text{ mm}$

Assumed overall depth at base of the stem, $D = 450 \text{ mm}$, effective, $d = 450 - 50 = 400 \text{ mm} \gg 242 \text{ mm}$

Taking 1m length of stem wall,



$b = 1000 \text{ mm}$, $d = 450 - \text{effective cover}$

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

$$= 40 + 12/2 = 46 \approx 50 \text{ mm}$$

$d = 450 - 50 = 400 \text{ mm} \gg 242 \text{ mm}$, hence safe

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

$d = 400 \text{ mm}$, $b = 1000 \text{ mm}$, $M_u = 160.6 \times 10^6 \text{ Nmm}$, $f_y = 415 \text{ N/mm}^2$,
 $f_{ck} = 20 \text{ N/mm}^2$

$A_{st} = 1184.6 \text{ mm}^2$

$A_{st, \text{min}} = 0.0012 \times b \times D = 0.0012 \times 1000 \times 450 = 540 \text{ mm}^2$

$A_{st} > A_{st, \text{min}}$, hence Ok.

Provide 12 mm ϕ bars as main steel

Spacing required, $s = \frac{1000 \times \frac{\pi}{4} \times 12^2}{1184.6} = 96 \text{ mm} \approx 100 \text{ mm}$ 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 = $0.12 \times 450 \times 1000/100 = 540 \text{ mm}^2$

Use 10 mm ϕ bars, spacing required

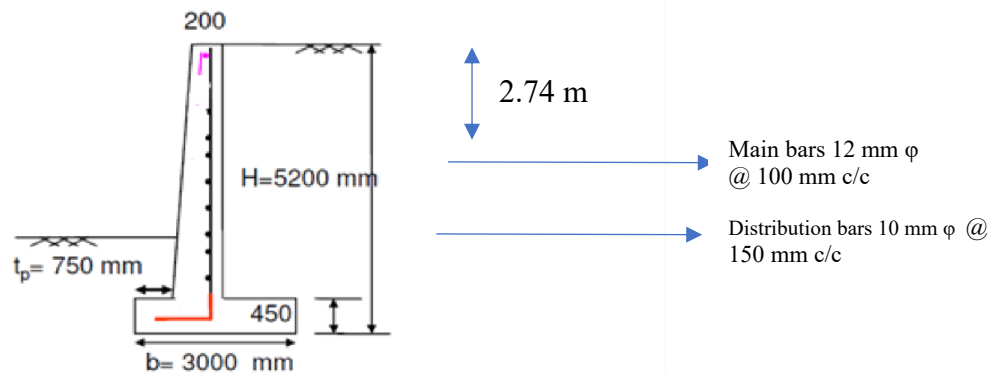
Spacing required, $s = \frac{1000 \times \frac{\pi}{4} \times 10^2}{540} = 145.4 \text{ mm} \approx 140 \text{ mm}$ or 150 mm

(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

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



Curtailement of bars

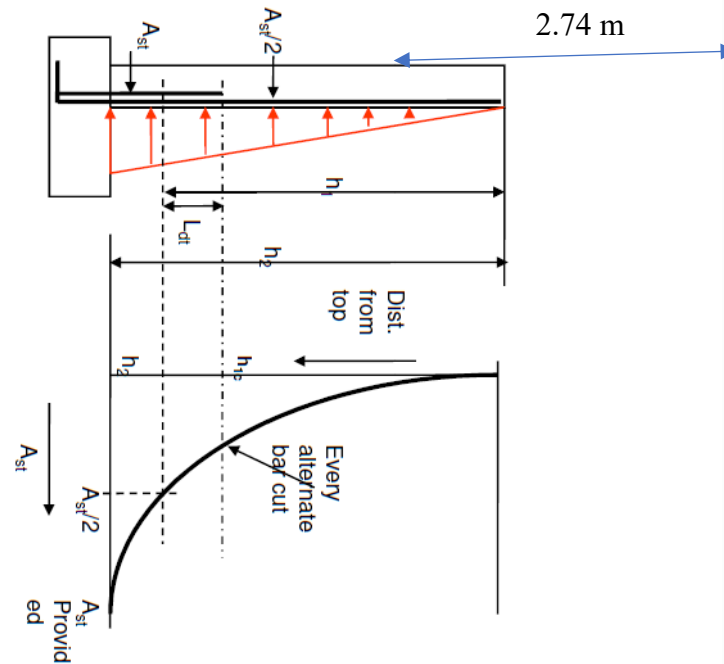
Curtail 50% steel from top, $A_{st} = \frac{50}{100} \times 1184.6 = 592.3 \text{ mm}^2$

