SEMESTER VI

17BECE602 DESIGN OF RC STRUCTURES II 3 2 0 4 100

OBJECTIVE

- To understand the design concept of various structures and detailing of reinforcements.
- To understand the design of underground and elevated liquid retaining structures.
- To study the design of material storage structures.
- To know the effect of temperature on concrete structures.
- To study the design of bridges subjected to IRC loading.

UNIT 1

YIELD LINE THEORY 12

Introduction-Assumptions - Characteristics of yield line - Determination of collapse load / plastic moment- Application of virtual work method - square, rectangular, circular and triangular slabs With point load and UDL (Simply support and Fixed support)- Design problems.

UNIT II

BUILDING FRAMES 12

Multi storeyed structures and framed structures-Elastic analysis, Suitable substitute frames for gravity loadings-Approximate analysis of single and two bay frames up to three storey using portal method and cantilever method.

UNIT III

RETAINING WALLS 12

Design of Cantilever retaining wall – Design of Counterfort Retaining walls-Stability Analysis.

UNIT IV

WATER TANKS 12

Classification-IS code provisions-Principles of design-Design of rectangular and circular water tanks , below ground level, tanks resting on ground and Elevated tanks – Intze type water tank (Theory only)

UNIT V

SPECIAL ELEMENTS 12

Design of staircases (Straight and doglegged) – Design of flat slabs – Design Principles of Mat foundation and box culvert.

TOTAL: 60 HRS

TEXT BOOKS:

Sl.No	Title of Book	Author of Book	Publisher	Year of Publishing
1.	R.C.C. Designs Reinforced Concrete Structures	Punmia B.C, Ashok Kumar Jain, Arun K.Jain	Laxmi Publications Pvt. Ltd., New Delhi	2006

REFERENCES:

Sl.No	Title of Book	Author of Book	Publisher	Year of Publishing
1.	Advanced Reinforced Concrete Design	Varghese.P.C	Prentice Hall of India Pvt. Ltd New Delhi.	2012
2.	Reinforced Concrete	Mallick, D.K. and Gupta A.P	Oxford and IBH Publishing Company, New York.	2003
3.	Design of Reinforced Concrete Structures	Gambhir.M.L	Prentice Hall of India Private Limited, New York.	2012

WEBSITES:

- http://www.icivilengineer.com
- http://www.engineeringcivil.com/
- http://www.aboutcivil.com/
- http://www.engineersdaily.com
- http://www.asce.org/
- http://www.cif.org/
- http://icevirtuallibrary.com/
- http://www.ice.org.uk/
- http://www.engineering-software.com/ce/

COURSE OUTCOMES

On completion of the course, the students will be able to:

- Apply the concepts of liquid retaining structures.
- Design material storage structures using various theories.
- Apply the concepts of environmental and transportation structures.
- Demonstrate the detailing of reinforcement.
- Draw the various RCC structures.

KARPAGAM ACADEMY OF HIGHER EDUCATION

(Established Under Section 3 of UGC Act, 1956)

COIMBATORE-641 021

17BECE602-DESIGN OF RC STRUCTURES II

Lecture Plan

Staff Name	**: Ms.P.PREETHI, M.E,**
Semester	**: 6 (2019-20 EVEN)**
Course Type	**: Core**
Number of credits	**: 4**
LTPC	**: 3 2 0 4**

S.No	Lecture Duration (Hour)	Topics to be covered	Support Materials
		UNIT I YIELD LINE THEORY	
1.	1	Introduction-Assumptions	T/534
2.	1	Characteristics of yield line	T/536
3.	1	Determination of collapse load / plastic moment	T/541
4.	1	Application of virtual work method	T/545
5.	1	square with point load and UDL	T/546
6.	1	circular with point load and UDL	T/547
7.	1	rectangular with point load and UDL	T/552
8.	1	Simply support and Fixed support	T/560
9.	1	Design problems.	T/561
		UNIT II BUILDING FRAMES	
10.	1	Multi storeyed structures	T/575
11.	1	framed structures	T/576
12.	1	Elastic analysis	T/578
13.	1	Suitable substitute frames for gravity loadings	T/580
14.	1	Approximate analysis of single and two bay frames up to three storeys	T/590
15.	1	portal method	T/591
16.	1	cantilever method	T/592
17.	1	Approximate analysis of single bay system	T/595
18.	1	cantilever method	T/598
		UNIT III RETAINING WALLS	
19.	1	Design of Cantilever retaining wall	T/671
20.	1	Design of Cantilever retaining wall	T/671
21.	1	Design of Cantilever retaining wall	T/671
22.	1	Design of Counterfort Retaining walls	T/686
23.	1	Design of Counterfort Retaining walls	T/686
24.	1	Design of Counterfort Retaining walls	T/686
25.	1	Stability Analysis	T/691
26.	1	Stability Analysis	T/691
27.	1	Stability Analysis	T/691

UNIT IV WATER TANKS			
28.	1	IS-code regulations	T/712
29.	1	Classification	T/714
30.	1	Principles of design	T/715
31.	1	Design of rectangular and circular water tanks	T/716
32.	1	Design of rectangular and circular water tanks , below ground level	T/717
33.	1	Design of tanks resting on ground	T/719
34.	1	Elevated tanks	T/721
35.	1	Intze type water tank (Theory only)	T/724
36.	1	Intze type water tank (Theory only)	T/724
UNIT V SPECIAL ELEMENTS			
37.	1	Design of staircases (Straight)	T/984
38.	1	Design of staircases (doglegged)	T/986
39.	1	Design of flat slabs	T/994
40.	1	Design Principles of Mat foundation	T/1008
41.	1	Design Principles of Mat foundation	T/1008
42.	1	Design Principles of Mat foundation	T/1008
43.	1	box culvert	T/1024
44.	1	box culvert	T/1024
45.	1	box culvert	T/1024
Revision			
46.	1	Revision	
47.	1	Previous year End Semester Question paper discussion	

TEXT BOOKS:

Sl. No	Title of Book	Author of Book	Publisher	Year of Publishing
1.	Limit State Design of Reinforced Concrete (As per IS 456:2000)	Dr.B.C.Punmia, Ashok Kumar Jain, Arun Kumar Jain	Laxmi Publications (P) Ltd	2010
2.	Reinforced Concrete Design	Unnikrishna Pillai & Devados Menon	Tata McGraw Hill Publishing Co, New Delhi	2012
3.	IS 456-2000 Indian Standard Code of practice for Reinforced Concrete.			
4.	SP-16 Design Aids for IS 456-1978. IS 875-1987-Code of Practice for Design Loads			

REFERENCE:

Sl. No	Title of Book	Author of Book	Publisher	Year of Publishing
1.	Reinforced Concrete	Mallick, S.K., and Gupta, A.P	Oxford & IBH Publishing Co., New Delhi	2008
2.	Reinforced Concrete Design	Sibha.S.N.	Tata McGraw-Hill Publishing Co, Ltd., New Delhi	2001
3.	Reinforced Concrete Mechanics and Design	Mac Gregor J.G	Prentice Hall, New Jersey	2008
4.	Reinforced Concrete limit state design.	Ashok K Jain	Nem Chand Bros, Roorkee	2012
5.	Limit State Design of R.C.Structures.	Varghese, P.C	PHI Learning Pvt. Ltd. New Delhi	2008

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- http://www.icivilengineer.com
- http://www.engineeringcivil.com/
- http://www.aboutcivil.com/
- http://www.engineersdaily.com
- http://www.asce.org/
- .http://www.cif.org/

20/11/19

FACULTY INCHARGE

20.11.19

HOD (Civil)

DEAN/FOE

Design of RC Structures II

UNIT – II

Yield Line Theory

Yield Line Theory:-

- It is one of the most important developments in the analysis and design of slab systems.
- It is the Ultimate load theory for the design of R.C. slabs.

Definition of Yield line:-

-It is defined as a line in the plane of slab across which reinforcing bars have yielded and about which excessive deformation (plastic rotations) under constant limit moment (Ultimate moment) continues to occur leading to failure.

~~Assumptions~~ (or) characteristics of yield lines:-

1. Yield lines are straight lines so that they may act as hinges of a collapse mechanism.
2. Yield lines ends at a slab boundary or at the intersection of other yield lines.

3. Yield lines act as axes of rotation for the movements of adjoining segments.

4. Each of the segments of the slab will tend to rotate in a rigid body motion.

5. If an edge is fixed (or) continuous, a yield line may form along the support.

6. Yield lines (or) yield lines produced, pass through the intersection of the axes of rotation of adjacent slab elements.

<u>Sign conventions for Yield Line Patterns and supports:-</u>

Free (or) unsupported edge

Simply supported edge

Fixed (or) continuous edge.

positive yield line

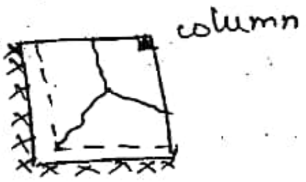
Negative yield line

Axis of rotation

Beam support

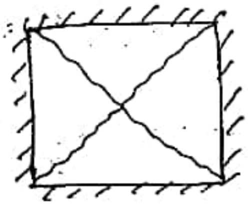
(or) column

o Point load

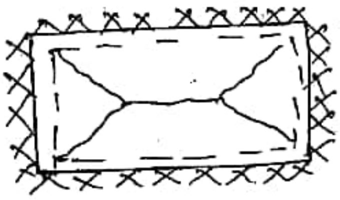


⇒ Square slab & adjacent sides are fixed supported.

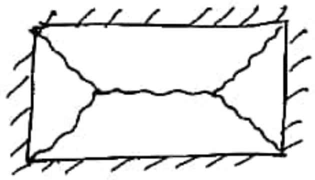
Yield Line Patterns in R.C. Slabs:-



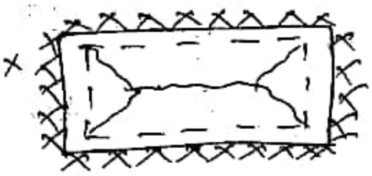
⇒ Square slab - simply supported



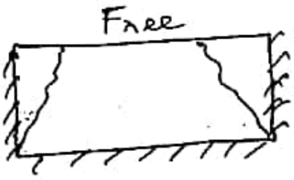
⇒ fixed support (-ve yield line (at bottom).



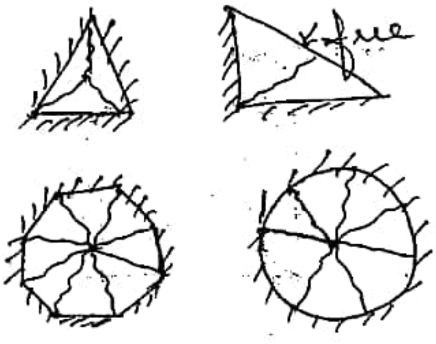
⇒ Rectangular slab simply supported.



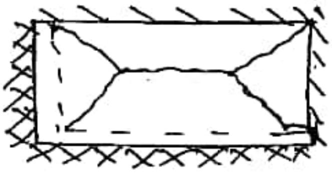
⇒ Rectangular slab - fixed (-ve yield lines).



⇒ rectangular slab - simply supported.



} Slabs of other shapes.

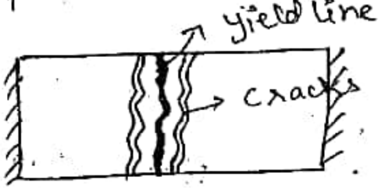


⇒ Rectangular 2 adjacent sides fixed & 2 adjacent sides simply supported.

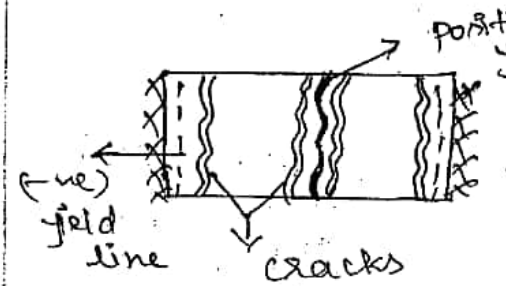
Yield line patterns:-

one way slab:-

In The one way slab yeild line (⊥) perpendicular to the direction of Reinforcement (R.F)



⇒ simply supported



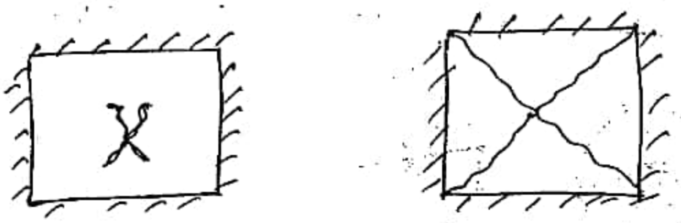
⇒ fixed supported

Two way slab:-

in two way slab yield lines are not perpendicular (⊥) to the direction of R.F.

square slabs:-

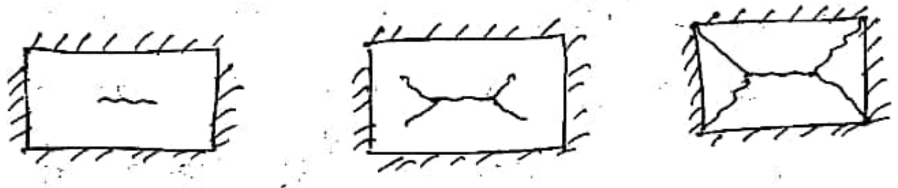
cracts developed small patches of yield of dignals @ mid span and spread towards corners.



square slabs.

Rectangular slab:-

Initial crack developed in a dirctn ⊥ to the short-span @ the mid-region of the slab s. Further increase in the load results shows fig below. in continuous growth of cracks as shown in fig. below.



Ultimate loads on slabs:-

After the yield line pattern has been assumed the ultimate load capacity of R.F concrete slab is found by 2 methods.

⇒ Virtual work method

⇒ Equilibrium method.

Virtual Work method:-

Load × deflection = mt × Rotation
(External work) (Internal work).

This method is based on principle that the work done by the external Forces in undergoing virtual displacement is equal to the internal work done (or) energy dissipated in rotation along the yield lines.

$$\Sigma\, m\theta_n l_0 = \Sigma\, w\Delta$$

$m \Rightarrow$ ult. mt across an yield line.

$l_0 \Rightarrow$ Length of yield line.

$\theta_n \Rightarrow$ Relative rotation of 2 adjacent plates, $\perp$ to the yield line.

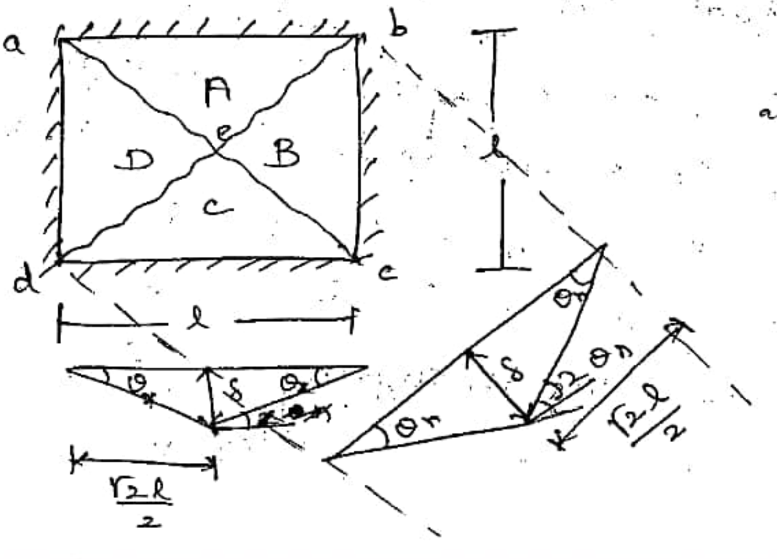
$W \Rightarrow$ Resultant load on each segment

$\Delta \Rightarrow$ corresponding displacement @ the centroid of load in each segment.

(1) Find the ult. load of the uniformly loaded isotropically R.F square slab, simply supported on all edges by virtual work method.

Sol:-

The yield line pattern is shown in fig.



displacement = δ (is given to centre of slab) ie @ 'e'

length of each diagonal yield line $l_0 = \sqrt{2}\,l$
from fig.

The total rotation of the diagonal segments $= 2\,\theta_n = 2 \dfrac{\delta}{\frac{\sqrt{2}\,l}{2\sqrt{2}}} = \dfrac{2\sqrt{2}\,\delta}{l}$

Internal work done on deb $= \Sigma\, m\, l_0\, \theta_n$

$= \Sigma\, m(l\sqrt{2})\left(2\dfrac{\sqrt{2}\,\delta}{l}\right)$

$= 4m\delta$

Internal work done on aec $= 4m\delta$

total Internal work done $= 4m\delta + 4m\delta$

$= 8m\delta$ — ①

If collapse load is w/unit area,
External work done = 4 (work done by each segments).

Virtual displacement $\Delta = \delta/3$

Loads
~~External~~ ~~work done~~ on each segments

$W_u = w \times$ Area of the segment

$= w \times \frac{1}{2} \times l \times \frac{l}{2} = \dfrac{wl^2}{4}$

acting through centroid
total external work done $= \Sigma W \Delta$

$= 4 \times \dfrac{wl^2}{4} \times \dfrac{\delta}{3} = \dfrac{wl^2\delta}{3}$

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By the principle of virtual work, ① = ②

i.e., total Internal work = total External work

$$8\,m\,\delta = \frac{wl^2\,\delta}{3}$$

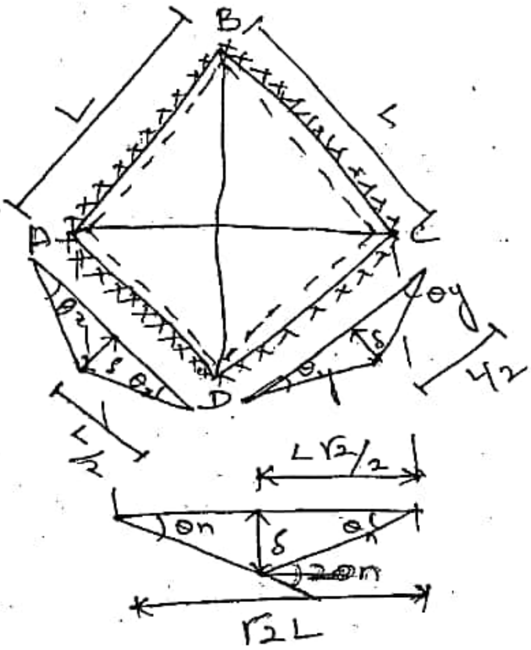
$$\boxed{\frac{24\,m}{l^2} = w}$$

$$\boxed{\text{ultimate moment} \quad m = \frac{wl^2}{24}}$$

Ans.

② Find the ult. load of the uniformly loaded isotropically R.F square slab, fixed supported on all edges by virtual work method.

Sol:-



-ve yield line length = L

diagonal yield line length = √2 L

(-)ve yield line:-

The rotation of negative yield lines

$$\theta_x = \theta_y = \theta$$

from fig. $\boxed{\theta_x = \frac{\delta}{L/2}}$

Internal work done by -ve yield line AB

$$= \Sigma m \theta l_0$$

$$= m \cdot \frac{\delta}{L/2} \cdot L = 2m\delta$$

||ly,
Internal work done by -ve yield line BC

$$= 2m\delta$$

||ly
Internal work done by -ve yield line CD

$$= 2m\delta$$

||ly
Internal work done by -ve yield line DA

$$= 2m\delta$$

total Internal work done by -ve yield lines AB, BC, CD and DA is

$$= 2m\delta + 2m\delta + 2m\delta + 2m\delta$$

$$= 8m\delta$$

(+) ve yield line :-

length of diagonal yield line $(l_0) = \sqrt{2}L$.

displacement $= \delta$

total rotation of the diagonal segments

$$= 2\,\theta_n = 2\,\frac{\delta}{\frac{\sqrt{2}L}{2}} = \frac{2\sqrt{2}\,\delta}{L}$$

Internal work done by 2 diagonal yield lines $= \Sigma\, m\, \theta_n\, l_0$

$$= 2m\,\frac{2\sqrt{2}\,\delta}{\cancel{L}}\cdot\sqrt{2}\cancel{L}$$

$$= 8\,m\,\delta$$

hence total Internal work done by (−ve) and (+) ve yield lines

$$= 8m\delta + 8m\delta$$

$$= 16\,m\,\delta \longrightarrow (1)$$

If the collapse load w/unit area.

External work done $= 4($work done by each segments$)$.

virtual displacement $\Delta = \delta/3$

~~External work done~~ Load on each segments

$= w \times$ area of one segments

$$= w \times \tfrac{1}{2} \times L \times \tfrac{L}{2} = \frac{wL^2}{4}$$

total external work done $= \Sigma w.\Delta$

$$= 4 \times \frac{wL^2}{4} \times \frac{\delta}{3} = \frac{wL^2 \cdot \delta}{3} \quad —(2)$$

By principle of virtual work

(1) = (2)

$$\Rightarrow \quad 16 m \cancel{\delta} = \frac{wL^2 \cancel{\delta}}{3}$$

$$\boxed{w = \frac{48\, m}{L^2}}$$

ultimate moment, $\boxed{m = \frac{wL^2}{48}}$ Ans.

* Isotropically Reinforced slab:-

The slab having equal R.F. in Both (or) two direction.

i.e ⇒ $m_{ux} = m_{uy} = m_u$.

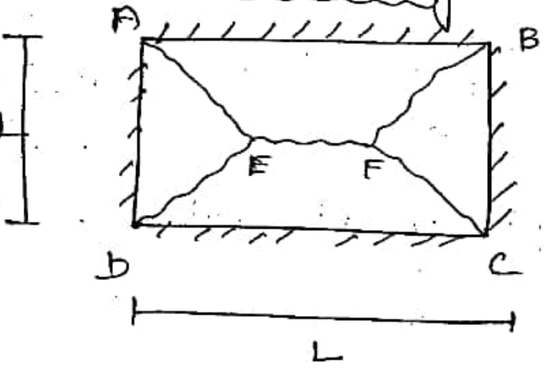
* Orthotropically Reinforced slab:-

If the Reinforcement in the two directions is not the same.

ie. $m_{ux} = m_u$, $m_{uy} = \mu\, m_u$

③ Rectangular slab, simply supported - ultimate Load

⇒ For Orthotropically Reinforced slab;



L = long span

αL = short span.

m_u = ultimate moment capacity in the short span direction

μm_u = ultimate moment capacity in the long span direction.

ultimate moment,

$$\boxed{m_u = \frac{w_u \alpha^2 L^2}{24}\left[\sqrt{3+\mu\alpha^2} - \alpha\sqrt{\mu}\right]^2}$$

⇒ For an Isotropically Reinforced slab,

$\mu = 1$

∴ ultimate moment,

$$\boxed{m_u = \frac{W_u \alpha^2 L^2}{24}\left[\sqrt{3+\alpha^2} - \alpha\right]^2}$$

Equilibrium Method:-

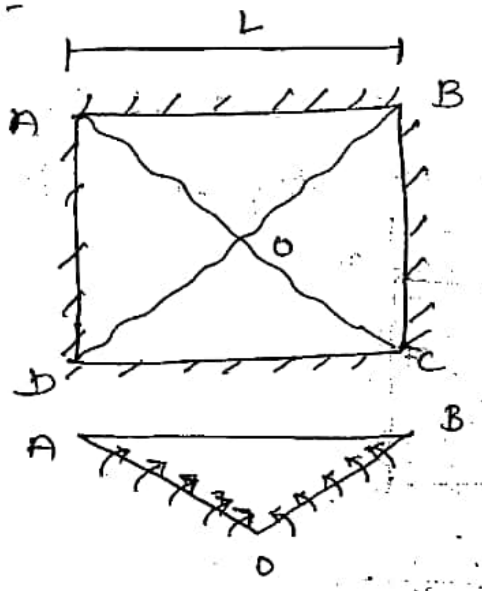
* In this method, the collapse load is calculated from the equilibrium of the individual segments of a mechanism.

⇒ In both virtual work and equilibrium method give the upper bound solution.

⇒ i.e. The computed collapse load basis of assumed yield line patterns bound to be larger than the actual collapse load.

④. Find the ultimate moment of Isotropically R.F square slab, simply supported along all edges by equilibrium method.

Sol:-



• Ultimate moment (m_u) will be the same along all the directions.

• The yield lines will be along the 2 diagonals.

Considering the equilibrium of the triangular sector ABO.

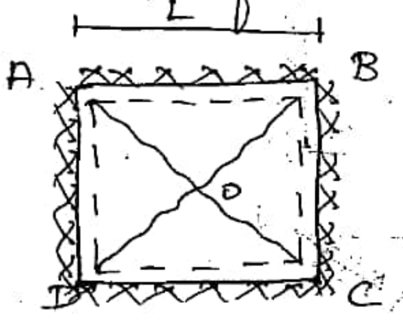
Taking moment about edge AB,

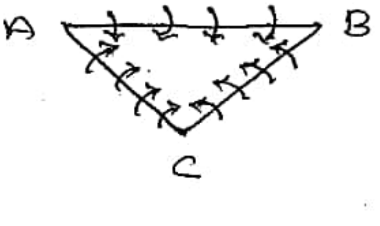
we get $m_u L = \frac{1}{2} L \cdot \frac{L}{2} \cdot w_u \frac{L}{6}$

$$m_u \cancel{L} = \frac{w_u L^{\cancel{3}2}}{24}$$

ultimate moment $\Big\}$, $\boxed{m_u = \frac{w_u L^2}{24}}$ //Ans.

⑤ Ultimate moment – Isotropically Reinforced square slab, fixed supported all the edges.





(+)ve yield line will be along the diagonals,
(-)ve yield line will be developed along the fixed edges.

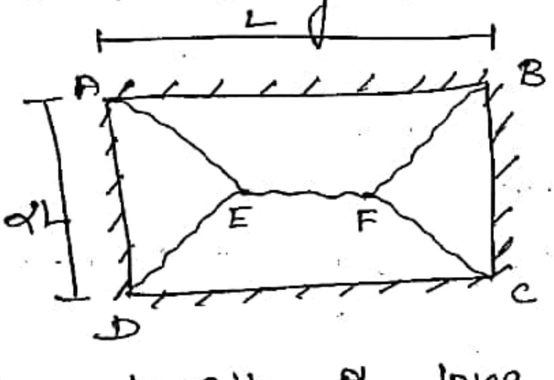
considering equilibrium of sector ABO,
taking moment about edge AB,

we get $m_u K + m_u K = \frac{1}{2} K \cdot \frac{L}{2} w_u \frac{L}{6}$

Ultimate moment } $\boxed{m_u = \frac{w_u L^2}{48} \text{ //Ans}}$

(b) Rectangular Slab

for orthotropically R.F slab with simply supported along the edges.



L = length of long span

αL = length of short span

m_u = Ulti. mt capacity in short span drtn.

μm_u = Ulti. mt capacity in long span drtn.

Ult. mt $\boxed{m_u = \frac{w_u \alpha^2 L^2}{24}\left[\sqrt{3+\mu\alpha^2} - \alpha\sqrt{\mu}\right]^2}$ //Ans.

for isotropically R.F ▭ lar slab,

$\mu = 1$

∴ ultimate moment,

$\boxed{m_u = \frac{w_u \alpha^2 L^2}{24}\left[\sqrt{3+\alpha^2} - \alpha\right]^2}$ //Ans.

