CE6PC01	CONCRETE STRUCTURE-II	PC-I	4-1-0	4 Credits

Pre-requisites: Strength of Materials

Detailed Syllabus:

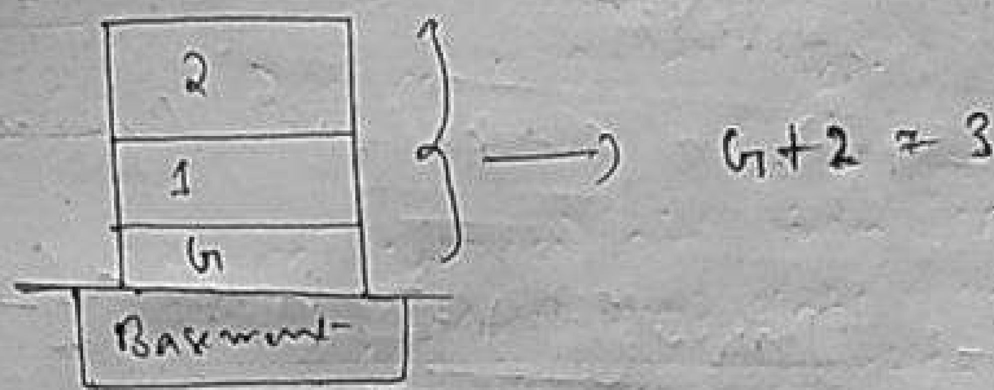
MODULE	CONTENTS	Hrs
1.	Design of Residential Buildings: fundamentals of multi-storey buildings, analysis of various loads: gravity, wind, earthquake loads., method of substitute frames, design examples, bending moments in columns, analysis of multistory frames subjected to horizontal loads.	12
2.	Design of RCC water tanks: Uncracked structures and determination of basic parameters, Revision of working stress design philosophies. Introduction to water tanks and their classifications, Important IS codes and its provisions, Analysis and design of Circular water tanks with flexible base and restrained base. Analysis and design of Rectangular water tanks, Analysis of Overhead tanks, Intze tank- basic geometrical configurations; analysis methods; design of top domes, cylindrical walls, ring beam.	12
3.	Design of Silos and Bunkers: Introduction, difference between bunker and silo, design of square or rectangular bunkers, design of circular bunkers, design examples, silos for storage of cement, design examples.	10
4.	Design of Simple Bridges: Bridges – basic definition, importance, classification., Site investigations for design of a bridge, Various loads and their combinations, Relevant IRC codes and its provisions, Introduction to RC bridge-, design of Culvertand T-beam bridge,.	12

Mod - 1 → Design of Building Structure.

Design of Multistorey Buildings :-

→ A multi-storey building is a building that has multiple storey's & typically containing vertical circulation in the form of ramps, stairs, & lifts.

→



Classification

- → low rise - < 5 storey
- → Mid rise → 5 to 10 storey.
- → High rise → >10 "
- → Sky scraper → >40 storey
- → Super tall → > 300 m
- → Mega tall → > 600 m.

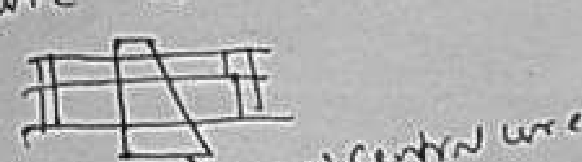
Structural types

i) Framed struc:-

→ Skeleton is made by beam & column.

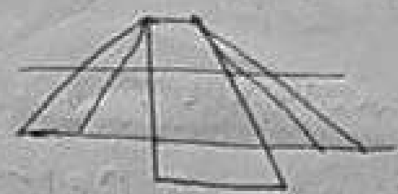
ii) propped struc:-

→ A central core is designed.



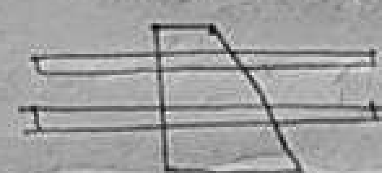
iii) <u>Suspended struc^n:-</u>

→ A central core is connected by cables in place of column.

→ 

iv) <u>Cantilever struc^n:-</u>

→ There is no column in central core.

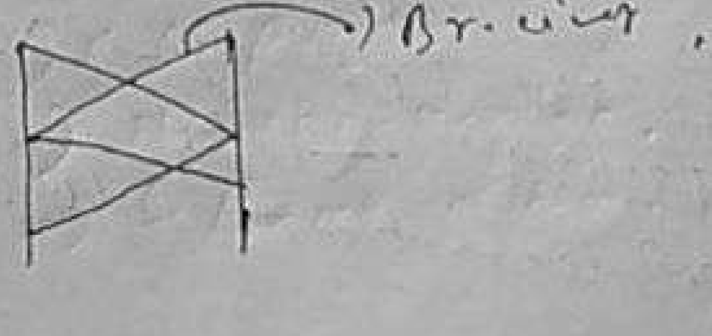
→ 

v) <u>Shear wall struc^n:-</u>

→ Shear wall is provided to counter lateral & wind pressure.

vi) <u>Braced struc^n:-</u>

→ Braced is provided with column to provide lateral stability.

→ 

④ Analysis & Design of Multistorey Building:-

Step-1:- Geometry of the Building.

Step-2:- Load & Load combinations.

Step-3:- Analysis.

Step-4:- Design & Detailing.

N.B:- Design carried out from Top to Bottom. Starts with slabs then Beam & then column & finally footings.

Step-1:- Geometry:-

i) Overall Dimension will be fixed.
ii) Depth of member can be obtained based on span to depth ratio by IS: 456 (clause 23.2).

Step-2 Loads:-

i) IS: 875 (part-1) → DL
ii) IS: 875 (part-2) → LL
iii) IS: 875 (part-3) → WL
iv) IS: 1893 → EL
v) Load combination → Table 18 (IS: 456)

Part-3 Analysis of Struc

- Determinate struc
- Indeterminate struc.
  - Vertical load
    - Force method
    - Slope & Deflection method
    - Moment distribution method
  - Horizontal load
    - portal method
    - canti lever method

Part-4 Design:

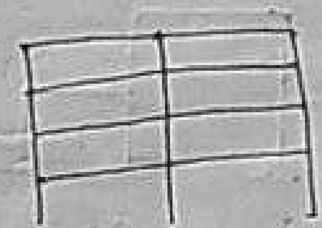
→ By IS: 456, SP-34, SP-16 charts.

Lecture-2

Framed structure:→

→ Network of columns and connecting Beams.

→



Framed structure

- Portal frame
- Building frame.

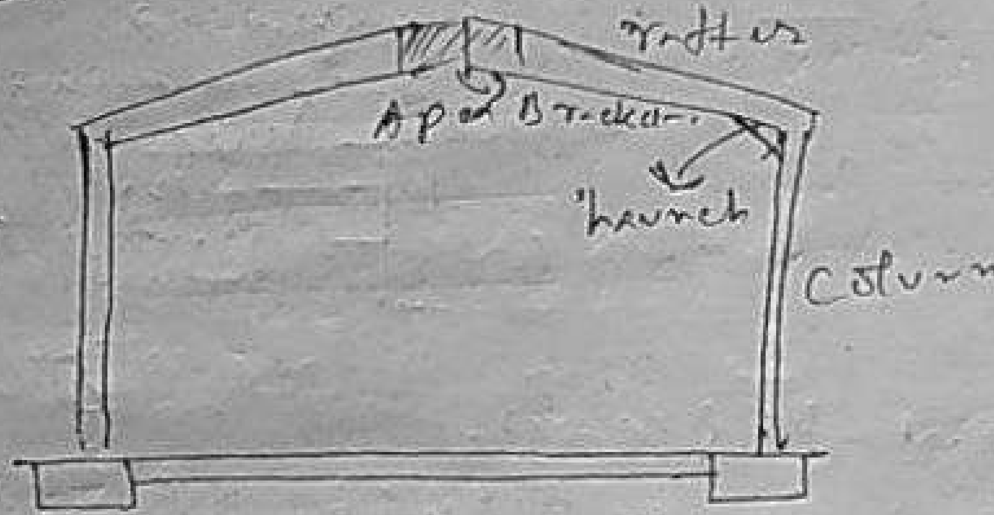
1) portal frame :-

→ Frame with rigid joints.

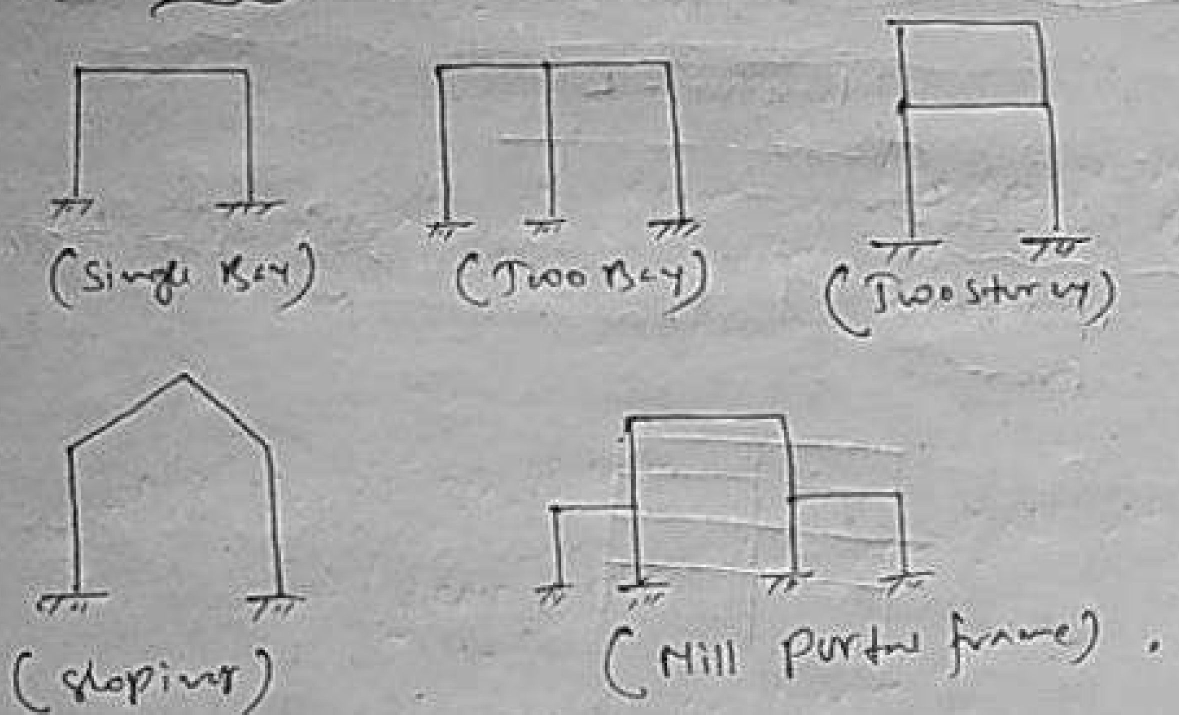
→ used for large span struc.

→ ex: Industrial shed

→



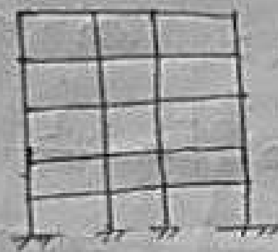
→ types :-



Building frame:-

→ Rigidly inter-connected Beam and columns of Multi Bay and multistoried .

→



Analysis & design of portal frame:-

→ via Moment Distribution method .

→ have to find design moment & shear force at critical section .

→ Step-1 Given data.

Step-2 Shape & Size of component of portal frame .

(i) Beam:- a) $d = l/12$ to $l/15$ (By IS:456).

b) width of Beam (b) = 1/2 or 1/3 of Depth.

c) Beam = b x D

(ii) column:- → Same as Beam dimension .

(iii) Slab:- l/d = 25 to 30 (for one way slab).

Step-3 Calculation of Load:-

1) For Slab:- D.L = (unit weight of RCC × depth). kN/m²

L.L = given.

floor finish = 0.6 kN/m².

∴ Total load on slab
= (D.L + L.L + F.F) kN/m².

(ii) For Beam:-

A) Load from slab = (Spacing × Total load on slab). kN/m.

b) Self weight of Beam = (b × d × unit weight of RCC)

∴ Total load on Beam
= (Load from slab + Self wt of Beam)

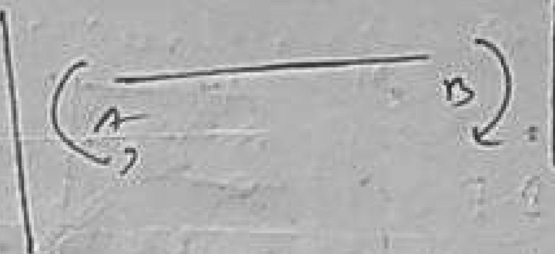
∴ Total Design load on Beam
= (PSF × working load)
= (1.5 × working load) kN/m.

Step-4 Analysis :-

i) Calculate length of column upto centre line of Beam.

ii) Fixed end moment at every point.

$$= \frac{wL^2}{12}$$



iii) Stiffness :- force or moment required to unit displacement or rotation.

a) First calculate the MOI for each member.

b) Now stiffness of each member.

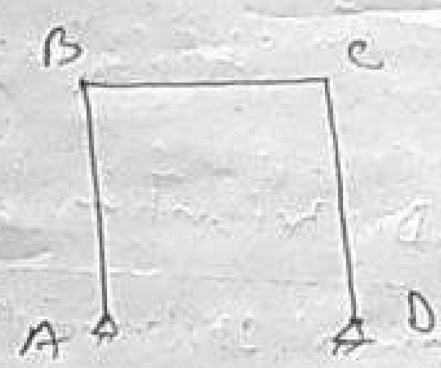
for Hing end, $k = \frac{3}{4} \frac{I}{L}$.

for fixed " , $k = \frac{I}{L}$

iv) Distribution factor :-

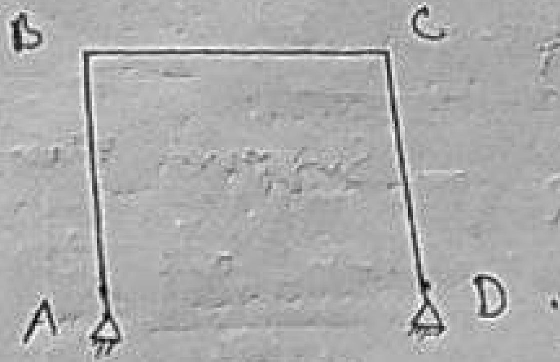
Joint	member	Stiffness	Total Stiffness	Distribution factor.
				$= \frac{\text{Stiffness of member}}{\text{total Stiffness}}$

iv) MDM



	A	B		C		D
DF		x	y	A	B	
FEM (Fixed end moment)		x'	y'	A'	B'	
		$x'' = [-(x'+y')] \times x$	y''	A''	B''	
COM (50%)			50%	50%		
[illegible]		+	−	+	−	

Ex:



$(DF)_{BA} = 0.416$, $(DF)_{BC} = 0.584$.

$(DF)_{CB} = 0.584$, $(DF)_{CD} = 0.416$.

$(M)_{BA} = 0$,

$(M)_{BC} = -261$

$(M)_{CB} = +261$

$(M)_{CD} = 0$.

	A	B		C		D
DF	0	0.416	0.584	0.584	0.416	0
FEM	0	0	−261	+261	0	0
		$= -\{0+(-261)\} \times 0.416 = 108.6$	$= -\{0+(-261)\} \times 0.584 = 152.4$	−152.4	−108.6	
COM (50%)		108.6	76.2	76.2	−108.6	
		–	–	–	–	
		–	–	–	–	
Final	0	153.28	−153.28	153.28	−153.28	0.

(ii) Width of Beam (b) = $\frac{1}{2} \times 600$

$= 300$ mm.

∴ So provide beam of $(b \times D) = (300 \times 650)$ mm.

(ii) Column :-

Let's provide column of

$= (300 \times 500)$ mm.

(iii) Slab :-

$l/D = 25$ to 30.

$l/D = 30$.

$\Rightarrow D = \frac{4000}{30} = 133$

∴ $D = 125$ mm.

$d = (125 - 25) = 100$ mm.

Step-3 Load calculation :-

(i) For slab :-

DL of slab = (Unit wt of RCC × Depth)

$= (25 \times .125)$

$= 3.125$ kN/m².

LL = 1.5 kN/m².

FF = 0.6 kN/m².

Total load on slab = $(D.L + L.L + F.F)$
$= (3.125 + 1.5 + 0.6)$
$= 5.225\ kN/m^2$.

(ii) Beam :-

a) Load from slab = (Spacing × total load on slab)
$= (4 \times 5.225)$
$= 20.9\ kN/m$.

b) Self weight of beam
$= (b \times d \times \text{unit wt of RCC})$
$= (0.3 \times 0.65 \times 25)$
$= 4.875\ kN/m$.

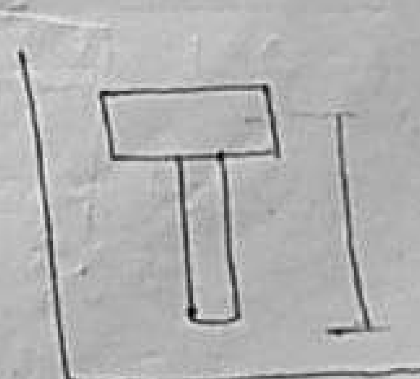
Total load on beam $= (20.9 + 4.875)$
$= 25.775\ kN/m$.

Design load on beam = (PSF × T.L)
$= (1.5 \times 25.775)$
$= 38.662\ kN/m$.

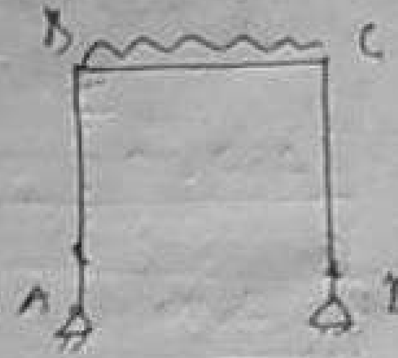
Step 4 Analysis :-

1) Length of column upto centre line of beam

$= 4.5 + 0.125 - \frac{0.65}{2}$
$= 4.3\ m$.



(ii) Fixed end moment:-



M_{FBC} =

$-wl^2/12$ $\qquad$ $+wl^2/12$

$$\therefore M_{FBC} = -\frac{wl^2}{12}$$

$$= \frac{-38.662 \times 9^2}{12}$$

$$= -261 \text{ kNm.}$$

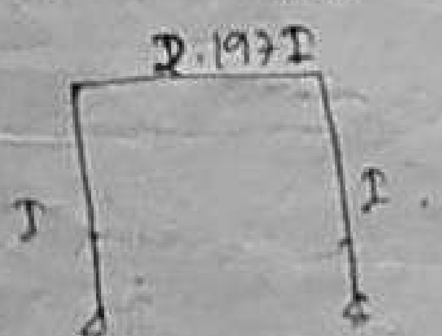
$$\therefore M_{FCB} = +wl^2/12$$

$$= +261 \text{ kNm.}$$

(iii) Stiffness:-

$$I_{AB} = \frac{bd^3}{12} = \frac{300 \times 500^3}{12} = I$$

$$I_{BC} = \frac{bd^3}{12} = \frac{300 \times 650^3}{12} = \bullet.$$



$$\frac{I_{AB}}{I_{BC}} = \left(\frac{500}{650}\right)^3$$

$\Rightarrow I_{BC} = 2.197 I_{AB}$

$\Rightarrow I_{BC} = 2.197I.$

So, $K_{BA} = \frac{3}{4} \frac{I_{AB}}{L_{AB}} = \frac{3}{4} \times \frac{I}{4.3} = 0.1744I$

$K_{BC} = \frac{I_{BC}}{L_{BC}} = 0.244I.$

$K_{CB} = 0.244I.$

$K_{CD} = 0.1744I.$

Joint	members	Stiffness	Total stiffness	DF
B	BA	0.1744I	0.4184 I	0.416
	BC	0.244I.		0.586
C	CB	0.244I	0.4184 I	0.586
	CD	0.1744I		0.416.

P.T.O

iv) MDM

	A	B		C		D
DF		0.416	0.584	0.584	0.416	
FEM	0		-261	+261		0
		108.6	152.4	-152.4	-108.6	
COM (50%)		108.6	-76.2	76.2	-108.6	
			–	–		
				–		
Total	0	153.28	-153.28	153.28	-153.28	0

So, moment at support = 153.28 kNm.

v) BM at Span :-

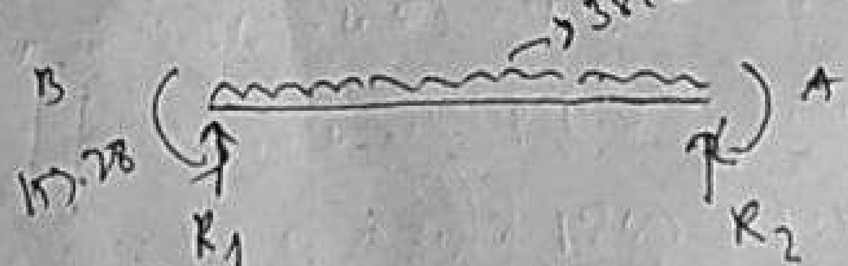
$$= \frac{wl^2}{8} - \text{support moment}$$

$$= \frac{wl^2}{8} - 153.28$$

$$= 238.18 \text{ kNm}.$$

vi) Shear force :-

a) SF at Beam :-



$\Sigma F_v = 0, \quad \Sigma M_B = 0.$

Now i) $\Sigma F_v = 0.$

$\Rightarrow R_1 + R_2 = 38.662 \times 9.$

ii) $\Sigma M_B = 0.$

$-153.28 + 38.66 \times 9 \times \frac{9}{2} - R_2 \times 9 + 153.22 = 0.$

$\Rightarrow R_2 = 174\ kN.$

$R_1 = 174\ kN.$

$\therefore SF = 174\ kN.$

b) SF at column:-

at column there is three force axial force, and horizontal & vertical force at hinge.