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

Actual point of cut off or cutting position = $3.36 - L_d = 3.36 - 0.564 = 2.74 \text{ m}$ from top.

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

Spacing of bars $12 \text{ mm } \phi @ 190 \text{ mm c/c} < 300 \text{ mm}$ and $3 \times$ effective depth . Hence it is ok

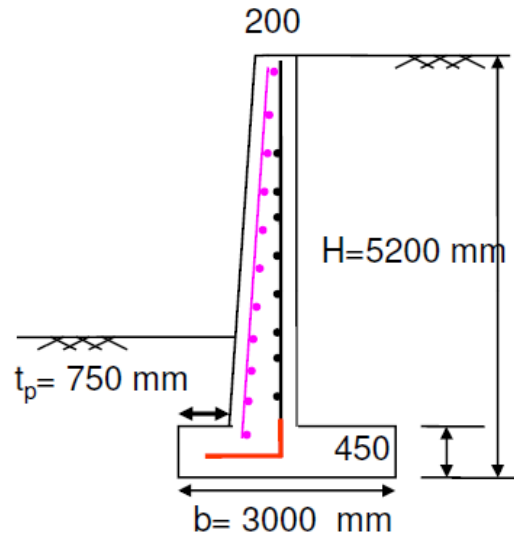


Secondary steel for stem at front (Temperature steel)

0.12% Gross Area = $0.12 \times 450 \times 1000/100 = 540 \text{ mm}^2$

This can be copied from distribution steel calculations

#10 @ 150 mm c/c < 450 mm and 5d ok

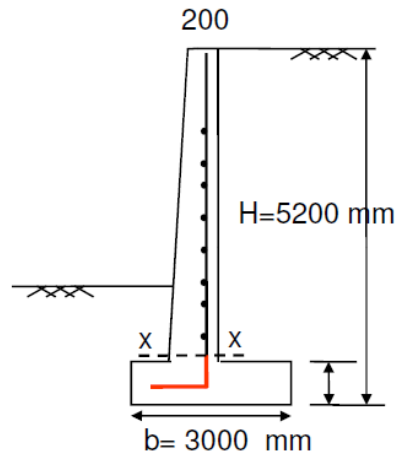


Check for shear

Max. Shear Force at Junction XX = $P_a = 67.68 \text{ kN}$ (Lateral earth pressure)

Ultimate Shear Force = $V_u = 1.5 \times 67.68 = 101.52 \text{ kN}$

Nominal shear stress = $\tau_v = V_u/bd = 101.52 \times 10^3 / 1000 \times 400 = 0.25 \text{ N/mm}^2$