Problems:-

① A square slab of side length 4m is simply supported @ the ends and carries a service L.L of 3kN/m². Design the slab. Use M_{20} concrete and Fe 415 steel bars.

Solution:-

given data:-

$L = 4m$

$f_{ck} = 20$, $f_y = 415$ N/mm²

Step :1:-

parameters calculation:-

$\frac{x_{u\,max}}{d}$ for Fe 415 steel

⇒ from SP-16 page 9.

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$$\boxed{\frac{x_{u\,max}}{d} = 0.479}$$

$$R_u = 0.36 f_{ck} \frac{x_{u\,max}}{d}\left(1 - 0.416 \frac{x_{u\,max}}{d}\right)$$

$$= 0.36 \times 20 \times 0.479 \left(1 - (0.416 \times 0.479)\right)$$

$$\boxed{R_u = 2.761}$$

Step 2:-

Effective depth calculation:-

from IS. 456:2000,

page: 39, clause: 24.1

$\frac{\text{span}}{\text{overall depth}} = 35$ ⇒ (simply supported slab)

$$\therefore \frac{L}{D} = 35$$

$$D = \frac{L}{35} = \frac{4000}{35} = 114 \text{ mm}$$

Let us provided over all depth

$$\boxed{D = 130 \text{ mm}}$$

using 10 mm dia bar and

clear cover = 15mm

$\therefore$ effective depth $d_{eff} = D - c.c - \phi$

$= 130 - 15 - 10$

eff depth, $\boxed{d_{eff} = 105\,mm}$

Step 3:-

Design Load Calculation:-

Self weight of slab = Tks × ut wt

$= 0.13 \times 25000$

$= 3250\ N/m^2$

assume, floor finish = 50 mm (Tk = Thick)

wt of F.F $= 0.05 \times 22000$

$= 1100\ N/m^2$

Live Load (L.L) = $3\ KN/m^2$

$= 3000\ N/m^2$

Total service load = $7350\ N/m^2$

Load factor = 1.5

$$\boxed{\begin{aligned} &\text{Ultimate Design Load} = 1.5 \times 7350 \\ &w_u = 11025\ N/m^2 \end{aligned}}$$

Step 4:-

moment Calculation:-

for square simply supported slab

ulti- mt, $\boxed{M_u = \dfrac{w_u L^2}{24}}$

$$= \frac{11025 (4)^2}{24} = 7350 \text{ N-m}$$

Ult. mt.

$$\boxed{Mu = 7.35 \times 10^6 \text{ N.mm}}$$ ✓

Limiting con balancing mt capacity of slab,

$$Mu, lim = Ru\, bd^2$$

from IS 456: 200, ANNEX G

clause: G-1.1, C

$$Mu, lim = 0.36 \frac{xu\,max}{d} \left(1 - 0.42 \frac{xu\,max}{d}\right) bd^2 fck$$

$$Mu, lim = Ru\, bd^2$$

$$= 2.761 \times (1000)(105)^2$$

$$\boxed{Mu, lim = 30.44 \times 10^6 \text{ N-mm}}$$

Since $Mu < Mu, lim$,

the slab is under-reinforced.

Step 5:-

Reinforcement calculation:-

from IS: 456, clause G-1.1 b), page: 96

$$Mu = 0.87 fy\, Ast\, d \left(1 - \frac{Ast\, fy}{bd\, fck}\right)$$

$$7.35 \times 10^6 = 0.87 \times 415 \times Ast \times 105 \left[1 - \frac{Ast \times 415}{1000 \times 105 \times 20}\right]$$

$$Ast^2 - 5600\ Ast + 0.981 \times 10^6 = 0$$

$$\boxed{Ast_{req} = 202\ mm^2}$$

Minimum steel Area,

Ast min = 0.12 % cross section.

$$Ast, min = \frac{0.12}{100} \times 1000 \times 130$$

$$\boxed{Ast, min = 156\ mm^2} < Ast = 202$$

$$\therefore Ast_{req} = 202\ mm^2$$

Use 8 mm dia bars,

$$\text{spacing} = \frac{1000 \times \text{Area of one steel bar}}{\text{steel required}}$$

$$Sv = \frac{1000 \times \frac{\pi \times 8^2}{4}}{202}$$

$$\boxed{Sv = 247\ mm.}$$

⇒ hence use 8 mm φ bars @ 240 mm c/c both ways.

(Q). A R.F concrete slab 5m x 5m is simply supported along the four edges and is R.F with 10mm dia. Fe 415 steel bars @ 150 mm c/c both ways. The average effective depth of the slab is 100 mm and the overall depth of the slab is 130 mm. The slab carries a flooring of 50 mm Thick having unit weight of 22 kN/m3. Determine the max. permissible service load. If M20 concrete is used.

Sol:-

$$\text{Area of Steel}, A_{st} = \frac{1000 \times \text{Area of 1 steel}}{\text{Spacing}}$$

$$A_{st} = \frac{1000 \times \frac{\pi \times 10^2}{4}}{150}$$

$$\boxed{A_{st} = 523.6 \ mm^2/m}$$

moment calculation:-

from IS 456 clause G 1.1 b).

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

$$= 0.87 \times 415 \times 523.6 \times 100 \left[1 - \frac{523.6}{1000 \times 100 \times 20}\right]$$

[∵ for 1m length slab & d = 100mm already as given]

$$\boxed{M_u = 16.85 \times 10^6 \text{ N-mm}}$$

Limiting Moment Calculation:-

from IS 456 clause G-1.1 c).

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

$$R_u = 0.36 \frac{x_{u\,max}}{d}\left(1 - 0.42 \frac{x_{u\,max}}{d}\right) f_{ck}$$

$R_u = ?$

for Fe 415 steel

$\frac{x_{u\,max}}{d} = 0.479$ (from SP-16, pag 9)

$$R_u = 0.36 \times 0.479 (1 - 0.42 \times 0.479) 20$$

$$\boxed{R_u = 2.761}$$

$$M_{u,lim} = R_u\, bd^2$$

$$= 2.761 \times 1000 \times 100^2$$

$$\boxed{M_{u,lim} = 27.61 \times 10^6 \text{ N-mm}}$$

since $M_u < M_{u,lim}$

the slab is under-reinforced.

$m_u = M_u = 16.85 \times 10^6$ N-mm

$m_u = 16.85$ KN-m

Service load calculation:-

for squar simply supported slab

$$\boxed{M_u = \frac{W_u L^2}{24}}$$

$$16.85 = \frac{W_u (5)^2}{24}$$

$$W_u = \frac{16.85 \times 24}{5^2}$$

$$\boxed{W_u = 16.176 \ KN/m^2}$$

Service load $= 16.176/1.5$

$= 10.78 \ KN/m^2$

D.L of slab $= 0.13 \times 25$

$= 3.25 \ KN/m^2$

note [for conrete slab ut wt $= 25 \ KN/m^3$).

D.L of finishing $= 0.05 \times 22$

$= 1.1 \ KN/m^2$

Total D.L $= 3.25 + 1.1$

$= 4.35 \ KN/m^2$

permissible service load $= 10.78 - 4.35$

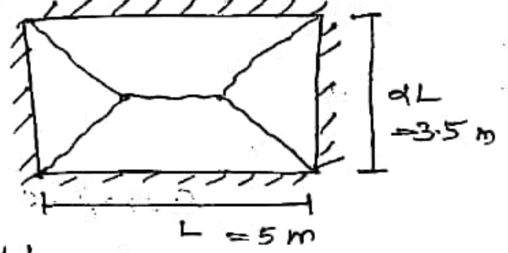
$= 6.43 \ KN/m^2$ Ans.

③. A rectangular slab 3.5×5m in size, simply supported @ the edges. The slab is expected to carry a service live load of 3 kN/m^2 and a floor finishing load of 1 kN/m^2. Use M20 concrete and Fe 415 steel. Design the slab if a). it is isotropically R.F, b). If it is orthotropically R.F with $\mu = 0.75$

Sol:-

Data given :-

long span L = 5 m.



Short span length = αL = 3.5

$$\boxed{\alpha = \frac{3.5}{5} = 0.7}$$

Step1:-

Effective depth calculation:-

from IS 456, clause 24.1

⇒ span to over all depth ratio for simply supported slab is,

$$\frac{L}{D} = 35$$

$$D = \frac{3500}{35} = 100 \text{ mm}$$

Over all depth required $D_{req} = 100$ mm.

hence provide over all depth of slab

$D_{pro} = 100$ mm.

assume clear cover = 15 mm.

Using 10 mm dia bars.

Then effective depth = d_{eff} = D − c.c − φ

= 100 − 15 − 10

$\boxed{d_{eff} = 75\ mm}$

step 2:-

Design Load Calculation:-

Dead wt of slab = 0.1 × 25 = 2.5 kN/m²

Dead wt of flooring = 1.0 kN/m²

L.L = 3.0 kN/m²

Total service load = 6.5 kN/m²

ultimate design Load = 1.5 × 6.5

$\boxed{W_u = 9.75\ kN/m^2}$

a). Isotropically Reinforced slab:- (μ=1).

Step 3:-

moment calculation:-

for ▭lar Isotropically R.F. slab

ulti. moment $M_u = \frac{W_u \alpha^2 L^2}{24}\left[\sqrt{3+\mu\alpha^2} - \alpha\sqrt{\mu}\right]^2$

μ=1, ∴ $M_u = \frac{9.75\,(0.7^2)\,(5^2)}{24}\left[\sqrt{3+(0.7)^2 \cdot 1} - 0.7\sqrt{1}\right]^2$

$\boxed{\begin{aligned} M_u &= 6.791\ kN\text{-}m/m \\ &= 6.791 \times 10^6\ N\text{-}mm/m \end{aligned}}$

step 4:-

Reinforcement Calculation:-

from IS 456, clause G-1.1, page: 96

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

$$6.791 \times 10^6 = 0.87 \times 415 A_{st} \times 75 \left[1 - \frac{A_{st} \times 415}{1000 \times 75 \times 20}\right]$$

$$A_{st}^2 - 3614 A_{st} + 906459 = 0$$

$$\boxed{A_{st\,req} = 272 \text{ mm}^2}$$

spacing calculation:-

use 8 mm ϕ bars.

$$\text{Spacing} = \frac{1000 \times \frac{\pi \times 8^2}{4}}{272}$$

$$\boxed{S_v = 185.5 \text{ mm}}$$

$\Rightarrow$ hence provide 8 mm ϕ bars @ 175 mm c/c both ways.

b). Orthotropically R.F Slab:-

$\mu = 0.75$

for $\square$lar orthotropically R.F slab

ulti. mt $m_u = \frac{w_u \alpha^2 L^2}{24}\left[\sqrt{3 + \mu\alpha^2} - \alpha\sqrt{\mu}\right]^2$

$$= \frac{9.75 (0.7)^2 (5)^2}{24}\left[\sqrt{3 + 0.75 (0.7)^2} - 0.7\sqrt{0.75}\right]^2$$

$$= 8.541 \text{ kN-m/m}$$

$$\boxed{M_u = 8.541 \times 10^6 \text{ N-mm/m}}$$

Step 5 :-

Reinforcement calculation

from IS 456 clause G-1-1

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

$$8.541 \times 10^6 = 0.87 \times 415 \times A_{st} \times 75 \left[1 - \frac{A_{st} \times 415}{1000 \times 75 \times 20}\right]$$

$$A_{st}^2 - 3614.4 A_{st} + 1140000 = 0$$

$$\boxed{A_{st} = 350 \text{ mm}^2}$$

Spacing calculation:-

In shorter direction:-

Use 8mm ϕ bars,

$$\text{spacing} = \frac{1000 \times \frac{\pi \times 8^2}{4}}{350} = 144 \text{ mm}$$

$\Rightarrow$ hence provide 8mm ϕ bars @ 140mm c/c in shorter direction.

In longer direction:-

use 8mm ϕ bars,

$$\text{spacing} = \frac{1000 \times \frac{\pi \times 8^2}{4}}{350 \times 0.75} = 192\text{mm}$$

$\Rightarrow$ hence provide 8mm ϕ bars @ 190mm c/c spacing in longer direction.

④. A rectangular slab 3.5 × 4.5m is isotropically Reinforced with 8mm dia. bars spaced @ 150 mm bothways. The average effective depth may be taken as 80 mm and the total depth of the slab is 100 mm. If Fe 415 steel and concrete of grade M 20 are used. deter.. the safe service Live load. The dead load of floor finishing may be assumed as 1.5 KN/m².

Sol:

given data:-

long span length $L = 4.5$m

Short span length $\alpha L = 3.5$m

$$\boxed{\alpha = 3.5/4.5 = 0.778}$$

since the slab is isotropically Reinforced,

$$\boxed{\mu = 1}$$

Area of Steel calculation:-

$$A_{st} = \frac{1000 \times \text{Area of single steel bar}}{\text{spacing}}$$

$$= \frac{1000 \times \frac{\pi \times 8^2}{4}}{150}$$

$$\boxed{A_{st} = 335\ mm^2/m}$$

ultimate moment calculation:-

from IS 456. clause G-1.1

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

$$= 0.87 \times 415 \times 335 \times 80 \left[1 - \frac{335 \times 415}{1000 \times 80 \times 20}\right]$$

$$= 8.835 \times 10^6\ N-mm$$

$$\boxed{M_u = 8.835\ kN-m}$$

The yield moment of the slab is,

$$\boxed{m_u = \frac{W_u \alpha^2 L^2}{24}\left[\sqrt{3+\alpha^2} - \alpha\right]^2}$$

$$W_u = \frac{24\, m_u}{\alpha^2 L^2 \left[\sqrt{3+\alpha^2} - \alpha\right]^2}$$

Service load calculation:-

ulti service load,

$$w_u = \frac{24 \times 8.835}{(0.778)^2 \times (4.5)^2 \left(\sqrt{3 + (0.778)^2} - 0.778\right]^2}$$

$$\boxed{w_u = 13.76 \text{ kN/m}^2}$$

total service load $= 13.76 / 1.5$

$= 9.17 \text{ kN/m}^2$

D.L of the slab $= 0.1 \times 25 = 2.5 \text{ kN/m}^2$

D.L of floor finishing $= 1.5 \text{ kN/m}^2$

Service L.L $= 9.17 - (2.5 + 1.5)$

$\boxed{\text{safe service Live load} = 5.17 \text{ kN/m}^2}$ // Ans.

Assumptions of yield lines:-

* The slab is under reinforced, so that there is tension failure.
* The yield lines are straight lines.
* Elastic deformation is negligible compared to plastic deformations
* After collapse mechanism is formed each of the segment of the slab may be treated as rigid body and the entire rotation is assumed to take place along the yield line.

Design of RC Structures II

UNIT - II

Building Frames

Framed structure:-

* A building frame may contain a no. of bays, and may have several storeys.
* A multi-storyed, multi-panelled frame is a complicated statically indeterminate structure.
* It consists of no. of beams and columns built monolithically.
* In framed structure floors and the walls are supported on beams which transmit the loads to the columns.
* Building frame is subjected to both vertical and horizontal loads.

⇒ Vertical Loads:

① Dead load → self wt of beams, slabs, columns etc.

② Live load.

⇒ horizontal loads:-

① Wind forces.

② Earth quake forces.

⇒ Practically all major buildings are framed structures. The building frames are highly Indeterminate structures

upto 2 storeys - Load bearing wall construction

upto 5 storeys - Approx. analysis procedures are useful.

exceeding 5 storeys - computer based analysis procedures are useful.

⇒ Building frames analysis:-

In earlier period to analyse the no. of storey of the building frames by * two cycle moment distribution method (or)

* substitute frame method

Substitute frame :-

* What ever be the no. of storey it is customary (and permissible) to analyse only a part of a frame, termed as substitute frame.

(or)

* A simple method of analysis, accurate enough for practical purpose, is used by analysing a small portion of the frame, called substitute frame rather than analysis of the whole frame.

⇒ It is based on the assumption that the moments in one floor have negligible effect of the moments of the floors above and below.

⇒ This substitute frame may be moved from floor to floor.

Analysis for vertical loads (or) gravity loads In substitute frame method:-

* A building structure may be assumed to be consisting of two sets of plane frames crossing each other at right angles.

* The vertical members are common to both these sets of frames.

* Each set of frames are analysed separately.

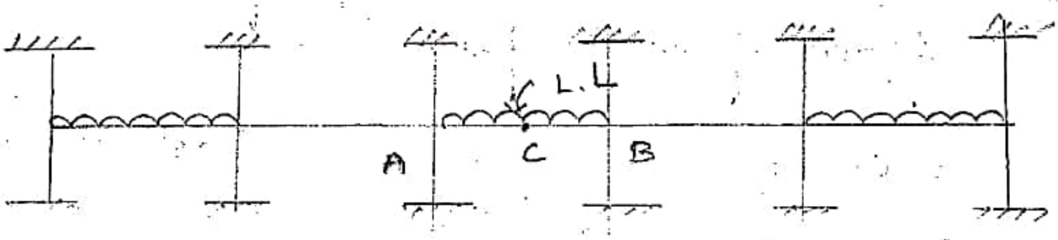
* Moments in the vertical members occur in two planes, the stresses in columns should be found for moments acting in two planes simultaneously and the corresponding vertical loads.

~~⇒ The beam should be loaded with Live Loads as follow~~

1) <u>Maximum Bending moments in beams:-</u>

The beam should be loaded with live loads as follows for maximum effects.

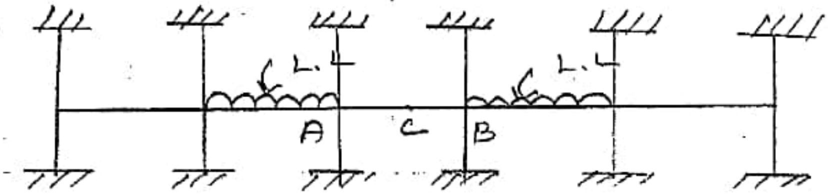
a) <u>For max. positive B.M @ mid span of C</u>



@ mid pt C of a span AB, the loads should be placed on the span and on alternative span as show in fig.

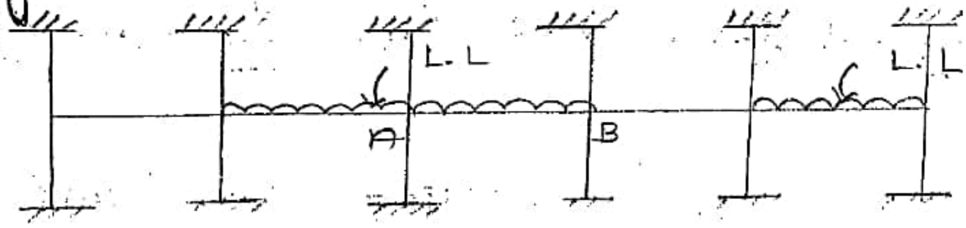
b). <u>For max. -ve B.M @ mid span of C:-</u>

@ the mid point C of a span AB, the span AB should be unloaded while load should be placed on spans adjacent to the span under consideration. as shown in fig.



c). For max. -ve B.M @ support A:-

for max -ve B.M @ support A, Loads should be placed on the two spans adjacent to the support, as shown in fig.

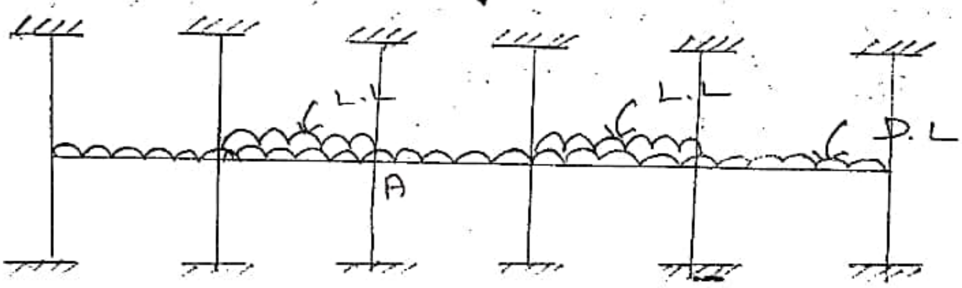


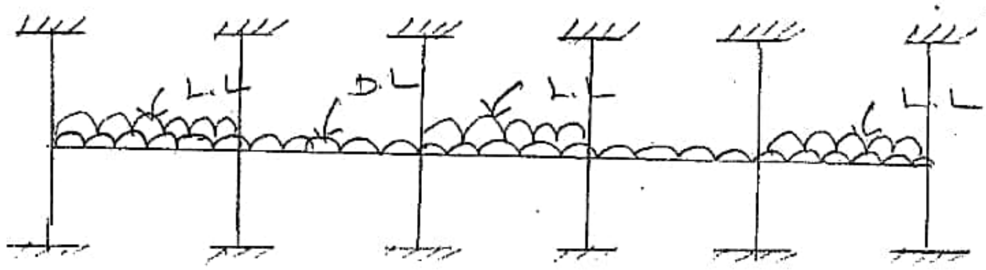
⇒ The B.M due to Dead Loads (D.L) are found separately.

⇒ The B.M for D.L and L.L are then added and the beam is designed.

ii). Max. B.M in columns:-

max. B.M in column @ A when the alternate spans are loaded. They are 2 sets of alternate loading as shown in fig.

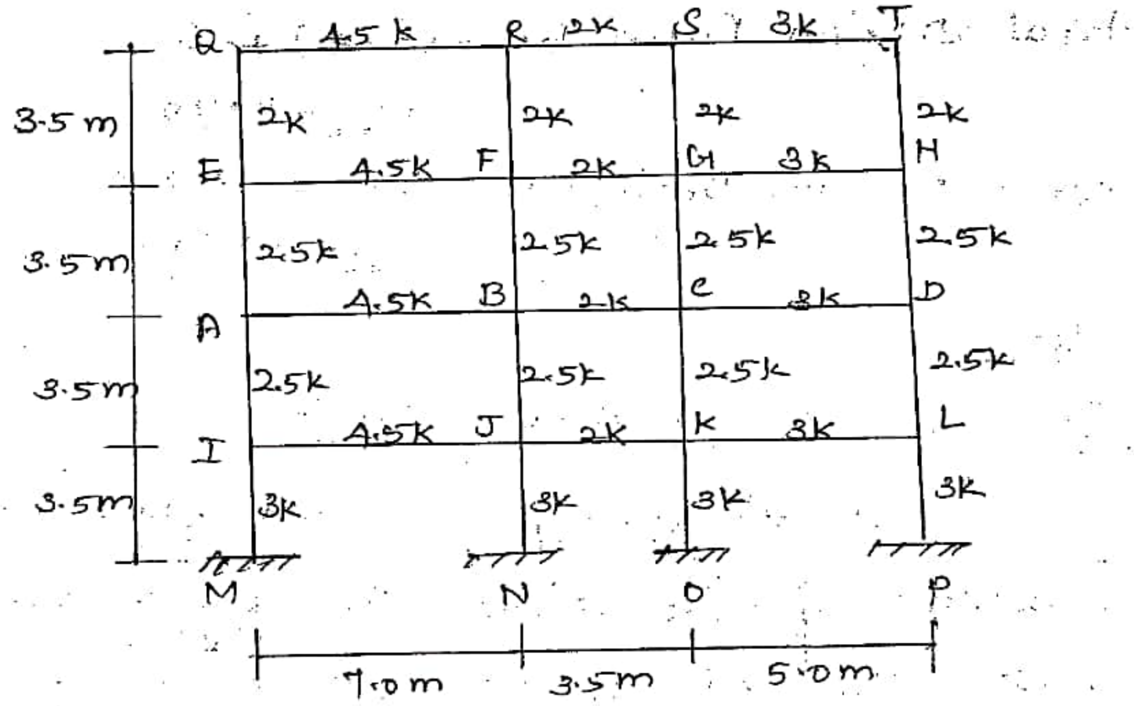




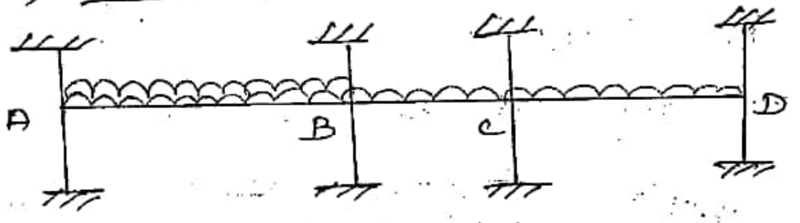
* The corresponding axial loads are found.
* The column is designed to resist the stresses provided by every combination of axial load and the corresponding mt.

① In multi-storey building, the frame shown in fig. are spaced @ 4 m intervals. Analysis the beam AB & BC, CD for mid span +ve BM taking L.L of 4 KN/m² and DL as 3 KN/m², 3.25 KN/m² and 2.75 KN/m² for the panel AB, BC & CD respectively. the self wt of the beam may be taken as,
beams of 7m span = 5 KN/m,
beams of 50m span = 3.5 KN/m,
beams of 3.5 m span = 2.5 KN/m. The relative stiffness of the members are marked on the fig. it self.

(4)



Sol:- i) max +ve B.M in mid-span of AB



Step1:- Calculation of Load

The frames are spaced @ 4m interval

the L.L transfored from the floor.