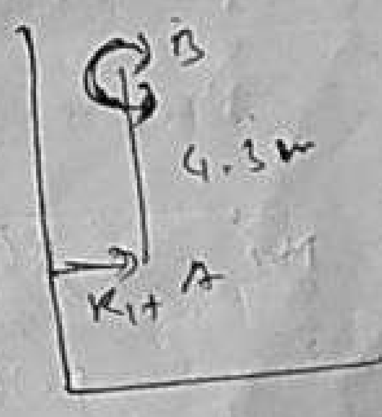
i) axial load on column = 174 kN.

ii) Horizontal force / shear force at hinge

$\Sigma M_B = 0$

$\Rightarrow -R_H \times 4.3 + 153.28 = 0$

$\Rightarrow R_H = 35.65\ kN.$

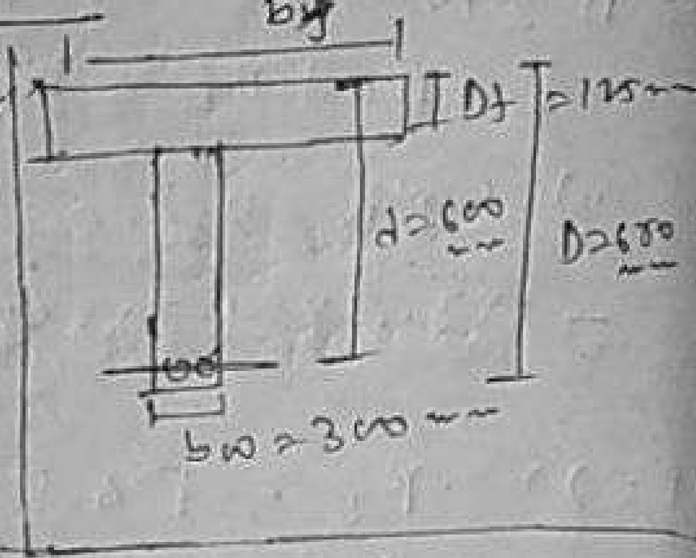


Summary

i) Max BM at Span of beam = 238.18 kNm.
ii) BM at support = 153.28 kNm.
iii) SF at beam end = 174 kN.
iv) Axial force on column = 174 kN.
v) SF at hinge = 35.65 kN.

Step 5 :- I) Design of Beam :-

A) For Span moment :-



I) $D_f = 125$ mm.
$b_w = 300$ mm.
$d = 600$ mm.

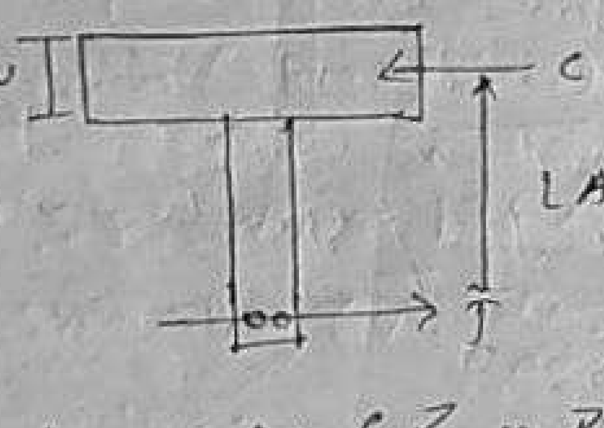
$b_f = b_w + 12 D_f$

$= 300 + 12 \times 125$

$= 1800$ mm.

II) Find NA :-
Let's assume NA is bottom fibre of flange.

∴



$\cancel{M_u' = 0.36 f_{ck} b_f D_f (d - 0.42 D_f)}$
$\cancel{= 0.36}$

$M_u' = 0.36 f_{ck} b_f x_u (d - 0.42 x_u)$
$= 0.36 f_{ck} b_f D_f (d - 0.42 D_f)$
$= 0.36 \times 20 \times 1800 \times 125 (600 - 0.42 \times 125)$

$M_u' = 886.95 \times 10^6$ Nmm
$= 886.95$ KNm.

& $M_u = 238.18$ kNm.

as $M_u < M_u'$, so NA will be in flange.

& section is under reinforced.

So, Area of r/f (steel)

$$A_{st} = \frac{0.5 f_{ck} b_f d}{f_y}\left[1-\sqrt{1-\frac{4.6 M_u}{f_{ck} b_f d^2}}\right]$$

$$= \frac{0.5 \times 20 \times 1800 \times 600}{415}\left[1-\sqrt{1-\frac{4.6 \times 238.18 \times 10^6}{20 \times 1800 \times 600^2}}\right]$$

$$\boxed{A_{st} = 1123 \text{ mm}^2}$$

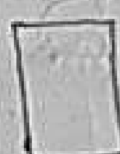
$\therefore$ No of bars for 20 mm ϕ rod

$$N = \frac{A_{st}}{A_1} = \frac{1123}{\pi/4 \times 20^2} = 3 \approx 4 \text{ bar}$$

b) <u>At support:-</u>

$M_u = 153.28$ kNm.

as flange is in tension so provide rectangular beam of 300×600 mm



300×600 mm.

So, $M_{ulim} = 0.138 f_{ck} b d^2$

$= 0.138 \times 20 \times 300 \times 600^2$

$= 298 \times 10^6$ N mm

$= 298$ kNm.

As $M_u < M_{ulim}$, so section is under reinforced.

So, $$A_{st} = \frac{0.5 f_{ck} b d}{f_y}\left[1-\sqrt{1-\frac{4.6 M_u}{f_{ck} b d^2}}\right]$$

$$= \frac{0.5 \times 20 \times 300 \times 600}{415}\left[1-\sqrt{1-\frac{4.6 \times 153.28 \times 10^6}{20 \times 300 \times 600^2}}\right.$$

$\boxed{A_{st} = 777 \text{ mm}^2}$

No of bar for 20 mm φ rod.

$$N = \frac{A_{st}}{A_1} = \frac{777}{\pi/4 \times 20^2} \approx 2.$$

$$\approx 3 \text{ bar}.$$

So, provide 3 - 20 mm φ at support.

c) Anchorage length :-

$$L_d = \frac{\phi f_y}{4 \tau_{bd}}$$

$$= \frac{20 \times 415}{4 \times 1.92}$$

$$= 1080 \text{ mm}.$$

d) Design for shear (Stirrup bars) :-

Factored S.F, $V_u = 174$ KN.

Nominal shear stress $\tau_v = \frac{V_u}{bd}$

$$= \frac{174 \times 10^3}{300 \times 600}$$

$$= 0.967 \text{ N/mm}^2.$$

% of shear r/f.

$$P_t = \frac{A_{st}(\text{support})}{bd} \times 100$$

$$= \frac{777}{300 \times 600} \times 100 = 0.431$$

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For $P_t = 0.431$ & M-20,
from table-19, IS-456-2000
Design shear stress $\tau_c = 0.48$.
as, $\tau_v > \tau_c$, so, Design of shear r/f is required,

$$V_{us} = V_u - \tau_c b d$$
$$= 176 \times 10^3 - 0.48 \times 300 \times 600$$
$$= 87600 \text{ N}$$

provide 2-legged 8mm ϕ bar as stirrup bar

$$A_{sv} = 2 \times \pi/4 \times 8^2 = 101 \text{ mm}^2$$

Spacing:

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

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

$$= \cancel{0.87 \times 87600}$$

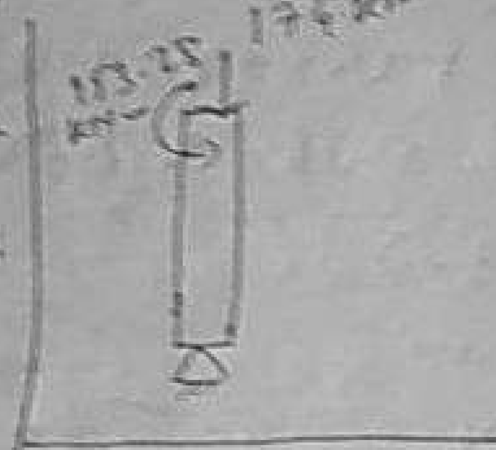
$$= \frac{0.87 \times 415 \times 101 \times 600}{87600}$$

$$\boxed{S_v = 248 \text{ mm}}$$

$$\text{Spacing } (S_v) \not> \min \begin{cases} \text{i) } 248 \\ \text{ii) } 0.75d \\ \text{iii) } 300 \text{ mm} \end{cases}$$

$$\therefore S_v = 248 \text{ mm}$$

(5) Design of column:-



i) Axial load (P_u) = 174 kN.

ii) moment (M_u) = 153.28 kN-m

for this case we have to interaction diagram.

for introduction diagram

i) steel grade → Fe 415

ii) Dimension → 300 × 500

iii) P_u & M_u

iv) $d'/D = \frac{50}{500} = 0.1$.

v) 2-phase steel arrangement.

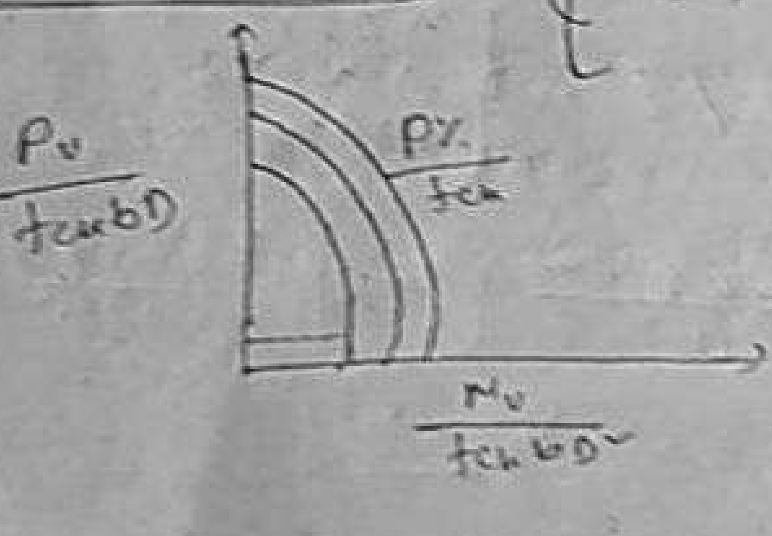
Shape
- Rect.
  - 2-phase
  - 4-phase
- circular

so, i) $\frac{P_u}{f_{ck} bD} = \frac{174 \times 10^3}{20 \times 300 \times 500} = 0.058$

ii) $\frac{M_u}{f_{ck} bD^2} = \frac{153.28 \times 10^6}{20 \times 300 \times 500^2}$

$= 0.102$.

Introduction Diagram:- {From SP-16 (IS:456-1978)}



For, Fe-415, & $d'/D = 0.1$, Rectangular two phase arrangement →
from chart - 32, page - 117 of SP-16

$$\frac{P\%}{f_{ck}} = 0.06$$

$\Rightarrow P\% = 0.06 \times 20$

$= 1.2\%$

So, $A_{sc} = \frac{P}{100} \times bD$

$= \frac{1.2}{100} \times 300 \times 500$

$= 1800 \ mm^2$

No of bar for 25 mm ϕ.

$$N = \frac{A_{sc}}{A_1} = \frac{1800}{\pi/4 \times 25^2}$$

$= 3. \approx 4$ bar

So, provide 4-25 mm ϕ as longitudinal r/f.

(4) Tie-up bar for column :- (Stirrup)

(i) dia of Tie-up bar

$d_s \geq$ Max $\begin{cases} i) \ \phi/4 \\ ii) \ 6mm \end{cases}$

here, $\phi/4 = 25/4 = 6.25$

$\therefore d_s \geq 6.25$ mm

$d_s = 8$ mm.

ii) Spacing of tie-up bar :-

Spacing (S) $\leq$ min $\begin{cases} \text{i) Least lateral dimension} \\ \text{ii) } 4 \times 16 \\ \text{iii) } 300 \text{ mm} \end{cases}$

Here,

i) LLD = 300 mm.

ii) $25 \times 16 = 400$

iii) 300 mm.

$\therefore$ S = 300 mm c/c.

So, provide 8 mm d. bar @ 300 mm c/c distance.

⊛ Design of footing :-

Step → Load → Dimension → r/f.

Ultimate load on column (Pu) = 174 kN.

$\therefore$ working load $= \frac{Pu}{PS.F}$

$= \frac{174}{1.5}$

$= 116$ kN.

Self weight of column

= vol × unit weight of RCC

$= 0.3 \times 0.5 \times 4.3 \times 25$

= 16.125 kN.

$\therefore$ Total load by column = 116 + 16.125

= 132.125 kN.

so, Self weight of footing

$= 10\%$ of column load

$= 132 \times 10/100$

$= 13.21$ KN,

so, Total load on soil

$=$ column load + Self wt. of footing

$= 132.125 + 13.21$

$= 145.3$ KN.

as, soil Bearing capacity $= 150\ kN/m^2$ (given)

so, plan area of footing

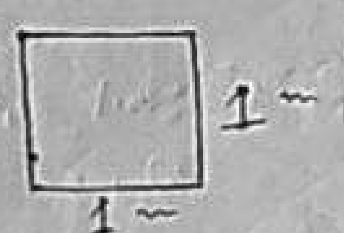
$$A = \frac{\text{total load}}{SBC}$$

$$= \frac{145.3}{150}$$

$= 0.98\ m^2$.

$\approx 1\ m^2$.

∴ so, plan is,

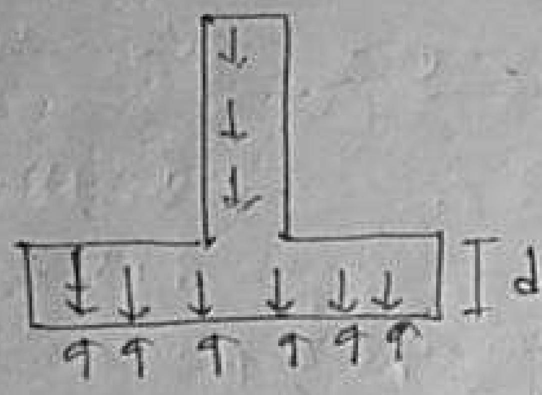


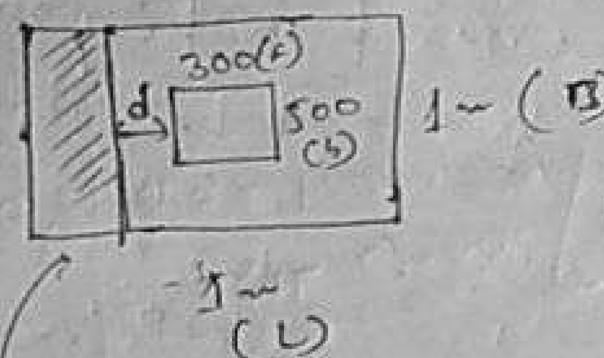
so Dimension of Square footing $= 1 \times 1$ m.

Depth of footing:-

→ By one way shear.

→ one way shear $\leq k\tau_c$.





$\left(\frac{L-a}{2} - d\right)$

→ Net soil pressure

$w_u = \frac{1.5P}{A}$ [P = column load]

$= \frac{1.5 \times 132.125}{1 \times 1}$

$= 200\ kN/m^2$.

$= 0.2\ N/mm^2$

So. one way shear $\leq k\tau_c$.

$k = 1$

τ_c = for 0.2 % of steel

$= 0.32\ N/mm^2$. (table-19).

$\therefore$ one way shear stress $\leq k\tau_c$

$$\Rightarrow \frac{\text{Shear load}}{\text{Resisting Area}} \leq k\tau_c$$

$$\Rightarrow \frac{\text{Soil pressure} \times \text{Area}}{\text{Resisting Area}} \leq k\tau_c$$

$$\Rightarrow \frac{w_u \times \left(\frac{L-a}{2} - d\right) \times 1m}{1 \times d} \leq 1 \times 0.32$$

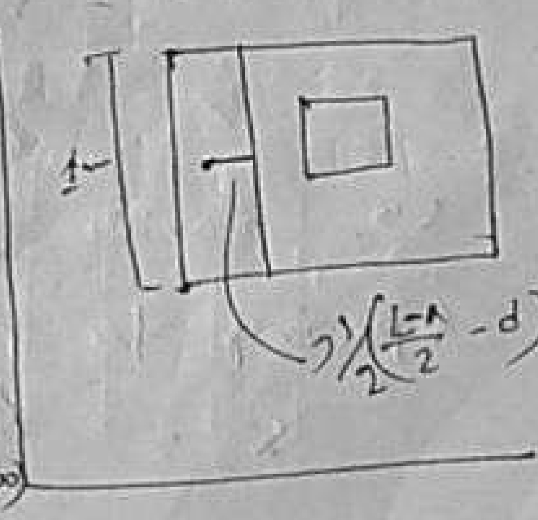
$\Rightarrow\ d = 187.5\ mm$

$\approx 200\ mm$

$\therefore\ D = 250\ mm$.

(*) **Bending moment :-**

$$\therefore M_u = w_u \times \left(\frac{L-a}{2} - d\right) \times 1 \times \frac{1}{2}\left(\frac{L-a}{2} - d\right)$$



$$M_u = 0.2\left(\frac{1000-300}{2} - 200\right) \times 1000 \times \frac{1}{2}\left(\frac{1000-300}{2} - 200\right)$$

$= 9 \times 10^6\ Nmm$

$= 9\ kNm$.

So, $M_{ulim} = 0.138\, f_{ck} b d^2$

$= 0.138 \times 20 \times 1000 \times 200^2$

$= 110 \times 10^6$ · Nmm

$= 110$ kNm

As, $M_u < M_{ulim}$ so, section is under reinforced.

So, Area of steel

$$A_{st} = \frac{0.5 f_{ck} b d}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck} b d^2}}\right]$$

$$= \frac{0.5 \times 20 \times 1000 \times 200}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 9 \times 10^6}{20 \times 1000 \times 200^2}}\right]$$

$$\boxed{A_{st} = 126.2\ mm^2}$$

& $(A_{st})_{min} = 0.12\%\ bd$

$= \frac{0.12}{100} \times 1000 \times 200$

$= 240\ mm^2$.

So, provide $A_{st} = 240\ mm^2$.

Spacing for 12 mm ϕ bar

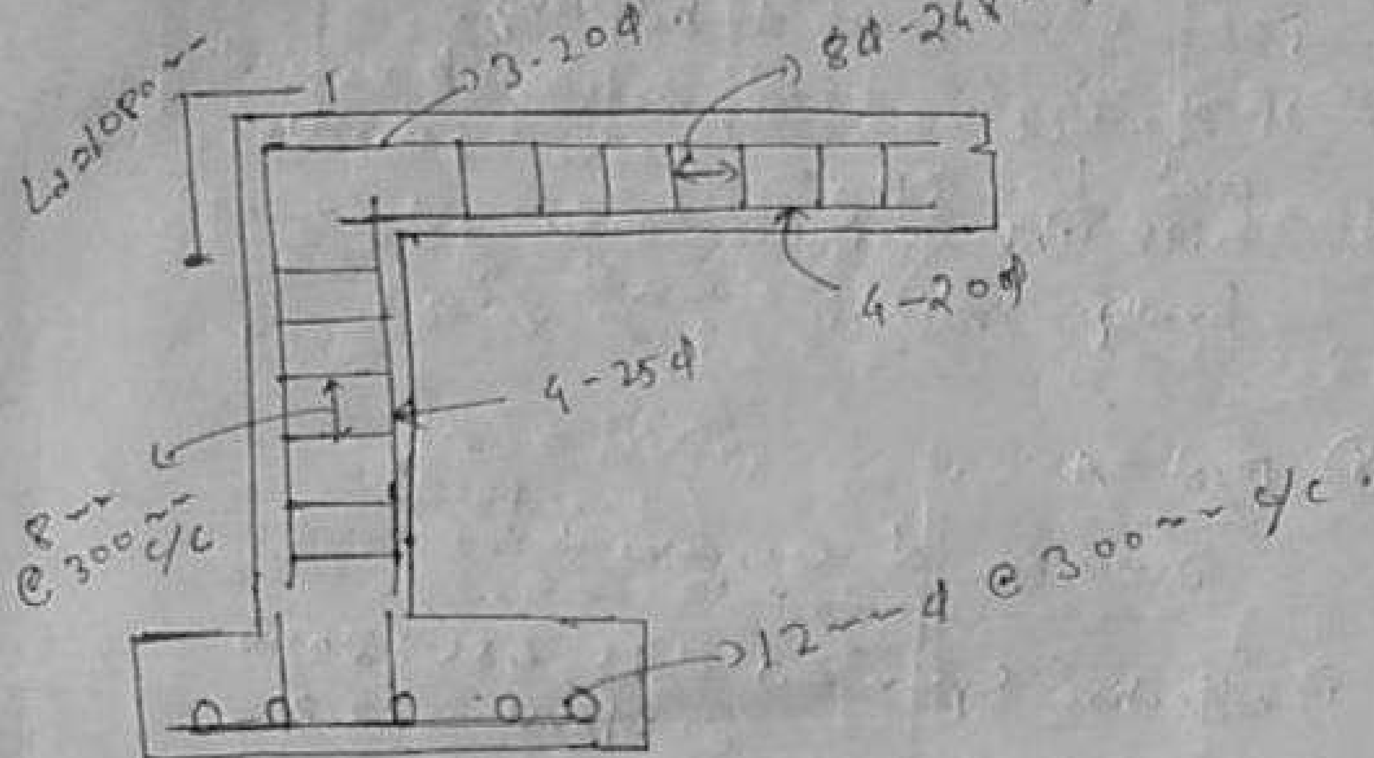
$S = \frac{A_1}{\text{total Area}} \times \text{width}$

$= \frac{\pi/4 \times 12^2}{240} \times 1000$

$= 471$ mm c/c.