To find τ_c , 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

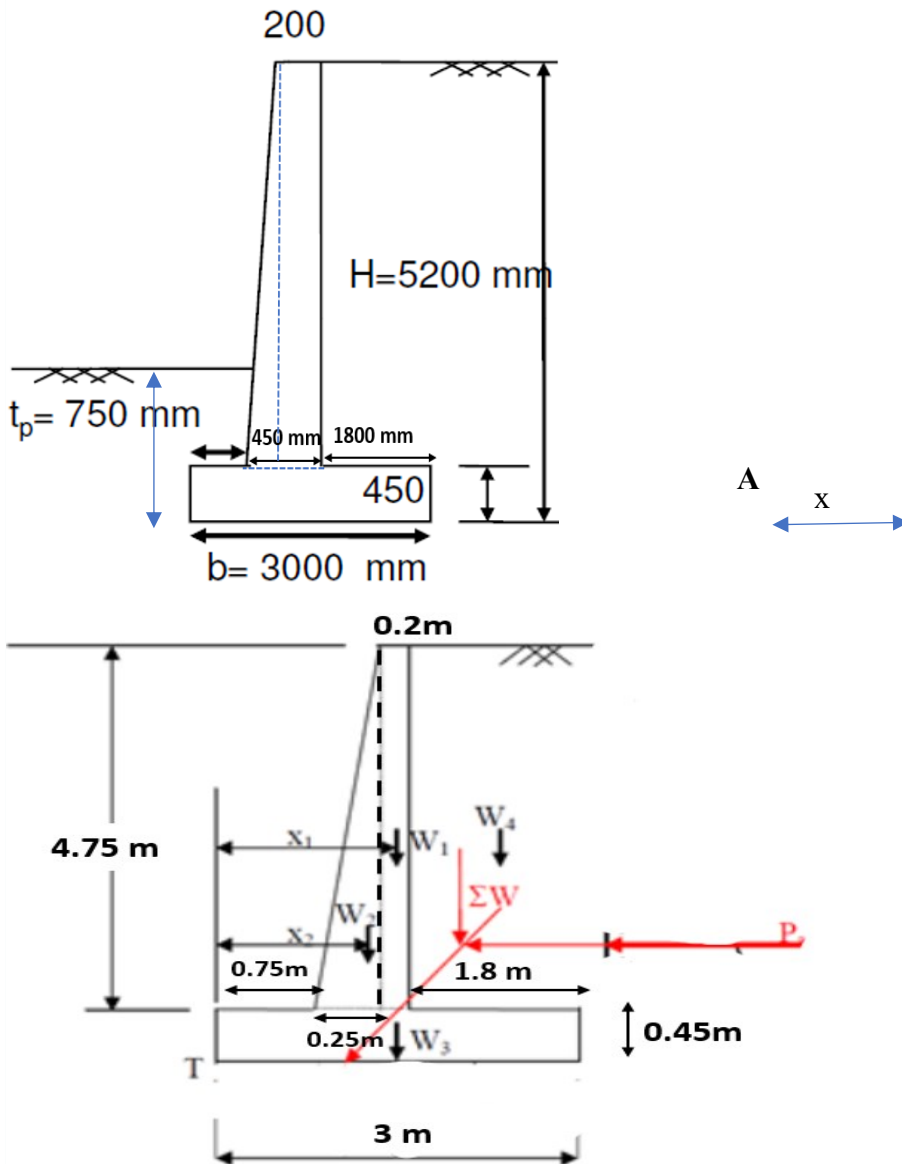
$\tau_c = 0.38 \text{ N/mm}^2$

Compare τ_v and τ_c , $0.25 < 0.38$

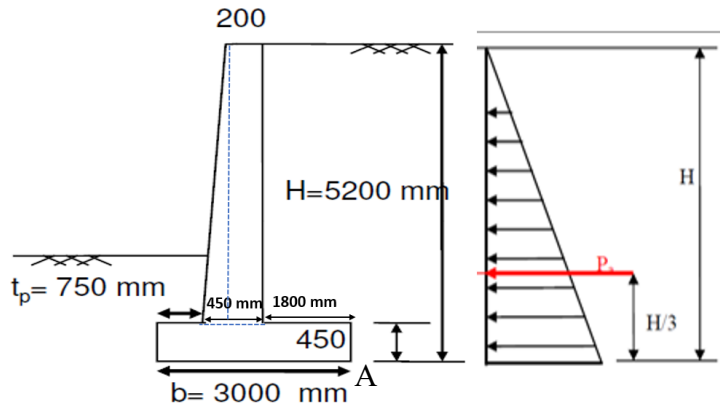
$\tau_v < \tau_c$ Hence safe in shear. No need of shear reinforcement.