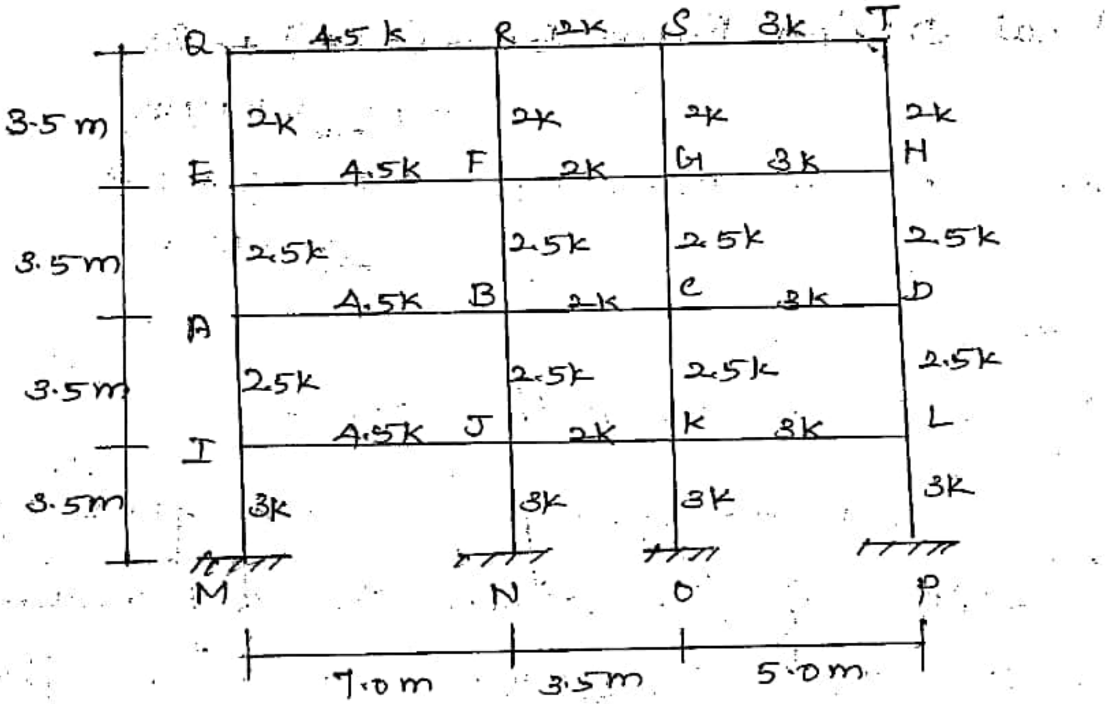
= L.L × spacing

= 4×4 = 16 kN/m

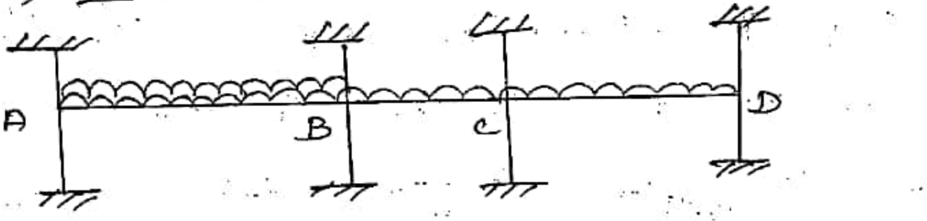
Total D.L on a beam = The D.L from the floors + D.L due to self wt of the beam.

total D.L on beam AB = (D.L × spacing) + self wt of beam

= (3×4) + 5 = 17 kN/m



Sol:- 1) max +ve B.M in midspan of AB



Step1:- Calculation of Load

The frames are spaced @ 4m interval the L.L transfored from the floor.

= L.L x spacing

= 4 x 4 = 16 kN/m

Total D.L on a beam = The D.L from the floors + D.L due to self wt of the beam.

total D.L on beam AB = (D.L x spacing) + self wt of beam

= (3 x 4) + 5 = 17 kN/m

for span CD total load = D.L + L.L

$= 14.5 + 16 = 30.5$ kN/m

$MF_{AB} = -\frac{wl^2}{12} = \frac{-33 \times 7^2}{12} = -134.75$ kN.m

$MF_{BA} = \frac{wl^2}{12} = 134.75$ kN.m

$MF_{BC} = -\frac{wl^2}{12} = -\frac{15.5 \times 3.5^2}{12} = -15.828$ kN.m

$MF_{CB} = \frac{wl^2}{12} = 15.823$ kN.m

$MF_{CD} = -\frac{wl^2}{12} = -\frac{(30.5) \times 5^2}{12} = -63.542$ kN.m

$MF_{DC} = \frac{wl^2}{12} = 63.542$ kN.m

Step 4:-

Distribution Factor (D.F) calculation:-

joint	members	Relative stiffness (K)	Σk	$D.F = \frac{k}{\Sigma k}$
A	AE	2.5k	9.5k	0.263
	AB	4.5k		0.474
	AI	2.5k		0.263
B	BF	2.5k	11.5k	0.217
	BC	2k		0.174
	BJ	2.5k		0.217
	BA	4.5k		0.391

C	CB CD CG CK	2k 3k 2.5k 2.5k	10k	0.2 0.3 0.25 0.25
D	DC DH DL	3k 2.5k 2.5k	8k	0.375 0.3125 0.3125

Step 5:-

Formation of mt distribution table:-

joint	A	B		C		D
Member	AB	BA	BC	CB	CD	DC
D.F	0.474	0.392	0.174	0.20	0.30	0.375
F.E.M	−134.75	+134.75	−15.83	+15.83	−63.542	+63.542
Balance	+63.87	−46.62	−20.692	+9.542	+14.314	−23.823
carry over	−23.31	+31.94	+4.771	−10.346	−11.9115	+7.157
Balance	+11.049	−14.39			+6.680	−2.684
Final mts	−83.141	+105.68			−54.45	+44.192

Balance:- (1st cycle)

$AB = (-134.75 \times 0.474) = -63.87$

$BA = (134.75 - 15.83) \times 0.392 = 46.62$

$BC = (134.75 - 15.83) \times 0.174 = 20.692$

$CB = (15.83 - 63.542) \times 0.2 = -9.542$

$CD = (15.83 - 63.542) \times 0.3 = -14.314$

Carry over:-

$AB = -\frac{46.62}{2} = -23.31$

$BA = +\frac{63.87}{2} = +31.94$

$BC = \frac{9.542}{2} = 4.771$

$CB = -\frac{20.692}{2} = -10.346$

$CD = \frac{-23.823}{2} = -11.9115$

$DC = +\frac{14.314}{2} = +7.157$

Balance :- (2nd cycle)

$AB = -23.31 \times 0.474 = -11.049$

$BA = (+31.94 + 4.77) \times 0.392 = +14.39$

$CB = -(10.346 + 11.9115) \times 0.2 = -4.45$

$CD = -(10.346 + 11.9115) \times 0.3 = -6.677$

$DC = 7.157 \times 0.375 = +2.684$

Final mts:-

$AB = -134.75 + 63.87 - 23.31 + 11.049$

$= -83.141$ kN.m

$BA = 134.75 - 46.62 + 31.94 - 14.39$

$= +105.68$ kN.m

ii). For max +ve B.M in mid span of CD :-

Let us analyse second floor ABCD substitute frame shown in fig. for max mid span +ve B.M in CD, the L.L is placed on AB & CD. The DLL placed on all the spans.

It is same in previous calculation method.

Final mts:- in

$CD = -63.542 + 14.314 - 11.9115 + 6.680$

$= -54.45$ kN.m

$DC = 63.542 - 23.823 + 7.157 - 2.684$

$= +44.20 \ kN \cdot m$

Step 6:- To Draw B.M Diagram

a). for span AB:-

Final Fixed end moments

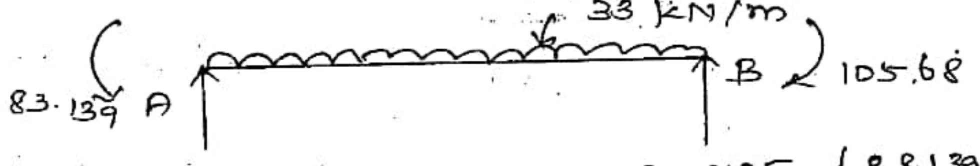
$MFAB' = -83.139$

$MFBC' = +105.68$

free BM @ mid span of $AB = \frac{wl^2}{8}$

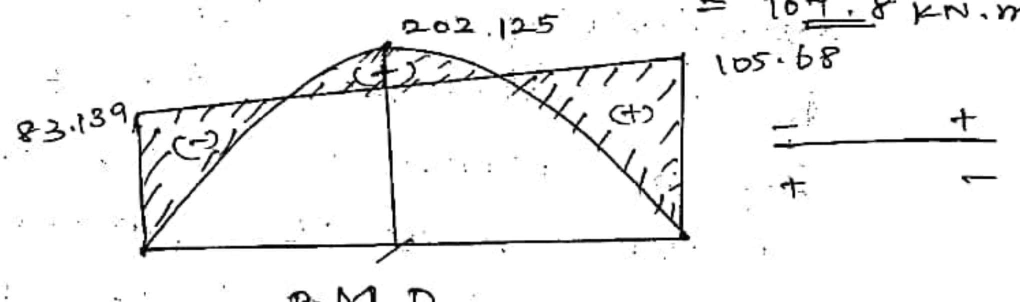
$= \frac{(D.L + L.L)l^2}{8}$

$= \frac{33 \times 7^2}{8} = 20.2125 \ kN\text{-}m$



Net B.M @ the centre of $AB = 20.2125 - \left(\frac{83.139 + 105.68}{2}\right)$

$= 107.8 \ kN.m$



B.M.D

b). for span CD:-

Final Fixed end moments

$MFCD' = -54.45 \ kN\text{-}m$

$MFDC' = +44.192 \ kN.m$

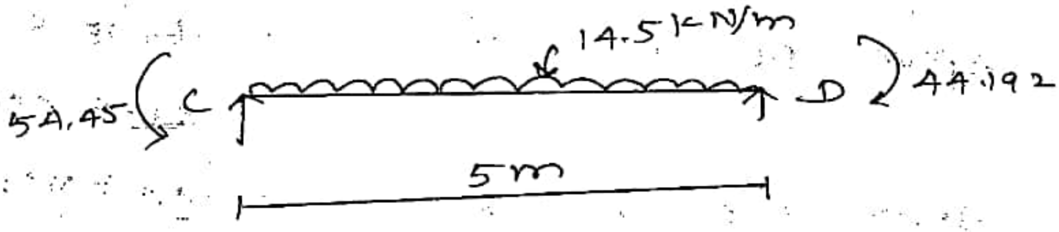
iii). Max +ve B.M in mid span of BC:-

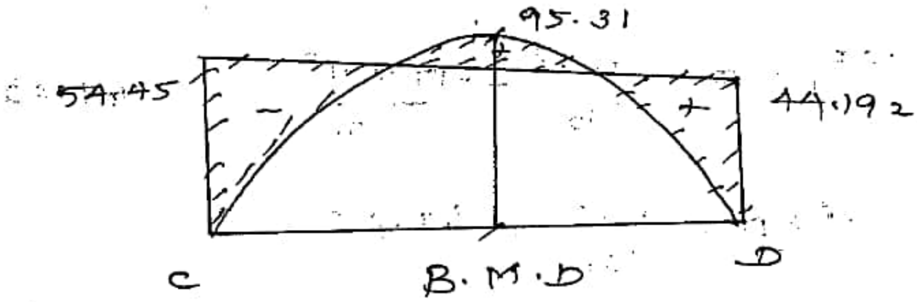
free B.M @ mid span of CD $= \frac{wl^2}{8}$

$= \frac{(14.5+16) \times 5^2}{8} = 95.31$ KN.m

Net B.M @ the centre of CD $= 95.31 - \frac{(54.45 + 44.192)}{2}$

$= 45.98$ KN.m





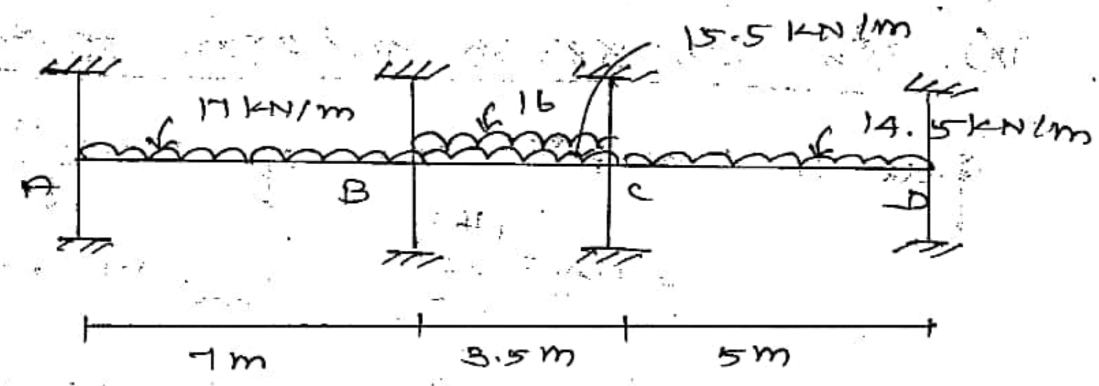
B.M.D

iii). Max (+ve) B.M in mid span of BC:-

Step 1:-

Formation of substitute frame:-

for max +ve B.M in mid span of BC. the L.L is placed on BC and D.L placed on all the spans.



Step 8:-

Fixed end mt calculation:-

span AB total load $w = 17 kN/m$

span BC " " $= 16 + 15.5$

$= 31.5 kN/m$

span CD " " $= 14.5 kN/m$

$$MF_{AB} = -\frac{wl^2}{12} = -\frac{17 \times 7^2}{12} = -69.42$$

$$MF_{BA} = \frac{wl^2}{12} = 69.42$$

$$MF_{BC} = -\frac{wl^2}{12} = -\frac{31.5 \times 3.5^2}{12} = -32.16$$

$$MF_{CB} = \frac{wl^2}{12} = 32.16$$

$$MF_{CD} = -\frac{wl^2}{12} = -\frac{14.5 \times 5^2}{12} = -30.21$$

$$MF_{DC} = \frac{wl^2}{12} = 30.21$$

Step 9:-

Distribution Factor (DF) calculation:-

joint	members	Relative stiffness K	ΣK	$D.F = \frac{K}{\Sigma K}$
A	AE AB AI	2.5k 4.5k 2.5k	9.5k	0.263 0.474 0.263
B	BF BC BJ BA	2.5k 2k 2.5k 4.5k	11.5k	0.217 0.174 0.217 0.391
C	CB CD CG CK	2k 3k 2.5k 2.5k	10k	0.2 0.3 0.25 0.25
D	DC DH DL	3k 2.5k 2.5k	8k	0.375 0.3125 0.3125

Step 10:-

Formation of mt distribution Table:-

joint	A	B		C		D
member	AB	BA	BC	CB	CD	DC
D.F	0.474	0.392	0.174	0.2	0.3	0.375
FEM	−69.42	+69.42	−32.16	+32.16	−30.21	+30.21
Balance	+32.9	−14.6	−6.48	−0.34	−0.585	−11.33
CO		+16.45	−0.2	−3.24	−5.66	
Balance			−2.83	−1.78		
Final mts			−41.67	+30.31		

Balance 1st cycle:-

AB = − 69.4 × 0.474 = − 32.9

BA = (+69.42 − 32.16) × 0.392 = 14.6

$BC = (69.42 - 32.16) \times 0.174 = 6.48$

$CB = (32.16 - 30.21) \times 0.2 = 0.39$

$CD = (32.16 - 30.21) \times 0.3 = 0.585$

$DC = 30.21 \times 0.375 = 11.33$

Balance (2nd cycle)

$BC = (-0.2 + 16.45) \times 0.174 = 2.83$

$CB = (-3.24 - 5.66) \times 0.2 = 1.78$

Step 11:-

to Draw B.M.D of BC :-

Final end moments:-

$M_{FBC}' = -41.67$ kN.m

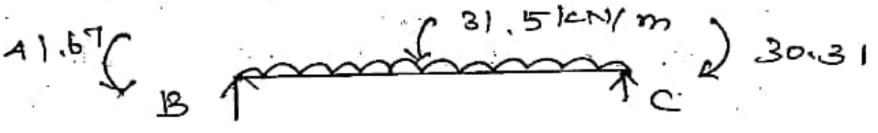
$M_{FCB}' = +30.31$ kN.m

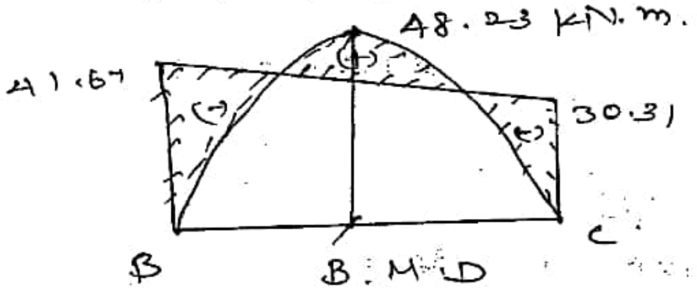
free B.M @ centre of span BC $= \frac{wl^2}{8}$

$= \frac{31.5 \times 3.5^2}{8} = 48.23$ kN.m

Net B.M @ centre BC $= 48.23 - \frac{41.67 + 30.31}{2}$

$= 12.24$ kN.m





B.M.D

—	+
+	—

Analysis of frames subjected to horizontal Forces

* A building frame is subjected to horizontal forces due to Wind pressure & seismic effects.

* These horizontal forces cause axial forces in columns and bending moment in all the members of the frame.

The following approximate methods are commonly used for the analysis of building frames subjected to lateral forces.

⇒ Portal method

⇒ Cantilever method.

⇒ Factor method.

Portal method :-

* It is more suited for low rise building frames.

In this method, following assumptions are made,

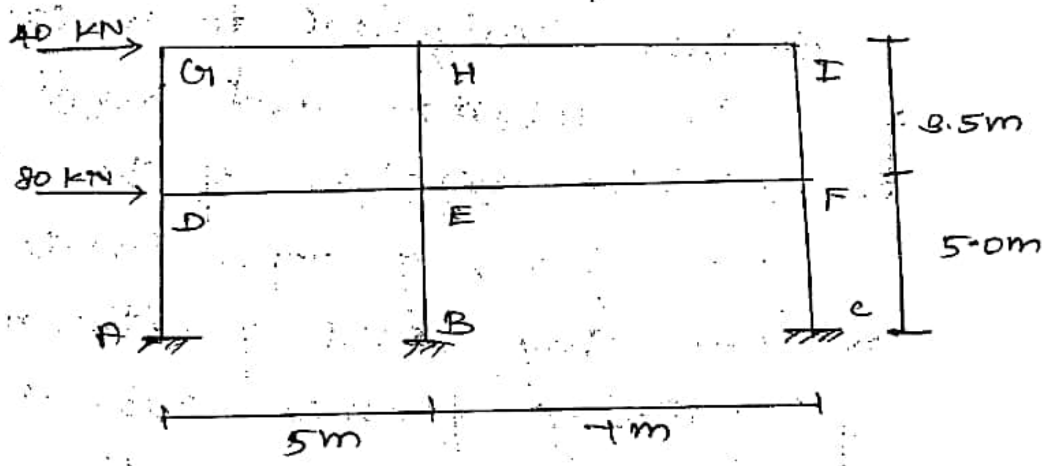
①. The point of contraflexure is located at the centre of each beam.

②. The point of contraflexure is located @ the 'c' of each column.

③. horizontal shear taken by each interior column is double the horizontal

Shear taken by each of exterior column.

② Analyse the building frame, subjected to horizontal forces, as shown in fig. Use portal method. Sketch the B.M.D.



⇒ Point of contraflexure (P.O.C) will be assumed to occur @ mid span / mid height of all the beams / columns.

Step 1:-

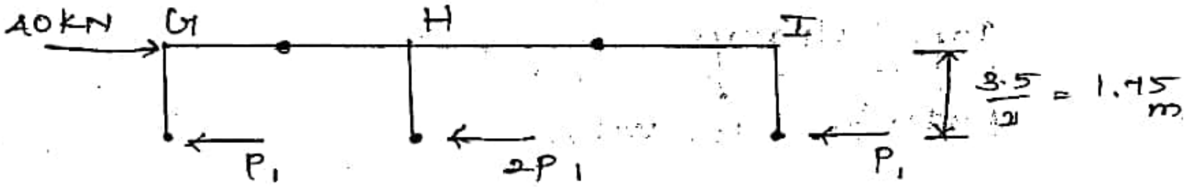
column shear (or) horizontal shear calculation:-

Horizontal shear in interior column is assumed to be twice that in the exterior columns.

$P_1, P_2 \ldots$ = horizontal shear in exterior columns of a storey.

$2P_1, 2P_2 \ldots$ = shear in interior columns of the respective storey.

For top storey:-



sum of shear in columns = Total external shear

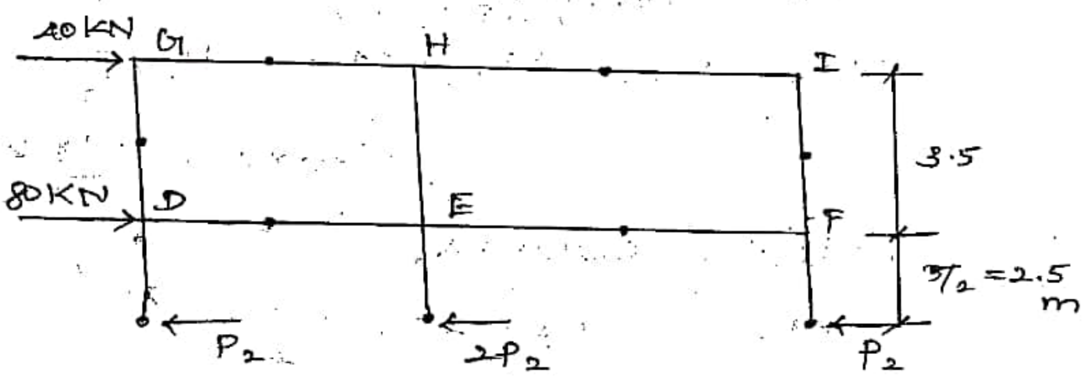
$\therefore P_1 + 2P_1 + P_1 = 40$ KN

$4P_1 = 40$

$\boxed{P_1 = 40/4 = 10 \text{ KN}}$

$\boxed{\therefore 2P_1 = 20 \text{ KN}}$

For the bottom storey:-



sum of shear in columns = total external shear

$P_2 + 2P_2 + P_2 = 40 + 80$

$4P_2 = 120$

$\boxed{P_2 = 30 \text{ KN}}$

$\boxed{2P_2 = 60 \text{ KN}}$

Step 2 :- moments @ the ends of columns :-

Top storey :-

Exterior columns,

$M_{GD} = M_{DG} = M_{IF} = M_{FI} = P_1 \times \frac{3.5}{2}$

$= 10 \times \frac{3.5}{2} = \underline{\underline{17.5}}$ kN.m (↺)

Interior columns,

$M_{HE} = M_{EH} = 2P_1 \times \frac{3.5}{2} = 20 \times 1.75 = \underline{\underline{35}}$ kN.m (↺).

For the bottom storey :-

Exterior columns

$M_{DA} = M_{AD} = M_{FC} = M_{CF} = P_2 \times \frac{5}{2}$

$= 30 \times 2.5 = \underline{\underline{75}}$ kN.m (↺)

Interior columns,

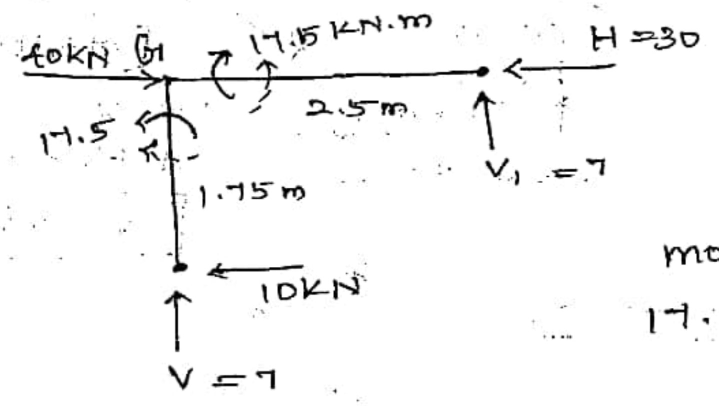
$M_{EB} = M_{BE} = 2P_2 \times \frac{5}{2} = 60 \times 2.5 = 150$ kNm (↺)

Step 3 :- Calculation of mts @ the ends of Beams and Beam shears :-

⇒ Beam shears are evaluated by considering various free bodies bounded by hinges (Σ rotational equilibrium).

While working out the support mts on each member, we have to remember that the support moment plus external moment is zero.

consider joint G,



$\Sigma H = 0;$

$40 - 10 + (-H) = 0$

$\boxed{H = 30\ kN}$ $(\leftarrow)$

moment,

$17.5 = V_1 \times 2.5$

$V_1 = \frac{17.5}{2.5} = 7\ kN\ (\uparrow)$

$\boxed{V_1 = 7\ kN}$

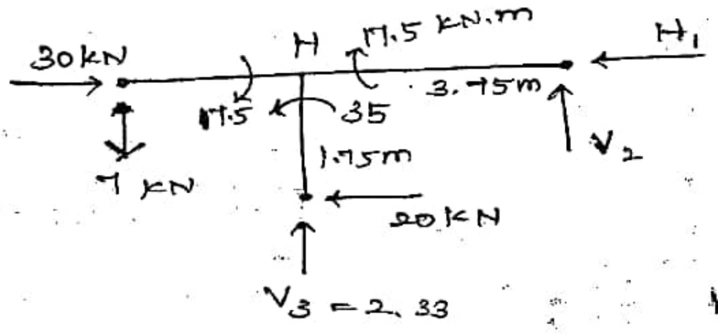
$\Sigma V = 0;$

$V_1 - V = 0$

$V = V_1$

$\boxed{V = 7\ kN}$

consider joint H, :-



$\Sigma H = 0,$

$30 - 20 - H_1 = 0$

$\boxed{H_1 = 10\ kN}$ $(\leftarrow)$

moment,

$17.5 = V_2 \times 3.75$

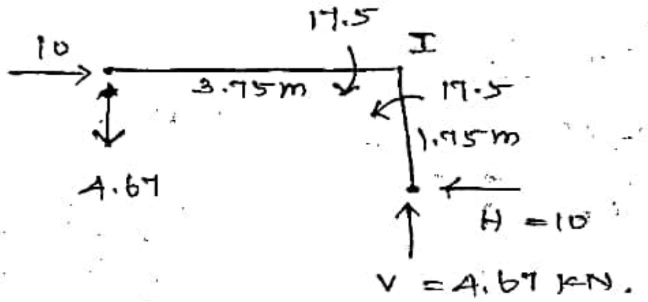
$\boxed{V_2 = 4.67\ kN}$

$\Sigma V = 0$

$-7 + 4.67 + V_3 = 0$

$\boxed{V_3 = 2.33\ kN}$

consider joint I:-



$\Sigma H = 0$

$10 - H = 0$

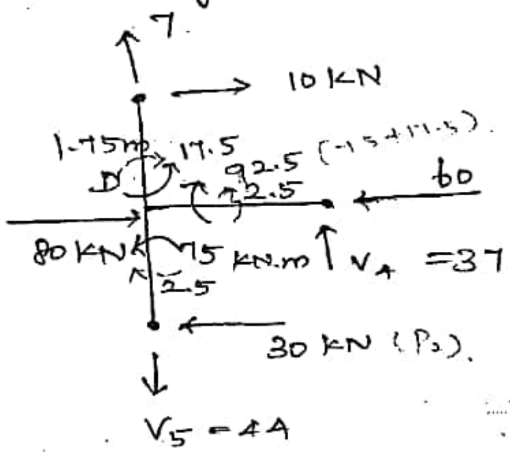
$\boxed{H = 10}$

$\Sigma V = 0$

$4.67 - V = 0$

$\boxed{V = 4.67}$

consider joint D:-



$92.5 = V_A \times 2.5$

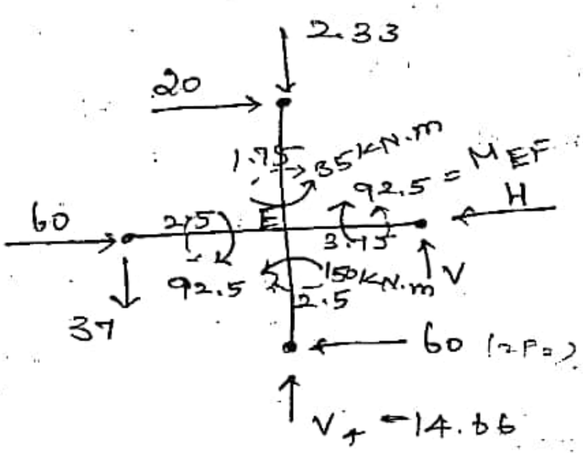
$\boxed{V_A = 37 \text{ KN}}$

$\Sigma V = 0$

$7 + 37 - V_5 = 0$

$\boxed{V_5 = 44 \text{ KN}}$

consider joint E:-



$-35 - 150 + 92.5 + M_{EF} = 0$

$\boxed{M_{EF} = 92.5 \text{ KN.m}}$

$92.5 = V \times 3.75$

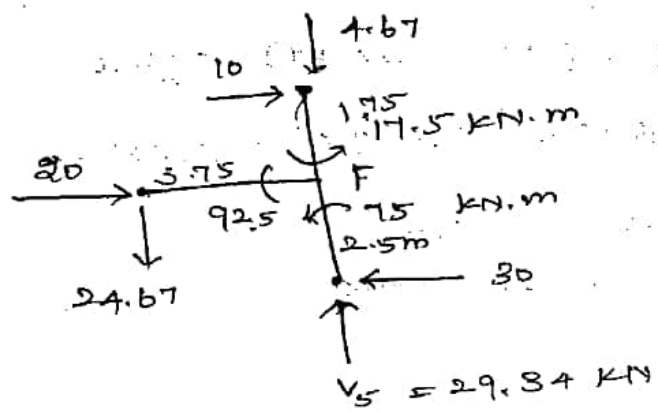
$\boxed{V = 24.67 \text{ KN}}$

$\Sigma V = 0$

$2.33 - V_4 + 37 - 24.67 = 0$

$\boxed{V_4 = 14.66 \text{ KN}}$

consider Joint F.