S = 300 mm c/c [assume S = 300 mm]

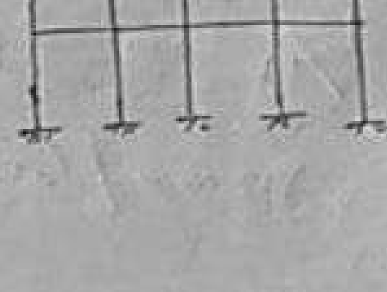
So, provide 12 mm dia bar @ 300 mm c/c on footing.



Building frame

→ Used for tall Buildings.

→ Rigidly inter-connected beam and columns of multi bay & multistoried are called building frame.



Analysis and design of Building frame :→

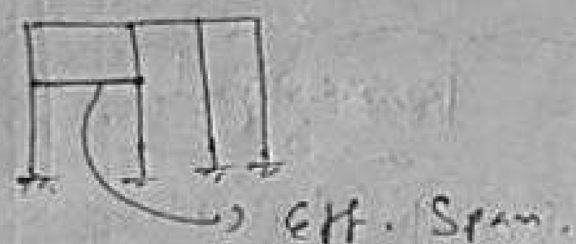
part - 1 → Geometry of the building.
part - 2 → Loads & load combinations.
part - 3 → Analysis.
part - 4 → Design & Detailing.

→ Analysis & design will be carried out from top to bottom. It starts with slab then Beam then Column and finally footing.

Part - A :→

Dimension of component :→

i) Effective Span → IS : 456 : 2000 Clause - 22.2



ii) Stiffness :→

$k = I/L$ (fixed end).

iii) Beam :→

$d = L/12$ to $L/15$

$d' = 50mm / 40mm$

a) $D = d + d'$
b) width $(b) = (1/2 \text{ to } 1/3)$ of D
c) $(b \times D)$ → overall dimension.

iv) **Column** :-) According to beam.

v) **Slab** :-)

One way → $l/d = 25-30$

Two way → $l/d = 30-35$.

$\boxed{D = d + d'}$.

Part - 2

Load Combination :-)

↓

DL, LL, WL, SL, EL.

↓

i) DL + LL
ii) DL + WL
iii) DL + EL
iv) DL + EL + WL

} IS-875, Part-5

Part - 3

Analysis :-)

a) Reaction parameter → SF, BM

↓

By
i) Stiffness matrix method
ii) Finite element analysis

Part - 4 Design :-

{ IS - 456 - 2000, SP - 16 } → Beam, column, footing,

* Part - 2 :- Load combination,

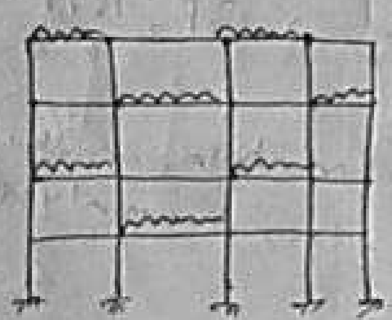
~~NO OF FLOOR~~ i) Reduction
ii) Placement of load for critical value.

i) Reduction :-

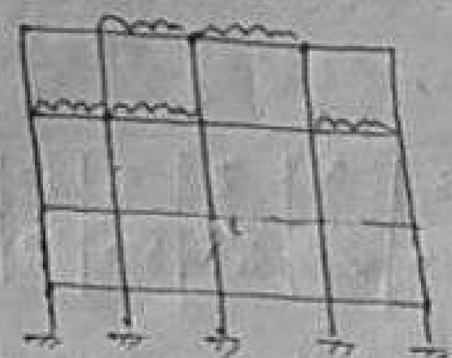
No of floor	Reduction (%) (Live Load)
1	0
2	10
3	20
4	30
5 - 10	40
> 10	50

ii) Placement of load :-

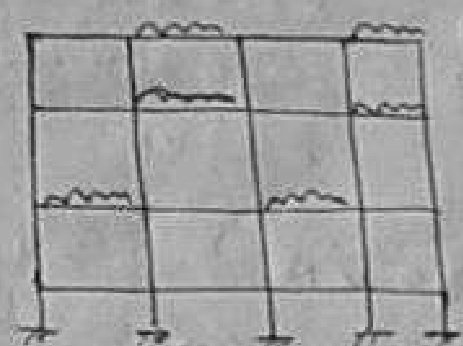
A) For max moment in span :-



b) Max moment at support :-



c) Max moment at column end :-



→ D- Box pattern

(*) Load combination :-

→ By IS : 875, part - 5

→ combination :-

i) (DL + LL) 1.5
ii) (DL + WL) 1.5
iii) (DL + EL) 1.5
iv) (DL + LL + EQ) 1.2
v) (DL + LL + WL) 1.2

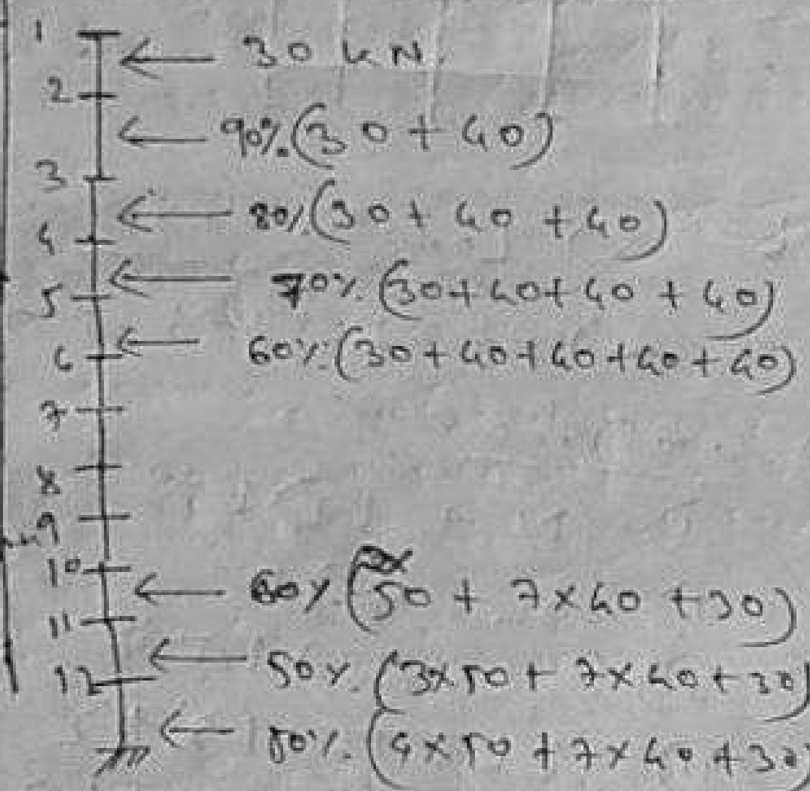
Q The total imposed load coming on column from different floors are given below

Floor No from top	1	2	3	4	5	6	7	8	9	10	11	12
Floor load in kN	30	40	40	40	40	40	40	40	50	50	50	50

Calculate the imposed load to be considered for the design of column members at different floor level.

S,

No of floor	Axial load
1	30
2	40
3	40
4	40
5	40
6	40
7	40
8	40
9	50
10	50
11	50
12	50



Q) A is the section at mid span of a beam, B is a section at end of the span and C is a section of column in 4th floor from top. The design forces obtained at these sections due to various forces is listed in table below for working load condition. Determine the design force at section A, B and C.

Load	A moment	A shear	B moment	B shear	C moment	C shear
DL	102	8	-132	90	24	360
LL	68	11	-96	36	22	264
WL	4	0	-24	8	16	48
EL	6	0	-28	12	20	28

Load Combination :-)

Load combination	A moment	A shear	B moment	B shear	C moment	C Axial
i) (DL + LL) 1.5	255	28.5	-342	249	69	936
ii) (DL+WL) 1.5	159	12	-234	147	60	612
iii) (DL + EL) 1.5	162	12	-240	153	66	582
iv) (DL+LL + WL) 1.2	208.8	22.8	-302.4	208.8	74.4	806.4
v) (DL+LL+EL) 1.2	211.2	22.8	-307.2	213.6	79.2	782.4

(8) Axial load on column without reduction
= 936 kN.

So After reduction the axial load on column = $936 \times 70/100$

= 655.2 kN.

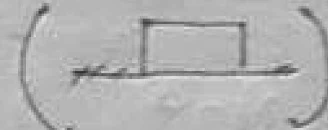
Now

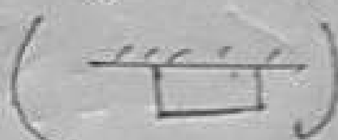
Max moment at A = 255 kN-m
Max S.F at A = 28.5 kN.
Max moment at B = -342 kN-m.
Max S.F at B = 249 kN.
Max moment at C = 79.2 kN-m.
Max axial force at C = 655.2 kN.
(after reduction).

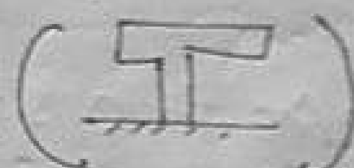
Mod-2 → Design of water tanks.

Design of water tank

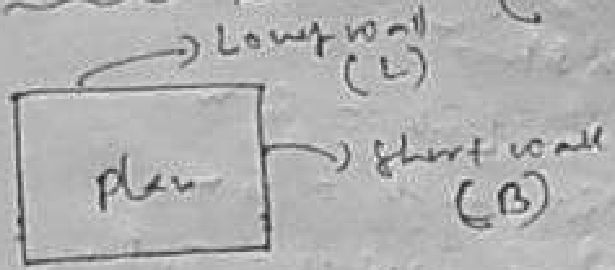
(*) Types of water tank :- (Based on location)

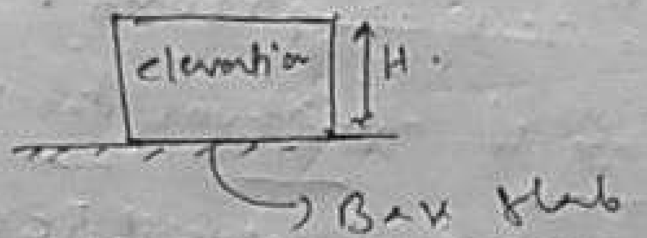
Rect / Cir. → Water tank resisting on ground. 

Rect / Cir. → Water tank below the ground. 

Rect / Cir. → elevated water tank 

⊛ Rectangular water tank :- (Resisting on ground). (IS: 3370).





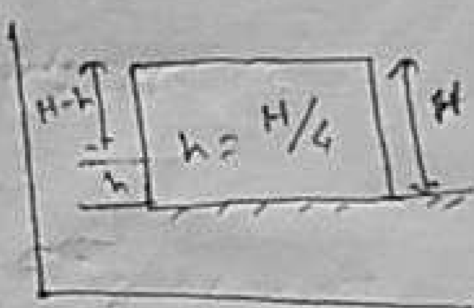
⊛ have to design → i) Long wall
ii) short wall.
iii) Base slab.

⊛ code consideration {By IS: 3370 (Part-V)}.

(i) If, $\boxed{L/B < 2}$ then,

a) (H − h) should be design as a continious frame.

b) h - should be design as a cantilever.



(ii) If $\boxed{L/B \geq 2}$ then,

a) Long (L) wall should be designed as a Cantilever.

b) h - should be design as cantilever.

c) (H − h) - should be design as a slab.

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Q) Design a rectangular water tank for the following data.

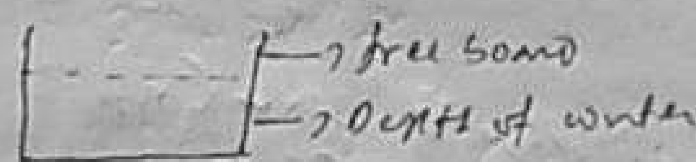
i) Length of tank = 6 m.
ii) width of tank = 4 m.
iii) Depth of water = 3.5 m.
iv) Tank rest on ground.

A) Step-1

→ Assume M30 concrete & Fe415 steel.

→ $L = 6m$, $B = 4m$, water depth = 3.5m.

→ consider free board = 0.20 m.



$\therefore H = 3.5 + 0.2 = 3.70 m$.

→ checking L/B ratio :-

$$L/B = 6/4 = 1.5 < 2.$$

So, According to code IS:3370. part-IV

1) (H-h) will be designed as a continious frame.

ii) $h = H/4 = 3.7/4 = 0.925 \approx 1m$ will be designed as a cantilever.

(42) Step 2 Design constant :-

i) σ_{cbc} → permissible stress in concrete.
ii) k → neutral axis co. eff.
iii) J → lever arm co. eff.
iv) m → modular ratio
v) Q → moment of resistance factor.
vi) σ_{st} → permissible stress in steel.

(i) σ_{cbc} = 10 N/mm² for M30 concrete.
{from, IS: 3370 (Part-2), table-2}.

(ii) $m = \frac{280}{3\sigma_{cbc}}$
$= \frac{280}{3\times10} = 9.33$.

(iii) σ_{st} = 130 N/mm² for HYSD bar
{from, IS: 3370 (Part-2) table-4}

(iv) $k = \frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}}$
$= \frac{9.33\times10}{9.33\times10+130} = 0.418$.

(v) $J = 1 - k/3 = 1 - \frac{0.418}{3} = 0.861$.

(vi) $Q = \frac{1}{2}\times\sigma_{cbc}\times k\times J = \frac{1}{2}\times10\times0.418\times0.861$
$= 1.80$.

For M30 & Fe 415 steel

$\sigma_{cbc} = 10 \ N/mm^2$

$\sigma_{st} = 130$ "

$k = 0.418$

$J = 0.861$

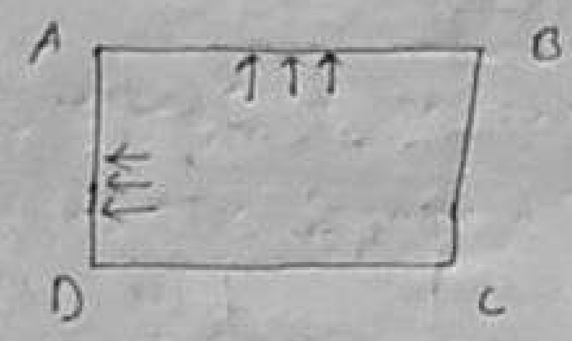
$Q = 1.80$

Step 3 Design of continuous frame (H-h)

i) water pressure at depth (H-h)

$$P = \gamma_w \times (H-h)$$
$$= 10 \times (3.7 - 1.0)$$
$$= 27 \ kN/m^2.$$

ii) Fixed end moments (FEM)



$$\rightarrow M_{FAB} = -\frac{pl^2}{12} = -\frac{27 \times 6^2}{12} = -81 \ kN\text{-}m$$

$$M_{FAD} = +\frac{pB^2}{12} = \frac{27 \times 4^2}{12} = 36 \ kN\text{-}m$$

iii) DF :-

Joint	member	K (Stiffness)	ΣK	$DF = \frac{K}{\Sigma K}$
A	AB	$\frac{4EI}{6} = 0.67 EI$	$1.67 EI$	0.4
	AD	$\frac{4EI}{4} = 1 EI$		0.6

iv) MDM :-

	D	A	B
DF	0.6	0.4	
FEM	36	-81	
	$-\frac{\{36+(-81)\}}{} \times 0.6$ $= 27$	$-(-81+36) \times 0.4$ $= 18$	
	63	-63	

$\therefore$ BM at Support $= 63$ kN-m.

v) BM at centre of long span (L) AB

$$= \frac{PL^2}{8} - M_A$$

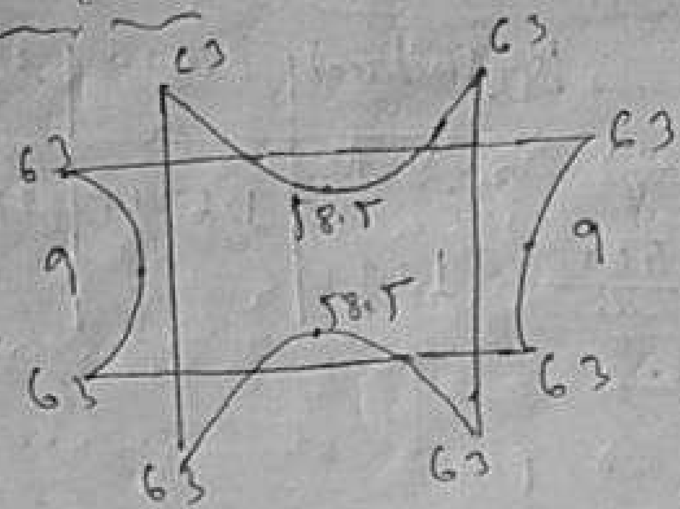
$$= \frac{27 \times 6^2}{8} - 63 = 58.5 \text{ kN-m}.$$

BM at centre of short span (B) AD

$$= \frac{PB^2}{8} - M_D$$

$$= \frac{27 \times 4^2}{8} - 63 = -9.0 \text{ kN-m}$$

Diagram of BM



→ Direct tension in longer wall

$= \frac{wl}{2} = \frac{PB}{2} = 27 \times 4/2$

$= 54$ kN.

→ Direct tension in short wall

$= \frac{wl}{2} = PL/2 = 27 \times 6/2$

$= 81$ kN/m.

vi) $M = Q \times bd^2$ (we know)

$63 \times 10^6 = 1.80 \times 1000 \times d^2$

$\Rightarrow d = 187.08$ mm

≈ 250 mm.

$D = d + 50$ (eff. cover)

$= 300$ mm.

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Step-4 **Design of longer wall :-**

(1) at support :-

$M = 63$ kN-m (Tension on liquid face)

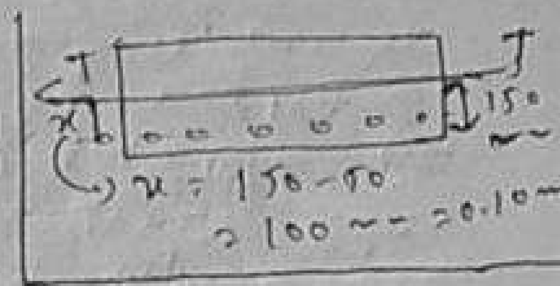
$T = 54$ kN.

$\Rightarrow$ Modified moment at support

$= 63 - T \times x$

$= 63 - (54 \times 0.10)$

$= 57.6$ kN.m



$\Rightarrow$ Ast for moment

$$= \frac{M}{\sigma_{st} \times J \times d}$$

$$= \frac{57.6 \times 10^6}{130 \times 0.861 \times 250}$$

$= 2058.42$ mm²

$\Rightarrow$ Ast for tension

$$= \frac{T}{\sigma_{st}}$$

$$= \frac{54 \times 10^3}{130} = 415.38 \text{ mm}^2$$

$\therefore$ Total Ast $= (2058.42 + 415.38)$

$= 2473.80$ mm²

provide 20 mm ϕ bars

$\therefore$ spacing $= \dfrac{A_{st}(1)}{A_{st}} \times 1000$

$= \dfrac{\pi/4 \times 20^2}{2473.80} \times 1000$

$= 126.93$

≈ 120 mm c/c. (Liquid face).

$\Rightarrow$ for remote face

$A_{st} = 0.24\%$.

(ii) centre :-

$\rightarrow M = 58.5$ kN-m (tension on remote face).

$\rightarrow T = 54$ kN.

$\rightarrow$ Modified moment at centre

$= M - (T \times x)$

$= 58.5 - (54 \times 0.10)$

$= 53.10$ kN-m.

$\rightarrow A_{st}$ for moment $= \dfrac{M}{\sigma_{st} \cdot j \cdot d}$

$= \dfrac{53.10 \times 10^6}{130 \times 0.861 \times 250}$

$= 1897.61$ mm^2

$\rightarrow$ Ast for Torsion $= \frac{T}{\sigma_{st}}$

$= \frac{54 \times 10^3}{130}$

$= 415.38 \, mm^2$.

$\therefore$ Total Ast required $= (1897.61 + 415.38)$

$= 2313 \, mm^2$.

$\rightarrow$ Now provide 20ϕ bars

$\therefore$ Spacing $= \frac{Ast(1)}{Ast} \times 1000$

$= \frac{\pi/4 \times 20^2}{2313} \times 1000$

$= 135.75$

$\approx 130 \, mm$ c/c (re-ste face)

$\rightarrow$ for liquid face :-

~~[illegible]~~

Ast = 0.24 %.

(iii) Distribution Steel :-

$\rightarrow$ Ast min $= 0.24\%$ (for Fe 415)

$= \frac{0.24}{100} \times bD$

$= \frac{0.24}{100} \times 1000 \times 300$

$= 720 \, mm^2$

→ provide 10 mm d bar. then

$$\text{Spacing} = \frac{A_{st}(1)}{A_{st}} \times 1000$$

$$= \frac{\pi/4 \times 10^2}{720} \times 1000$$

$$= 109.02$$

$$\approx 100 \text{ mm } c/c.$$

Step-5 Design of shorter wall :-

(1) At support :-

$M = 63$ kN-m (Tension on liq. face)

$T = 81$ kN.