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	$0.2 \times 4.75 \times 1 \times 25 = 23.75$	$(0.75 + 0.25 + 0.2/2) = 1.1$
Stem W2	$\frac{1}{2} \times 0.25 \times 4.75 \times 1 \times 25 = 14.84$	$0.75 + \frac{2}{3} \times 0.25 = 0.916$
Base slab W3	$3.0 \times 0.45 \times 1 \times 25 = 33.75$	$3/2$
Back fill, W4	$1.8 \times 4.75 \times 1 \times 18 = 153.9$	$0.75 + 0.45 + 1.8/2 = 2.1$
Total	$\Sigma W = 226.24$ 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	$P_H = \frac{1}{2} \times \frac{1}{3} \times 18 \times 5.2^2 = 81.12$ 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 $\Sigma M_R / M_O > 1.55$

$\Sigma M_R = 413.55$ 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:**

$\Sigma W = 226.24$ kN

$P_H = 81.12$ kN

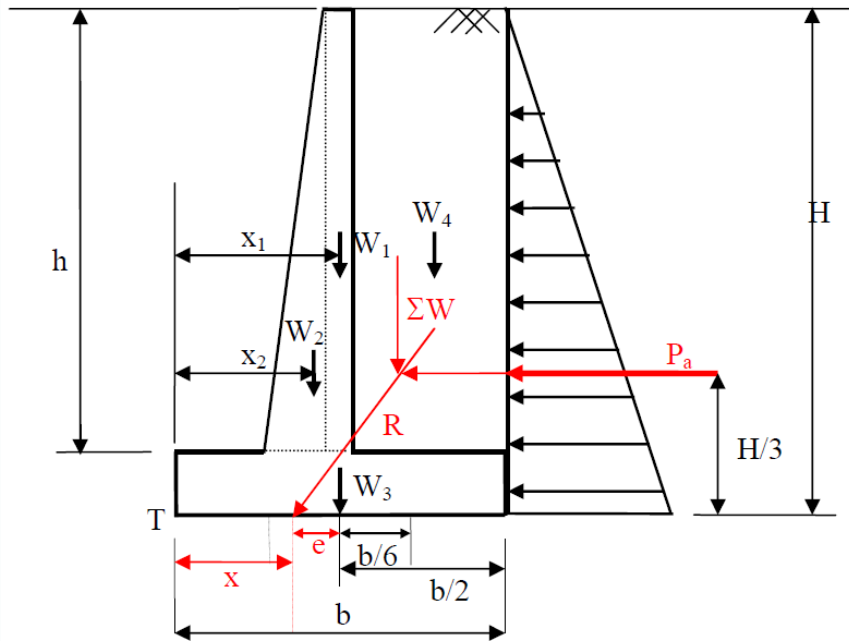
As per IS: 456:2000, (F.S) sliding should satisfy condition that $\mu \Sigma W / P_H \geq 1.55$

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

(F.S) sliding = $1.67 \geq 1.55$ 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,
 $x = \Sigma M / \Sigma W$, where $\Sigma M =$ Net moments about toe = $\Sigma M_R - M_O = 413.55 - 140.05 = 273.5$ kNm



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

- **Eccentricity $e = b/2 - x = 3/2 - 1.2 = 1.5 - 1.2 = 0.3\text{m} < b/6$** ($0.3 < 0.5$)(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