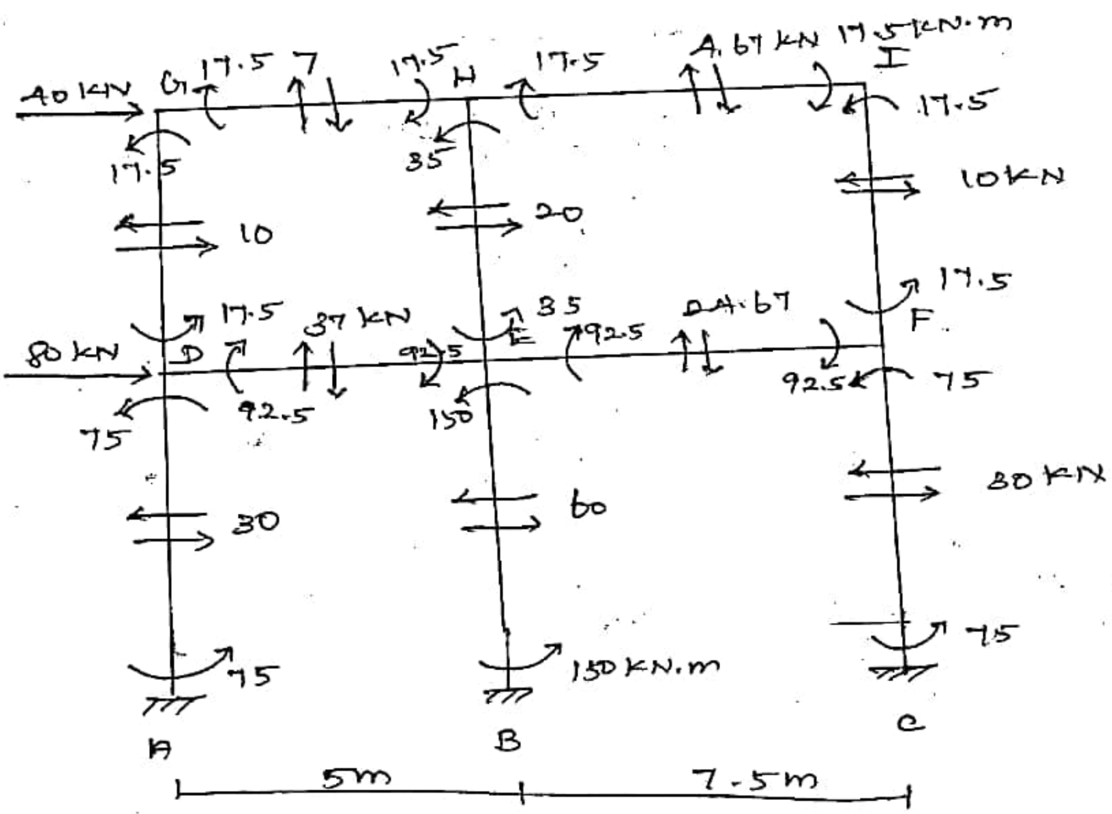


$\Sigma V = 0$

$4.67 - V_5 + 24.67 = 0$

$\boxed{V_5 = 29.34 \text{ kN}}$

Step 4:- calculation of Axial Forces in column:-



Axial Forces in columns:-

P_{GD} = Shear in Beam GH = 7 kN (tension)

$P_{HE} = F_{HG} - F_{HI} = 7 - 4.67 = 2.33$ kN (comp.)

P_{IF} = shear in Beam IH = 4.67 kN (comp.)

P_{DA} = Axial Force in GD + shear in DE

= 7 + 37 = 44 kN (tens.)

$P_{EB} = P_{HE} + (F_{DE} - F_{EE})$

$= 2.33 + (37 - 24.67) = \underline{14.66}$ KN (comp.)

P_{FC} = axial Force in IF + shear in FE

$= 4.67 + 24.67$

$= \underline{29.34}$ kN (comp)

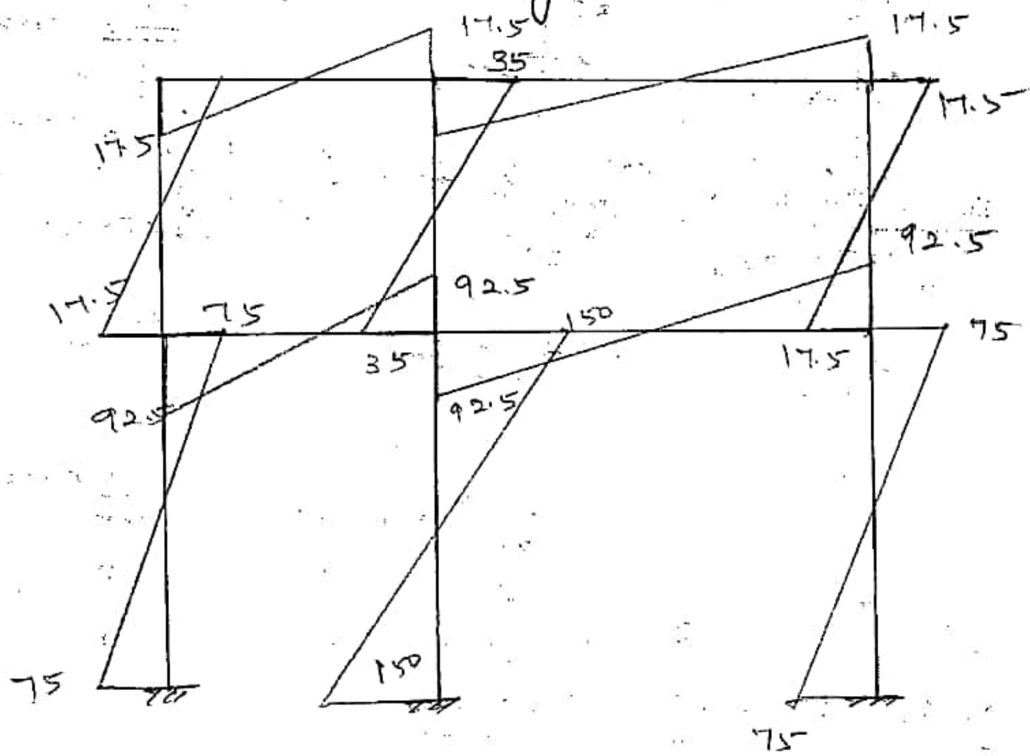
check:-

Total axial F @ the base,

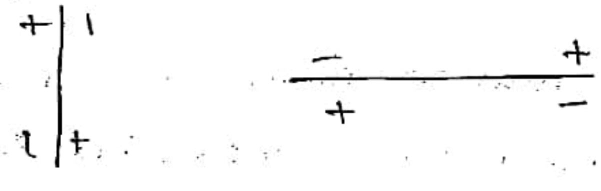
$= +44 - 14.66 - 29.34$

$= 0$ (zero).

Step 5:- B. M Diagram:-



sing convention:-



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Cantilever Methods:-

This method assumes the building frame as a vertical cantilever fixed @ the base and free @ the top and subj. to Lateral Loads. Hence the col.s on the windward side will be in tension and those on the leeward side will be in compression.

Assuming the wind to blow from left to right, the wind load will cause a C.W over turning mt. For equilibrium an equal and opposite mt will have to be developed by the frame. This will be made available by axial forces in the col.s, which will be tensile in the windward col.s & comp. in the leeward col.s and will be of such magnitude as to create an Anti C.W moment required for the equilibrium.

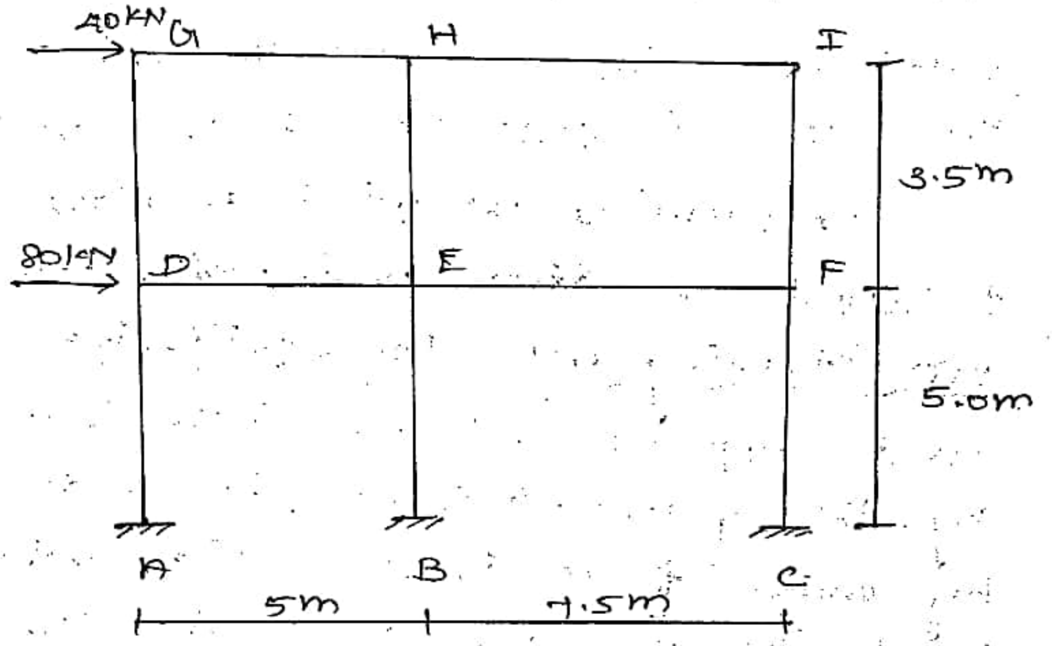
In this method the following assumptions are made in the analysis:-

① There is a P.O.C @ the centre of each beam.

② There is a P.O.C @ the centre of each column.

③ The direct stresses (axial stress) in the columns due to horizontal Forces, are directly proportional to their distance from the centroidal vertical axis of the frame.

② Analyse the frame subj. to horizontal Forces as shown in fig. below by cantilever method assuming that all the col.s have the same area of c/s.



Sol:-

Step 1:-

Location of centroidal axis of the col.s:-

Let the centroidal axis be @ a dist. of $\bar{x}$ from the windward (W.W) col. GDA. Taking mt of areas of the col.s about GDA. The c/s area of all the col.s are assumed to be same area 'A'.

$$\bar{x} = \frac{\Sigma x A}{\Sigma A}$$

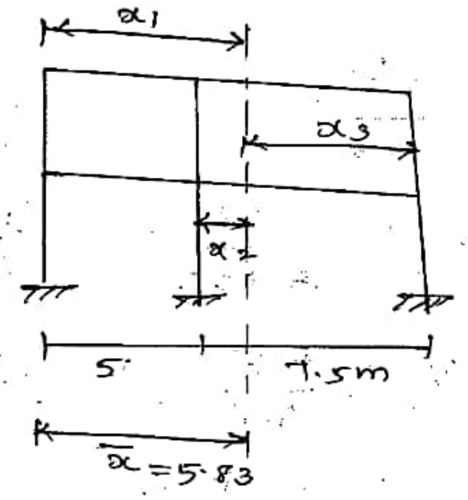
$$= \frac{(0 \times A) + (5 \times A) + (12.5 \times A)}{A + A + A} = \frac{17.5 \cancel{A}}{3 \cancel{A}}$$

$$\boxed{\bar{x} = 5.83 \text{ m}}$$

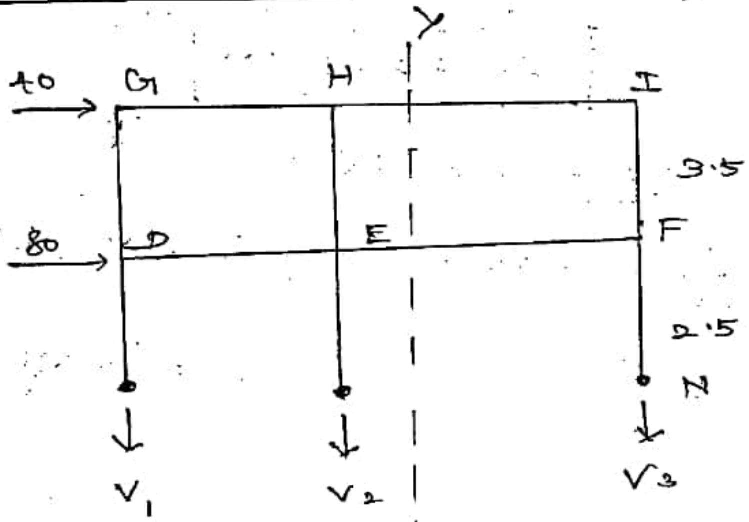
$\therefore x_1 = 5.83\,m$

$x_2 = 5.83 - 5 = 0.83\,m$

$x_3 = 7.5 - 0.83 = 6.67\,m$



Step 2:-
Axial Forces in the col.s of First storey:-



taking mt about 'N' (+↺)

$(40 \times \overset{(3.5+2.5)}{6}) + (80 \times 2.5) - V_1 \times \overset{(5+7.5)}{12.5} - V_2 \times 7.5 = 0$

$$\boxed{240 + 200 - 12.5\,V_1 - 7.5\,V_2 = 0} \quad -(1)$$

Let the axial Force in the col. DA,

$$\boxed{V_1 = V}$$

Since the areas are equal the A.F in the other col.s will be in ~~por~~ proportion to their dist. from the centroidal axis.

$$\frac{V}{x_1} = \frac{V_2}{x_2} = \frac{V_3}{x_3}$$

$$\therefore V_2 = \left(\frac{x_2}{x_1}\right) V = \frac{0.83}{5.83} V \quad (\downarrow).$$

$$V_3 = \left(\frac{x_3}{x_1}\right) V = \left(\frac{6.67}{5.83}\right) V \quad (\uparrow)$$

Sub. V_2 & V_3 in eqn ①

$$\Rightarrow 240 + 200 - 12.5 V - 7.5\left(\frac{0.83}{5.83}\right) V = 0$$

$$\boxed{V = 32.43 \text{ kN}}$$

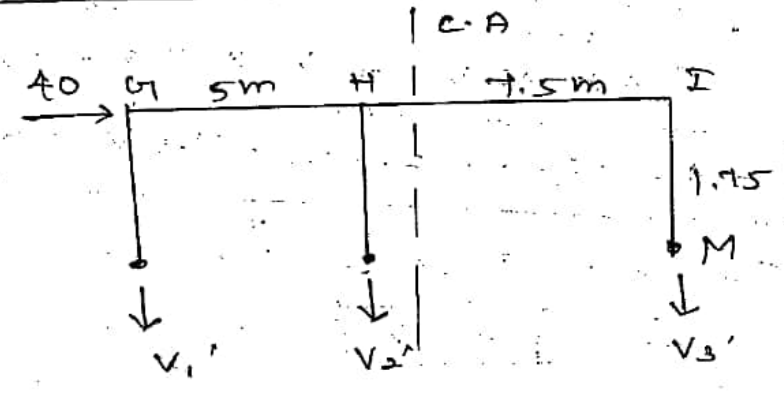
$$V = V_1 = 32.43 \text{kN} \quad (\downarrow).$$

$$V_2 = \frac{0.83}{5.83} \times 32.43 = 4.62 \text{kN} \quad (\downarrow)$$

$$V_3 = \frac{6.67}{5.83} \times 32.43 = 37.102 \text{ kN} \quad (\uparrow).$$

Step 3 :-

Calculation of A.F in the col s of 2nd storey :-



taking mt about M, (+↻)

$$40 \times 1.75 - V_1' \times 12.5 - V_2' \times 7.5 = 0$$

(5+7.5)

$$\boxed{70 - 12.5\,V_1' - 7.5\,V_2' = 0} \quad \text{--- (2)}$$

Let $V_1' = V_1$ = A.F in the col. AF
axial Force:

$$\frac{V_1}{x_1} = \frac{V_2'}{x_2} = \frac{V_3'}{x_3}$$

$$V_2' = \left(\frac{x_2}{x_1}\right) V_1 \quad (\downarrow), \quad = \left(\frac{0.83}{5.83}\right) V_1$$

$$V_3' = \left(\frac{x_3}{x_1}\right) V_1 \quad (\uparrow), \quad = \left(\frac{6.67}{5.83}\right) V_1$$

sub. V_2' & V_3' value in eq (2),

$$70 - 12.5\,V_1 - 7.5\left(\frac{0.83}{5.83}\right) V_1 = 0$$

$$\boxed{V_1 = 5.16\ kN} \ (\downarrow).$$

$$\boxed{V_2' = 0.735\ kN \ (\downarrow)}$$

$$\boxed{V_3' = 5.903\ kN \ (\uparrow)}$$

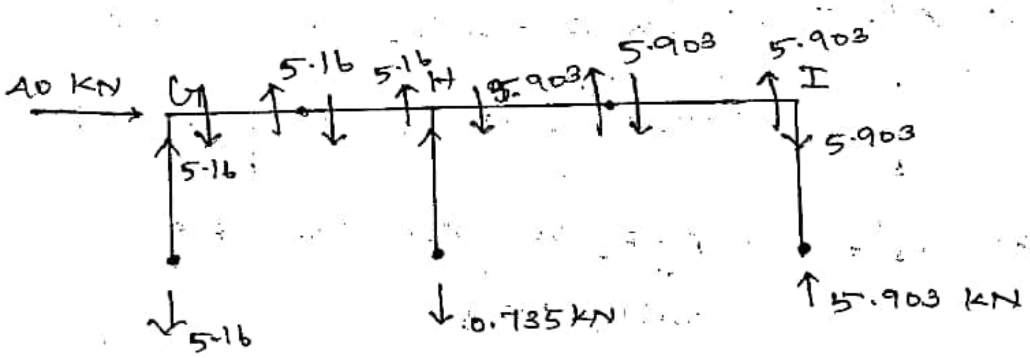
Step. 4:-

Calculation of S.F in the colums:-

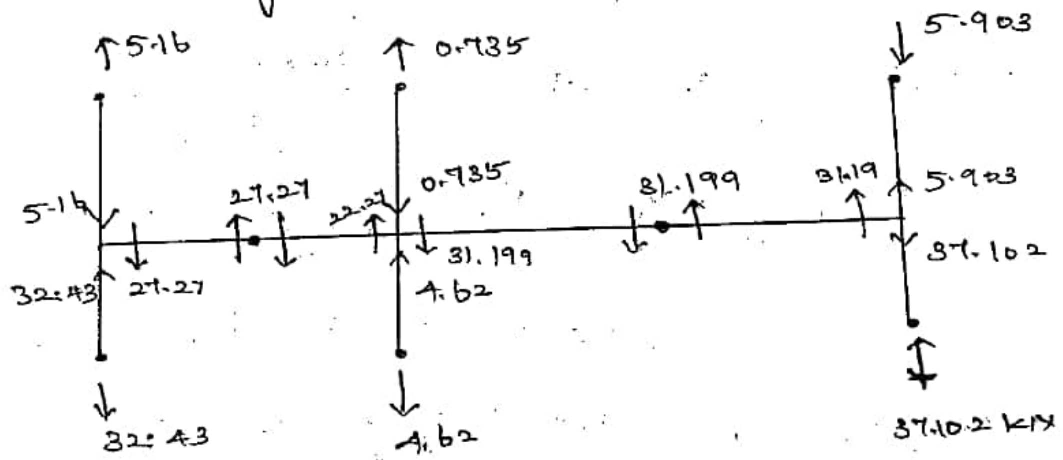
It can be determined from the free bodies of the beams / columns as shown in fig. below.

The beams shear @ the ends of each beams are also the shears @ the hinges @ mid-span.

IInd storey:-



First storey :-



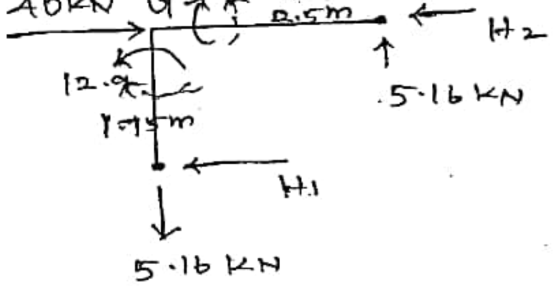
The S.F @ all the joints are calculated and marked above.

Step 5:-

Calculation of B.M, S.F in Beams & columns:-

consider the free body diagram of each joint apply equilibrium equations.

consider joint G,



$12.9 = H_1 \times 1.75$

$\boxed{H_1 = 7.37 \text{ KN}}$

$M_{GH} = M_{HG} = 5.16 \times 2.5$
$= \underline{\underline{12.9 \text{ KN.m}}}$ (↻)

$M_{GD} = M_{DG} = \underline{12.9} \text{ KN.m}$ (↻)

$\therefore$ C.

$\Sigma H = 0$

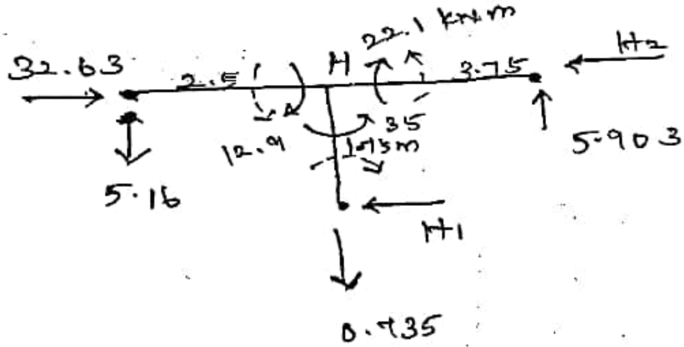
$40 - 7.37 - H_2 = 0$

$$\boxed{H_2 = 32.63\ kN}$$

Consider : joint, H :-

$M_{IH} = M_{HI} = 5.903 \times 3.75$

$= 22.1\ kN.m$



$M_{HE} = M_{EH}$

$= 22.1 + 12.9$

$= 35$

$35 = H_1 \times 1.75$

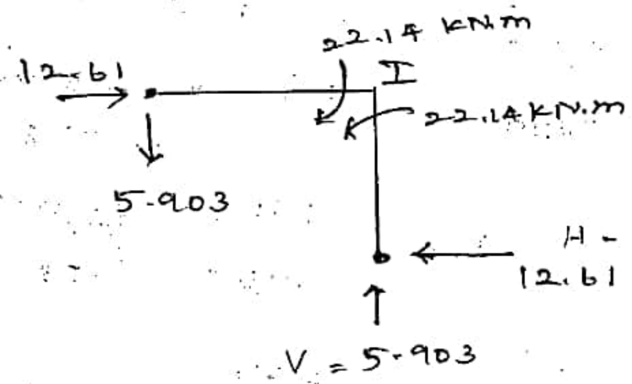
$$\boxed{H_1 = 20.02\ kN}$$

$\Sigma H = 0$

$32.63 - 20.02 = H_2$

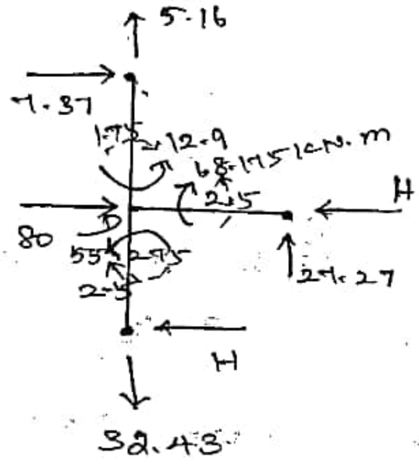
$$\boxed{H_2 = 12.61\ kN}$$

joint I :-



$M_{FI} = M_{IF} = 22.14\ kN.m$

consider joint D:-



$M_{ED} = M_{DE} = 27.27 \times 2.5$

$= 68.175\ kN.m$ (↻)

$M_{DA} = M_{AD}$

$= 68.175 - 12.9$

$= 55.275\ kN.m$

$55.275 = H \times 2.5$

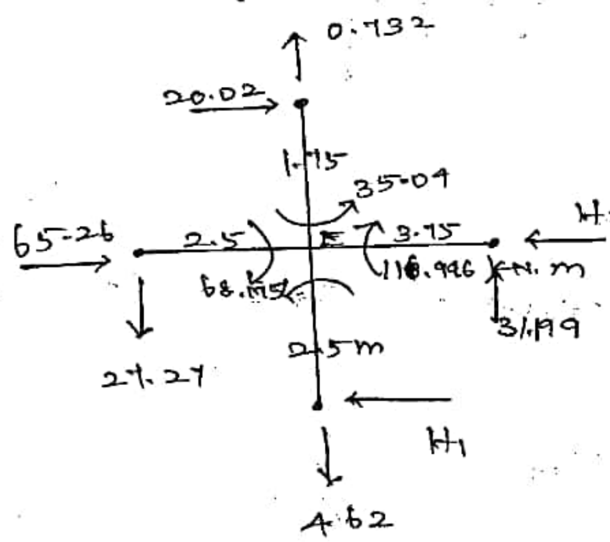
$\boxed{H = 22.11\ kN}$

$\Sigma H = 0$

$80 + 7.37 - 22.11 - H_1 = 0$

$\boxed{H_1 = 65.26\ kN}$ (←)

consider joint E:-



$M_{EF} = 31.99 \times 3.75$

$= 119.996\ kN m$

$M_{EF} = M_{FE}$ (↻)

$\Sigma M_E = 0$

$116.996 - 35.04 + 68.175 = M_{EB}$

$M_{EB} = 150.131\ kN m$ (↻)