⇒ modified moment $= M - T \times x$

$= 63 - (81 \times 0.10)$

$= 54.9$ kN-m

⇒ Ast for moment $= \frac{M}{\sigma_{st} \times J \times d}$

$$= \frac{54.9 \times 10^6}{130 \times 0.861 \times 250}$$

$= 1961.44$ mm²

⇒ Ast for tension $= \frac{T}{\sigma_{st}}$

$$= \frac{81 \times 10^3}{130}$$

$= 623$ mm².

∴ Total Ast required

$= (1961.94) + (623)$

$= 2584.94 \ mm^2$

∴ ~~provide 20 mm φ @ 120 mm c/c~~

∴ provide 20 mm d bars then,

$$\text{spacing} = \frac{Ast(1)}{Ast} \times 1000$$

$$= \frac{\pi/4 \times 20^2}{2584.94} \times 1000$$

$= 121.47$

≈ 120 mm c/c

(Liquid face)

⇒ for remote face :-)

$Ast = 0.24\%$ of bD.

(ii) At centre :-)

→ $M = -9.0$ kN·m (Tension on liquid face)

→ $T = 81$ kN.

→ modified moment $= M - T \cdot x$

$= -9 - (81 \times 0.10)$

$= -17.1$ kN·m.

→ Ast for moment $= \dfrac{M}{\sigma_{st} \cdot j \, d}$

$$= \frac{17.1 \times 10^6}{130 \times 0.861 \times 250}$$

$= 611.10 \ mm^2$

→ Ast for tension $= \frac{T}{\sigma_{st}}$

$$= \frac{81 \times 10^3}{130} = 623 \text{ mm}^2$$

→ Total Ast $= (611.10 + 623)$

$$= 1234.10 \text{ mm}^2$$

Now provide 16 mm ϕ bar, so

$$S = \frac{A_{st}(1)}{A_{st}} \times 1000$$

$$= \frac{\pi/4 \times 16^2}{1234.10} \times 1000$$

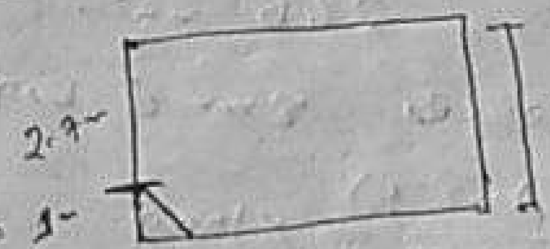
$$= 162.83$$

$$\approx 160 \text{ mm } c/c \text{ (Liquid face)}$$

→ Now for remote face :-

provide Min Ast $= 0.24\%$ bD.

iii) Cantilever action of bottom h = 1 m :-



→ pressure at base $= \gamma_w \cdot H = 10 \times 3.7 = 37 \text{ kN/m}^2$

→ B.M at base $= \frac{1}{2} \times (\gamma_w \cdot H) \times h \times h/3$

$$= \frac{1}{2} \times 37 \times 1 \times 1/3$$

$$= 6.16 \text{ kN.m (Tension on liquid face)}$$

$\rightarrow$ $A_{st} = \dfrac{M}{\sigma_{st} \cdot J \times d}$

$$= \frac{6.16 \times 10^6}{130 \times 0.861 \times 250}$$

$= 220.13\ mm^2 < A_{st}$ min $(0.24\%\ bD)$

So provide $0.24\%\ bD = 360\ mm^2\ A_{st}$.

Step-6 **Design of Base Slab:** $\rightarrow$

$\longrightarrow$ provide min 200 mm thick slab (Assumed)

$\longrightarrow$ provide steel only on top face.

$\Longrightarrow$ provide min A_{st} $(0.24\%\ bD)$.

So, thickness of Surface zone

$$= \frac{200}{2} = 100\ mm.$$

only upper Surface will be reinforced

$$\therefore A_{st} = \frac{0.24}{100} \times 1000 \times 100$$

$$= 240\ mm^2.$$

$\rightarrow$ Now provide 8 mm ϕ bar, So,

$$S = \frac{A_{st}(1)}{A_{st}} \times 1000$$

$$= \frac{\pi/4 \times 8^2}{240} \times 1000$$

$$= 209.33 \approx 200\ mm\ c/c.$$

✱ Design of circular water tank (Resting on ground) :-

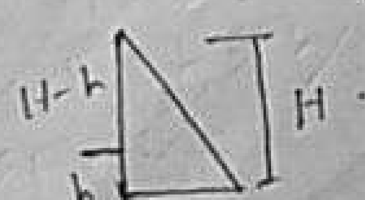
⊛ Circular water tank
- → Flexible Base
- → Rigid Base. (Restrained Base).

Flexible Base :- → Wall free to expand or contract at some point. (Shear & Stress is low).

Rigid Base :- → Wall is not free to expand or contract. (Shear & Stress is high).

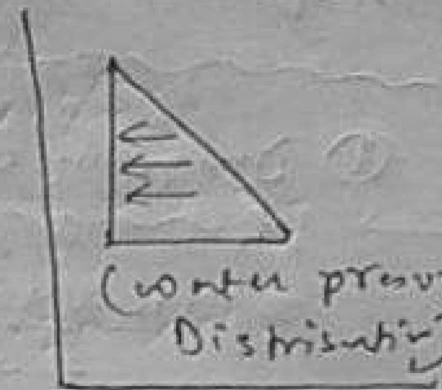
✱ Criteria for Flexible Base & Rigid Base :-

Flexible Base	Rigid Base
i) Max pressure at Base $P_A = \gamma_w \times H$.	i) Cantilever moment at Base $= \left(\frac{1}{2} \times \gamma_w \times H \times h \times \frac{h}{3}\right)$
ii) Hoop tension at Base $T = \gamma_w \times H \times D/2$ $= P_A \times D/2$ [D = Dia of the tank]	ii) Max hoop tension at H−h, $T = \gamma_w \cdot (H-h) \times D/2$ where, $h = H/3$ or 1 m (whichever is greater)



⊛ Imp. points :-

→ According to water pressure Distribution in tank max pressure intensity is in the Base of the tank.



→ So provide max r/f in Bottom & towards up we can reduce the r/f.

⊛ Design Steps :-

i) Design the wall at different height.
ii) calculate thickness of wall.
iii) vertical r/f or Distribution steel.
iv) Design of Base slab.

Q. Design a circular water tank for the following data.

i) Dia of tank = 6m.
ii) Depth of water = 3.75m
iii) Tanks rest on ground.
iv) Wall & slab are not monolithic.
v) Use M30 concrete & Fe415 steel.

→ Depth of water = 3.75 m.
→ Assume free board = 0.25 m.
$\therefore H = 3.75 + 0.25 = 4m$.
→ Dia of tank (D) = 6m

Step-1 ~~Hoop tension~~

→ water pressure at Base (P_w) = $\gamma_w \cdot H$
$= 10 \times 4$
$= 40\ kN/m^2$.

→ Hoop tension at Base
$T = P_w \times D/2$
$= 40 \times 6/2 = 120\ kN$.

→ Ast req = $\frac{T}{\sigma_{st}}$
$= \frac{120 \times 10^3}{130} = 923.07\ mm^2$.

→ provide 12 mm-ϕ bar so,
$S = \frac{Ast(1)}{Ast} \times 1000$
$= \frac{\pi/4 \times 12^2}{923} \times 1000$
$= 122.47 \approx 120\ mm\ c/c$.

→ Now at 2m from top (H = 2m)

→ $P_A = \gamma_w \times H$

$= 10 \times 2 = 20\ kN/m^2$.

→ $T = P_A \times D/2$

$= 20 \times 6/2 = 60\ kN$.

→ $A_{st} = \frac{T}{\sigma_{st}}$

$= \frac{60 \times 10^3}{130} = 461.53\ mm^2$.

→ provide 12 mm ϕ bar so,

$S = \frac{A_{st}(1)}{A_{st}} \times 1000$

$= \frac{\pi/4 \times 12^2}{461} \times 1000$

$= 245\ mm$

$\approx 250\ mm\ c/c$.

Step 2 thickness of wall :→ (t)

we know

$$\sigma_{ct} = \frac{T}{A_g + (m-1) A_{st}} \quad [IS:456]$$

where, σ_{ct} = permissible tensile stress in concrete.

T = tension (N/m)

A_g = gross Area = $b \times t$

m = modular ratio

A_{st} = area Area of Steel.

Now

$$\sigma_{ct} = \frac{T}{A_g + (m-1) A_{st}}$$

$$\Rightarrow 1.5 = \frac{120 \times 10^3}{1000 \times t + (9.33-1) \times 923}$$

$$\Rightarrow t = 70.55 \text{ mm}$$

$$\approx 100 \text{ mm}$$

for M30,
$\sigma_{ct} = 1.5$
(IS: 3370, P-2)

$m = \frac{280}{3\sigma_{cbc}}$
$= \frac{280}{3 \times 10}$
$= 9.33$
(IS: 456, Pg-81)

Step-3 vertical r/f / Distribution steel :->

$P_{t\,min} = 0.24\%$ of bD

$\therefore A_{st}$ min $= \frac{0.24}{100} \times 1000 \times 100$

$= 240\ mm^2$.

-> Now provide 8 mm ϕ bar so,

$S = \frac{A_{st}(1)}{A_{st}} \times 1000$

$= \frac{\pi/4 \times 8^2}{240} \times 1000$

≈ 200 mm c/c.

Step-4

Bar slab :->

-> Assume thickness of bar slab = 150 mm.

-> And provide 0.24% steel.

$\therefore A_{st} = \frac{0.24}{100} \times 1000 \times 75$ [for upper side].

$= 180\ mm^2$.

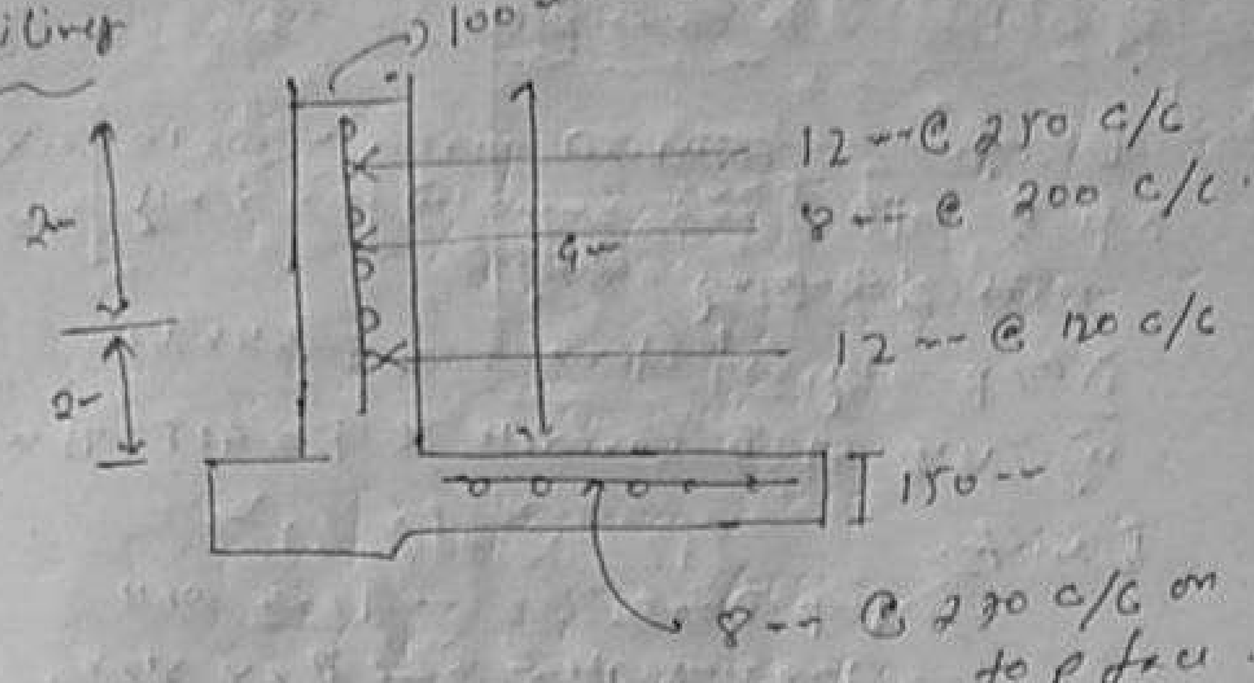
-> provide 8 mm ϕ bar so,

$S = \frac{A_{st}(1)}{A_{st}} \times 1000$

$= \frac{\pi/4 \times 8^2}{180} \times 1000$

≈ 270 mm c/c.

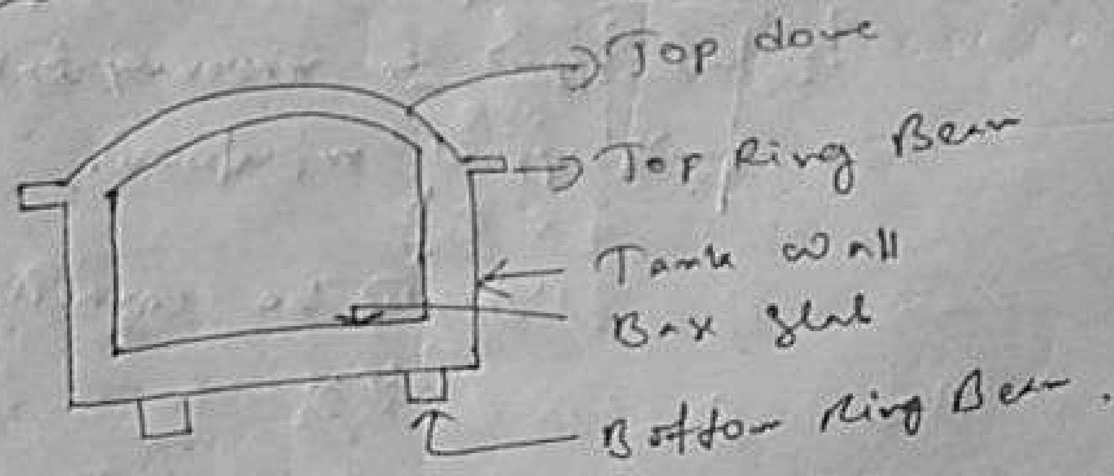
Detailing



* elevated water tank :-

- Rectangular / Square
- circular
- Intze
- Conical

* circular elevated water tank :-



Example

Q Design an overhead circular water tank with flat bottom and supported on ring beam with following data.

i) Capacity of tank = 4.5 lakh lit.

ii) Use M30 concrete & Fe 415 steel

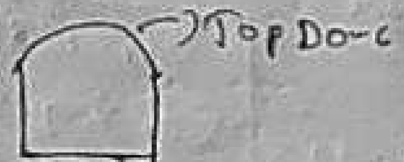
Design,

— Top Dome — Tank wall

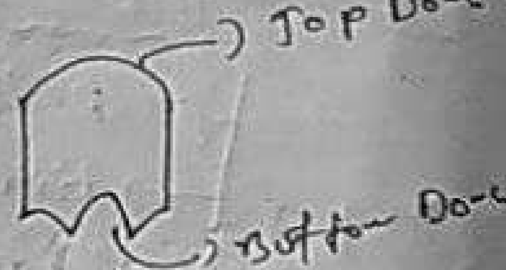
— Top ring Beam — Base slab.

— Bottom ring Beam.

⊛ Bottom of tank :-



(flat Bottom)



(Intze tank).

⊛ Support :-

→ Support on masonry wall

→ Support on column.

↓

should be connected on ring Beam.

Solution :-

1) Dimension :-

$V = A \times H$ | $V = 4.5$ lakh lit

$= \pi/4 \times D^2 \times H$ | $= 450\ m^3$

$D = 12\ m$ (Assumed)

$\therefore 450 = \pi/4 \times 12^2 \times H$

$\Rightarrow H = 3.97$

$\approx 4\ m.$

Design :-

(1) Design of top Dome :-

- i) Meridional force (T_1)
- ii) Hoop tension (T_2)

* Meridional force :- Tension force in longitudinal direction in spherical structure or Dome.

$$\Rightarrow \boxed{T_1 = \frac{w \times R}{1 + \cos\theta}}$$

w = load on dome

= LL + S.W. of Dome.

Now, $L.L = 1.5\ kN/m^2$ (Assumed)

S.W = thickness × unit weight

$= 0.10 \times 25$ [thickness of Dome = 75 to 100 mm]

$= 2.5\ kN/m^2.$

$\therefore W = 1.5 + 2.5 = 4\ kN/m^2.$

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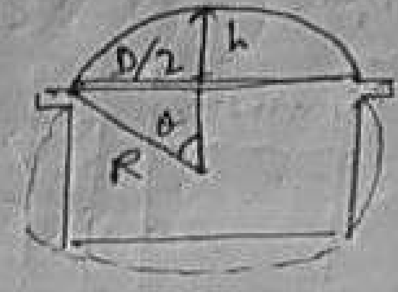
$$\Rightarrow \boxed{R = \frac{(D/2)^2 + h^2}{2 \times h}}$$



h = Rise of Dome

$= 0.2 \times D$ (Assumed)

$= 0.2 \times 12$

$= 2.4$ m.

$$\Rightarrow R = \frac{(12/2)^2 + (2.4)^2}{2 \times 2.4}$$

$= 8.7$ m.

$$\Rightarrow \sin\theta = \frac{D/2}{R}$$

$$\sin\theta = 6/8.7$$

$$\Rightarrow \theta = \sin^{-1} 6/8.7 = 43.60^\circ$$

$\therefore \cos\theta = 0.724.$

Now,

$$T_1 = \frac{w \times R}{1 + \cos\theta}$$

$$= \frac{4 \times 8.7}{1 + 0.724} = 20.18 \text{ kN/m}.$$

Meridional Stress $= F/A$

$$= \frac{20.18 \times 10^3}{1000 \times 100} = 0.202 \text{ N/mm}^2$$

→ In IS : 3370 (part - 2), table - 2, For M30 concrete, permissible stress in concrete = 8 N/mm²

→ 0.202 < 8 N/mm² — (Safe)

→ So provide 0.24 % min r/f.

$$\therefore Ast = \frac{0.24}{100} \times 1000 \times 100$$

$$= 240 \text{ mm}^2$$

→ provide 8 ϕ mm bars.

$$S = \frac{Ast(1)}{Ast} \times 1000$$

$$= \frac{\pi/4 \times 8^2}{240} \times 1000$$

$$= 200 \text{ mm c/c.}$$

For Hoop tension (T_2) :-

→ $$T_2 = w \times R \left[\cos\theta - \frac{1}{1+\cos\theta}\right]$$

$$= 4 \times 8.7 \left[0.724 - \frac{1}{1+0.724}\right]$$

$$= 5 \text{ kN/m.}$$

→ Hoop stress $= P/A = \dfrac{5 \times 10^3}{1000 \times 100}$

$= 0.05 < 8$ N/mm²

(Safe).

$\therefore$ provide min r/t (0.24 %)

$\therefore$ Let provide 8 mm ϕ bar

$\therefore A_{st} = \frac{0.24}{100} \times 1000 \times 100$

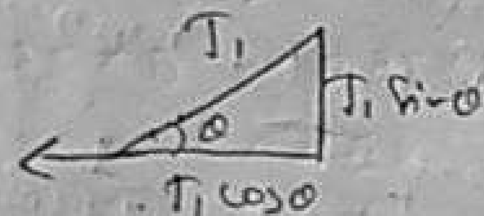
$= 240 \ mm^2$.

$\therefore S = \frac{A_{st}(1)}{A_{st}} \times 1000$

$= \frac{\pi/4 \times 8^2}{240} \times 1000$

$= 200 \ mm \ c/c$.

2) Design of top ring Beam :-

→ 

→ top ring Beam has to be designed to counter the Horizontal component of Meridional tension.

→ $w = T_1 \cos\theta$

$= 20.18 \times 0.224 = 14.61 \ kN/m$

→ Total hoop tension in beam

$= w \times D/2 = 14.61 \times 12/2$

$= 87.66 \text{ kN}$

→ Ast for hoop tension

$= \frac{T}{\sigma_{st}} = \frac{87.66 \times 10^3}{130}$

$= 674.30 \text{ mm}^2$

→ Let provide 12 mm ϕ bars

$$S = \frac{A_{st}(\phi)}{A_{st}} \times 1000$$

$$= \frac{\pi/4 \times 12^2}{674.30} \times 1000$$

≈ 150 mm c/c.

→ Find out Dimension of Ring Beam :-

(IS: 456, page - 80)

$$\sigma_{ct} = \frac{T}{A_g + (m-1)A_{st}}$$

$\Rightarrow 1.5 > \frac{87.66 \times 10^3}{(200 \times D) + (9.33-1) \times 674}$

$\Rightarrow 87.66 \times 10^3 < 375D + 9408.33$

$\Rightarrow D > 208.67$

$\therefore D = 300$ mm

σ_{ct} = permissible tensile stress in concrete (for M30 concrete $\sigma_{ct} < 1.5 \text{ N/mm}^2$)

A_g = gross area = $b \times D$ (Assume $b = 250$ mm)

$m = \frac{280}{3\sigma_{cbc}} = 9.33$ (for M30)

A_{st} = Area of Steel.

T = hoop tension.