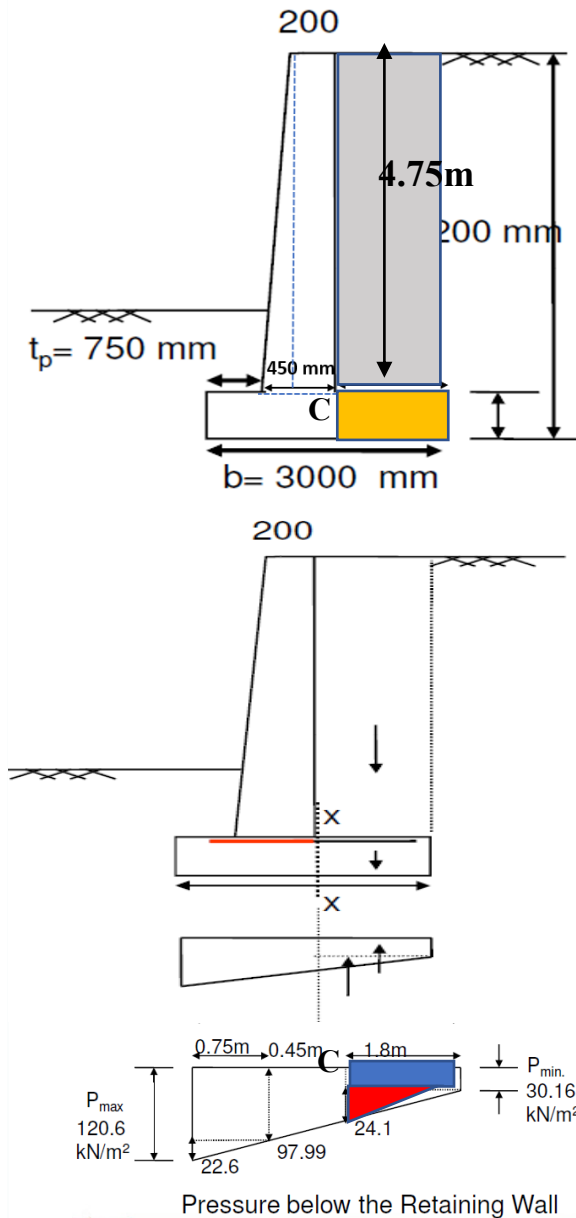
$$\text{Max. pressure} = P_{\text{max}} = \frac{\sum W}{b} \left[1 + \frac{6e}{b} \right]$$

120.66 kN/m² < SBC, safe

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

30.16 kN/m² > 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/m²) . Hence it is safe.






Using similar triangles,

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

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	$1.8 \times 4.75 \times 1 \times 18 = 153.9$	$1.8/2 = 0.9$	=
Heel slab 	$0.45 \times 1.8 \times 25 = 20.25$	$1.8/2 = 0.9$	=
Pressure distribution, (below heel slab) rectangle 	$- 30.16 \times 1.8 = -$	$1.8/2 = 0.9$	-

Pressure distribution, Triangle 	$\frac{1}{2} \times -24.1 \times 1.8 = -21.69$	$\frac{1}{3} \times 1.8 = 0.6$	-1
Total Load at junction C	98.17	Total BM at Junction C	Σ

$$\Sigma M_C = 94.87$$

$$M_u = 1.5 \times 94.87 = 142.3 \text{ kNm} = 142.3 \times 10^6 \text{ N mm}$$

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

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

$$A_{st} = 1041.5 \text{ mm}^2$$

Use 16 mm ϕ bars (it is base slab) (You can choose 12 mm also)

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

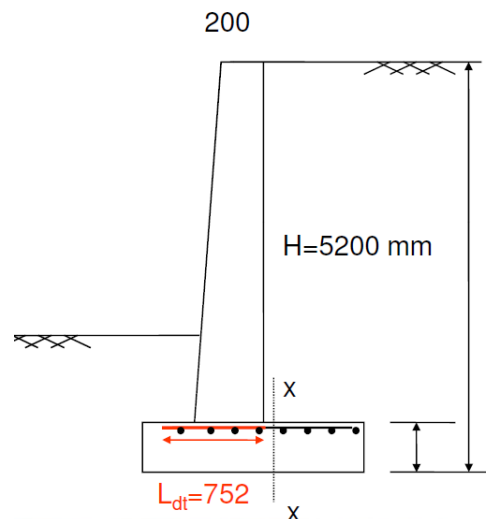
Main steel #16 mm Φ @ 190mm c/c < 300 mm and 3d ok. Hence it is safe.