$M_{EB} = M_{BE}$

$150.131 = H_1 \times 2.5$

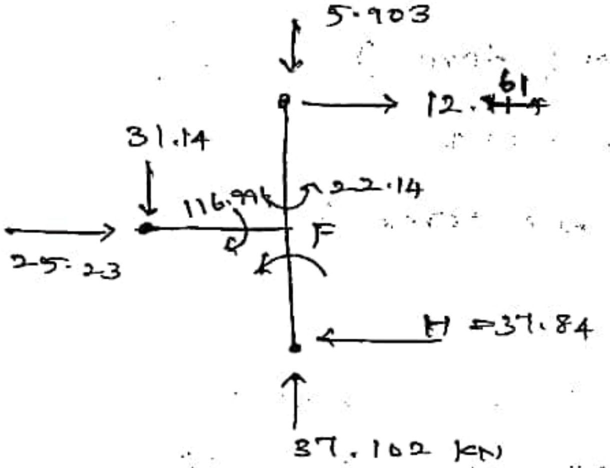
$\boxed{H_1 = 60.05\ kN}$

$\Sigma H = 0$

$65.26 + 20.02 - 60.05 = H_2$

$\boxed{H_2 = 25.23\ kN}$

Consider joint F:-



$M_{FC} = 116.8996 - 22.14$

$= 94.86$ kN m

$\Sigma H = 0$

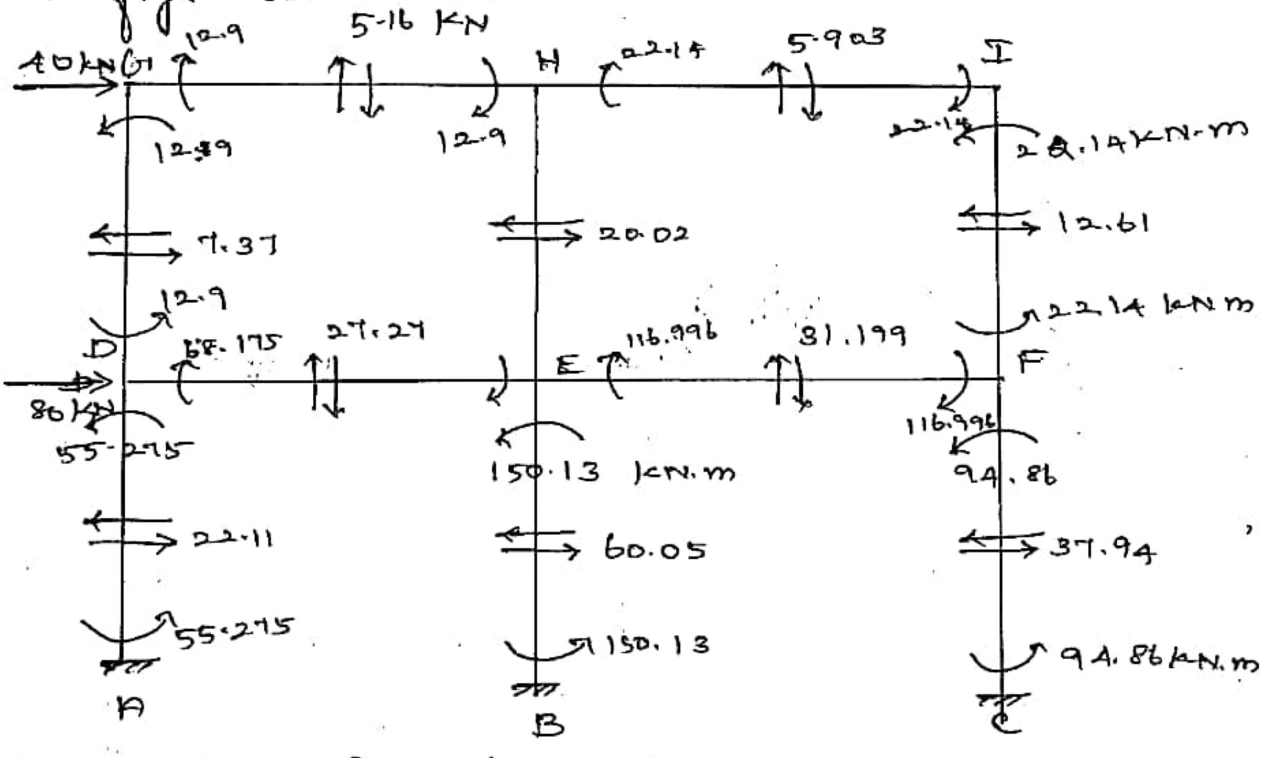
$25.23 + 12.61 - H = 0$

$\boxed{H = 37.84 \text{ kN}}$

Step 6:-

Calculation of A.F in the columns:-

mts & shears in cols & Beams are shown in fig. below.



Axial Forces in columns:-

$P_{GD} = 5.16$ kN (tension)

$P_{HE} = 0.735$ kN (tens.)

$P_{IF} = 5.903$ kN (comp.)

$P_{DA} = 32.48$ kN (tens.)

$P_{EB} = 4.62$ kN (tens.)

$P_{FC} = 37.102$ kN (comp.)

Check :-

total axial Force @ base.

$= -32.43 - 4.62 + 37.102$

$= 0.052 \simeq 0.$ (zero)

DESIGN OF REINFORCED CONCRETE AND BRICK MASONRY STRUCTURES.

UNIT-I

RETAINING WALLS.

1, Design a cantilever Retaining wall to retain Earth embankment 4m high above ground level. The unit weight of the earth is 18 KN/m³ and its angle of repose is 30°, The embankment is horizontal at its top. The safe bearing capacity of soil is 200 KN/m² and the co-efficient of friction between soil and concrete is 0.5. Adopt M20 concrete and Fe 415 steel. Take Factor of safety against overturning and sliding as 1.40.

Given data:

* Height of earth embankment = 4m above ground level.
* SBC of soil, $P = 200\ KN/m^2$
* unit weight of soil (Density), $\gamma = 18\ KN/m^3$
* co-efficient of friction, $\mu = 0.5$
* $f_{ck} = 20\ N/mm^2$; $f_y = 415\ N/mm^2$.

S/:

S-I : DIMENSIONS OF RETAINING WALL:

* Minimum depth of foundation : $\frac{P}{\gamma} \times ka^2$

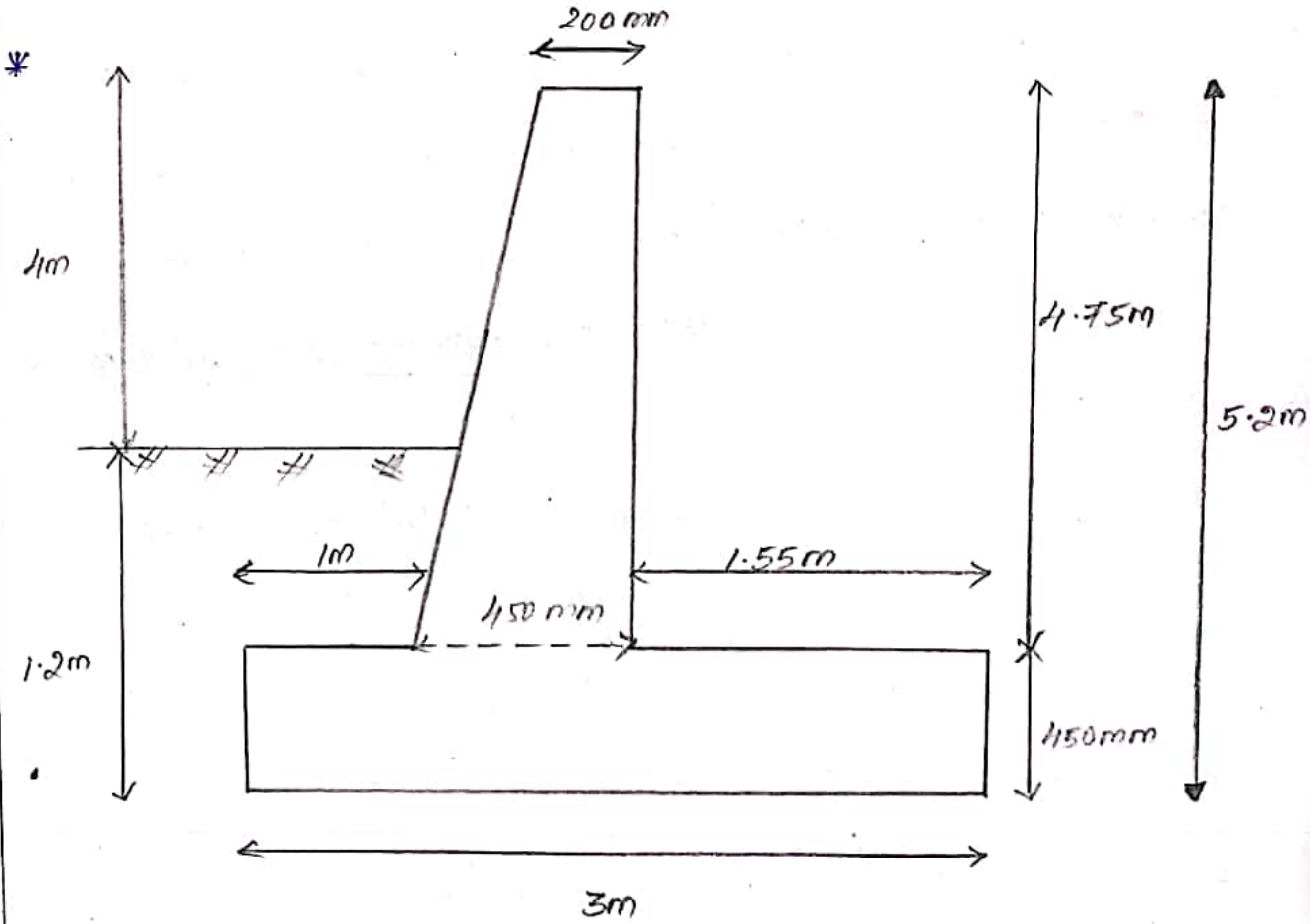
$$= \frac{P}{\gamma} \times \left[\frac{1-\sin\varphi}{1+\sin\varphi}\right]^2$$

$$= \frac{200}{18} \times \left[\frac{1-\sin 30}{1+\sin 30}\right]^2$$

$= 1.23\,m$

Depth of foundation may be taken as 1.2 m

*



* Overall Height of Retaining wall = Height of embankment above G.L + Depth of foundation

$= 4m + 1.2 = 5.2m$

* Thickness of Base slab $= H/12$

$= 5.2/12 = 0.43m = 430mm$

∴ Thickness of Base slab taken as 450 mm

* Width of Base slab = $0.5H$ to $0.6H$

$= (0.5 \times 5.2)$ to (0.6×5.2)

$= 2.6$ to $3.12m$

∴ Width of Base slab may be taken as 3m

* Height of the stem = Overall height of the Retaining wall − Thickness of Base slab

$= 5.2 - 0.45 = 4.75m$

* ~~Height~~ width of the stem at bottom = Thickness of the stem

$= 450mm$

* ~~Height~~ width of the stem at top = 200 mm

* Width of toe slab = 1m $= 6/3 = 3/3 = 1$

(4)

STEP : 2 : DESIGN OF STEM :

* Maximum moment at the bottom of the stem, M

$$= K_a(\gamma h^3/6)$$

$$= 0.33 \times (18 \times 4.73^3/6)$$

$\left[K_a = \frac{1-\sin 30}{1+\sin 30} = 0.3\right]$

$$M = 107.17 \text{ KN.m}$$

Factored moment, $M_u = 1.5 \times 107.17$

$$M_u = 160.75 \text{ KN.m}$$

* Effective depth = Over Thickness of stem − cover

$$= 450 - 50 = 400 \text{ mm}$$

$\Rightarrow d = \sqrt{M_u / 0.138 f_{ck} b} = \sqrt{\frac{160.75 \times 10^6}{0.138 \times 20 \times 1000}} = 241 \text{ mm}$

Hence the ∴ Effective depth is more than the limiting value

* Ast can be calculated as follows,

(i) $M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{f_{ck} b d}\right]$ [From IS 456;2000, Clause 38.1 pg No : 96]

$$160.75 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 400}\right]$$

$$A_{st} = 1186 \text{ mm}^2$$

using 16 mm ϕ bars, spacing of bars $= \frac{201}{1186} \times 1000$ ($\rightarrow \pi/4 \times 16^2$)

$$= 169 \simeq 170 \text{ mm}$$

∴ provided 16 mm ϕ bars @ 170 mm c/c on both the faces of the retaining wall as vertical reinforcement.

(ii) Distribution reinforcement = 0.12 % of ~~bd~~

$$= \frac{0.12}{100} \times 1000 \times 450$$

$$= 540\ mm^2$$

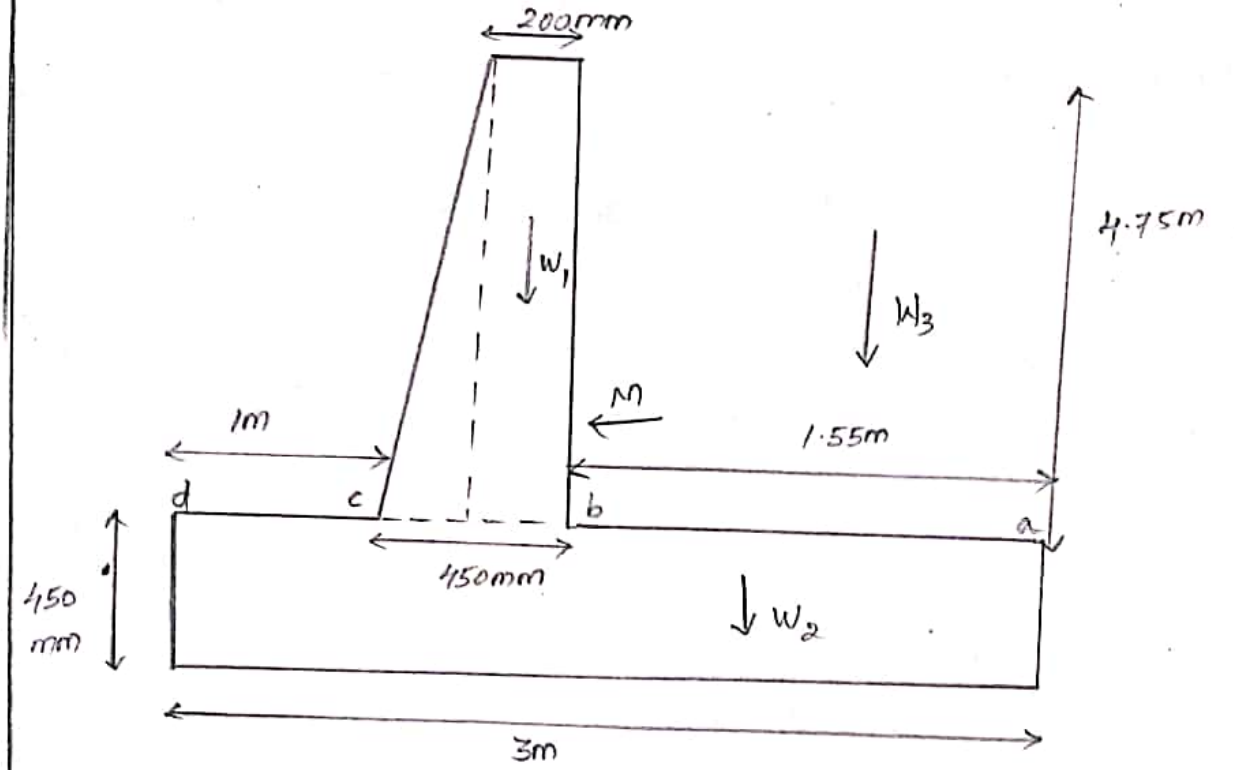
Using 10 mm φ bars, spacing of bars = $\frac{\pi/4 \times 10^2}{540} \times 1000$

$= 145\ mm$.

∴ provided 10 mm φ bars, @ 150 mm c/c on both faces of the stem as distribution reinforcement.

STEP : 3 : STABILITY CALCULATIONS :

(*) soil pressure below base slab :



W_1 = Self weight of stem

W_2 = Self weight of base slab

W_3 = weight of soil above heel slab

M = moment acting at the bottom of the stem considering 1m run of the retaining wall.

Load	Magnitude of load (KN)	Distance from end 'a'	Moment (KN.m)
i) Self weight of stem			
* $25\times0.2\times4.75$	23.75	$1.55+0.2/2 = 1.65$	39.18
* $25\times0.25\times4.75\times\frac{1}{2}$	14.84	$1.55+0.2+0.25/3 = 1.83$	27.20
ii) Self weight of base slab			
$25\times3\times0.45$	33.75	$3/2 = 1.5$	50.62
iii) Self weight of soil above heel slab			
$18\times1.55\times4.75$	132.5	$1.55/2 = 0.775$	102.68
iv) Moment acting at the bottom of the stem			
$\Rightarrow K_a\times\gamma h^3/6$	-	-	107.17
	$\Sigma W = 204.84$ KN		$\Sigma M = 326.85$ KN.m

The distance of resultant force from end 'a', $z = \frac{\Sigma W}{\Sigma M} \; \frac{\Sigma M}{\Sigma W}$

$$= \frac{204.9}{326.8} \; \frac{326.8}{204.9}$$

$$\simeq 1.6\,m$$

Eccentricity, $e = z - B/2$ [$\therefore$ B = Width of base slab]

$$= 1.6 - 3/2$$

$$e = 0.1\,m$$

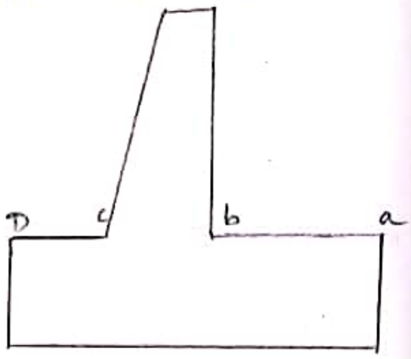
$$b/6 = 3/6 = 0.5$$

$\therefore$ $e < b/6$, Hence safe.

The soil pressure below the base slab is compressive.

$\therefore$ P_{max}, Maximum pressure will occur at end 'd'

P_{min}, Minimum pressure will occur at end 'a'

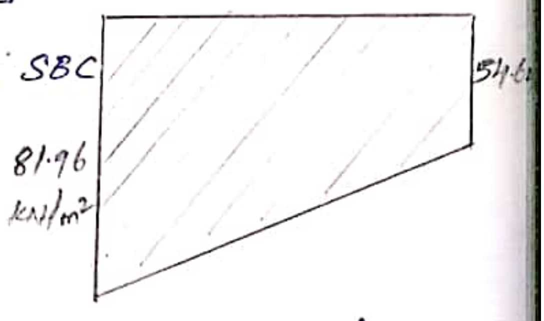


$$P_{max} = \frac{\Sigma W}{B}\left[1 + \frac{6e}{B}\right]$$

$$= \frac{204.9}{3}\left[1 + \frac{(6 \times 0.1)}{3}\right]$$

$\therefore$ $P_{max} = 81.96\ KN/m^2 < SBC$

Hence safe



$$P_{min} = \frac{\Sigma W}{B}\left[1 - \frac{6e}{B}\right]$$

$$= \frac{204.9}{3}\left[1 + \frac{(6 \times 0.1)}{3}\right] = 54.64\ KN/m^2 > 0$$

$\therefore$ Hence safe

* Check for safety against sliding ~~Overturning~~ Overturning (OR) Factor of Safety against overturning :

$$\text{Factor of safety} = 0.9 \times \frac{M_R}{M_O} > 1.4$$

where,

M_R = Resisting moment

M_o : Overturning moment.

$$M_R = \Sigma W [B - z]$$

$$= 204.9 [3 - 1.6] = 286.86 \text{ KN.m}$$

$$M_o = K_a(\gamma h^3/6)$$

$$= 107.17 \text{ KN.m}$$

$$FOS = 0.9 \times \frac{286.86}{107.17}$$

$$= 2.4 > 1.4$$

∴ Hence safe

* check for safety against sliding (OR) Factor of safety against sliding :

$$\text{Factor of safety} = 0.9 \times \frac{F_R}{F_S}$$

where,

F_R = Resisting Force ; F_s = Sliding Force

$F_R = \mu \Sigma W$

$= 0.5 \times 204.9 = 102.45$ KN

$F_S = K_a \gamma H^2/2$

$= 0.33 \times \dfrac{18 \times 5.2^2}{2} = 80.30$ KN

$FOS = 0.9 \times \dfrac{F_R}{F_S} > 1.4$

$= 0.9 \times \dfrac{102.45}{80.30}$

$= 1.14 < 1.4$

∴ Hence unsafe, A shear key has to be provided at the base slab.

* Design of shear key :

Assume the depth of shear key as 600 mm

The intensity of passive Earth pressure infront of shear key

$P_p = K_p \times \gamma h_s^2/2$

Where k_p = co-efficient of passive Earth pressure

h_s = Depth of shear key + Thickness of base slab

$= 0.6 + 0.45 = 1.05$m

$$K_p = 1/K_a = \frac{1+\sin 30}{1-\sin 30} = 3.03$$

$$P_p = 3.03 \times 18 \times \frac{1.05^2}{2}$$

$$P_p = 30.06 \text{ KN/m}^2$$

$$FOS = 0.9 \times \left[\frac{F_R + P_p}{F_S}\right]$$

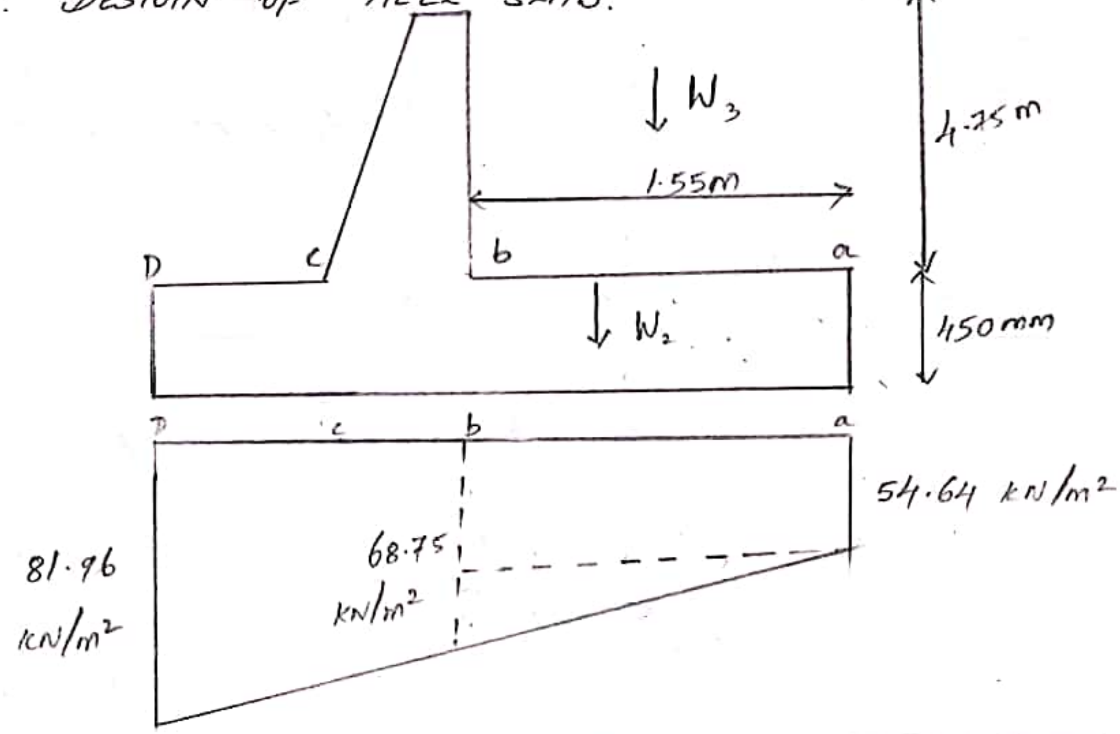
$$= 0.9 \times \left[\frac{102.45 + 30.06}{80.30}\right]$$

$$FOS = 1.48 > 1.4$$

$\therefore$ Hence safe

The Retaining wall is safe against failure due to sliding. The reinforcement in stem is Extended upto the shear key.

STEP 4: DESIGN OF HEEL SLAB.



The soil pressure at the fixed end of the heel slab can be calculated as follows:

$$* \quad P_{min} + \left[\frac{(P_{max} - P_{min})}{\text{width of base slab}} \times \text{width of heel slab} \right]$$

$$\Rightarrow 54.64 + \left[\frac{(81.96 - 54.64)}{3} \times 1.55 \right]$$

$$\Rightarrow 68.75 \ kN/m^2.$$

Load	Magnitude of load (kN)	Distance from end 'a'	Moment (kN.m)
(i) Self weight of heel slab * $25 \times 1.55 \times 0.45$	17.43 (↓)	$1.55/2 = 0.775$	13.50 (↓)
(ii) Self wt of soil above heel slab * $18 \times 4.75 \times 1.55$	132.52 (↓)	$\frac{1.55}{2} = 0.775$	102.70 (↓)
(iii) upward soil pressure			
i) Rectangular portion * 1.55×54.64	84.69 (↑)	$\frac{1.55}{2} = 0.775$	65.63 (↑)
(ii) Triangular portion * $1/2 \times 1.55 \times (68.75 - 54.64)$	10.94 (↑)	$\frac{1.55}{3} = 0.52$	5.68 (↑)

$\Sigma W = 54.32$ KN $\Sigma M = 44.89$ KN.m

Ultimate Bending moment $: 1.5 \times 44.89$

$= 67.33$ KN.M.

Ast can be calculated as follows,

$$Mu = 0.87 fy \, Ast \cdot d \left[1 - \frac{fy \, Ast}{fck \, bd}\right]$$

$$67.33 \times 10^6 = 0.87 \times 415 \times Ast \times 400 \left[1 - \frac{415 \times Ast}{20 \times 1000 \times 400}\right]$$

$Ast = 478 \ mm^2$

$\therefore$ using 12 mm ϕ bars, spacing of bars $= \frac{\pi/4 \times 12^2}{478} \times 1000$

$= 236$ mm

provided 12 mm ϕ bar @ 200 mm c/c on both the faces of heel slab as main reinforcement.

* Distribution reinforcement :

using 10 mm ϕ bars, spacing of bar =

- Ast = 0.12 % of bd

$= \frac{0.12}{100} \times 1000 \times 450 = 540 \ mm^2$

ast/Ast $\times$ 1000

3d

300 mm

Using 10mm dia. bars, spacing of bars $= \frac{\pi/4 \times 10^2}{540} \times 1000$

$= 145$ mm

$= 78.53$

provided 10 mm ϕ bars @ 150 mm c/c on both the faces of heel slab as distribution reinforcement.