∴ Size of Beam = (250 × 300) mm

→ provide min shear reinforcement.
8 mm ϕ - 2 legged vertical stirrups

→ According to IS: 456-2000, page-48

$$S_v \text{ (Spacing)} = \frac{0.87 f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times (\pi/4 \times 8^2) \times 2}{0.4 \times 250}$$

$$= 362.96 \text{ mm}$$

→ Spacing limit ⇒

Min
i) $S_v = 362$ mm
ii) $0.75 \times D = 0.75 \times 300 = 225$ mm
iii) 300 mm

∴ provide 8 mm ϕ - 2 legged vertical stirrup at 225 mm c/c.

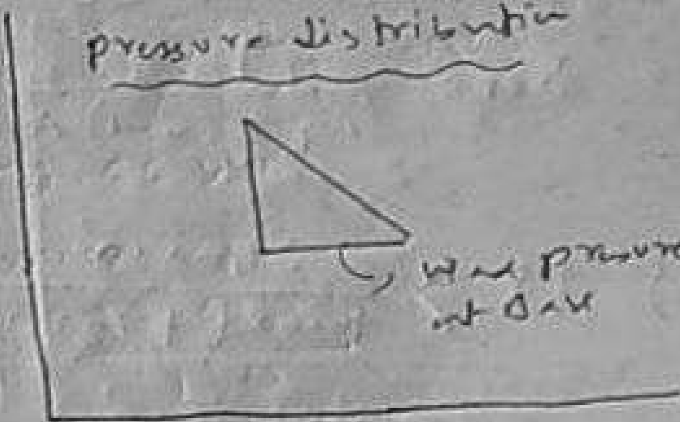
3) Design of tank wall:-

→ Max hoop tension at Base

$$T = \frac{\gamma_w \times H \times D}{2}$$

$$= \frac{10 \times H \times 12}{2}$$

$$= 60\,H \ \text{kN/m}.$$



$$\therefore A_{st} = T/\sigma_{st} = \frac{60 \times H}{130} \times 10^3$$

$$= 461.54\,H \ \text{mm}^2.$$

→

Depth from top (H) in m	Area required Ast (mm^2) (461.54H)	Reinforcement provided
1	461.54	Let provide 8φ mm bars, $S = \frac{A_{st}(1)}{A_{st}} \times 1000$ $= \frac{\pi/4 \times 8^2}{461.54} \times 1000 \approx 100$ mm c/c.
2	923	10φ @ 100 mm c/c.
3	1384.62	10φ @ 75 mm c/c.
4	1846.16	12φ @ 75 mm c/c.

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⇒ Thickness of wall :→

$\sigma_{ct} = \dfrac{T}{A_g + (m-1) A_{st}}$

$T = 60 \times H$

$= 60 \times 4 = 240$ kN.

$\Rightarrow 1.5 > \dfrac{240 \times 10^3}{1000 \times t + (8.33) \times 1846}$

$\sigma_{ct} \leq 1.5$

$A_g = 1000 \times t$

$(m-1) = 9.33 - 1$

$= 8.33$

$A_{st} = \text{max } A_{st}$

$= 1846.16$

$\Rightarrow t > 144.30$

∴ provide $t = 200$ mm (at base)

$t = 150$ mm (at top)

∴ Avg thickness of wall $= \dfrac{200 + 150}{2}$

$= 175$ mm.

4) Design of Base slab :→

⇒ load from Dome $= T_1 \sin\theta \times 2\pi \times D/2$

$= 20.18 \sin 43.60° \times 2\pi \times 12/2$

$= 524.64$ kN.

⇒ Load from top Ring Beam

$= (b \times D) \times \pi \times D \times 25$

$= (0.25 \times 0.30) \times \pi \times 12 \times 25$

$= 70.68$ kN.

⇒ Load of wall

$= 0.175 \times (4-0.3) \times \pi \times 12.175 \times 25$

$= 619.15$ kN.

∴ Total load $= (524.64 + 70.68 + 619.15)$

$= 1214.47$ kN.

2 Tb. → this load act on the edge of slab.

→ And the water force act as a UDL in middle of the slab.

⇒ Weight of water

$= (\pi/4 \times D^2 \times H) \times \gamma_w$

$= (\pi/4 \times 12^2 \times 4) \times 10$

$= 4523.90$ kN.

→ Self weight of slab

$= t \times 25$

~~= 350 × 25~~

$= 0.35 \times 25$

$= 8.75$ kN/m².

Assume

$t = D/35$

$= 12/35 = 0.342$

≈ 350 mm

→ Total weight of slab $= (\pi/4 \times D^2) \times S.W$

$= \pi/4 \times (12.4)^2 \times 8.75$

$= 1056.67$ kN.

↓ dia $(D) = 12 + 2 \times 0.2$

$= 12.4$

⇒ Finished load $= (\pi/4 \times 12^2) \times 0.6$

$= 67.85$ kN

assumed finishing load intensity $= 0.6$ kN/m²

∴ Load on slab $= 5648.42$ kN.

∴ Total downward force on bottom ring Beam

$=$ ~~total~~ $=$ Load on edge + Load on centre

$= 1214.47 + 5648.42$

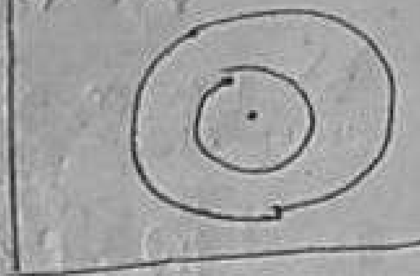
$= 6863$ kN.

(*) Case-1 → circular slab simply supported and subjected to water pressure plus self weight of slab.

Case-2 :- circular slab simply supported and subjected to upward ring load.

(*) Case-1 :- water pressure + S.W of slab :-

i) M_r = radial moment
$= \frac{3}{16} \times q \, (a^2 - r^2)$



ii) M_θ = circumferential moment
$= \frac{3}{16} \times q \times a^2 - \frac{q \times r^2}{16}$

where :- q = load/m²
a = radius of slab.
r = distance from centre.

Here, $q = \frac{5648.42}{\pi/4 \times 12.4^2} = 46.77$ kN/m².

$a = 12.4/2 = 6.2$ m.

Now,

r(m)	0	2.25	4.5	6.2
Mr(kN.m)	337.09	292.68	159.51	0
Mθ(kN.m)	337.09	322.29	277.90	244.72

(8) For case-II :-)

b = radius of beam = $\frac{0.75 \times \text{Dia of tank}}{2}$

$= \frac{0.75 \times 12}{2}$

$= 4.5\ m$

a = radius of slab = $\frac{\text{Dia of wall}}{2}$

$= \frac{12.4}{2} = 6.2\ m$

W = 6863 kN

Now if $r \le b$:-)

$$M_r = M_\theta = \frac{W}{8\pi}\left[2\log\frac{a}{b} + 1 - \left(\frac{b}{a}\right)^2\right]$$

At $r > b$

$$M_r = \frac{W}{8\pi}\left[2\log\frac{a}{r} - \left(\frac{b}{a}\right)^2 + \left(\frac{b}{r}\right)^2\right]$$

$$M_\theta = \frac{W}{8\pi}\left[2\log\frac{a}{r} - \left(\frac{b}{r}\right)^2 + 2 - \left(\frac{b}{a}\right)^2\right]$$

Now.

r(m)	0	2.25	4.5	6.2
M_r (kN·m)	-205.23	-205.23	-205.23	0
M_θ (kN·m)	-205.23	-205.23	-205.23	-258.44

∴ Net moment or design moment :-

↳ Case-1 - Case-2

r	0	2.25	4.5	6.2
M_r	131.86	83.65	-45.72	0
M_θ	131.86	117.06	72.67	-13.72

∴ Design moment = Max moment

$= \text{at } r = 0$

$= 131.86 \text{ kN-m}$

→ $\boxed{M = Q \cdot b d^2}$

Now, $Q = \frac{1}{2} \times \sigma_{cbc} \times k \times J$

$= \frac{1}{2} \times 10 \times 0.418 \times 0.861$

$= 1.80$

$131.86 \times 10^6 = 1.80 \times 1000 \times d^2$

$\Rightarrow \boxed{d = 270.66} \text{ mm}$

∴ provide $d = 300 \text{ mm}$

∴ overall depth $(D) = 300 + 50$

$= 350 \text{ mm}$

∴ $A_{st} = \frac{M}{\sigma_{st} \times J \times d}$

$= \frac{131.86 \times 10^6}{130 \times 0.861 \times 300} = 3926.86 \text{ mm}^2$

∴ provide 25 mm ϕ bar,

$$\therefore S = \frac{Ast(1)}{Ast} \times 1000$$

$$= \frac{\pi/4 \times 25^2}{3926.86} \times 1000$$

$$\approx 110 \text{ mm } c/c.$$

5) Design of Bottom Ring beam :-

→ Design with respect to
- → Shear force.
- → Bending moment.
- → Torsional moment.

→ Dimension of beam

i) width of beam = 350 mm (Assumed)

ii) Depth of beam = $\frac{\text{Dia of beam}}{15}$

$$\approx \frac{9}{15} = 0.6 \text{ m}$$

$$= 600 \text{ mm}.$$

→ Shear force :-

$$\boxed{F_\phi = w \times R (\theta - \phi)}$$

where, w = load per m length

$$= \frac{6863}{\pi \times d} \text{ [for circular tank load]}$$

$$= \frac{6863}{\pi \times 9} = 242.72 \text{ kN/m}.$$

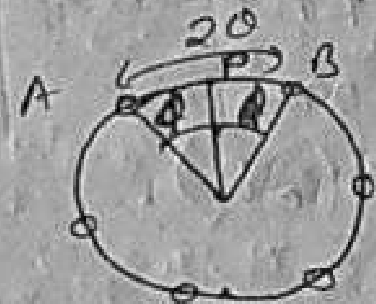
Now, Self weight of beam $= (0.35 \times 0.60) \times 25$

$= 5.26 \simeq 6$ kN/m.

$\therefore$ Total $w = 242.72 + 6$

$= 249$ kN/m.

b) $2\theta =$ Angle b/w two adjacent support or column.



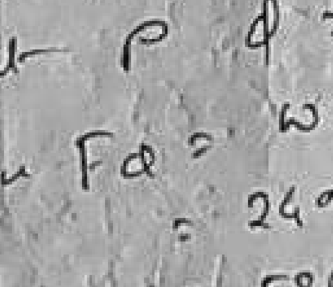
Now $2\theta = \dfrac{360^\circ}{n}$ [n = no of column = 6]

$\Rightarrow \theta = 30^\circ = \pi/6$.

c) R = Radius of beam

$= 9/2 = 4.5$ m.

d) $\phi =$ Angle measured from A to P.



Now at A, $\phi = 0^\circ$ (Support).

& at P $\phi = 30^\circ = \theta$

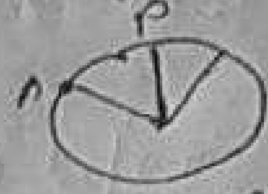
$\therefore$ Max shear force $F_\phi = w \times R(\theta - \phi)$

$= 249 \times 4.5 (\pi/6 - 0)$

$= 586.70$ kN.

→ Bending moment :-

- → at Support
- → at mid Span



A = Support.
P = Mid-Span.

a) Bending moment at Support :-

$$M_0 = wR^2(\theta \cdot \sin\phi + \theta \cdot \cot\theta \cdot \cos\theta - 1)$$

or

$$M_0 = C_1 \times wR^2 (2\theta).$$

No of Supports	2θ	C_1	C_2	C_3	ϕ_m
4	90°	0.137	0.070	0.021	19.25°
5	72°	0.108	0.054	0.014	15.25°
6	60°	0.089	0.045	0.009	12.75°
8	45°	0.066	0.030	0.005	9.33°
10	36°	0.054	0.023	0.003	7.50°
12	30°	0.045	0.017	0.002	6.25°

$$M_0 = C_1 \times wR^2 (2\theta)$$

$$= 0.089 \times 249 \times 4.5^2 \times 60°$$

$$= 469.94 \text{ kN-m}.$$

⇒ b) B.M at Mid Span :-

$$M_0 = C_2 \times wR^2 (2\theta)$$

$$= 0.045 \times 249 \times 4.5^2 \times 60$$

$$= 237.61 \text{ kN-m}.$$

c) Torsional moment :-

$T_d = C_3 \times wR^2 (2\theta)$

$= 0.009 \times 249 \times 4.5^2 \times 60$

$= 47.52$ kN-m

⇒ Reinforcement at Support :-

Design the beam as doubly R.C. beam.

$M_u = 1.5 \times 469.43 = 704.15$ kN-m (Hogging)

$V_u = 1.5 \times 586.70 = 880$ kN

$T_u = 0$.

→ beam size = 350 × 600 mm.

Let eff. cover = 50 mm (d')

∴ $d = 600 - 50 = 550$ mm

→ $d'/d = 50/550 = 0.09 \approx 0.10$

$$\frac{M_u}{bd^2} = \frac{704.85 \times 10^6}{350 \times 550^2} = 6.65$$

from (SP 16, pg - 88, Table - 52) -

find p_t, p_c

∴ $p_t = 2.205\%$, $p_c = 0.822\%$.

→ $A_{st} = \frac{2.205}{100} \times 350 \times 550 = 4244.62$ mm²

provide 28 φ mm bar,

$S = 140$ mm c/c.

$\rightarrow$ $Asc = \frac{0.822}{100} \times 350 \times 550$

$\Rightarrow 1582.35 \, mm^2$

Let, provide, 20ϕ mm bar.

$\therefore S = 160$ mm c/c.

$\rightarrow$ R/f at mid span :-)

$\rightarrow$ have to find singly or Doubly.

$\rightarrow$ If $M_u < M_{u lim} \rightarrow$ Singly R.C

$M_u > M_{u lim} \rightarrow$ Doubly R.C

$\rightarrow$ $M_u = 1.5 \times 237.94$

$= 356.91$ kN-m (Sagging)

$M_{u lim} = 0.138 \, f_{ck} b d^2$

$= 0.138 \times 30 \times 350 \times 550^2$

$= 438.32$ kN.m

$\therefore$ $M_u < M_{u lim} \rightarrow$ Design as Singly R.C. beam

$\rightarrow$ $\frac{M_u}{bd^2} = \frac{356.91 \times 10^6}{350 \times 550^2} = 3.37$.

Now from SP-16, P-50, for M30

$P_t = 1.105 \%$.

$\therefore Ast = \frac{1.105}{100} \times 350 \times 550 = 2127.12 \, mm^2$

$\therefore$ provide 25ϕ bar, $S = 200$ mm c/c.

Check for torsion :-

$M_e = m + m_t$

$= m + T_u \frac{(1 + D/b)}{1.7}$

[$\phi_{max} = 12.75°$

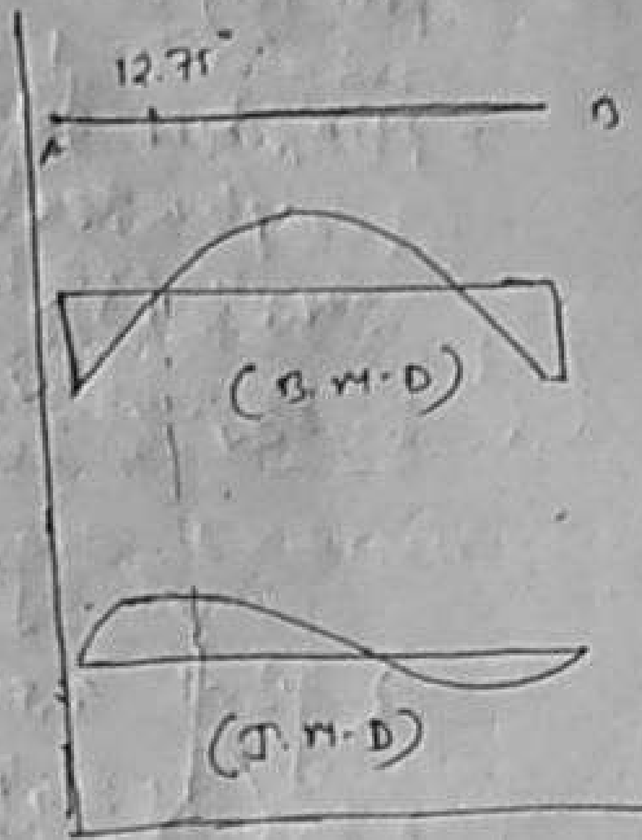
$= 0.2225$ radians.

$\therefore$ distance from support

$= R \times \phi$

$= 4.5 \times 0.2225$

$= 1\,m$]



Now, $T_u = 1.5 \times 48.40 = 72.6$ kN-m.

$\therefore$ ~~$M_e = 0 + 72$~~

$\therefore M_e = m + m_t$

$= 0 + T_u \frac{(1 + D/b)}{1.7}$

$= 0 + 72.6 \frac{(1 + 600/500)}{1.7}$

$= 115.92$ kN-m.

$\therefore$ Fact. $M_e = 1.5 \times 115.92 = 173.88$ kN-m.

Now, $173.88 < 356.91$ (BM at mid span)

So, provided bars of 25ϕ are enough.

(4) Shear r/f :-

→ Max shear force at support

$V = 586.70$ kN.

$\therefore V_u = 1.5 \times 586.70 = 880$ kN.

$$\therefore \tau_v = \frac{V_u}{bd} = \frac{880 \times 10^3}{350 \times 550}$$

$$= 4.57 \text{ N/mm}^2.$$

→ But $4.57 > 3.5$ N/mm²

(IS : 456, p. 72, Table - 20).

So, Increase the section as 400 mm × 800 mm.

$$\therefore \tau_v = \frac{880 \times 10^3}{400 \times 750} = 2.93 \text{ N/mm}^2.$$

→ Now, pt at support = 2.205%.

from IS : 456 table - 19.

$\tau_c = 0.87$ N/mm²

$$\therefore V_{us} = V_u - \tau_c \times bd$$

$$= 880 \times 10^3 - 0.87 \times 400 \times 750$$

$$= 619000 \text{ N}.$$

$\therefore$ provide 10 ϕ Lm - 4 legged stirrup.

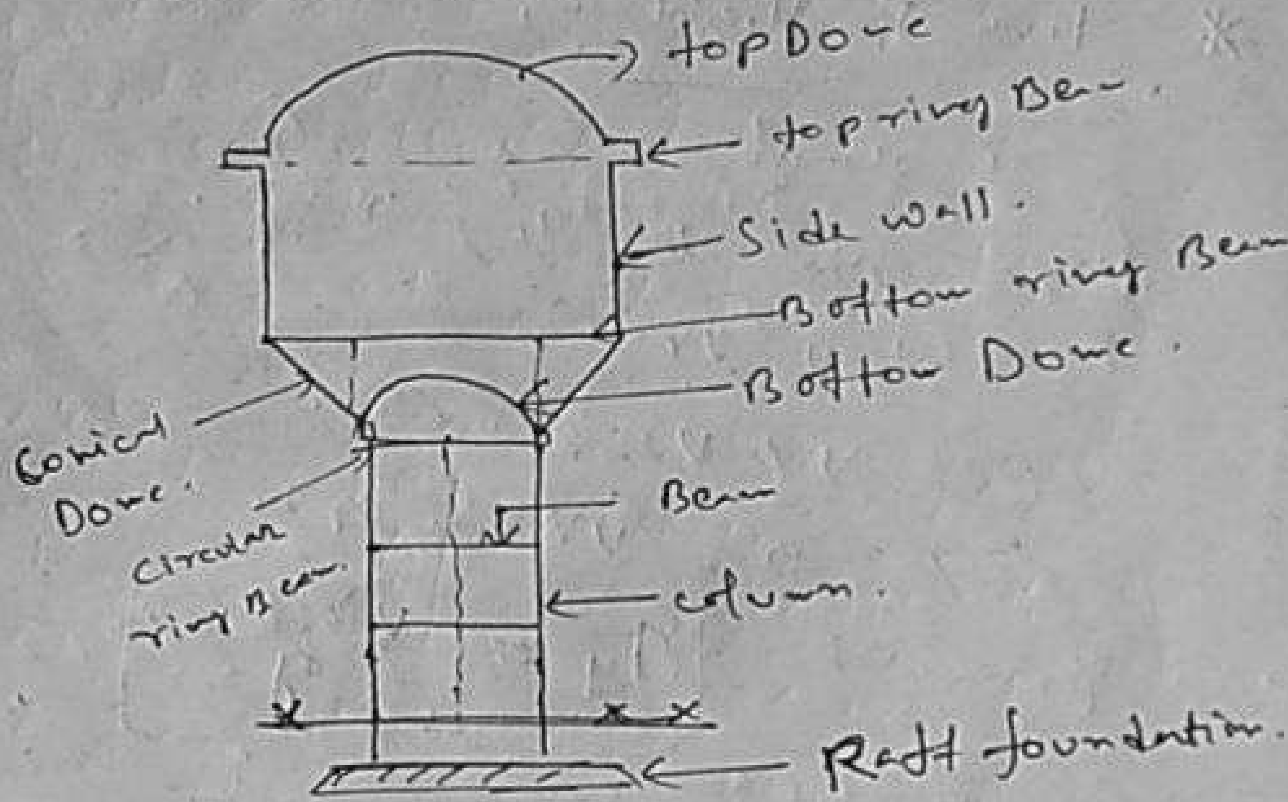
$$\therefore A_{sv} = \frac{\pi/4 \times 10^2 \times 4}{}$$

$$= 314 \text{ mm}^2.$$

✲ Design of Intze water tank :->

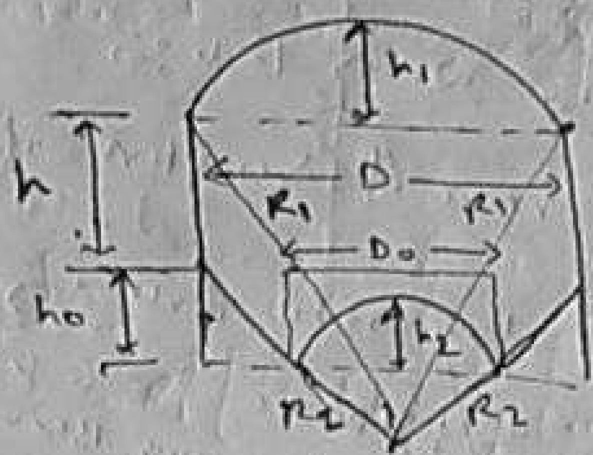
↳ By the name of Scientist "otto Intze".