Development length

$$L_d = 47 \phi_{bar} = 47 \times 16 = 752 \text{ mm}$$

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.

$$\text{Maximum shear} = V = 98.17 \text{ kN}, V_u, \max = 98.17 \times 1.5 = 147.255 \text{ kN}$$

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

$$\tau_v = 0.368 \text{ N/mm}^2$$

$$p_t = \frac{100 \times 1041.5}{1000 \times 400} = 0.260 \% , \text{ Find } \tau_c$$

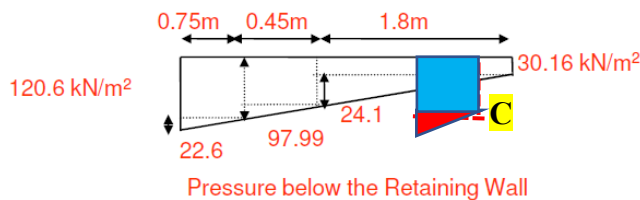
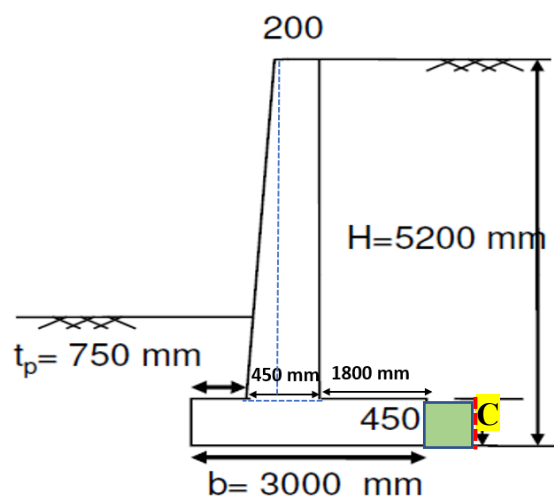
Use IS:456-2000, Page 73, Table 19, $p_t = 0.26 \%$

$$\tau_c = 0.365 \text{ N/mm}^2$$

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



Load	Magnitude, kN	Distance from C,
Self wt of Toe slab	$0.75 \times 0.45 \times 25 = 8.44$	$0.75/2$
Pressure distribution, rectangle	$-97.99 \times 0.75 = -73.49$	$0.75/2$
Pressure distribution, Triangle	$\frac{1}{2} \times -22.6 \times 0.75 = -8.474$	$\frac{2}{3} \times 0.75 =$
		Total BM at Junction C

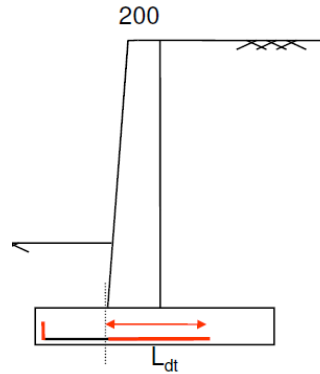
$$M_u = 1.5 \times 28.63 = 43 \text{ kN-m,}$$

$$A_{st} = 302.48 \text{ mm}^2$$

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:

$$L_d = 47 \phi_{bar} = 47 \times 10 = 470 \text{ mm}$$



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

$$\text{Net shear force for toe slab } V = -\frac{(120.6 + 110.04)}{2} \times 0.35 + 0.45 \times 0.35 \times 25 = -36.42 \text{ kN}$$

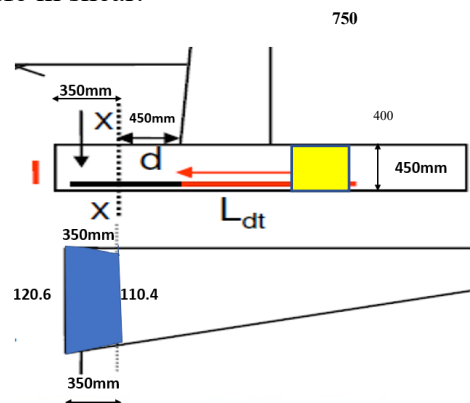
$$V_{U,max} = 36.42 \times 1.5 = 54.63 \text{ kN}$$