Dead wt of soil over toe slab (1.2 - 0.45) ×1×18	13.5		
(ii) upward soil pressure.			
* Rectangular portion			
1×72.85	72.85 (↑)	$1/2 = 0.5$	36.42 (↑)
* Triangular portion			
$1/2 \times (81.96 - 72.85) \times 1$	4.55 (↑)	$2/3 \times (1) = 0.67$	3.04 (↑)
	$\Sigma W = 66.15$		$\Sigma M = 33.84$ ~~kN/m~~

Factored Bending Moment = 1.5×33.84

$= 50.76$ KN.m

Ast can be calculated as follows,

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

$$50.76 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 400}\right]$$

$$A_{st} = 358 \text{ mm}^2$$

Using 12mm φ bars, spacing of bar = $\frac{314}{358} \times 1000$

$= 315$ mm

∴ provided 12mm φ bar @ 300 mm c/c on both the faces of toe slab as main reinforcement.

* Distribution reinforcement:

using 10 mm φ bars, .: A_s

$$= 0.12\ \%\ \text{of}\ bD$$

$$= \frac{0.12}{100} \times 1000 \times 450 = 540\ N/mm^2$$

using 10 mm φ bars, spacing of bars = 145 mm

.: provided 10mm φ bars @ 150 mm c/c on both the faces of toe slab as distribution reinforcement.

* STEP 6: CHECK FOR SHEAR STRESS AT JUNCTION OF STEM AND BASE SLAB:

Net working shear force, $V = (1.5P - \mu W)$

$$= [(1.5 \times 80.30) - 102.45]$$

$$V = 18\ KN$$

Factored shear force, $V_u = 1.5 \times 18 = 27\ KN$

Nominal shear stress, $\tau_v = \frac{V_u}{bd}$ [Pg No : 72]

$$= \frac{27 \times 10^3}{400 \times 1000} = 0.067\ N/mm^2.$$

To find τ_c, $\frac{100\ A_{st}}{bd} = \frac{100 \times 1\cancel{2}86}{1000 \times 400} = 0.3$

(PgNo : 73) (11)

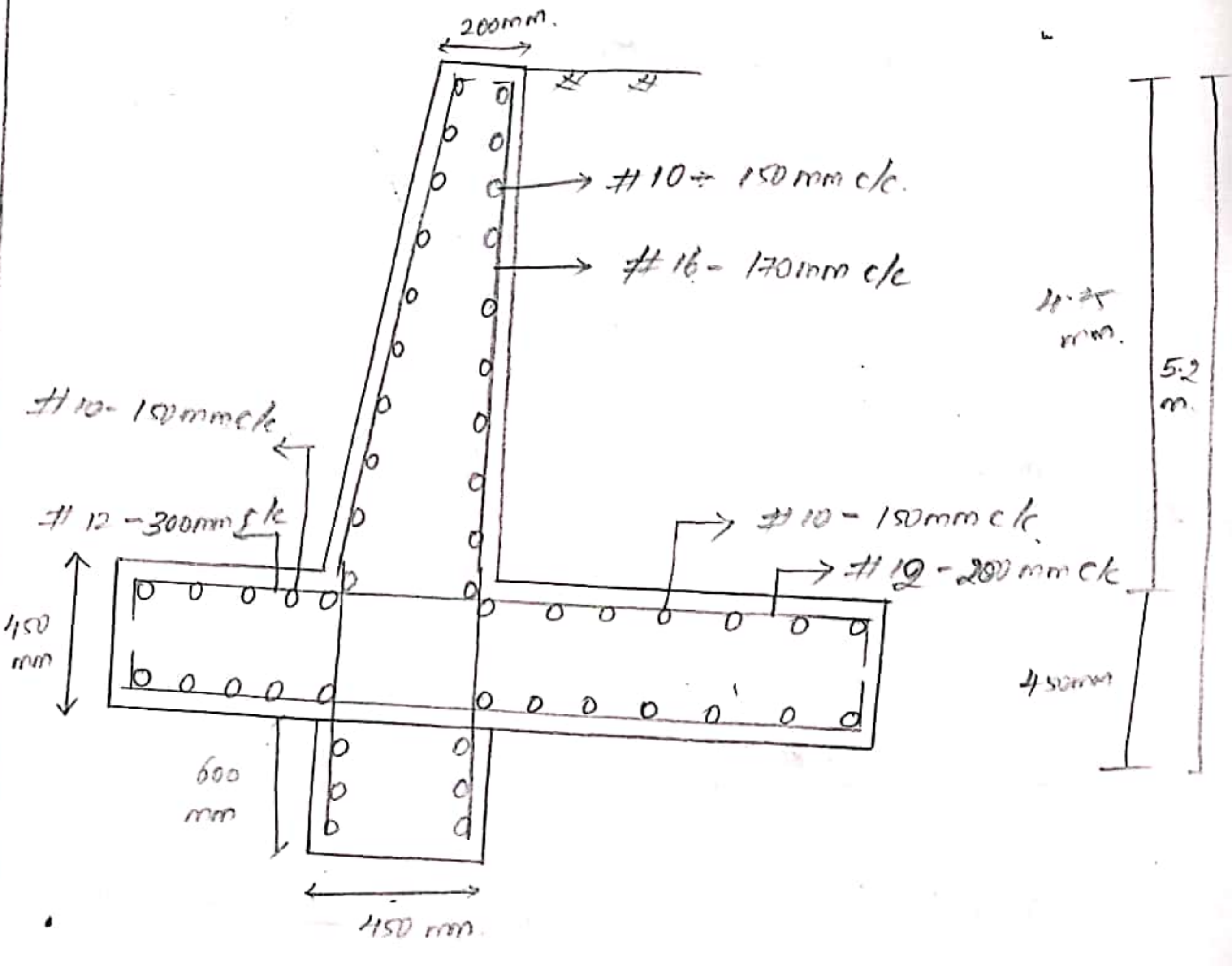
From table 19 of IS 456, the permissible shear stress

$\tau_c = 0.42$

$\tau_c' > \tau_c$

Hence shear stress are within safe permissible limits.

STEP 7 : REINFORCEMENT DETAILING:



Design of counterfort Retaining wall:

As stated earlier counterfort retaining wall are economical for wall height greater than 6m. The design of counterfort retaining wall comprises of

⇒ Design of counterfort

⇒ Design of vertical slab

⇒ Design of toe slab

⇒ Design of Heel slab.

a) General Features:

* Retaining walls of height over 6m are usually provided with counterforts

* The counterfort retaining walls consists of the stem (or) upright slab, toe slab, heel slab and the counterfort spaced at regular intervals

* Spacing of counterforts = L = 3 to 3.5m

* Thickness of base slab = $2LH$ cm

L = spacing of counterfort in m

H = overall height of the retaining wall (m)

* Base width = $0.6H$ to $0.7H$

* Toe projection = $1/4$ width of base slab

b) Design principles:

i) Stem (or) upright slab is designed as continuous slab to span b/w counterforts

$$Max.\ BM = \left(\frac{PL^2}{12}\right)$$

where,

P = pressure Intensity at base

$= K_a \gamma h$

ii) Toe slab is designed for soil pressure and dead weight of slab.

iii) Heel slab is designed as continuous slab supported b/w counterforts to resists of soil and upward pressure at base.

iv) counterforts thickness is the same as the base slab counterforts are designed to take lateral earth pressure

Max BM in counterfort: $K_a \cdot \frac{w h^3}{6} \cdot L$

where,

h = height of retaining wall above base

L = spacing of counterfort.

2. Design a counterfort type retaining wall to suit the following data.

⇒ Height of wall above ground level = 6m

⇒ SBC of soil at site, P = 160 kN/m²

⇒ Angle of internal friction = 30°

⇒ Density of soil, γ = 16 kN/m³

⇒ Spacings of counterforts = 3m c/c

Materials M_{20} grade concrete & Fe415 HYSD Bars.

Sol:

S-I : Dimensions of Retaining wall :

* Minimum depth of foundation $= \frac{P}{\gamma} \times Ka^2$

$$= \frac{P}{\gamma}\ \frac{160}{16}\left(\frac{1-\sin 30}{1+\sin 30}\right)^2$$

$$d = 1.2m$$

Overall height of wall = 1.2 + 6

$$H = 7.2m$$

Thickness of Base slab : 2LH cm

$$= 2\times3\times 7.2 = 43.2\ cm$$

$$b = 450\ mm$$

Provided 450mm thick base slab

Base width, $B = 0.6H$ to $0.7H$

$= (0.6 \times 7.2)$ to (0.7×7.2)

$= 4.32$ to 5.04

$\therefore$ B = 4.- Width of the base slab can be taken as 4.5m

$\therefore$ Toe projection = $1/4 \times$ width of base slab

$= 1/4 \times 4.5 = 1.12$ m

$\therefore$ Width of toe slab can be taken as 1m.

Height of the stem = Overall height of the retaining wall - thickness of base slab

$= 7.2 - 0.45 = 6.75$ m

* STEP - 2: Design of stem:

The stem has to be designed as continuous slab spanning between counterforts:

$\therefore$ The pressure intensity at the bottom of stem,

$P = K_a \gamma h$

$= 0.33 \times 16 \times 6.75$

$P = 35.64$ KN/m^2

Scanned with CamScanner

Bending moment at the stem, $= \frac{Pl^2}{12}$

$$= \frac{35.64 \times 3^2}{12}$$

$$= 26.73 \text{ kN-m}$$

$\therefore$ ultimate moment, $M_u = 1.5 \times 26.73 = 40.09$ kN-m

$\therefore$ Thickness of stem can be calculated as follows :

Effective depth, $d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$

$$= \sqrt{\frac{40.09 \times 10^6}{0.138 \times 20 \times 1000}}$$

$$d = 120.52 \text{ mm}$$

$\therefore$ Thickness of stem can be taken as 200mm (uniform thickness from top to bottom)

$\therefore$ $D = 200$ mm, $d = 150$ mm.

Ast can be calculated as follows,

$$M_u = 0.87 \times f_y \times A_{st} \times d \left[1 - \frac{f_y A_{st}}{f_{ck} bd}\right]$$

$$40.09 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 150}\right]$$

$$A_{st} = 837 \text{ mm}^2.$$

Using 12mm φ bars, spacing of bars : $\frac{\pi/4 \times 12^2}{837} \times 1000$

$= 135$ mm.

∴ provided 12mm φ bars @ 150 mm c/c on both the faces of the stem as main reinforcement.

* Distribution reinforcement = 0.12 % of bD

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

$= 240 \ mm^2$

Using 10 mm φ bars = 327 mm

∴ provided 10 mm φ bars @ 300 mm c/c on both the faces of stem as Distribution Reinforcement.

STEP : 3 : STABILITY CALCULATIONS:

⇒ Considering 1m run of the wall.

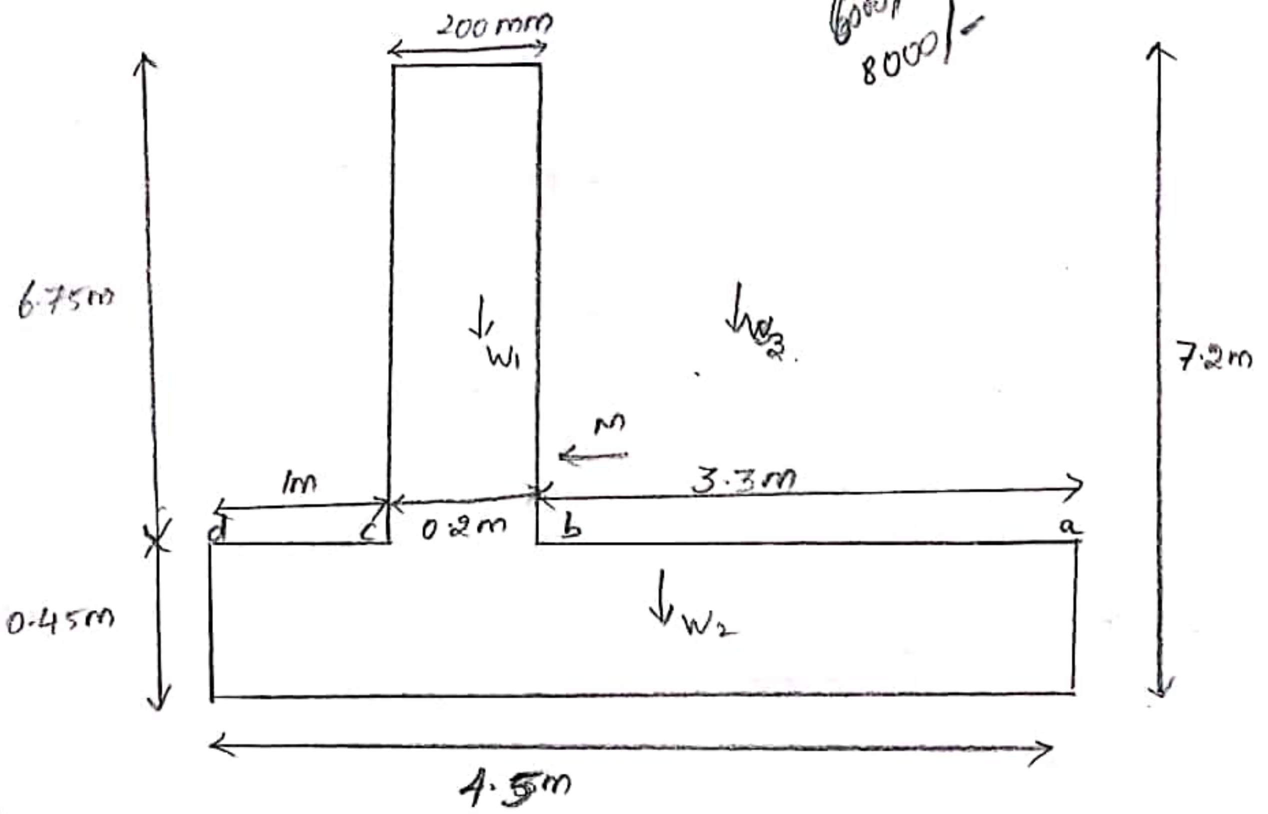
Load	Magnitude of load (kN)	Distance from end 'a' (m)	Moment kN-m.
i) Self weight of stem ⇒ $25 \times 0.2 \times 6.75$	33.75	$3.3 + 0.2/2 = 3.4$	114.75.
ii) Self weight of base slab ⇒ $25 \times 4.5 \times 0.45$	50.62	$4.5/2 = 2.25$	113.89
iii) weight of soil above heel slab ⇒ $16 \times 6.75 \times 3.3$	356.4	$3.3/2 = 1.65$	588.06

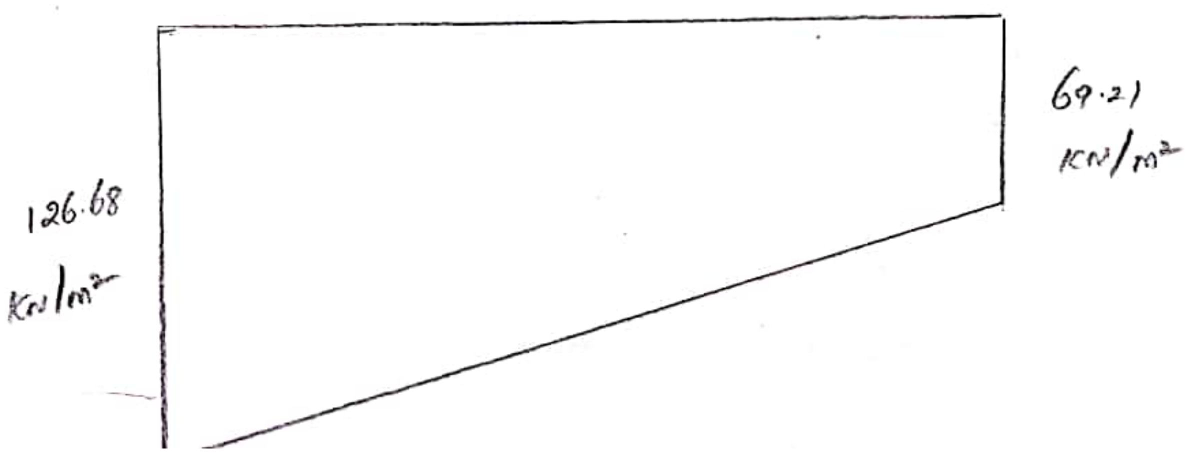
iv) Moment of earth pressure $\Rightarrow K_a \times \gamma h^3/6$ $\Rightarrow 0.33 \times (16 \times 6.75^3/6)$ $\left(\frac{1-\sin 30}{1+\sin 30}\right)$	–	–	273.37 KN-m

$\Sigma W = 440.77$ KN

$\Sigma M = 1090.07$ KN-m

6000/-
8000/-





The distance from resultant force end 'a', $x = \frac{\Sigma M}{\Sigma W}$

$$= \frac{1090.07}{440.77}$$

$$= 2.47\,m$$

$\therefore$ Eccentricity, $e = x - B/2$

$$= 2.47 - 4.5/2$$

$$e = 0.22$$

$$b/6 = \frac{4.5}{6} = 0.75$$

$\therefore$ $e < b/6$, Hence safe.

The soil pressure below the slab is compressive

$\therefore$ P_{max}, Maximum pressure will occur at end 'd'

P_{min}, Minimum pressure will occur at end 'a'

$$P_{max} = \frac{\Sigma W}{B}\left[1 + \frac{6e}{B}\right]$$

$$= \frac{440.77}{4.5}\left[1 + \frac{(6 \times 0.22)}{4.5}\right]$$

$$P_{max} = 126.68\ kN/m^2 < SBC$$

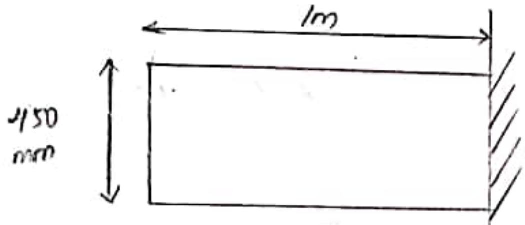
$$P_{min} = \frac{\Sigma W}{B}\left[1 - \frac{6e}{B}\right]$$

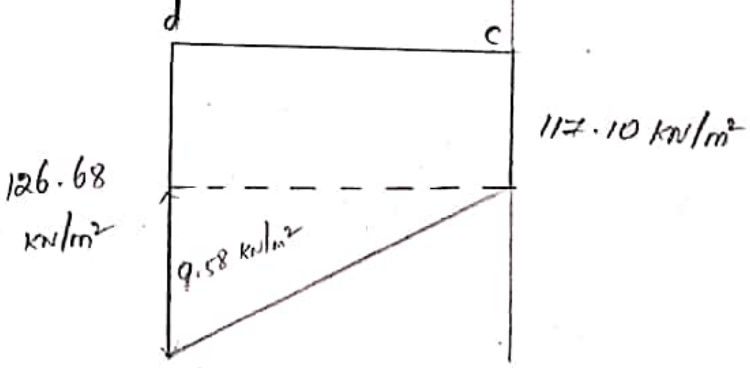
$$= \frac{440.77}{4.5}\left[1 - \frac{(6 \times 0.22)}{4.5}\right] = 69.21\ kN/m^2 < 0$$

$\therefore$ Hence safe.

STEP : 4 : DESIGN OF TOE SLAB :

LOADS	MAGNITUDE OF LOADS (KN)	DISTANCE FROM 'C' (KN)	MOMENT AT 'C' KN-m.
* Self weight of toe slab ⇒ 0.45 × 1 × 25	11.25	$1/2 = 0.5$	5.625
* weight of soil above toe slab ⇒ 0.75 × 1 × 16	12	$1/2 = 0.5$	6
upward pressure :			
* Rectangular portion ⇒ 1 × 117.10	117.10	$1/2 = 0.5$	58.55
* Triangular portion ⇒ $\frac{1}{2}$ × 9.58 × 1	4.79	$2/3 = 0.67$	3.20
	ΣW = KN (↑−↓) 98.64 KN		ΣM = 50.12 KN-m.





The pressure at the fixed end of the toe slab can be calculated as follows;

$$\Rightarrow P_{min} + \left[\frac{(P_{max} - P_{min})}{\text{width of base slab}} * (\text{width of heel slab} + \text{thickness of } \cancel{\text{stem}} \text{ base slab})\right]$$

$$\Rightarrow 69.21 + \left[\frac{(126.68 - 69.21)}{4.5} \times (3.3 + 0.45)\right]$$

$$\Rightarrow 117.10 \ KN/m^2$$

Ultimate moment = $1.5 \times 50.12 = 75.18$ KN.m

→ Effective depth of toe slab = $450 - 50 = 400$ mm.

Ast can be calculated as follows,

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

$$75.18 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 400}\right)$$

$$A_{st} = 535 \ mm^2$$

Using 12 mm φ bars, spacing of bars = 211 mm

∴ provided 12 mm φ bars @ 220 mm c/c on both the faces of toe slab as main reinforcement.

Distribution reinforcement $= 0.12\%$ of bD

$$= \frac{0.12}{100} \times 1000 \times 450 = 540 \, mm^2$$

$\therefore$ provided 10 mm ϕ bars @ 145 mm c/c on both the faces of the slab as distribution reinforcement.

STEP 5: DESIGN OF HEEL SLAB:

Consider 1m wide strip (Continuous slab)

Load	Magnitude
1) weight of soil on strip $1 \times 6.75 \times 16$	108
2) Self weight of heel slab strip $1 \times 0.45 \times 25$	11.25
upward pressure	
i) 1×69.21	69.21

$$\Sigma(\uparrow \sim \downarrow) = 50.04 \text{ KN}.$$

$\therefore$ Spacing of counterforts $= 3m$

Maximum Negative Service BM at counterfort

$$M = \frac{PL^2}{12} = \frac{50.04 \times 3^2}{12} = 37.53 \text{ KN.m}$$

Factored B.M $= 1.5 \times 37.53 = 56.29$ KN.m

Ast can be calculated as follows.

$$M_u = 0.87 \, f_y \, A_{st} \, d \left[1 - \frac{f_y A_{st}}{f_{ck} \, b d} \right]$$

$$59.29 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{415 \, A_{st}}{20 \times 1000 \times 400} \right]$$

$$A_{st} = 419 \text{ mm}^2$$

∴ provided. 12 mm φ bars @ 270 mm c/c on both the faces on heel slab as main reinforcement.

* Distribution Reinforcement $= 0.12\%$ of bD

$$= \frac{0.12}{100} \times 1000 \times 450 = 540 \text{ mm}^2$$

∴ provided 10 mm φ bars @ 145 mm c/c on both the faces of heel slab as distribution reinforcement.

STEP 6: DESIGN OF COUNTERFORTS!

Thickness of the counterfort = 2 × thickness of the stem

$$\therefore b = 2 \times 200 = 400 \text{ mm}$$

Overall depth of the counterfort at the bottom

= width of the base slab - width of toe slab

= 4.5 - 1

= 3.5 m = 3500 mm

Effective depth, d = 3450 mm

∴ Maximum working moment in counterfort

$$M = Ka \cdot \frac{\gamma h^3}{6} \cdot L$$

$$= \frac{1}{3} \cdot \frac{16 \times 6.75^3}{6} \cdot 3$$

$$M = 820.12 \text{ KN-m}$$

Factored moment, M_u = 1230.18 KN-m.

Reinforcement at the bottom

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

$$1230.18 \times 10^6 = 0.87 \times 415 \times A_{st} \times 3450 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 3450}\right]$$

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

But, Min reinforcement as per IS 456 - 2000

$$\text{Min } A_{st} = \frac{0.85 bd}{f_y}$$

$$= \frac{0.85 \times 400 \times 3450}{415}$$

$$\therefore \text{Min } A_{st} = \cancel{2826}\ 2826 \text{ mm}^2$$

Using 25 mm φ bars; No of bars $= \frac{2826}{\pi/4 \times 25^2} = 6$ bars

∴ provided 6 Nos of 25 mm φ bars for counterforts

(OR)

Counterfort has to be designed as vertical cantilever beam

The Area of steel from BM will be less than that of minimum Area of steel given in IS-456 code.

∴ Area of steel can be calulated as follows, based on IS 456- code.

$$\text{Min } A_{st} = \frac{0.85\,bd}{f_y} \qquad [\text{Pg No}: 47]$$

$$\therefore \text{Min } A_{st} = \frac{0.85 \times 400 \times 3450}{415} = 2865 \text{ mm}^2$$

STEP 7: Connection between the stem and the counterforts:

Consider bottom 1m heighth of the retaining wall.

The pressure acting at the bottom of the stem

$$= K_a \gamma h$$

$$= 0.33 \times 16 \times 6.75 = 35.64 \text{ kN/m}^2.$$

Total load transfer to the counterforts $= 35.64 \times 1 \times 3$ → ht → spacing of counterforts.

$$= 106.92 \text{ kN}.$$

Factored force $= 1.5 \times 106.92$

$$= 160.38 \text{ kN}$$

Area of steel, $A_{st} = \frac{F_u}{0.87 f_y}$ IS 456 : 2000 [pg No : 73]

$$= \frac{160.38 \times 10^3}{0.87 \times 415} = 444.20\ mm^2$$

using 2 legged 10 mm ϕ links.,

Spacing of links $= \frac{2 \times 78.53}{444.20} \times 1000 = 353\ mm$

$\therefore$ provided 2 legged 10mm ϕ Horizontal links @ 300 mm c/c.