✲ Component of intze water tank :->



Q. Fix the basic dimensions of elevated intze water tank to store 6 lakhs litre water. Design and detail all structural components. Take live load = 1.5 kN/m². Use M 30 grade concrete and Fe 415 steel.

(,) Dimensions :-



(1) Dimensions :-

Let Dia of tank (D) = 12 m.

Dia of Bottom R. B. $(D_0) = 0.6 \times 12$

$= 7.2$ m

≈ 8 m.

Rise of top Dome $(h_1) = 0.2 \times 12$

$= 2.4$

≈ 2 m.

Rise of Bottom Dome $(h_2) = 1.5$ m.

height of conical Dome $(h_0) = 2$ m.

(1.5 to 2.5 m)

height of cylindrical portion (h)

= ?

Now, capacity of tank :-)

= vol of top + vol of conical Dome − vol of Bottom Dome.

$= (\pi/4 \times D^2 \times h) + \pi/12 \times h_0 (D^2 + D_0^2 + D \cdot D_0)$

$- \pi/3 \times h_2^2 (3R_2 - h_2)$.

$= (\pi/4 \times 12^2 \times h) + \pi/12 \times 2 (12^2 + 8^2 + 12 \cdot 8)$

$- \pi/3 \times 1.5^2 (3 \times 6.08 - 1.5)$

$= (113.09 \times h) + (159.17) - (39.44)$

$\therefore 600 = (113.09h) + (159.17) - (39.44)$

$\Rightarrow h = 4.24m$

$\approx 5m$ (say)

R_2 = Radius of curvature of Bottom Dome.

$R_2 = \dfrac{(D_0/2)^2 + h_2^2}{2 \times h_2}$

$= \dfrac{(8/2)^2 + 1.5^2}{2 \times 1.5}$

$= 6.08m$.

(2) Design of top Dome :-)

- Meridional force
- Hoop force

1) Meridional force (T_1) :-)

$T_1 = \dfrac{wR}{1 + \cos\theta_1}$

$R_1 = \dfrac{(D/2)^2 + h^2}{2 \times h_1}$

$= \dfrac{(12/2)^2 + 2^2}{2 \times 2} = 10m$.

w = L.L + S.W of Dome.

= 1.5 + (thickness × unit wt)

= 1.5 + (0.1 × 25)

= 1.5 + 2.5 = 4 kN/m².

$\Rightarrow \sin\theta_1 = \frac{D/2}{R} = 6/10 = 0.6$

$\theta_1 = 36.86°$.

$\therefore \cos\theta_1 = 0.80$.

$\Rightarrow T_1 = \frac{wR}{1+\cos\theta_1} = \frac{4 \times 10}{1+0.80} = 22.22$ kN/m.

$\therefore$ Meridional stress $= F/A = \frac{22.22 \times 10^3}{1000 \times 100}$

$= 0.22$ N/mm² < 8 (OK).

ii) Hoop force →

$T_2 = wR_1 \left[\cos\theta_1 - \frac{1}{1+\cos\theta_1}\right]$

$= 4 \times 10 \left[0.80 - \frac{1}{1+0.80}\right]$

$= 9.78$ kN/m.

$\therefore$ Hoop stress $= \frac{9.78 \times 10^3}{1000 \times 100} = 0.0978$ N/mm² < 8 (OK).

provide min r/f (0.24%)

$\therefore A_{st} = \frac{0.24}{100} \times 1000 \times 100 = 240$ mm².

$\therefore$ Let provide 8 mm bar.

S = 200 mm c/c.

$\boxed{\text{For both } T_1 \text{ \& } T_2}$

(3) Design of top ring Beam:-

→ Have to design due to horizontal component of meridional force (T_1)

$\therefore W = T_1 \cos\theta_1 = 22.22 \times \cos\theta_1 = 17.77$ kN/m.

→ Total Hoop tension in Beam.

$$= \frac{W \times D}{2} = \frac{17.77 \times 12}{2} = 106.62 \text{ kN}.$$

$\therefore$ Ast for hoop tension $= \frac{T}{\sigma_{st}}$

$$= \frac{106.62 \times 10^3}{130}$$

$$= 820 \text{ mm}^2.$$

$\therefore$ Let provide 12 ϕ mm bar

$\therefore$ S = 130 mm c/c

(b) Dimensions:-

$$\sigma_{ct} = \frac{T}{A_g + (m-1) A_{st}}$$ [IS: 456, pg-80]

$$\Rightarrow 1.5 \geq \frac{106.62 \times 10^3}{bD + (9.33-1) \times 869}$$

$$\Rightarrow 1.5 \geq \frac{106.62 \times 10^3}{300 \times D + (9.33-1) \times 869}$$ [Let b = 300 mm]

$\Rightarrow D \geq 212.80$

provide D = 300 mm.

Size of Beam = 300 × 300 mm

⇒ Stirrups :-

provide 8φ - 2 legged stirrups.

$$\therefore S_v = \frac{0.87 \times f_y \times A_{sv}}{0.4 \times b}$$

$$= \frac{0.87 \times 415 \times \pi/4 \times 8^2 \times 2}{0.4 \times 300}$$

$$= 300 \text{ mm}$$

$$\therefore \text{Spacing} = \text{Min} \begin{cases} \text{i) } 0.75\,D = 0.75 \times 300 = 225 \\ \text{ii) } 300 \text{ mm} \\ \text{iii) } S_v = 300 \text{ mm} \end{cases}$$

∴ provide 8φ - 2 legged vertical stirrups @ 225 c/c.

4) Design of side walls :-

$$T = \frac{\gamma_w \times h \times D}{2} = \frac{10 \times h \times 12}{2} = 60\,h \text{ kN/m}$$

$$\therefore A_{st} = T/130 = \frac{60h}{130} \times 10^3$$

Depth from top	Ast (mm²)	R/f provided
1	461.54	8φ @ 210 c/c
3	1384.62	10φ @ 110 c/c
5	2322.70	16φ @ 170 c/c

⇒ Thickness of Wall :-

$$\sigma_{ct} = \frac{T}{A_g + (m-1)A_{st}}$$

$T = 60h$
$= 60 \times 5 = 300\ kN.$
$A_g = 1000 \times t$

$$\Rightarrow 1.5 \geq \frac{300 \times 10^3}{1000 \times t + (9.33-1) \times 2322.20}$$

$\Rightarrow t \geq 177.92$

provide $t = 250$ mm. at base &
200 mm at top.

$$\text{Avg } t = \frac{250+200}{2} = 225 \text{ mm.}$$

5) Design of Bottom ring Beam :-

→ Load due to top dome = $T_1 \sin\theta$
$= 22.22 \times 0.6$
$= 13.33\ kN/m.$

→ Load due to top R.B = $b \times D \times \gamma$
$= 0.3 \times 0.3 \times 25$
$= 2.25\ kN/m.$

→ Load due to wall = (thickness × height × γ)

$h = 5 - 0.3 - 0.5 = 4.2$ m.

[Assume size of Bottom Beam = 1000 × 500 mm]

$= 0.225 \times 4.2 \times 25 = 23.625\ kN/m.$

→ Load due to self weight
$= 1 \times 0.5 \times 25 = 12.5\ kN/m.$

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∴ Total load on Beam $(w_1) = 51.70$ kN/m.

Now Horizontal component of w_1

$H_1 = w_1 \times \tan\beta = 51.70 \times \tan 45°$ [$\beta = 45°$]

$= 51.70$ kN/m.

water pressure, $H_2 = \gamma_w \times h \times d$

$= 10 \times (5 - 0.35) \times 0.5$

$= 23.25$ kN/m.

[Here, h = height of water

d = depth of Beam]

$\therefore H = H_1 + H_2 = 75.45$ kN/m.

∴ Hoop tension $= \dfrac{H \times D}{2}$

$= \dfrac{75.45 \times 12}{2}$

$= 452.7$ kN.

→ $A_{st} = T/\sigma_{st} = \dfrac{452.7 \times 10^3}{130}$

$= 3482.30$ mm².

∴ provide 20 φ mm bar.

$S = 200$ mm c/c.

→ Checking of Dimension :-

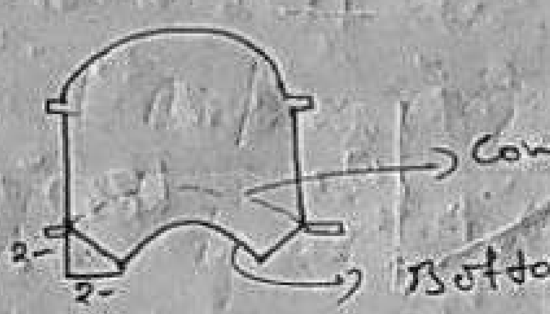
$$\sigma_{ct} = \frac{T}{A_g + (m-1) A_{st}}$$

$$= \frac{452.7 \times 10^3}{(1200 \times 500) + (9.33-1)(3482.39)}$$

$= 0.85\ N/mm^2 < 1.5\ N/mm^2$

(Safe).

6) Design of conical Dome :→



→ Avg dia of conical dome $= \frac{12+8}{2} = 10m$.

→ Avg depth of water $= \frac{5+7}{2} = 6m$.

→ weight of water $= (\pi \times D \times 6 \times \text{width}) \times 10$

$= (\pi \times 10 \times 6 \times 2) \times 10$

$= 3770$ KN.

→ width of slab $= \sqrt{2^2 + 2^2}$

$= 2.83m$

→ weight of slab $= (\pi \times D) \times \text{width} \times \text{thickness} \times 25$

$= (\pi \times 10) \times 2.83 \times .50 \times 25$

$= 1112$ KN.

Assume $t = 500mm$

$\Rightarrow$ weight from top (Dome, wall, beam)

$$= (\pi \times D) \times w_1 = (\pi \times 12) \times 51.70$$
$$= 1949 \text{ kN}$$

$\therefore$ Total load $= (3370 + 1112 + 1949) = 6831$ kN.

$\therefore$ Load per m (w_2) at the base of slab

$$w_2 = \frac{6831}{\pi \times \text{bottom dia}} = \frac{6831}{\pi \times 8}$$
$$= 272 \text{ kN/m}.$$

$\Rightarrow$ Meridional force (T_3)

$$\therefore \cos\beta = \frac{w_2}{T_3}$$

$$\Rightarrow T_3 = \frac{w_2}{\cos\beta} = \frac{272}{\cos 45^\circ} = 385 \text{ kN}.$$

$\therefore$ Meridional stress $= F/A = \dfrac{385 \times 10^3}{1000 \times 500}$

$$= 0.77 \text{ N/mm}^2 < 8 \text{ N/mm}^2 \text{ (ok)}.$$

$\Rightarrow$ Hoop tension :-

$$H_T = (P \operatorname{cosec}\theta + q \cot\theta) \times D/2.$$

P = water pressure $= \gamma_w \times h$

q = S.W. of slab $= 1 \times 0.5 \times 25 = 12.5$ kN/m².

$\theta = 45^\circ$.

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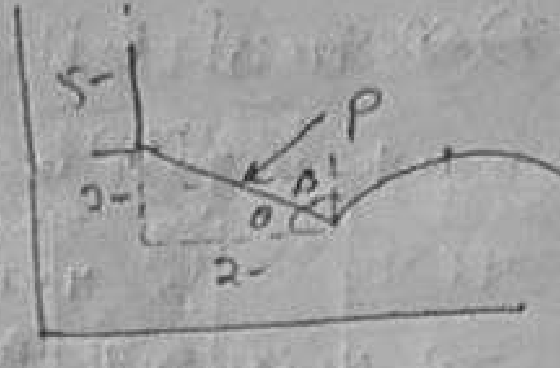
⇒ H_T at top of conical Dome



$H_T = (p\ cosec\theta + q \cot\theta) \times D/2$

$= (\gamma_w \times h\ cosec\theta + 12.5 \cot\theta) \times D/2$

$= (10 \times 5\ cosec 45^\circ + 12.5 \cot 45^\circ) \times 12/2$

$= 499.26$ kN.

⇒ At centre, ($h = 5 + 2/2 = 6$ m, & $D = 10$ m)

$\therefore H_T = (10 \times 6\ cosec 45^\circ + 12.5 \cot 45^\circ) \times 10/2$

$= 486.76$ kN.

⇒ At bottom ($h = 5 + 2 = 7$ m, $D = 8$ m)

$\therefore H_T = [(10 \times 7)\ cosec 45^\circ + 12.5 \cot 45^\circ] \times 8/2$

$= 446$ kN.

∴ Max hoop tension is in at top $= 499.26$ kN.

$\therefore A_{st} = \frac{T}{\sigma_{st}} = \frac{499.26 \times 10^3}{130} = 3840.46$ mm².

∴ Let provide 16 Φ mm bar, ∴ $S = 100$ mm c/c.

⇒ Distribution steel :- (min 0.24%)

$\therefore A_{st} = \frac{0.24}{100} \times 1000 \times 500 = 1200$ mm².

∴ provide 10 Φ bar, ∴ $S = 120$ mm c/c.

⇒ Diversion check :-

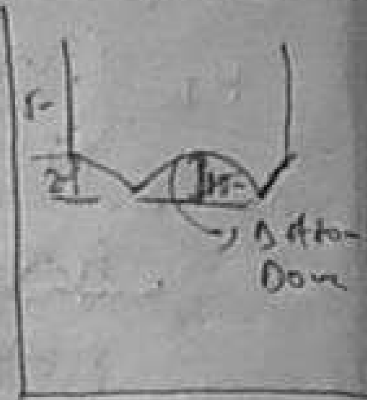
$\sigma_{ct} = \frac{T}{A_g + (m-1)A_{st}} = \frac{499.26 \times 10^3}{(1000 \times 500) + (8.33) \times 1200}$

$= 0.978$ N/mm² < 1.5 N/mm²

(ok).

7) Design of Bottom Spherical Dome :-

Note :- there is no hoop tension So we have to design only for Meridional tension.



⇒ Meridional force $(T_4) = \dfrac{w \times R}{1+\cos\theta}$

wt of water = width × ht × γ_w

$= 1 \times (5+2-1.5) \times 10$

$= 55 \text{ kN/m}^2$

→ S.W $= (1 \times 0.25 \times 25)$ [Assume t = 250mm]

$= 6.25 \text{ kN/m}^2$

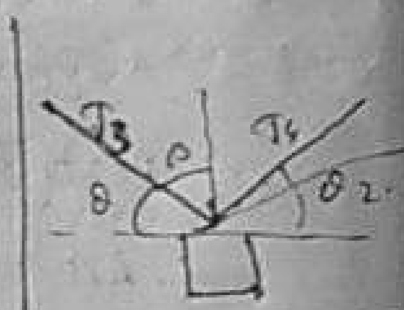
∴ Total load (w) = 55 + 6.25 = 61.25 kN/m².

→ R_2 = Radius of curvature

$= \dfrac{(D_o/2)^2 + h_2^2}{2 \times h_2} = \dfrac{(8/2)^2 + 1.5^2}{2 \times 1.5} = 6.08 \text{ m}$.

→ $\sin\theta_2 = \dfrac{D_o/2}{R_2} = 4/6.08 = 0.657$.

∴ $\theta_2 = 41.07°$



∴ $T_4 = \dfrac{w \times R}{1+\cos\theta} = \dfrac{61.25 \times 6.08}{1+\cos 41.07°}$

$= 212.30 \text{ kN/m}$.

∴ Meridional stress $= \dfrac{212.30 \times 10^3}{1000 \times 250} = 0.849 < 8 \text{ N/mm}^2$ (ok)

∴ provide $P_{t\,min} = 0.24\%$.

∴ $A_{st} = \dfrac{0.24}{100} \times 1000 \times 250 = 600 \text{ mm}^2$.

∴ Let provide 10mm ∴ S = 130 mm c/c.

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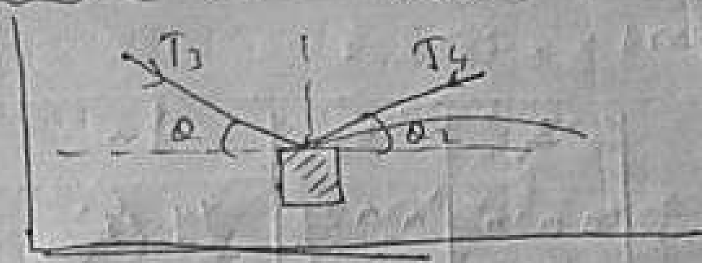
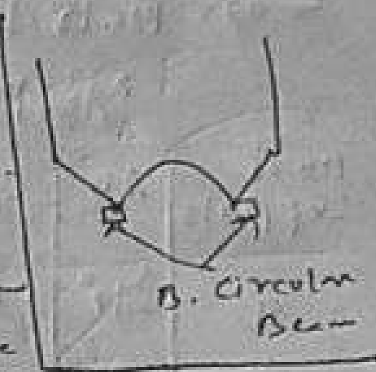
8) Design of Bottom Circular Beam:-





→ Inward thrust from Conical Dome

$= T_3 \cos\theta = 385 \cos 45° = 272.24$ kN/m.

→ Outward thrust from Spherical Dome.

$= T_4 \cos\theta_2 = 210.42 \cos 41.09° = 159$ kN/m.

∴ Net inward thrust $= (272.24 - 159)$

$= 113.24$ kN/m.

→ Hoop compression on beam $= \dfrac{w \times D}{2}$

$= \dfrac{113.24 \times 8}{2} = 452.96$ kN.

→ Hoop stress $= P/A$

$= \dfrac{452.96 \times 10^3}{500 \times 1000}$ | Assume size of beam $= 500 \times 1000$ mm

$= 0.905$ N/mm² < 8 N/mm². (Safe).

→ Vertical load on beam,

$= T_3 \sin\theta + T_4 \sin\theta_2$

$= 385 \sin 45° + 210.92 \sin 41.09°$

$= 410.80$ kN/m.

→ S.W. of beam $= 0.5 \times 1 \times 25 = 12.5$ kN/m.

∴ Total vertical load $= 410.80 + 12.5$

$= 423.30$ kN/m.

∴ Total load $= (\pi \times D) \times 423.30$

$= \pi \times 8 \times 423.30 = 10639$ kN.

→ Now we provide '8' no of column as a support.

No of supports	2θ	C_1	C_2	C_3	ϕ_m
4	90°	0.137	0.070	0.021	19.25°
5	72°	0.108	0.054	0.014	15.25°
6	60°	0.089	0.054	0.009	12.75°
8	45°	0.066	0.030	0.005	9.33°
10	36°	0.054	0.027	0.003	7.50°
12	30°	0.045	0.017	0.002	6.25°

→ Max −ve B.M at support

$= C_1 w R^2 (2\theta)$

$= 0.066 \times 423.3 \times 4^2 \times (\pi/4)$ [R = Radius of beam]

$= 351.08$ kNm

→ Max +ve B.M at mid span.

$= C_2 w R^2 (2\theta)$

$= 0.030 \times 423.30 \times 4^2 \times (\pi/4)$.

$= 159.58$ kNm.

→ Max torsional moment

$= C_3 w R^2 (2\theta)$

$= 0.005 \times 423.30 \times 4^2 \times \pi/4$

$= 26.6$ kN.m

$\rightarrow$ Max S.F at support $= \dfrac{W\ (\text{total load})}{2 \times \text{no of column}}$

$$= \frac{10639}{2 \times 8} = 665\ kN.$$

$\Rightarrow$ <u>NOW</u> $M = Q \cdot bd^2$

$$351.08 \times 10^6 = 1.80 \times 500 \times d^2$$

$\Rightarrow$ $d = 624.57\ mm.$

$\therefore$ provide $d = 950\ mm$, cover $= 50\ mm$, $D = 1000\ mm$.
(# what we assumed (500×1000) is correct)

$\Rightarrow$ $Ast = \dfrac{M}{\sigma_{st} \cdot J \cdot d} = \dfrac{351.08 \times 10^6}{130 \times 0.861 \times 950}$

$= 3302\ mm^2$.

$\therefore$ provide 28ϕ, $S = ?$

$\Rightarrow$ Shear check :-

$\rightarrow$ τ_v (nominal shear stress) $= \dfrac{V}{bd}$

$$= \frac{665 \times 10^3}{500 \times 950} = 1.4\ N/mm^2$$

$\Rightarrow$ $P(t) = \dfrac{100 Ast}{bd} = \dfrac{100 \times 3302}{500 \times 1000} = 0.75.$

$\therefore$ $\tau_c = 0.38\ N/mm^2$ [According to IS:456, pg-84 with respect to P(t)]

$\therefore$ $\tau_v > \tau_c$, $\therefore$ it is to be designed for shear.