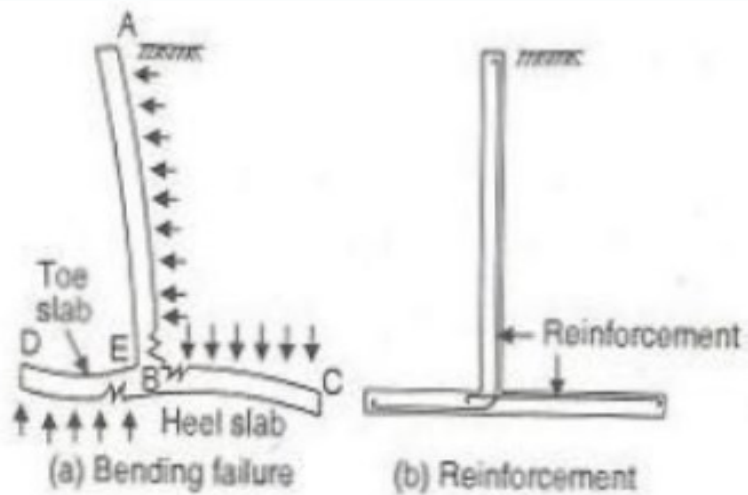
$$\zeta_v = \frac{(54.63 \times 1000)}{(1000 \times 400)} = 0.13 \text{ N/mm}^2$$

$$p_t = \frac{(100 \times 302.48)}{(1000 \times 400)} = 0.075$$

From IS:456-2000, Page 73, $p_t \leq 0.15\%$, $\zeta_c = 0.28 \text{ N/mm}^2$

$\zeta_v < \zeta_c$, Hence safe in shear.





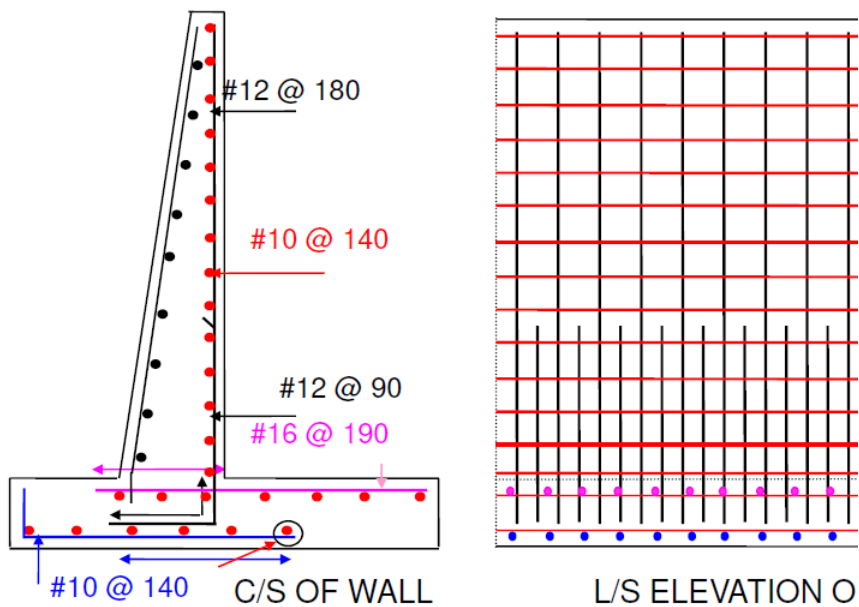
Construction joint

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

Drainage

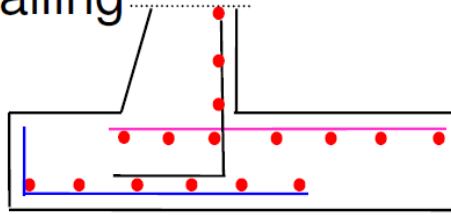
100 mm dia. pipes as weep holes at 3m c/c at bottom. Also provide 200 mm gravel blanket at the back of the stem for back drain.

Drawing and detailing



Drawing and detailing

BASE SLAB DETAILS



PLAN OF BASE SLAB