STEP 8: Connection b/w counterfort and heel slab:

Consider 1m run of the heel slab at the end A.

pressure acting on the heel slab, $P_{min} = 69.21\ kN/m^2$

Force transfer to the counterfort $= 69.21 \times 1 \times 3$

$= 207.63\ kN.$

Factored load $= 1.5 \times 207.63$

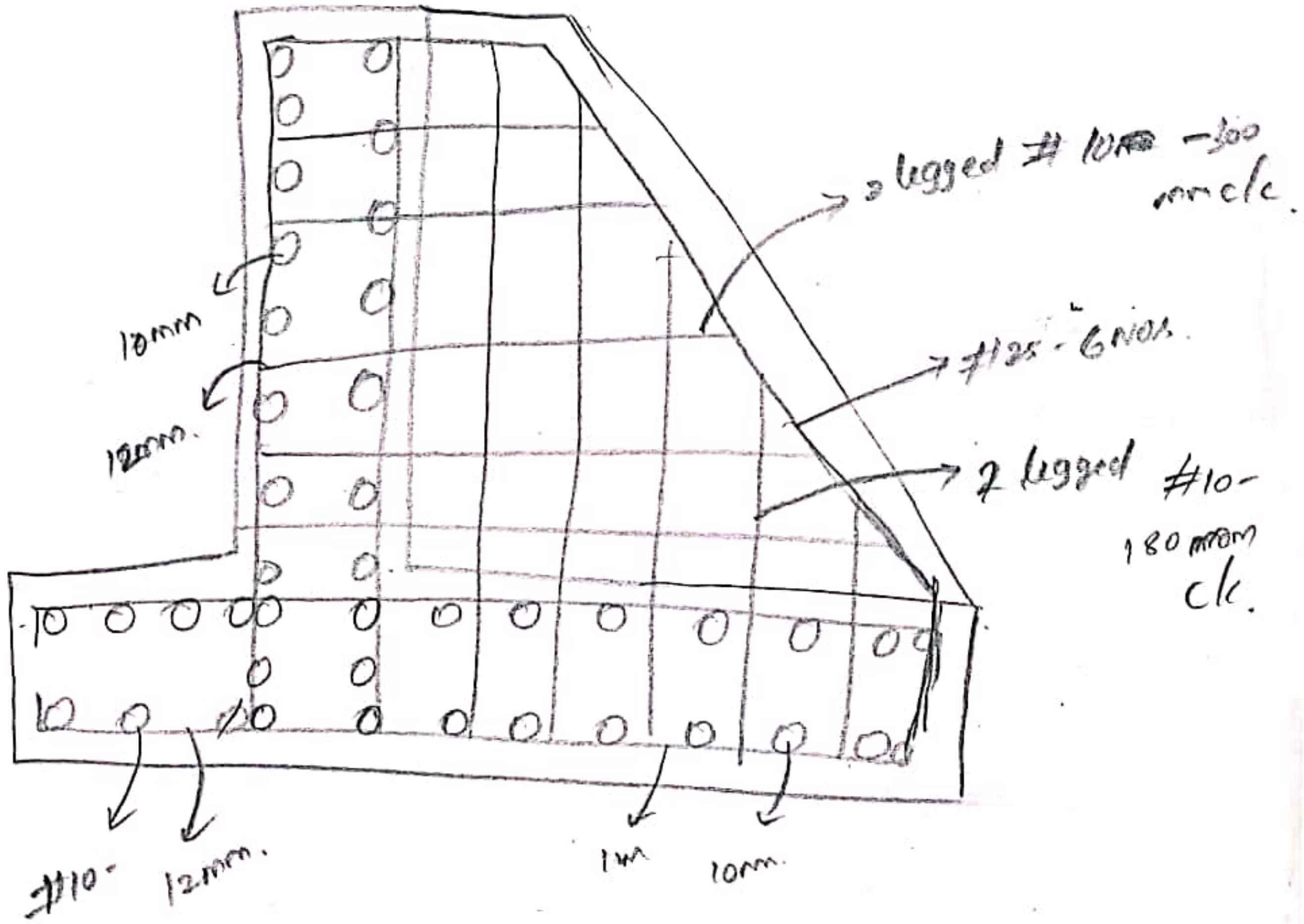
$= 311.45\ kN.$

$$A_{st} = \frac{311.45 \times 10^3}{0.87 \times 415} = 862.60\ mm^2$$

using 2 legged 10 mm ϕ links,

Spacing of links $= \frac{2 \times 78.53}{862.60} = 182\ mm$

$\therefore$ provided 2 legged 10 mm ϕ vertical links @ 180 mm c/c.



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DESIGN OF REINFORCED CONCRETE AND BRICK MASONRY STRUCTURES.

UNIT-I

RETAINING WALLS.

1, Design a cantilever Retaining wall to retain Earth embankment 4m high above ground level. The unit weight of the earth is 18 kN/m³ and its angle of repose is 30°, The embankment is horizontal at its top. The safe bearing capacity of soil is 200 kN/m² and the co-efficient of friction between soil and concrete is 0.5. Adopt M20 concrete and Fe 415 steel. Take Factor of safety against overturning and sliding as 1.40.

Given data:

* Height of earth embankment = 4m above ground level.
* SBC of soil, $P = 200\ kN/m^2$
* unit weight of soil (Density), $\gamma = 18\ kN/m^3$
* co-efficient of friction, $\mu = 0.5$
* $f_{ck} = 20\ N/mm^2$; $f_y = 415\ N/mm^2$.

S/:

S-I : DIMENSIONS OF RETAINING WALL:

* Minimum depth of foundation : $\frac{P}{\gamma} \times ka^2$

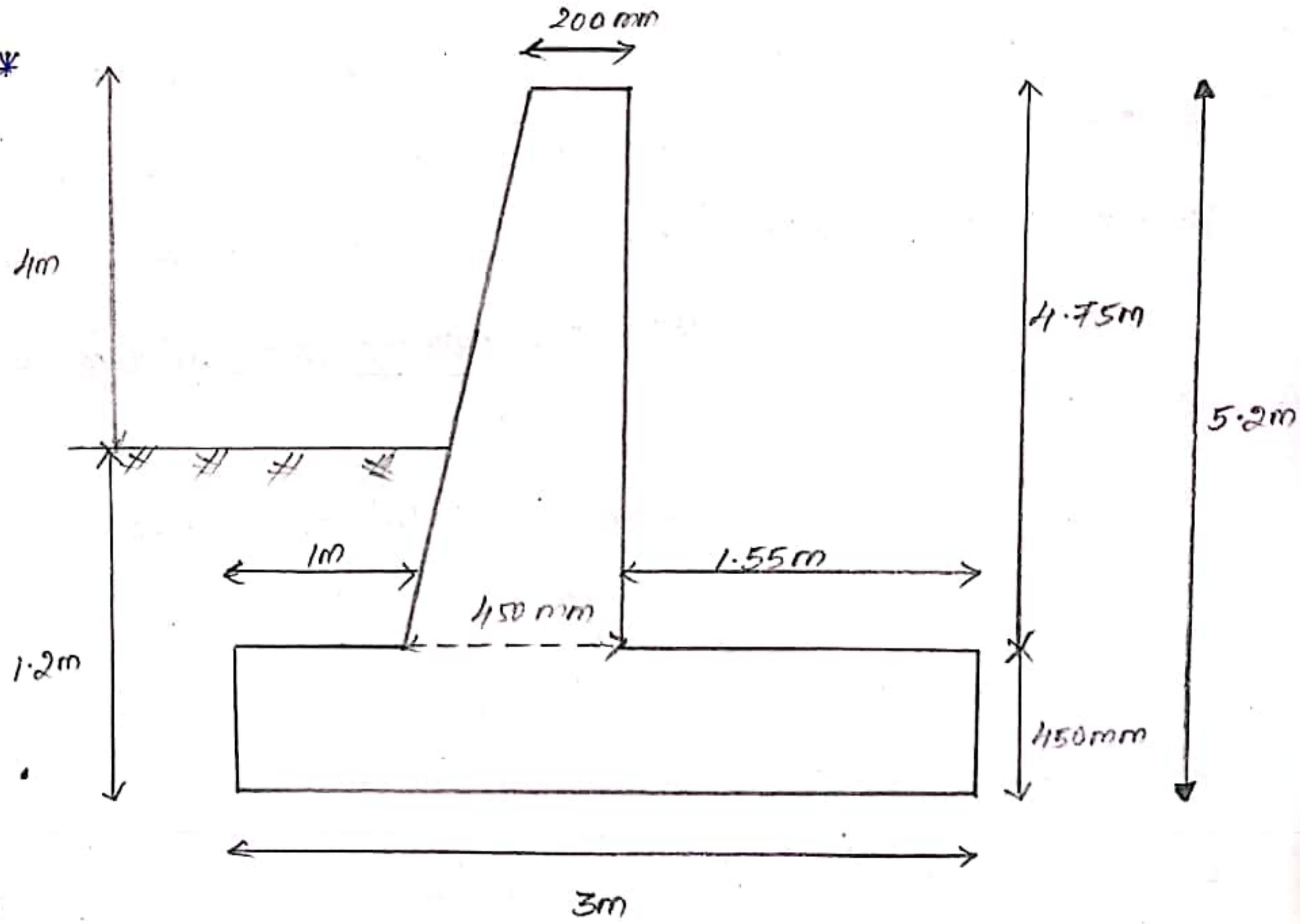
$$= \frac{P}{\gamma} \times \left[\frac{1-\sin\varphi}{1+\sin\varphi}\right]^2$$

$$= \frac{200}{18} \times \left[\frac{1-\sin 30}{1+\sin 30}\right]^2$$

$$= 1.23\,m$$

Depth of foundation may be taken as 1.2 m

*



* Overall Height of Retaining wall = Height of embankment above G.L + Depth of foundation

= 4m + 1.2 = 5.2m

* Thickness of Base slab = $H/12$

= $5.2/12$ = 0.43m = 430mm

∴ Thickness of Base slab taken as 450 mm

* Width of Base slab = 0.5 H to 0.6 H

= (0.5 × 5.2) to (0.6 × 5.2)

= 2.6 to 3.12m

∴ Width of Base slab may be taken as 3m

* Height of the stem = Overall height of the Retaining wall − Thickness of Base slab

= 5.2 − 0.45 = 4.75m

* ~~Height~~ width of the stem at bottom = Thickness of the stem

= 450 mm

* ~~Height~~ width of the stem at top = 200 mm

* width of toe slab = 1m = $6/3$ = $3/3$ = 1

STEP : 2 : DESIGN OF STEM :

* Maximum moment at the bottom of the stem, M

$$= K_a (\gamma h^3/6)$$

$$\left[K_a = \frac{1 - \sin 30}{1 + \sin 30} = 0.3 \right]$$

$$= 0.33 \times (18 \times 4.73^3/6) \quad \left[\frac{1-\sin 30}{1+\sin 30}\right]$$

$$M = 107.17 \text{ KN.m}$$

Factored moment, $M_u = 1.5 \times 107.17$

$$M_u = 160.75 \text{ KN.m}$$

* Effective depth = Overall Thickness of stem − cover

$$= 450 - 50 = 400 \text{mm}$$

$$\Rightarrow d = \sqrt{M_u / 0.138 f_{ck}\, b} = \sqrt{\frac{160.75 \times 10^6}{0.138 \times 20 \times 1000}} = 241 \text{mm}$$

∴ Hence the effective depth is more than the limiting value

* Ast can be calculated as follows,

(i) $$M_u = 0.87 f_y A_{st}\, d \left[1 - \frac{f_y A_{st}}{f_{ck}\, b d}\right]$$ [From IS 456:2000, clause 38.1 pg No: 96]

$$160.75 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 400}\right]$$

$$A_{st} = 1186 \text{ mm}^2$$

using 16mm ϕ bars, spacing of bars $= \frac{201}{1186} \times 1000$ $\rightarrow \pi/4 \times 16^2$

$$= 169 \simeq 170 \text{mm}$$

∴ provided 16 mm ϕ bars @ 170 mm c/c on both the faces of the retaining wall as vertical reinforcement.

(ii) Distribution reinforcement = ~~0.12~~ % of ~~Ad~~

$$= \frac{0.12}{100} \times 1000 \times 450$$

$$= 540 \text{ mm}^2$$

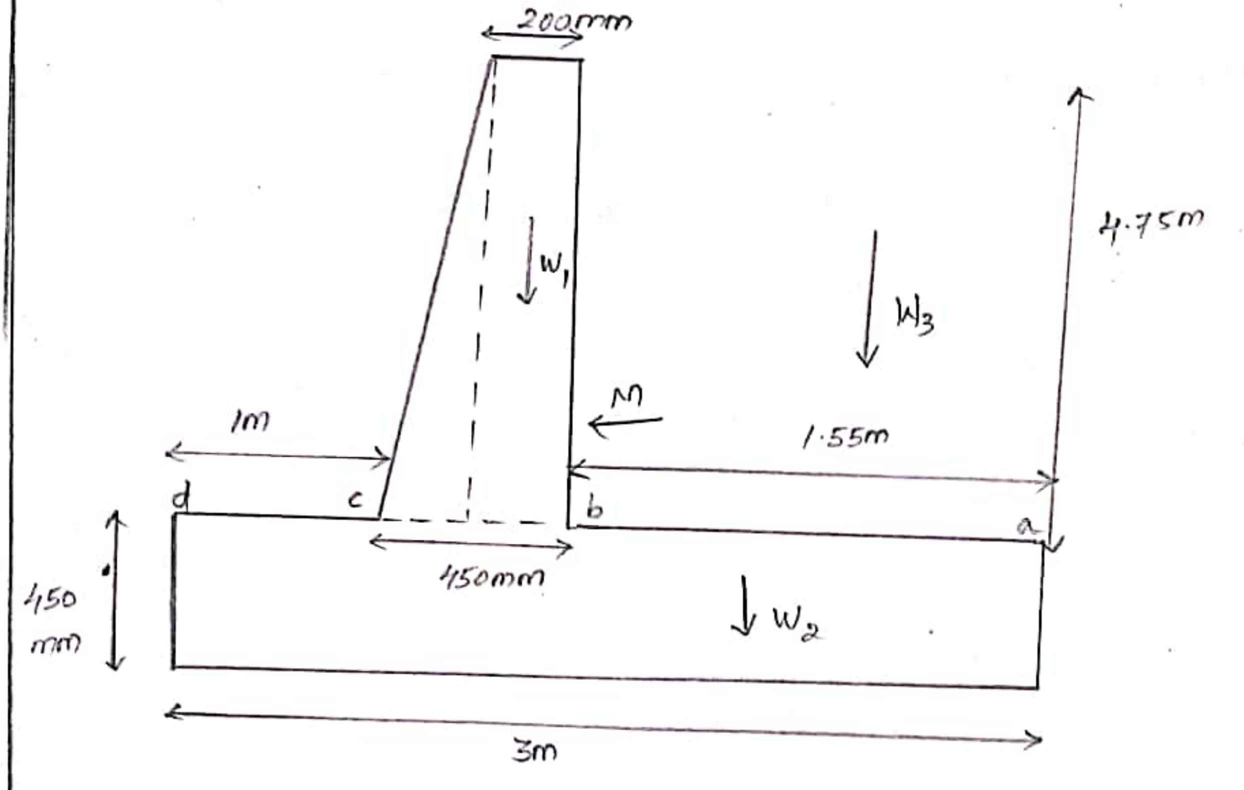
Using 10 mm φ bars, spacing of bars = $\frac{\pi/4 \times 10^2}{540} \times 1000$

$= 145 \text{ mm}$

∴ provided 10 mm φ bars, @ 150 mm c/c on both faces of the stem as distribution reinforcement.

STEP : 3 : STABILITY CALCULATIONS :

(*) soil pressure below base slab :



W_1 = Self weight of stem

W_2 = Self weight of base slab

W_3 = weight of soil above heel slab

M = moment acting at the bottom of the stem considering 1m run of the retaining wall.

Load	Magnitude of load (KN)	Distance from end 'a'	Moment (KN.m)
i) Self weight of stem			
* $25\times0.2\times4.75$	23.75	$1.55+0.2/2 = 1.65$	39.18
* $25\times0.25\times4.75\times\frac{1}{2}$	14.84	$1.55+0.2+0.25/3 = 1.83$	27.20
ii) Self weight of base slab			
$25\times3\times0.45$	33.75	$3/2 = 1.5$	50.62
iii) Self weight of soil above heel slab			
$18\times1.55\times4.75$	132.5	$1.55/2 = 0.775$	102.68
iv) Moment acting at the bottom of the stem			
$\Rightarrow K_a\times\gamma h^3/6$	-	-	107.17
	$\Sigma W = 204.84$ KN		$\Sigma M = 326.85$ KN.m

The distance of resultant force from end 'a', $z = \frac{\Sigma W}{\Sigma M} \quad \frac{\Sigma M}{\Sigma W}$

$= \frac{204.9}{326.8} \quad \frac{326.8}{204.9}$

$\simeq 1.6m$

Eccentricity, $e = z - B/2$ [∴ B = Width of base slab]

$= 1.6 - 3/2$

$e = 0.1m$

$b/6 = 3/6 = 0.5$

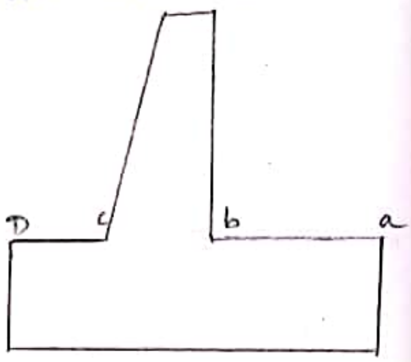
∴ $e < b/6$, Hence safe.

The soil pressure below the base slab is compressive.

∴ P_{max}, Maximum pressure will occur at end 'd'

P_{min}, Minimum pressure will occur at end 'a'

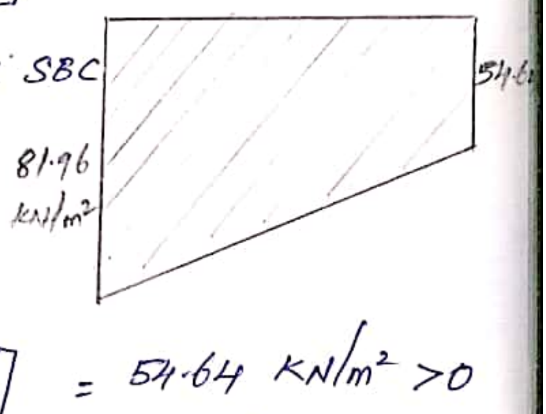
$P_{max} = \frac{\Sigma W}{B}\left[1 + \frac{6e}{B}\right]$



$= \frac{204.9}{3}\left[1 + \frac{(6 \times 0.1)}{3}\right]$

∴ $P_{max} = 81.96\ KN/m^2 < SBC$

Hence safe



$P_{min} = \frac{\Sigma W}{B}\left[1 - \frac{6e}{B}\right]$

$= \frac{204.9}{3}\left[1 + \frac{(6 \times 0.1)}{3}\right] = 54.64\ KN/m^2 > 0$

∴ Hence safe

* Check for safety against sliding Overturning (OR) Factor of Safety against overturning :

$$\text{Factor of safety} = 0.9 \times \frac{M_R}{M_O} > 1.4$$

where,

M_R = Resisting moment

M_o : Overturning moment.

$$M_R = \Sigma W[B-z]$$

$$= 204.9[3-1.6] = 286.86 \text{ KN.m}$$

$$M_o = K_a(\gamma h^3/6)$$

$$= 107.17 \text{ KN.m}$$

$$FOS = 0.9 \times \frac{286.86}{107.17}$$

$$= 2.4 > 1.4$$

∴ Hence safe

* check for safety against sliding (OR) Factor of safety against sliding :

$$\text{Factor of safety} = 0.9 \times \frac{F_R}{F_s}$$

where,

F_R = Resisting Force ; F_s = Sliding Force

$F_R = \mu \Sigma W$

$= 0.5 \times 204.9 = 102.45$ KN

$F_S = K_a \gamma H^2/2$

$= 0.33 \times \frac{18 \times 5.2^2}{2} = 80.30$ KN

$FOS = 0.9 \times \frac{F_R}{F_S} > 1.4$

$= 0.9 \times \frac{102.45}{80.30}$

$= 1.14 < 1.4$

∴ Hence unsafe, A shear key has to be provided at the base slab.

* Design of shear key :

Assume the depth of shear key as 600 mm

The intensity of passive Earth pressure infront of shear key

$P_p = K_p \times \gamma h_s^2/2$

where K_p = co-efficient of passive Earth pressure

h_s = Depth of shear key + Thickness of base slab

$= 0.6 + 0.45 = 1.05$ m

$$K_p = 1/K_a = \frac{1+\sin 30}{1-\sin 30} = 3.03$$

$$P_p = 3.03 \times 18 \times \frac{1.05^2}{2}$$

$$P_p = 30.06 \ kN/m^2$$

$$FOS = 0.9 \times \left[\frac{F_R + P_p}{F_S}\right]$$

$$= 0.9 \times \left[\frac{102.45 + 30.06}{80.30}\right]$$

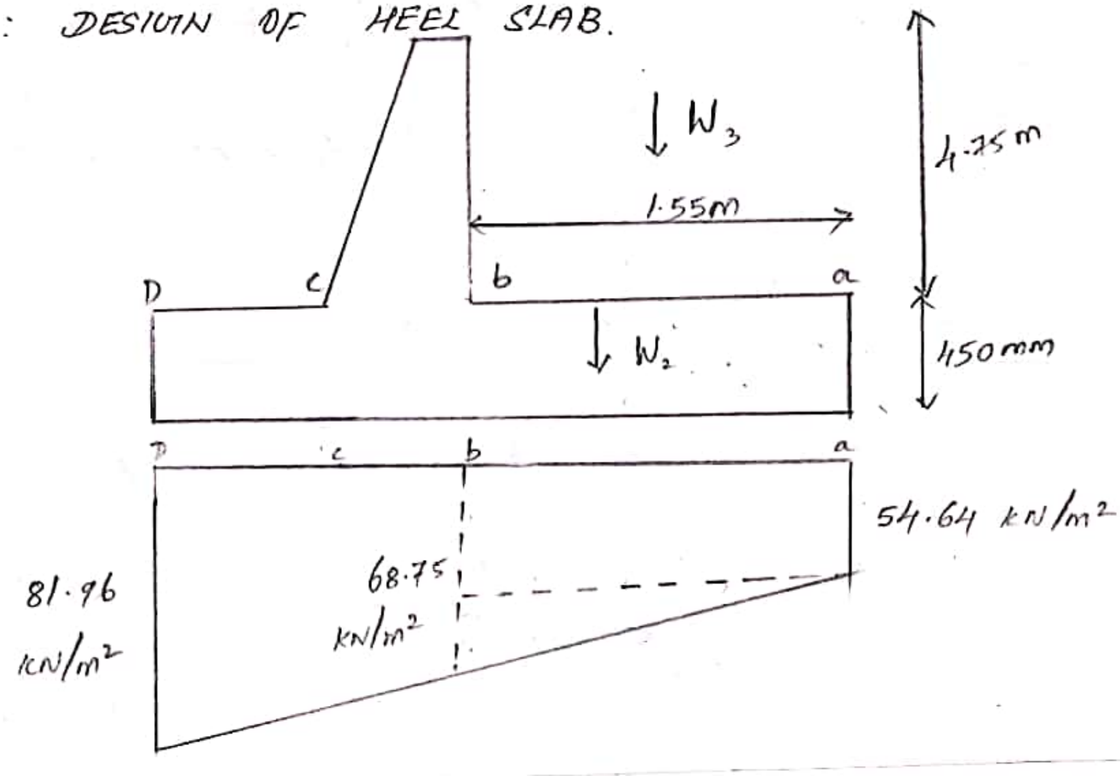
$$FOS = 1.48 > 1.4$$

$\therefore$ Hence safe

The Retaining wall is safe against failure due to sliding.

The reinforcement in stem is Extended upto the shear key.

STEP 4: DESIGN OF HEEL SLAB.



The soil pressure at the fixed end of the heel slab can be calculated as follows:

$$* \quad P_{min} + \left[\frac{(P_{max} - P_{min})}{\text{width of base slab}} \times \text{width of heel slab} \right]$$

$$\Rightarrow 54.64 + \left[\frac{(81.96 - 54.64)}{3} \times 1.55 \right]$$

$$\Rightarrow 68.75 \ kN/m^2.$$

Load	Magnitude of load (kN)	Distance from end 'a'	Moment (kN.m)
(i) Self weight of heel slab * 25 × 1.55 × ~~0.50~~ 0.45	17.43 (↓)	$1.55/2 = 0.775$	13.50 (↓)
(ii) Self wt of soil above heel slab * 18 × 4.75 × 1.55	132.52 (↓)	$\frac{1.55}{2} = 0.775$	102.70 (↓)
(iii) upward soil pressure			
i) Rectangular portion * 1.55 × 54.64	84.69 (↑)	$\frac{1.55}{2} = 0.775$	65.63 (↑)
(ii) Triangular portion * ½ × 1.55 × (68.75−54.64) ~~(81.96−68.75~~	10.94 (↑)	$\frac{1.55}{3} = 0.52$	5.68 (↑)

$\Sigma W = 54.32$ KN $\qquad \Sigma M = 44.89$ KN.m

Ultimate Bending moment $= 1.5 \times 44.89$

$= 67.33$ KN.M.

Ast can be calculated as follows,

$$Mu = 0.87\, fy\, Ast \cdot d \left[1 - \frac{fy\, Ast}{fck\, bd}\right]$$

$$67.33 \times 10^6 = 0.87 \times 415 \times Ast \times 400 \left[1 - \frac{415 \times Ast}{20 \times 1000 \times 400}\right]$$

$Ast = 478\ mm^2$

$\therefore$ using 12 mm ϕ bars, spacing of bars $= \dfrac{\pi/4 \times 12^2}{478} \times 1000$

$= 236$ mm

provided 12 mm ϕ bar @ 200 mm c/c on both the faces of heel slab as main reinforcement.

* Distribution reinforcement :

using 10 mm ϕ bars, spacing of bar =

• $Ast = 0.12\,\%$ of bD

$= \dfrac{0.12}{100} \times 1000 \times 450 = 540\ mm^2$

ast/Ast × 1000

3d

300 mm

using 10mm dia. bars, spacing of bars $= \dfrac{\pi/4 \times 10^2}{540} \times 1000$

$= 145$ mm

$= 78.53$

provided 10 mm ϕ bars @ 150 mm c/c on both the faces of heel slab as distribution reinforcement.

Dead wt of soil over toe slab (1.2 × 0.45) (ii) upward soil pressure	13.5		
* Rectangular portion 1 × 72.85	72.85 (↑)	1/2 = 0.5	36.42 (↑)
* Triangular portion 1/2 × (81.96 − 72.85) × 1	4.55 (↑)	2/3 × (1) = 0.67	3.04 (↑)
	$\Sigma W = 66.15$		$\Sigma M = 33.84$ ~~kN/m~~

Factored Bending Moment = 1.5×33.84

$= 50.76$ KN.m

Ast can be calculated as follows,

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

$$50.76 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left[1 - \frac{415 \times A_{st}}{20 \times 1000 \times 400}\right]$$

$$A_{st} = 358 \text{ mm}^2$$

Using 12mm φ bars, spacing of bar = $\frac{314}{358} \times 1000$

$= 315$ mm

∴ provided 12mm φ bar @ 300 mm c/c on both the faces of toe slab as main reinforcement.

* Distribution reinforcement:

using 10 mm φ bars, i.e.

$= 0.12\%$ of bD

$= \frac{0.12}{100} \times 1000 \times 450 = 540 N/mm^2$

using 10 mm φ bars, spacing of bars = 145 mm

∴ provided 10mm φ bars @ 150mm c/c on both the faces of toe slab as distribution reinforcement.

* STEP 6: CHECK FOR SHEAR STRESS AT JUNCTION OF STEM AND BASE SLAB:

Net working shear force, $V = (1.5P - \mu W)$

$= [(1.5 \times 80.30) - 102.45]$

$V = 18 KN$

Factored shear force, $V_u = 1.5 \times 18 = 27 KN$

Nominal shear stress, $\tau_v = \frac{V_u}{bd}$ [Pg NO: 72]

$= \frac{27 \times 10^3}{400 \times 1000} = 0.067 N/mm^2$.