$\rightarrow$ Shear taken by concrete $= 0.38 \times 500 \times 950$
$= 180.5\ kN.$

$\therefore$ Balance shear / Net shear $= 665 - 180.5$
$= 484.5\ kN.$

→ Use 12Φ - 4 legged vertical stirrups.

$A_{sv} = \pi/4 \times 12^2 \times 4$

$= 452 \text{ mm}^2$

$S_v = \dfrac{A_{sv} \times \sigma_{sv} \times d}{V_s}$ [σ_{sv} = permissible ~~shear~~ tensile stress in shear r/f = not more than 230 (pg-85, IS:456)]

$= \dfrac{452 \times 175 \times 950}{484.5 \times 10^3}$

$= 155 \text{ mm}$

$\approx 150 \text{ mm c/c}$

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Mod-3 → Design of bunker & silos.

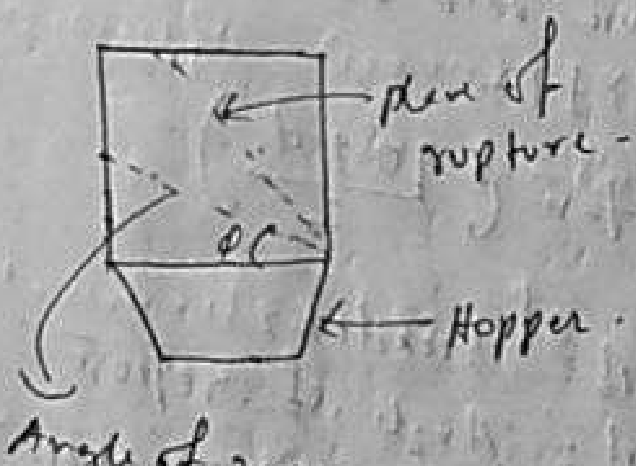
Design of Bunkers & Silos :-

→ Bunkers & Silos are those structures used for storage of materials like, grain, coal, cement etc.

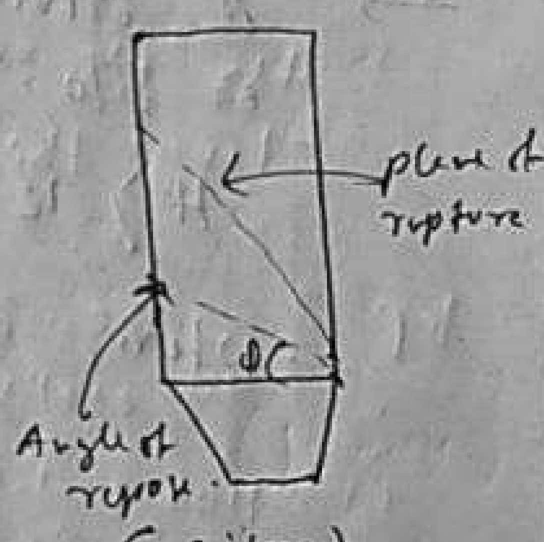
→ Both Bunkers & Silos are commonly called as bin.

① Bunkers :-

If depth & breadth of the bin are such that if the plane of rupture meets the surface of material before it strikes the opposite side of the bin then it is called "shallow bin" or bunkers.



(Bunkers)



(Silos)

* Angle of repose :- certain angle upto this material does not slide.

② Silos :-

If the plane of rupture does not intersect the surface of material & touches the side of bin then it called "deep bin" or silos.

→ Generally a bin is said to be silo if the depth is greater than <u>twice</u> the breadth.

→ Hooper are rectangular bin with bottom floor consisting of four sloping sides.

→ In bunkers side walls resist lateral pressure and the total load of material is supported by the floor of bunker.

→ the intensity of lateral pressure on the sides is determined by Rankine's theory.

→ A structure will be classified as silo, if –

$$H > B \tan\left(\frac{90^\circ + \phi}{2}\right).$$

where, B = Breadth.
ϕ = Angle of repose.
H = height of structure.

* Design guidelines for circular Bunkers :-

Required data :-

i) weight of material to be stored in bunker.
ii) Density of material.
iii) Angle of repose.
iv) Grade of material - steel & concrete.

Step-1 Dimension of bunker

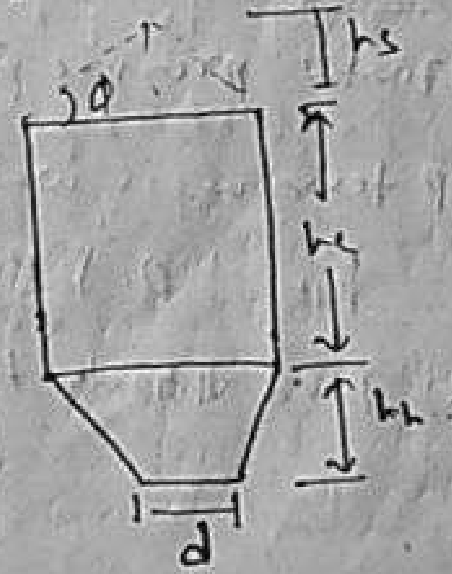
(i) Vol. of bunker = w/γ

where, w = weight of material.

γ = Unit wt of material.

* Assume the Dimensions initially :-

(i) Bunker Diameter (D)
(ii) Height of cylindrical part (h_c)
(iii) Depth of hopper bottom (h_h)
(iv) Dia of hopper (d)



* hs → height of surcharge

Step-2 :- check capacity of bunker :-

(i) vol. of Surcharge $= \frac{1}{3}\left(\frac{\pi D^2}{4} \times h_s\right)$

(ii) vol. of cylindrical portion
$= \pi/4 \, (D^2) \times h_c$.

(iii) vol. of frustrum of cone -

$$= \frac{\pi h_h}{12}\left[D^2 + D.d + d^2\right]$$

$\therefore$ Total vol $=$ (i) + (ii) + (iii)

* Now if total vol $\geq W/\gamma$,
then we can proceed, otherwise
change the dimension.

Step 3 Design of cylindrical walls :-

(i) Horizontal pressure $(p) = \gamma h_c \cos\theta$

(ii) Hoop tension in wall per metre height

$F_H = 0.5 PD$.

→ Find ultimate hoop tension
$F_{Hu} = 1.5 F_H$.

(iii) Reinforcement :-

$$A_{st} = \frac{F_{Hv}}{0.87 f_y}$$

* Check with minimum r/f.

A_{st} = 0.12% of cross-sectional Area.

= 0.12% t × b.

where, t = thickness of wall (Assumed) [100-200 mm]

b = 1 m.

Step-4

Design of hopper bottom :-

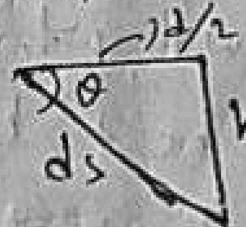
(i) Assume thickness of sloping slab (100 to 250 mm)

(ii) Weight of coal = (γ × vol. of hopper)

(iii) Weight of sloping bottom

$$= \gamma_c \cdot ds \left[\pi \left(\frac{D+d}{2} \right) + ds \right]$$

where, ds = depth of sloping bottom.



D = Dia of bunker.

d = dia of hopper.

γ_c = density of concrete.

(iv) Total load (W) = (ii) + (iii)

(IV) Tension in sloping slab at centre.

$$T = \frac{W}{n \times \left[\frac{D+d}{2}\right]} \operatorname{cosec}\theta$$

Now, find ultimate tension,

$$T_u = T \times 1.5$$

(V) Reinforcement for direct tension,

$$A_{st} = \frac{T_u}{0.87 f_y}$$

* This r/f to be provided in direction of sloping slab.

Step-5

Design of hoop r/f :→

(I) Normal component of coal pressure at centre of sloping side.

$$P_{n_1} = \gamma h \left(\cos^2\theta + \cos^2\phi \cdot \sin^2\theta\right)$$

γ = unit wt. of material.

$h = h_c + \frac{h_h}{2} + \frac{h_s}{2}$ [Mean height]

θ = Angle of hopper bottom.

ϕ = Angle of repose.

(ii) Normal component due to weight of sloping Side, $P_{n_2} = W_d \cos\theta$.

where $W_d = \gamma_c \; t \times 1$.

γ_c = unit weight of concrete.

t = thickness of hopper.

(iii) Total Normal pressure

$$P = P_{n1} + W_d \cos\theta$$

(iv) Hoop tension per metre

$$F_{HT} = \frac{1}{2} \times P \cdot d_m.$$

$$d_m = \text{mean dia} = \frac{D + d}{2}$$

* Find ultimate hoop tension $F_{HU} = 1.5\, F_{HT}$.

(v) Area of Hoop r/f.

$$A_{st} = \frac{F_{HU}}{0.87 f_y}$$

Q) Design a circular cylindrical bunker to store 20 tonne of coal. Density of coal is = 9 kN/m². Angle of repose = 30°. Use limit state method of design and adopt characteristics strength of concrete and steel as 15 & 415 N/mm². Sketch the details reinforcement.

Given data,

wt. of coal = 20 tonne
= 20,000 kg.
= 200 kN.
Density of coal = 9 kN/m².
Angle of repose (ϕ) = 30°.
f_{ck} = 15 N/mm².
f_y = 415 N/mm².

Step-1 Dia. of bunker :-

(i) vol. of bunker $= \frac{W}{\gamma}$

$= \frac{200}{9} = 22.2 m^3$

(ii) Adopt the dia of bunker = 2.8 m (D)

(iii) Depth of hooper (h_h) = 1.2 m.

(iv) height of cylindrical portion (h_c) = 3 m.

(v) Dia of hopper = 500 mm (d)

(vi) height of Surcharge = 800 mm (h_s)

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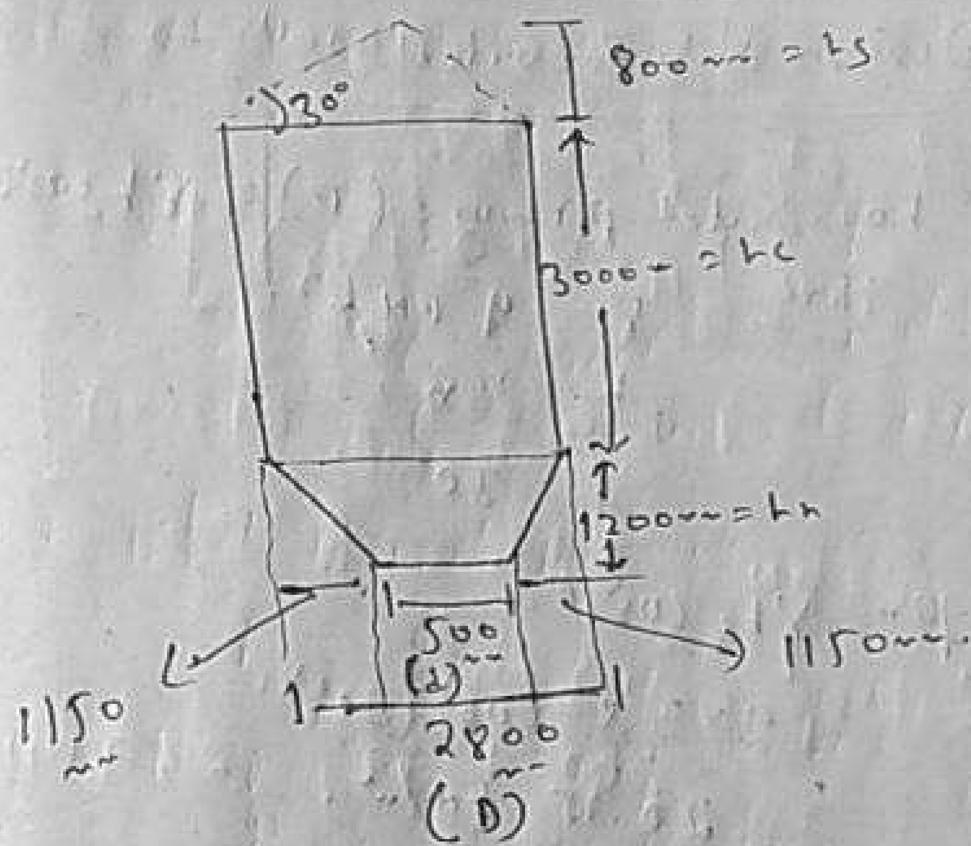


Step-2 Capacity of bunker

(i) vol. of surcharge $= \frac{1}{3}\left[\frac{\pi D^2}{4} \times h_s\right]$

$= \frac{1}{3}\left[\frac{\pi \times 2.8^2}{4} \times 0.8\right]$

$= 1.64\ m^3$.

(ii) vol. of cylindrical portion

$= \pi/4\,(D^2) \times h_c$

$= \pi/4 \times 2.8^2 \times 3$

$= 18.46\ m^3$

(iii) vol. of hopper $= \frac{\pi h_h}{12}\left[D^2 + D.d + d^2\right]$

$= \frac{\pi \times 1.2}{12}\left[2.8^2 + 2.8 \times 0.5 + 0.5^2\right]$

$= 2.9\ m^3$.

$\therefore$ Total vol $=$ (i) + (ii) + (iii) $= 23\ m^3 > 22.2$ (ok).

Step 3 Design of cylindrical wall :→

(I) horizontal pressure $(P) = \gamma h_c \cos^2\phi$.

Here, $\gamma = 9\ kN/m^3$.

$h_c = 3m$

$\phi = 30°$

$\therefore P = \gamma h_c \cos^2\phi$

$= 9 \times 3 \times \cos^2(30°)$

$= 20.25\ kN/m^2$.

(II) Hoop tension in cylindrical wall per metre height

$F_H = 0.5\ PD$.

Here, $P = 20.25\ kN/m^2$.

$D = 2.8\ m$.

$\therefore F_H = 0.5 \times 20.25 \times 2.8$

$= 28.35\ kN$.

$\therefore$ Ultimate hoop tension $(F_{Hu}) = 1.5 \times 28.35$

$= 42.53\ kN$.

(III) Reinforcement :→

$A_{st} = \frac{F_{Hu}}{0.87 f_y} = \frac{42.53 \times 10^3}{0.87 \times 415}$

$= 118\ mm^2$.

checking minimum r/f :-)

$= 0.12\%$ of gross Area

$= \frac{0.12}{100} \times t \times b$.

$= \frac{0.12}{100} \times 150 \times 1000$ [Assumed $t = 150$ mm]

$= 180\ mm^2$.

∴ Use 8 mm dia bar at 250 c/c distance.

Step-4 Design of hopper bottom :-).

(i) Assumed the thickness of sloping slab $= 180$ mm (t).

(ii) weight of coal $= \gamma \times$ vol. of hopper

$= 9 \times 23$

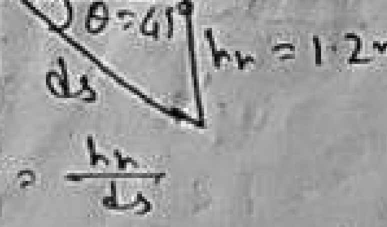
$= 207$ kN.

(iii) weight of sloping bottom

$$= \gamma_c d_s \left[\pi\left(\frac{D+d}{2} + d_s\right)\right]$$

Here, $\gamma_c = 24\ kN/m^3$

d_s = depth of sloping bottom



$\sin 45° = \frac{h_n}{d_s}$

$\Rightarrow d_s = h_n\sqrt{2} = 1.2 \times \sqrt{2}$.

$D = 2.8 m, \quad d = 0.5 m$

$\therefore$ Wt of sloping bottom

$$= \gamma_c d_s \left[\pi \left(\frac{d+D}{2} \times d_s\right)\right]$$

$$= 24 \times 1.2 \times \sqrt{2} \left[\pi \left(\frac{2.8+0.5}{2}\right) \times (1.2 \times \sqrt{2})\right]$$

$$= 42 \text{ kN}.$$

$\therefore$ Total load = (ii) + (iii)

$$= 207 + 42 = 249 \text{ kN}.$$

(iv) Tension in sloping slab at centre :-

$$T = \frac{W}{\pi \times \left[\frac{D+d}{2}\right]} \operatorname{cosec} 45^\circ$$

$$= \frac{249}{\pi \times 1.85} \times \operatorname{cosec} 45^\circ$$

$$= 60.6 \text{ kN}$$

$\therefore$ Ultimate tension $(T_u) = 60.6 \times 1.5$

$$= 90.9 \text{ kN}$$

(v) Reinforcement for direct tension :-

$$A_{st} = \frac{T_u}{0.87 f_y} = \frac{90.9 \times 10^3}{0.87 \times 415}$$

$$= 250 \text{ mm}^2$$

$\therefore$ Use 8 mm dia bar 200 mm c/c distance

& this r/t is to be provide in sloping side.

Step-5 :- Design of hoop reinforcement :-

(i) Normal component of coal pressure at centre of sloping side.

$$P_n = \gamma h (\cos^2\theta + \cos^2\phi \cdot \sin^2\theta).$$

where $\gamma = 9 \ kN/m^2$.

$h = h_c + \frac{h_h}{2} + h_s/2$

$= 3 + 1.2/2 + 0.8/2$

$= 4m$

$\theta = 45°$

$\phi = 30°$

$\therefore P_n = 9 \times 4 \left[\cos^2 45° + \cos^2 30° \cdot \sin^2 45°\right]$

$= 31.5 \ kN/m^2$.

(ii) Normal component due to weight of sloping side.

$= W_d \cdot \cos\theta$.

where $W_d = \gamma_c \cdot t \times 1$.

$= 24 \times 0.18 \times 1 \quad [t = 180 \ m]$

$= 4.32 \ kN$.

$\therefore W_d \cos\theta = 4.32 \times \cos 45°$

$= 3 \ kN/m^2$

(iii) Total normal pressure

$$P = P_n + W_d \cos\theta$$

$$= 31.5 + 3$$

$$= 34.5 \text{ kN/m}^2$$

NOW, mean dia at centre

$$d_m = \frac{D+d}{2} = 1.85 \text{ m}$$

(iv) Hoop tension per metre

$$= \frac{1}{2} \times P \cdot d_m$$

$$= \frac{1}{2} \times 34.5 \times 1.85$$

$$= 32.8 \text{ kN}$$

(v) Ultimate hoop tension,

$$= 1.5 \times 32.8 \text{ kN}$$

$$F_{HTU} = 49.2 \text{ kN}$$

(vi) Area of hoop r/f

$$A_{st} = \frac{F_{HU}}{0.87 f_y}$$

$$= \frac{49.2 \times 10^3}{0.87 \times 415} = 137 \text{ m}^2$$

* checking for Min r/f.

= 0.12% of gross Area

$= \frac{0.12}{100} \times t \times b$

$= \frac{0.12}{100} \times 180 \times 1000$

$= 216 \text{ mm}^2$

∴ Use 8 mm dia bar with 220 mm c/c.

(*) Design of Silos :-

Janssen's theory :-

→ In Janssen theory it is assumed that the large portion material weight is supported by friction between material & wall and only a small portion of weight is transferred to hopper bottom.

→ Horizontal pressure is given by :-

$$P_h = \frac{\gamma R}{\mu'}\left[1 - \exp\left(-\frac{\mu' n h}{R}\right)\right]$$

where, γ = density of material.

R = hydraulic mean Radius

$= D/4$. [D = Dia of Silo]

μ' = friction co.eff. = $\tan\phi'$

n = Ratio of (P_h/P_v) or $= \left[\frac{1-\sin\phi}{1+\sin\phi}\right]$

ϕ = Angle of repose.

h = height of cylindrical portion.

Now, vertical pressure intensity is indicated by P_v.

$$P_v = \frac{\gamma R}{2\mu' n}\left[1 - \exp\left[-\frac{2\mu' n h}{r}\right]\right]$$

Where, μ' = co-eff. of friction.

γ = density of material.

$n = \left(\frac{1-\sin\phi}{1+\sin\phi}\right)$

ϕ = Angle of repose

r = Radius of cylindrical wall

$= D/2$.

R = hydraulic mean Radius

$= r/2$ or $D/4$

(*) Airy's theory :-

→ It is based on Coulomb's wedge theory of earth pressure.

→ By this theory it is possible to calculate the horizontal pressure per unit length of the periphery & the position of the plane of rupture.

→ Depending upon the plane of rupture, two case are considered

Case-1 :- Shallow bin.

Case-2 Deep bin.

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Case - I Shallow bin :-

→ plane of rupture cuts the top horizontal surface.

→ $$P_h = \frac{\gamma h^2}{2}\left[\frac{1}{\sqrt{1+\mu^2} + \sqrt{\mu(\mu+\mu')}}\right]^2$$

where,
- $\mu' = \tan\phi'$
- $\phi' =$ Angle of friction.
- $\phi =$ Angle of repose.
- $\gamma =$ density of material.
- $h =$ height of cylindrical portion.
- $\mu =$ co-eff of friction.

* vertical load carried by the wall is $\boxed{P_v = \mu' P_h}$

Case - II (Deep Bin)

→ plane of rupture cuts opposite sides.

→ $$P_h = \left[\frac{\gamma b}{\mu+\mu'}\right]\left[1 - \frac{\sqrt{1+\mu^2}}{\sqrt{\frac{2h}{b}(\mu+\mu') + (1-\mu\mu')}}\right]$$

where,
- $\gamma =$ density of material.
- $b =$ width of silo / dia of silo
- $\mu =$ co-eff of friction
- $\mu' = \tan\phi'$
- $\phi' :$ Angle of friction.

* vertical load carried by wall $\boxed{P_v = \mu' P_h}$

Steps for design of Silos :→

Step-1 Given data.

Step-2 Allowable stress.

$$\left\{\begin{array}{l} \sigma_{cbc} = \\ \sigma_{ct} = \\ \sigma_{st} = \end{array}\right. \text{from IS: 456.}$$

Step-3 Dia of Silos :→

i) Dia of Silos - (D).
ii) Height of cylindrical portion (h_c)
iii) Depth of hopper bottom (h_h)
iv) Dia of opening of hopper bottom (d_h).

Step-4 Design of cylindrical wall

Janssen's theory :→

$$\text{Horizontal pressure } (P_h) = \frac{\gamma R}{\mu'}\left[1 - \exp\left(1 - \frac{\mu' k}{R}\right)\right]$$

where γ = density of material.

R = Hydraulic mean Radius
$= D/4$.

μ' = co-eff of friction b/w wall & material

P_v = vertical intensity of pressure

$n = \frac{1 - \sin\phi}{1 + \sin\phi}$ [ϕ = Angle of repose]

h = height of cylindrical portion.

(ii) Hoop tension in cylindrical wall per metre height.

$$F_{HT} = \frac{P_h D}{2}$$

where, P_h = horizontal pressure.

D = Dia of silos.

(iii) Area of hoop r/f

$$A_{st} = \frac{F_{HT}}{\sigma_{st}}$$

* check with min r/f = 0.12% of gross Area.

(iv) Tensile stress in concrete

$$= \frac{F_{HT}}{A_c + m A_{st}}$$

where, A_c = Area of concrete.

$= b \times t$ [t = Assumed, b = 1m]

$$m = \frac{280}{3\sigma_{cbc}}$$

Now the distribution steel

$A_{st}DS = 0.12\%$ of c/s Area

$= \frac{0.12}{100} \times t \times b.$

Step-6 **Design of hopper bottom :-**

(i) Surcharge load on hopper bottom

$$= \left[\gamma h - \frac{4 P_h \cdot \mu'}{D} \right]$$

where,

γ = Density of material.

h = height of cylindrical portion,

P_h = Horizontal pressure intensity.

μ' = coeff of friction,

D = Dia of silos.

(ii) Weight of slopping bottom,

$$= \pi \left[\left(\frac{D+d}{2} \right) + h_s \right] h_s \cdot \gamma$$

$$= \gamma \cdot h_s \left[\pi \left(\frac{D+d}{2} + h_s \right) \right]$$

where

γ = density of material

h_s = height of sloping side

$= h\sqrt{2}.$

D = dia of silos

d = dia of hopper opening.

iii) Total load (W) = (i) + (ii)

iv) Tension in sloping slab.

$$T = \frac{W}{\pi\left(\frac{D+d}{2}\right)} \operatorname{cosec}\theta$$

where θ = Angle of hopper with horizontal.

W = total load

D = Dia of silos.

d = Dia of hopper opening.

v) Steel r/f to direct tension

$$A_{st} = \frac{T}{\sigma_{st}}$$

vi) Surcharge load per meter run on hopper bottom $= \frac{P_s}{\pi D^2}$

* check with max. allowable pressure.

vii) Normal pressure :-

(a) Normal pressure intensity

$$P_n = \left(1.35 \cos^2\theta + P_h \sin^2\theta\right)$$

where, P_h = max. horizontal pressure.

(b) Normal component due to self weight

$= W_d \cos\theta$.

where $W_d = \gamma \times t \times 1$.

∴ Total normal pressure = (a) + (b)

viii) Hoop tension per metre

$$F_{HT} = \frac{1}{2} P d_m$$

P = total normal pressure

d_m = mean dia $= \frac{D+d}{2}$.

ix) Area of hoop r/f-

$$A_{st} = \frac{F_{HT}}{\sigma_{st}}$$

Q A cylindrical silo has an internal dia of 6m and 20m deep cylindrical portion with a conical hopper bottom. The material stored is wheat with a density of 8 kN/m³. The co-eff of friction between wall & material is 0.444. The ratio of horizontal to vertical pressure intensity is 0.40. Angle of repose is 25°. Design the r/f in silo walls - Adopt M-15 grade concrete & Fe 415 steel. Adopt Janssen theory for pressure calculation.

Mod-4 → Design of Bridges

Bridge :→ A bridge is a structure built to span physical obstacles without closing the way underneath such as body of water, valley, or road for the purpose of providing passage over the obstacle.

⊛ Classification :→

According to Superstructure.

- RCC Bridges
- Pre-stressed Concrete Bridges
- Steel Bridges
- Steel concrete composite Bridges
- Cable supported Bridges.

⊛ Component of Road Bridges :→

i) Road vehicle.

ii) **Bridge Deck** :-

The load bearing floor of a bridge which carries and spreads the loads to the main beams. It is either of rcc, pre-stressed concrete, welded steel etc.

RCC- Bridges

- Solid Slab Bridges.
- RCC T- Beam Briges.

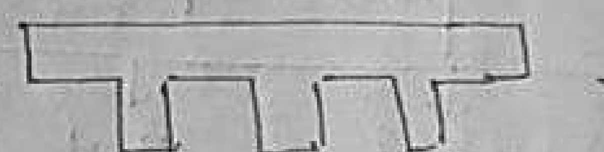
Slab Bridges:-

→ used for upto 12m Span Bridges

→ 

R.C.C. T- Beam Briges:-

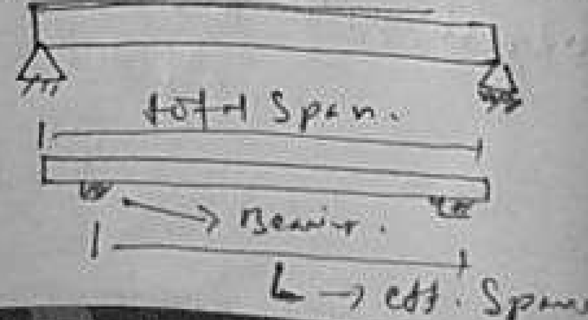
→ used for 12 to 28m Span Bridges.

→ 

General features of Design and vehicle loading :-

I) **Deck:-** Carrigeway, footpath & crash barrier.

II) **Span:-**



(*) Code of practice for Road Bridges :-

IRC-5 - Section I → Gen. features of design.

IRC-6 - Section II → Loads & Stresses.

(*) The width of carriageway shall not be less than -

a) 4.25 m for single Bridge.

b) 7.5 m for two lane "

* Shall be increased by 3.5 m for every additional lane.

* footpath → 1.5 m.

(*) Design of Slab Bridges :-

→ Hydraulic design → when Bridges crosses the water Body.

→ Structural design :- → Applicable for all Bridges.

Q. Design a RCC slab bridge for a state highway for the following data.

i) Carriage way (2 lane) → 7.5 m wide.

ii) Material grade, M20, Fe 415

iii) footpath (kerb) - 600 m wide.

iv) wearing coat - 80 mm.

v) clear span - 6 m.

vi) width of Bearing - 0.4 m.

vii) loading - IRC class A or AA. whichever gives worst effect

→ Step-1 permissible stresses :-

$\sigma_{cbc} = 7\ N/mm^2$

$\sigma_{st} = 190 N/mm^2$ (IS:456, table 22)

$m = \frac{280}{3\sigma_{cbc}} = 13.$

$k = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = 0.3238.$

$J = 1 - k/3 = 0.89$

$Q = 1/2\ J.k.\sigma_{cbc} = 1.008.$

Step-2 Depth of slab & eff. span :-

→ Assume the thickness 80 mm per 1 m.

So, overall thickness = 80 × 6

= 480 ≈ 500 mm.

using 25 mm ϕ bars Assuming 25 mm clear cover, eff. depth = $500 - (25 + 25/2)$

$= 462.5$ mm.

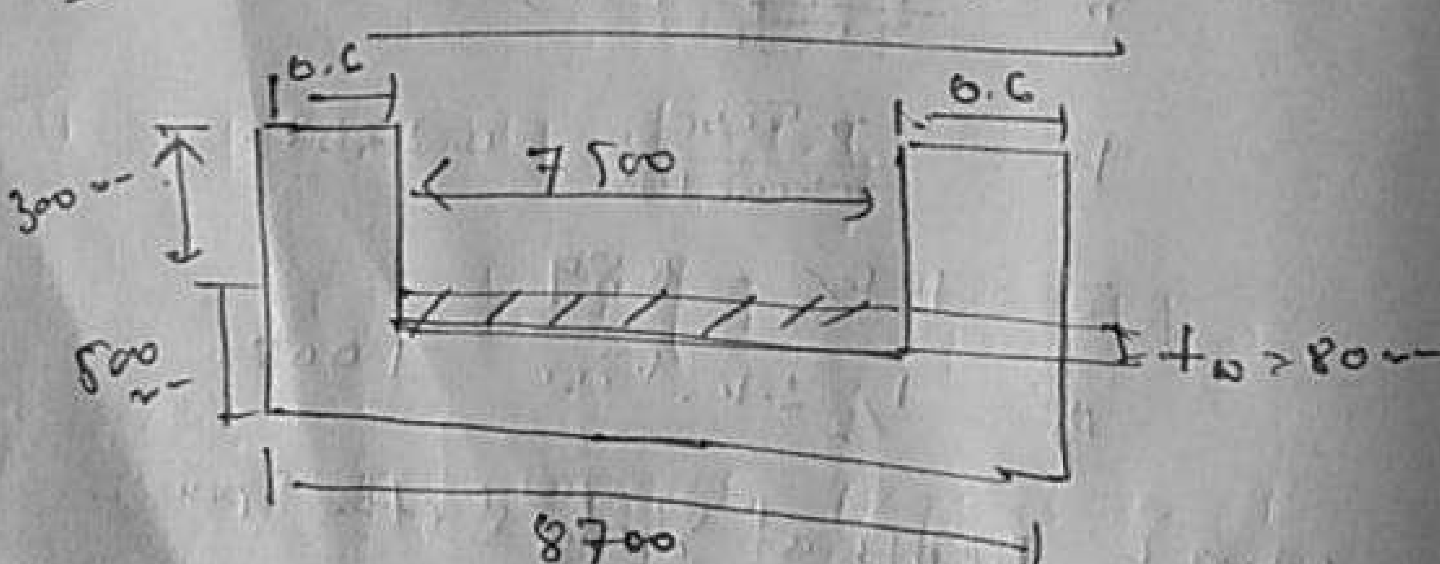
Given, width of bearing = 0.4 m.

(b) eff. span :-

= least of
1) clear span + eff. depth
$= 6 + 0.462$
$= 6.462$ m.
ii) centre to centre of bearing
$= 6 + \frac{0.4}{2} + \frac{0.4}{2}$
$= 6.4$ m

∴ So eff. span = $L_{eff} = 6.4$ m.

Step-3 Draw cross-section of Bridge :-



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Step-4 Dead load Bending moment :-

(i) Dead weight of slab = $\gamma_c \times t_s$.

where, $\gamma_c = 24\ kN/m^2$

$t_s = 500\ mm$

$= 0.5\ m$.

$\therefore$ Dead weight of slab $= 24 \times 0.5$

$= 12\ kN/m^2$

(ii) Wearing coat $= \gamma_w \times t_w$

$= 22 \times 0.08$

$= 1.76\ kN/m^2$.

$\therefore$ total load $= 12 + 1.76 = 13.76\ kN$.

$\therefore$ Dead load B.M

$\Rightarrow M_{dead} = \dfrac{w_d \cdot L_e^2}{8}$

$= \dfrac{13.76 \times 6.4^2}{8}$

$= 70.4\ kN.m$

Step-5 Live load Bending moment :-

→ Class AA tracked vehicle

(i) Impact factor (IF)

$= \left[25 - \frac{15}{4}(l_e - 5)\right]$

$= \left[25 - \frac{15}{4}(6.4 - 5)\right]$

$= 19.7\ \%$.

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(ii) eff length of road :-

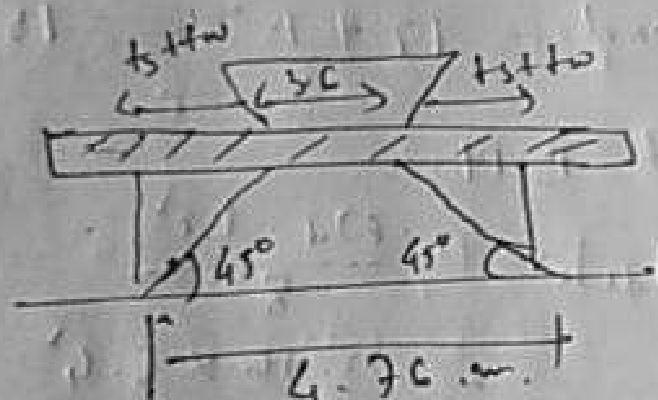
$$L_R = \{ l_t + 2(t_s + t_w) \}$$

where, l_t = Length of tracked vehicle
= 3.6 m.

$t_s = 0.5$ m.

$t_w = 0.08$ m.

$\therefore L_R = [3.6 + 2(0.5 + 0.08)]$
= 4.76 m.



(iii) eff. width of slab :-

$$b_e = k \cdot x \left[1 - \frac{x}{L_e} \right] + b_w$$

where k = constant Depends on (B/L) ratio of Reqd table. of IRC21

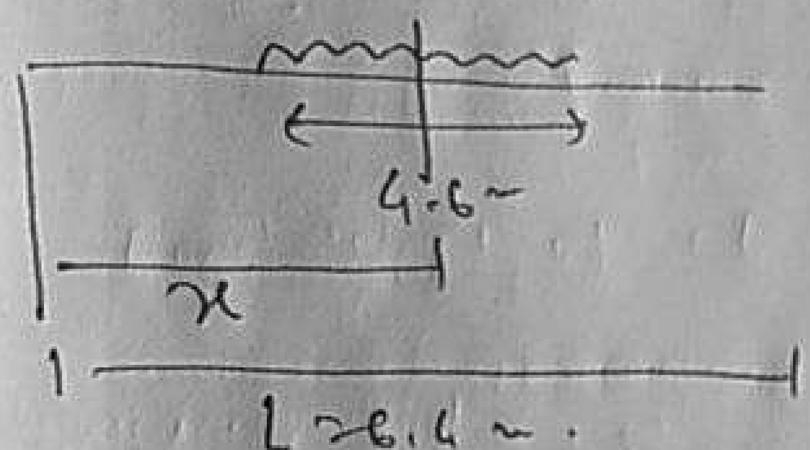
or

= 2.77 for SSB.

L_e = eff. length = 6.4 m.

x = distance from C G of load from nearest support.

$$= \frac{6.4}{2} = 3.2 \text{ m}.$$



b_w : width of vehicle $= \{b_v + 2t_w\}$

$$= \{0.85 + 2(0.08)\}$$ [for class AA vehicle $b_v = 0.85$ m]

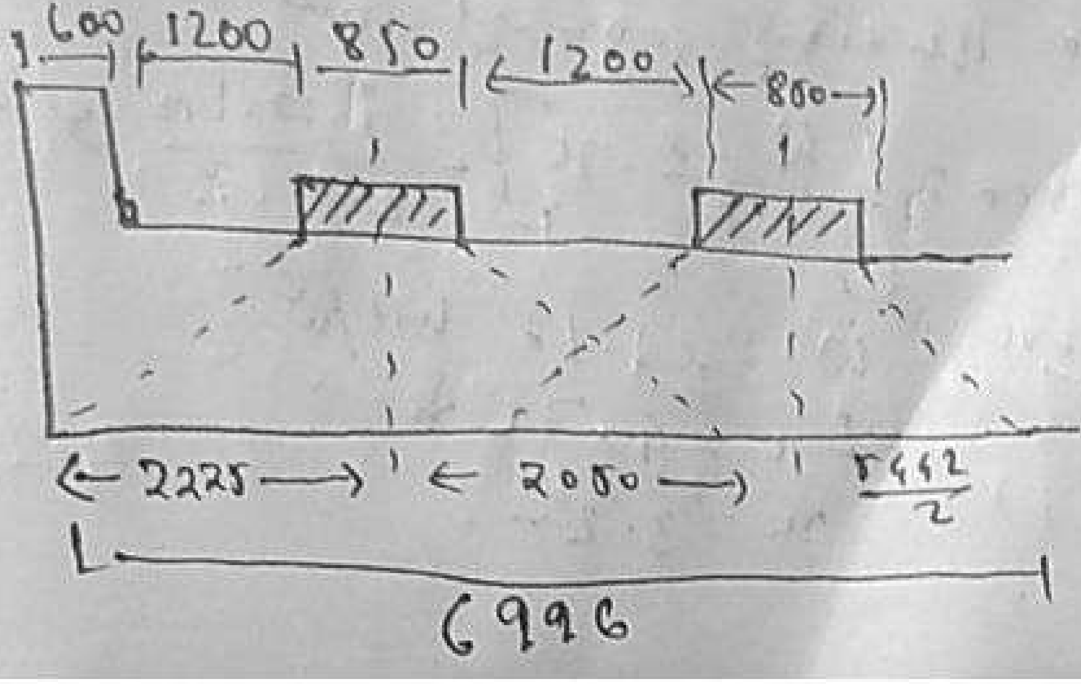
$$= 1.01 \text{ m}.$$

$$\therefore b_e = k \cdot x\left[1 - \frac{x}{l_e}\right] + b_w$$

$$= 2.77 \times 3.2\left[1 - \frac{3.2}{6.4}\right] + 1.01$$

$$= 5.442 \text{ m}.$$

But according to code for Minimum Clearance eff. width will be.



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So, when the vehicle is placed with minimum clearance, effective width will change.

from diagram, net eff. width

$b_e = 6996\ mm$

$= 6.996\ m.$

(iv) Total load of two tracks with Impact.

$= 35T + 35T + (1 + I_f)$

where, $1T = 10\ kN.$

$I_f = 19.7\%$

$= 0.197.$

$\therefore \quad = 350 + 350 + (1 + 0.197)$

$W_T = 838\ kN.$

Average intensity of load

$$w = \frac{W_T}{Area} = \frac{W_T}{L_R \times b_e}$$

$$= \frac{838}{4.76 \times 6.996}$$

$= 25.17\ kN/m^2.$

(v) Max Bending moment due to live load

$$M_{max} = \left[\frac{w \cdot L_R}{2} \cdot x\right] - \left[\frac{w \cdot L_R}{2} \times \frac{L_R}{4}\right]$$

we have, $w = 25.17\ kN/m^2$

$L_R = 4.76\ m$

$x = 3.2\ m.$

$M_{wk} = \left\{\frac{25.17\times 4.26}{2}\times 3.2\right\} - \left\{\frac{25.17\times 4.26}{2}\times\frac{4.26}{4}\right\}$

$= 120.35$ kN-m.

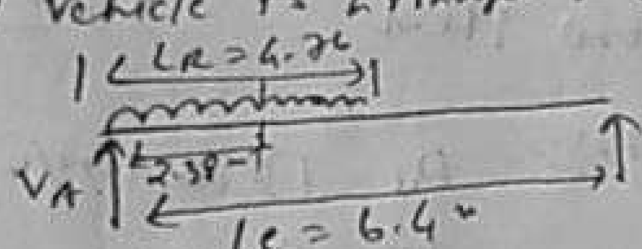
Total design moment $M = M_{dl} + M_{LL}$

$= 70.4 + 120.36$

$= 191$ kNm.

Step-6 Shear due to class AA tracked vehicle :→

The max shear at support, IRC class AA tracked vehicle is arranged as shown.



Then eff. width

$b_e = kx\left(1 - \frac{x}{l_e}\right) + b_w$

We have, $k = 2.77$

$b_w = 1.01$

$l_e = 6.4$ m.

$x = 2.38$ m.

$\therefore b_e = 2.77 \times 2.38\left(1 - \frac{2.38}{6.4}\right) + 1.01$

$= 5.16$ m.

Width of depression $= \cancel{22.25 + 20}$

$= 225 + 2050 + \frac{5160}{2}$

$= 6.855$ m.

From b_e new value of load intensity

$w = \frac{W}{l_R \times b_e} = \frac{838}{4.26 \times 6.855}$

$= 25.60$ kN/m².

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(ii) Shear force $(V_R) = \frac{W \times L_R \times (Le - x)}{Le}$

We have

$W = 25.68$

$L_R = 4.76$

$Le = 6.4m$

$x = 2.38m$

$V_R = \frac{25.68 \times 4.76\,(6.4 - 2.38)}{6.4}$

$= 76.80$ kN

(iii) Dead load shear $= \frac{W_d \times Le}{2}$

We have $W_d = 13.76$ kN/m

$Le = 6.4m$

∴ Dead load shear $= \frac{13.76 \times 6.4}{2}$

$= 43.75$ kN

Total design shear $= 76.80 + 43.75$

$= 121$ kN

Step-7 Design of deck slab :-

Eff. depth required

$d = \sqrt{\frac{M}{Q.b}}$

$= \sqrt{\frac{191 \times 10^6}{1.008 \times 1000}}$ [b=1m]

$= 435.3$ mm

Eff. depth provided $= 462.5$ mm

So proceed with with $d = 462.5$ mm

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R/f :-) $A_{st} = \frac{M}{\sigma_{st} + j \cdot d}$

Adopt 25 mm ϕ bar

$A_{st} = \frac{191 \times 10^6}{190 \times 0.89 \times 462.5}$

$= 2442$ mm^2.

Spacing of 25 mm dia bar

$= \left[\frac{1000 \times a_{st}}{A_{st}}\right]$

$= \frac{1000 \times \pi/4 \times 25^2}{2442}$

$= 201$ mm.

$\simeq 200$ mm c/c.

(Ans).

Summary of Annotations