To find τ_c, $\frac{100 A_{st}}{bd} = \frac{100 \times 1\cancel{78}6}{1000 \times 400} = 0.3$

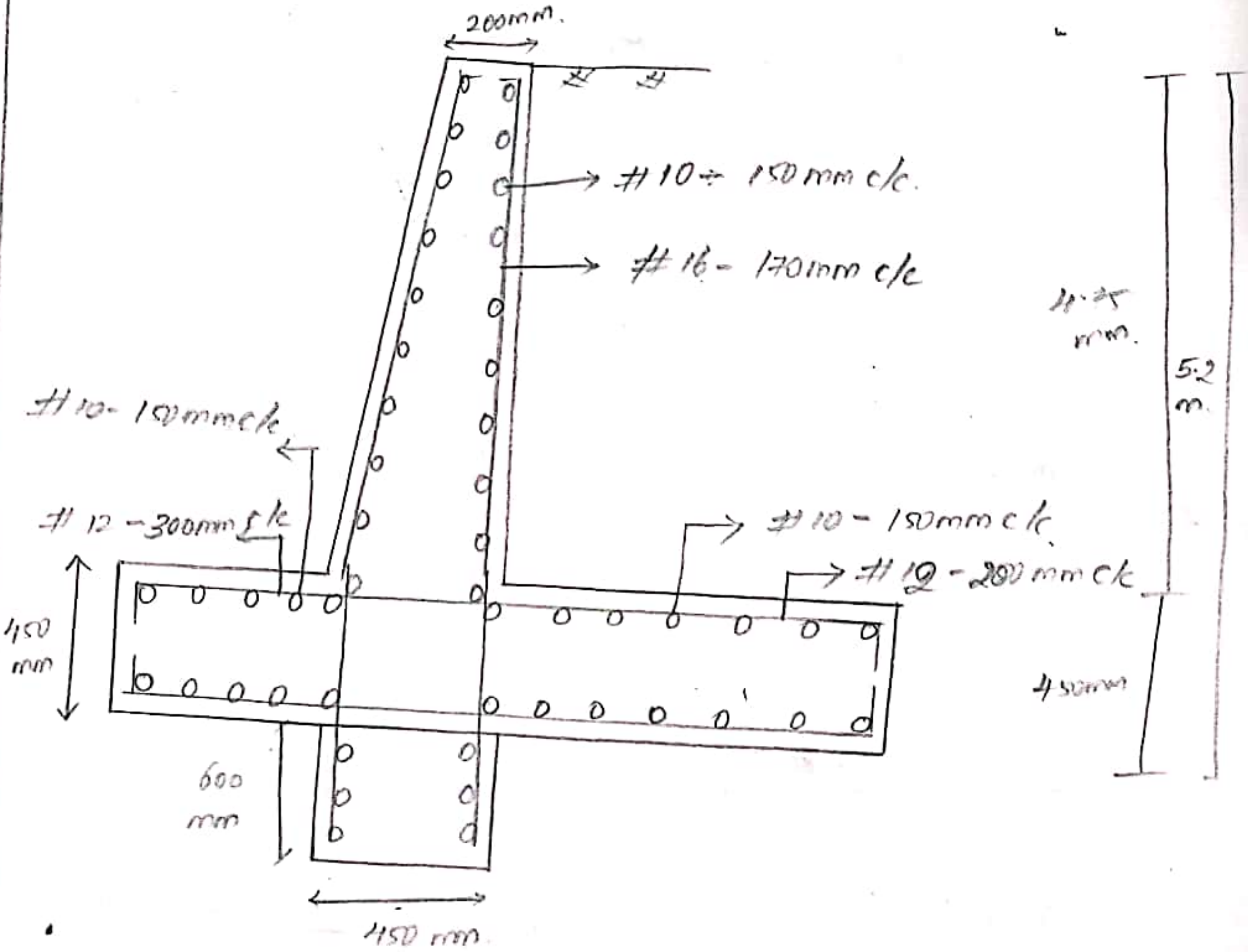
From table 19 of IS 456, the permissible shear stress

$\tau_c = 0.42$

$\tau_c' > \tau_c$

Hence shear stress are within safe permissible limits.

STEP 7 : REINFORCEMENT DETAILING:



Design of counterfort Retaining wall:

As stated earlier counterfort retaining wall are economical for wall height greater than 6m. The design of counterfort retaining wall comprises of

⇒ Design of counterfort

⇒ Design of vertical slab

⇒ Design of toe slab

⇒ Design of Heel slab.

a) General Features:

* Retaining walls of height over 6m are usually provided with counterforts

* The counterfort retaining walls consists of the stem (or) upright slab, toe slab, heel slab and the counterfort spaced at regular intervals

* Spacing of counterforts = L = 3 to 3.5m

* Thickness of base slab = $\sqrt{LH}$ cm

L = spacing of counterfort in m

H = Overall height of the retaining wall (m)

* Base width = $0.6H$ to $0.7H$

* Toe projection = 1/4 width of base slab

b) Design principles:

i) Stem (or) upright slab is designed as continuous slab to span b/w counterforts

$$Max \cdot BM = \left(\frac{PL^2}{12}\right)$$

where,

P = pressure Intensity at base

$= K_a \gamma h$

ii) Toe slab is designed for soil pressure and dead weight of slab.

iii) Heel slab is designed as continuous slab supported b/w counterforts to resists of soil and upward pressure at base.

iv) counterforts thickness is the same as the base slab counterforts are designed to take lateral earth pressure

Max BM in counterfort: $K_a \cdot \frac{wh^3}{6} \cdot L$

where,

h = height of retaining wall above base

L = spacing of counterfort.

2. Design a counterfort type retaining wall to suit the following data.

⇒ Height of wall above ground level = 6m

⇒ SBC of soil at site, $P = 160\ kN/m^2$

⇒ Angle of internal friction = 30°

⇒ Density of soil, $\gamma = 16\ kN/m^3$

⇒ Spacings of counterforts = 3m c/c

Materials M_{20} grade concrete & Fe_{415} HYSD Bars.

S/:

S-I : Dimensions of Retaining wall :

* Minimum depth of foundation $= \frac{P}{\gamma} \times Ka^2$

$$= \frac{P}{\gamma}\ \frac{160}{16}\left(\frac{1-\sin 30}{1+\sin 30}\right)^2$$

$$d = 1.2m.$$

Overall height of wall = 1.2 + 6

$$H = 7.2m.$$

Thickness of Base slab = 2LH cm

$$= 2\times3\times7.2 = 43.2\ cm$$

$$b = 450\ mm$$

provided 450 mm thick base slab

Base width, $B = 0.6H$ to $0.7H$

$= (0.6 \times 7.2)$ to (0.7×7.2)

$= 4.32$ to 5.04

∴ B = 4.- Width of the base slab can be taken as 4.5m

∴ Toe projection = $1/4 \times$ width of base slab

$= 1/4 \times 4.5 = 1.12$ m

∴ width of toe slab can be taken as 1m.

Height of the stem = overall height of the retaining wall - thickness of base slab

$= 7.2 - 0.45 = 6.75$ m

* STEP - 2: Design of stem:

The stem has to be designed as continuous slab spanning between counterforts:

∴ The pressure intensity at the bottom of stem,

$P = K_a \gamma h$

$= 0.33 \times 16 \times 6.75$

$P = 35.64$ KN/m²

Bending moment ~~at~~ the stem, $= \frac{Pl^2}{12}$

$= \frac{35.64 \times 3^2}{12}$

$= 26.73$ kN-m

∴ ultimate moment, $M_u = 1.5 \times 26.73 = 40.09$ kN-m

∴ Thickness of stem can be calculated as follows :

Effective depth, $d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$

$= \sqrt{\frac{40.09 \times 10^6}{0.138 \times 20 \times 1000}}$

$d = 120.52$ mm

∴ Thickness of stem can be taken as 200mm (uniform thickness from top to bottom)

∴ $D = 200$ mm, $d = 150$ mm.

Ast can be calculated as follows,

$$M_u = 0.87 \times f_y \times A_{st} \times d \left[1 - \frac{f_y A_{st}}{f_{ck} b d}\right]$$

$$40.09 \times 10^6 = 0.87 \times 415 \times A_{st} \times 150 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 150}\right]$$

$A_{st} = 837$ mm^2.

Using 12mm ϕ bars, spacing of bars : $\frac{\pi/4 \times 12^2}{837} \times 1000$

$= 135$ mm.

$\therefore$ provided 12mm ϕ bars @ 150 mm c/c on both the faces of the stem as main reinforcement.

* Distribution reinforcement = 0.12% of bD

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

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

Using 10 mm ϕ bars = 327 mm

$\therefore$ provided 10 mm ϕ bars @ 300 mm c/c on both the faces of stem as Distribution Reinforcement.

STEP : 3 : STABILITY CALCULATIONS:

$\Rightarrow$ Considering 1m run of the wall.

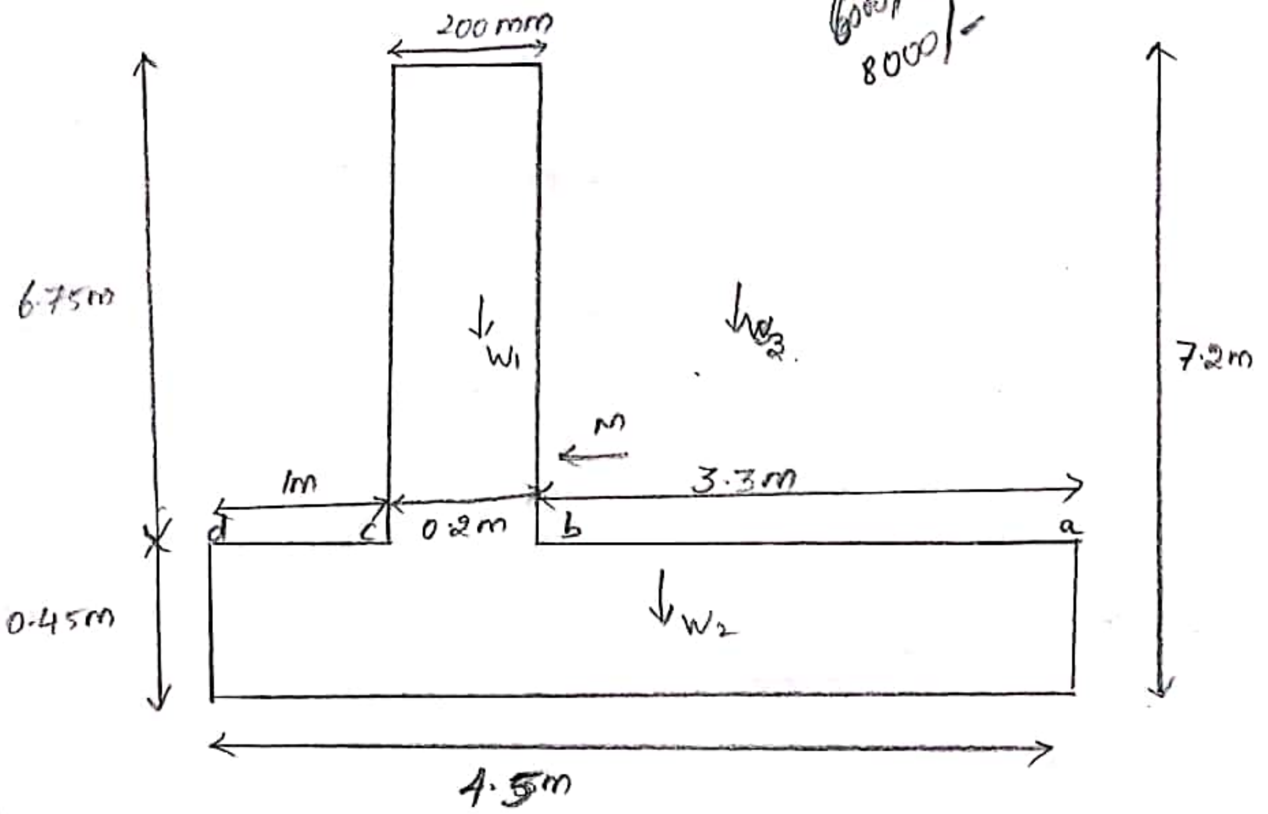
Load	Magnitude of load (KN)	Distance from end 'a' (m)	Moment KN-m.
i) Self weight of stem $\Rightarrow 25 \times 0.2 \times 6.75$	33.75	$3.3 + 0.2/2 = 3.4$	114.75.
ii) Self weight of base slab $\Rightarrow 25 \times 4.5 \times 0.45$	50.62	$4.5/2 = 2.25$	113.89
iii) weight of soil above heel slab $\Rightarrow 16 \times 6.75 \times 3.3$	356.4	$3.3/2 = 1.65$	588.06

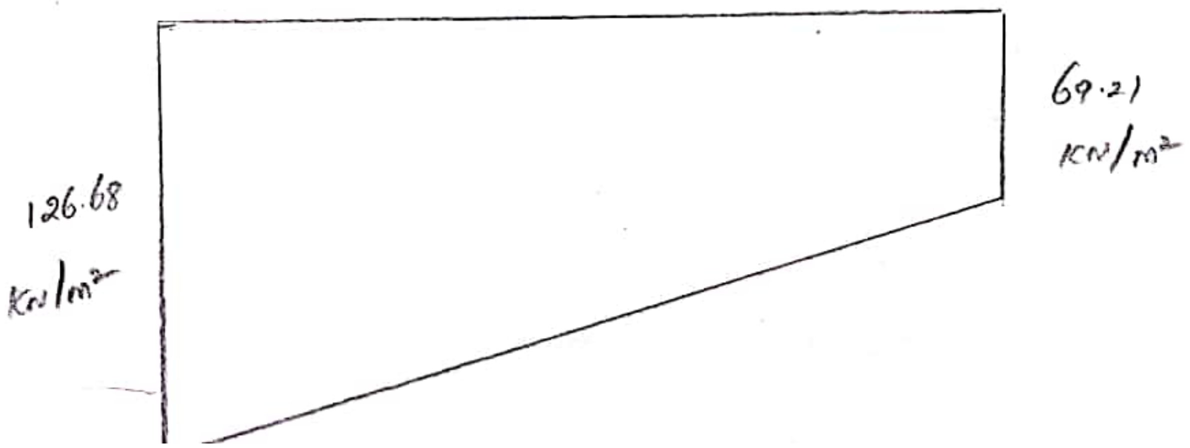
iv) Moment of earth pressure $\Rightarrow K_a \times \gamma h^3/6$ $\Rightarrow 0.33 \times (16 \times 6.75^3/6)$ $\left(\frac{1-\sin 30}{1+\sin 30}\right)$	–	–	273.37 KN-m

$\Sigma W = 440.77$ KN

$\Sigma M = 1090.07$ KN-m ✓

6000/-
8000/-





The distance from resultant Force end 'a', $x = \frac{\Sigma M}{\Sigma W}$

$$= \frac{1090.07}{440.77}$$

$$= 2.47\,m$$

$\therefore$ Eccentricity, $e = x - B/2$

$$= 2.47 - 4.5/2$$

$$e = 0.22$$

$$b/6 = \frac{4.5}{6} = 0.75$$

$\therefore$ $e < b/6$, Hence safe.

The soil pressure below the slab is compressive

$\therefore$ P_{max}, Maximum pressure will occur at end 'd'

P_{min}, Minimum pressure will occur at end 'a'

$$P_{max} = \frac{\Sigma W}{B}\left[1 + \frac{6e}{B}\right]$$

$$= \frac{440.77}{4.5}\left[1 + \frac{(6 \times 0.22)}{4.5}\right]$$

$$P_{max} = 126.68\ kN/m^2 < SBC$$

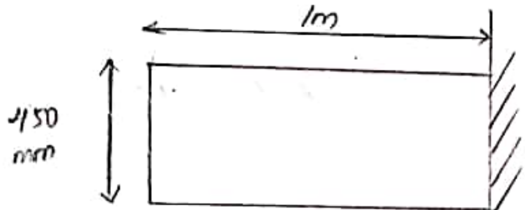
$$P_{min} = \frac{\Sigma W}{B}\left[1 - \frac{6e}{B}\right]$$

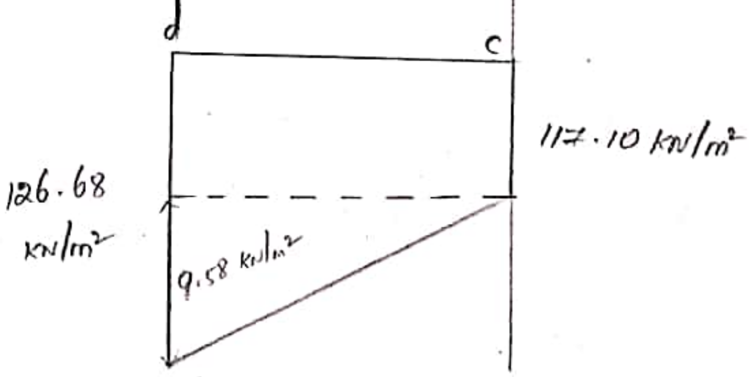
$$= \frac{440.77}{4.5}\left[1 - \frac{(6 \times 0.22)}{4.5}\right] = 69.21\ kN/m^2 < 0$$

$\therefore$ Hence safe.

STEP : 4 : DESIGN OF TOE SLAB :

LOADS	MAGNITUDE OF LOADS (KN)	DISTANCE FROM 'C' (KN)	MOMENT AT 'C' KN-m.
* Self weight of toe slab ⇒ 0.45 × 1 × 25	11.25	$\frac{1}{2}$ = 0.5	5.625
* weight of soil above toe slab ⇒ 0.75 × 1 × 16	12	$\frac{1}{2}$ = 0.5	6
upward pressure : * Rectangular portion ⇒ 1 × 117.10	117.10	$\frac{1}{2}$ = 0.5	58.55
* Triangular portion ⇒ $\frac{1}{2}$ × 9.58 × 1	4.79	$\frac{2}{3}$ = 0.67	3.20
	ΣW = ~~[illegible]~~ KN (↑−↓) 98.64 KN		ΣM = 50.12 KN-m.





The pressure at the fixed end of the toe slab can be calculated as follows;

$$\Rightarrow P_{min} + \left[\frac{(P_{max} - P_{min})}{\text{width of base slab}} * (\text{width of heel slab} + \text{thickness of } \cancel{\text{stem}} \text{ base slab}) \right]$$

$$\Rightarrow 69.21 + \left[\frac{(126.68 - 69.21)}{4.5} \times (3.3 + 0.45) \right]$$

$$\Rightarrow 117.10 \ KN/m^2$$

Ultimate moment = $1.5 \times 50.12 = 75.18$ KN·m

→ Effective depth of toe slab = $450 - 50 = 400$ mm.

Ast can be calculated as follows,

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

$$75.18 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400 \left(1 - \frac{415 A_{st}}{20 \times 1000 \times 400}\right)$$

$$A_{st} = 535 \ mm^2$$

Using 12 mm φ bars, spacing of bars = 211 mm

∴ provided 12 mm φ bars @ 220 mm c/c on both the faces of toe slab as main reinforcement.

Distribution reinforcement = 0.12 % of bD

$$= \frac{0.12}{100} \times 1000 \times 450 = 540 \text{ mm}^2$$

∴ provided 10 mm φ bars @ 145 mm c/c on both the faces of the slab as distribution reinforcement.

STEP 5: DESIGN OF HEEL SLAB:

Consider 1m wide strip (Continuous slab)

Load	Magnitude
1) weight of soil on strip $1 \times 6.75 \times 16$	108
2) Self weight of heel slab strip $1 \times 0.45 \times 25$	11.25
upward pressure	
i) 1×69.21	69.21

$\Sigma(\uparrow \sim \downarrow) = 50.04$ KN.

∴ Spacing of counterforts = 3m

Maximum Negative service BM at counterfort

$$M = \frac{pL^2}{12} = \frac{50.04 \times 3^2}{12} = 37.53 \text{ KN.m}$$

Scanned with CamScanner

Factored B.M = 1.5 × 37.53 = 56.29 kN.m

Ast can be calculated as follows.

$$M_u = 0.87\, f_y\, A_{st}\, d\left[1 - \frac{f_y A_{st}}{f_{ck}\, b d}\right]$$

$$59.29 \times 10^6 = 0.87 \times 415 \times A_{st} \times 400\left[1 - \frac{415\, A_{st}}{20 \times 1000 \times 400}\right]$$

$$A_{st} = 419\ mm^2$$

∴ provided. 12 mm φ bars @ 270 mm c/c on both the faces on heel slab as main reinforcement.

* Distribution Reinforcement = 0.12 % of bD

$$= \frac{0.12}{100} \times 1000 \times 450 = 540\ mm^2$$

∴ provided 10 mm φ bars @ 145 mm c/c on both the faces of heel slab as distribution reinforcement.

STEP 6: DESIGN OF COUNTERFORTS!

Thickness of the counterfort = 2 × thickness of the stem

∴ b = 2×200 = 400 mm

Overall depth of the counterfort at the bottom

= width of the base slab - width of toe slab

= 4.5 - 1

= 3.5 m = 3500 mm

Effective depth, $d = 3450$ mm

$\therefore$ Maximum working moment in counterfort

$$M = Ka \cdot \frac{\gamma h^3}{6} \cdot L$$

$$= \frac{1}{3} \cdot \frac{16 \times 6.75^3}{6} \cdot 3$$

$$M = 820.12 \text{ KN-m}$$

Factored moment, $M_u = 1230.18$ KN-m.

Reinforcement at the bottom

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

$$1230.18 \times 10^6 = 0.87 \times 415 \times A_{st} \times 3450 \left[1 - \frac{415 A_{st}}{20 \times 1000 \times 3450}\right]$$

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

But, Min reinforcement as per IS 456 - 2000

$$\text{Min } A_{st} = \frac{0.85 bd}{f_y}$$

$$= \frac{0.85 \times 400 \times 3450}{415}$$

$$\therefore \text{Min } A_{st} = \cancel{2826}\ 2826 \text{ mm}^2$$

Using 25 mm ϕ bars; No of bars $\dfrac{2826}{\pi/4 \times 25^2} = 6$ bars

$\therefore$ provided 6 Nos of 25 mm ϕ bars for counterforts

(OR)

Counterfort has to be designed as Vertical cantilever beam

The Area of steel from BM will be less than that of minimum Area of steel given in IS-456 code.

∴ Area of steel can be calulated as follows, based on IS 456- code.

$$\text{Min Ast} = \frac{0.85\,bd}{f_y} \quad \text{[Pg No: 47]}$$

$$\therefore \text{Min Ast} = \frac{0.85 \times 400 \times 3450}{415} = 2865\ mm^2$$

STEP 7: Connection between the stem and the counterforts:

Consider bottom 1m height of the retaining wall.

The pressure acting at the bottom of the stem

$= K_a \gamma h$

$= 0.33 \times 16 \times 6.75 = 35.64\ KN/m^2$.

Total load transfer to the counterforts $= 35.64 \times 1 \times 3$ → ht → spacing of counterforts.

$= 106.92\ KN$.

Factored force $= 1.5 \times 106.92$

$= 160.38\ KN$

IS 456 : 2000
[pg NO : 73]

Area of steel, $A_{st} = \frac{f_u}{0.87 f_y}$

$= \frac{160.38 \times 10^3}{0.87 \times 415} = 444.20 \ mm^2$

using 2 legged 10 mm ϕ links.,

Spacing of links : $\frac{2 \times 78.53}{444.20} \times 1000 = 353 \ mm$

$\therefore$ provided 2 legged 10mm ϕ Horizontal links @ 300 mm c/c.

STEP 8 : Connection b/w counterfort and heel slab:

Consider 1m run of the heel slab at the end A.

pressure acting on the heel slab, $P_{min} = 69.21 \ kN/m^2$

Force transfer to the counterfort $= 69.21 \times 1 \times 3$

$= 207.63 \ kN.$

Factored load $= 1.5 \times 207.63$

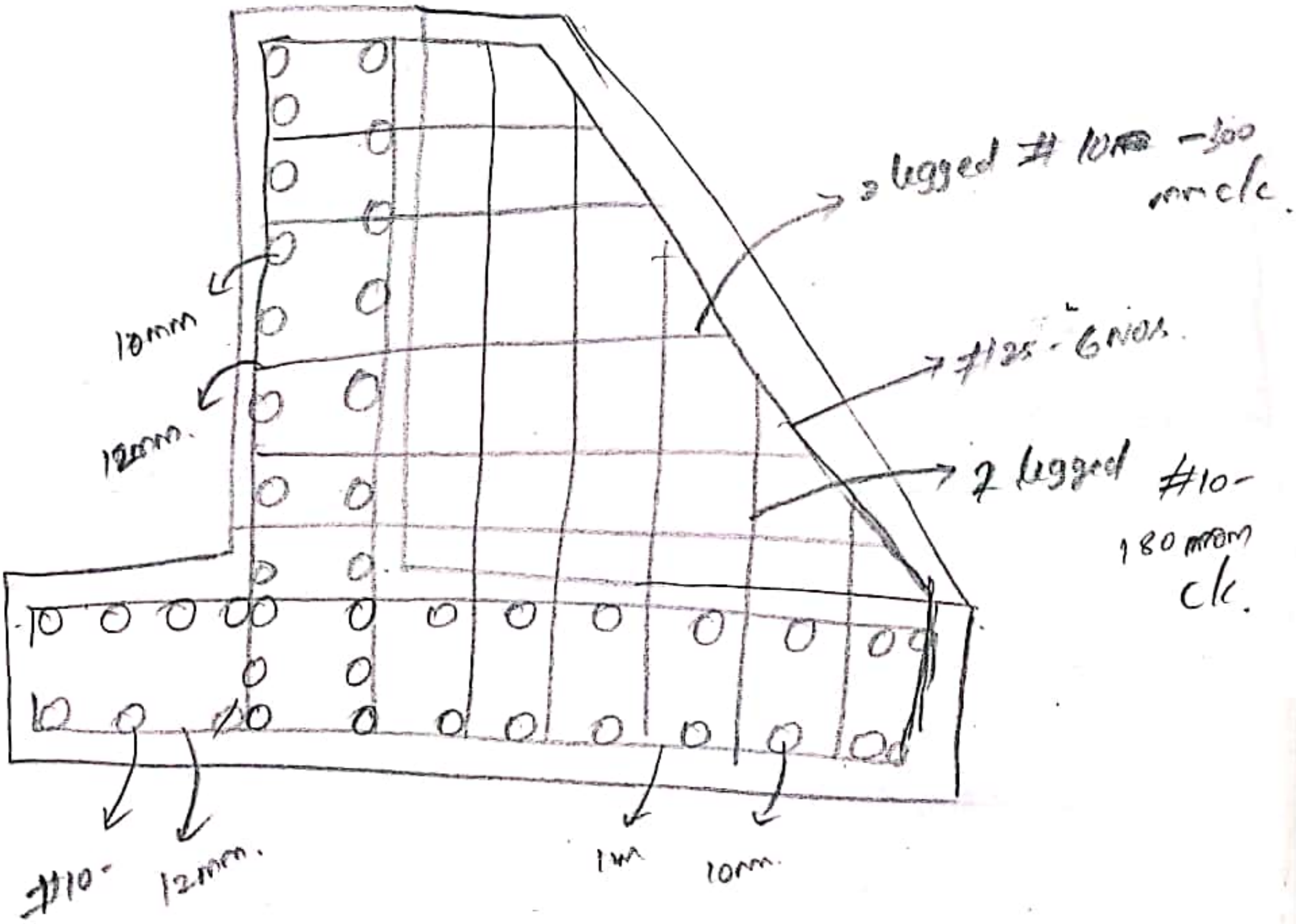
$= 311.45 \ kN.$

$A_{st} = \frac{311.45 \times 10^3}{0.87 \times 415} = 862.60 \ mm^2$

using 2 legged 10 mm ϕ links,

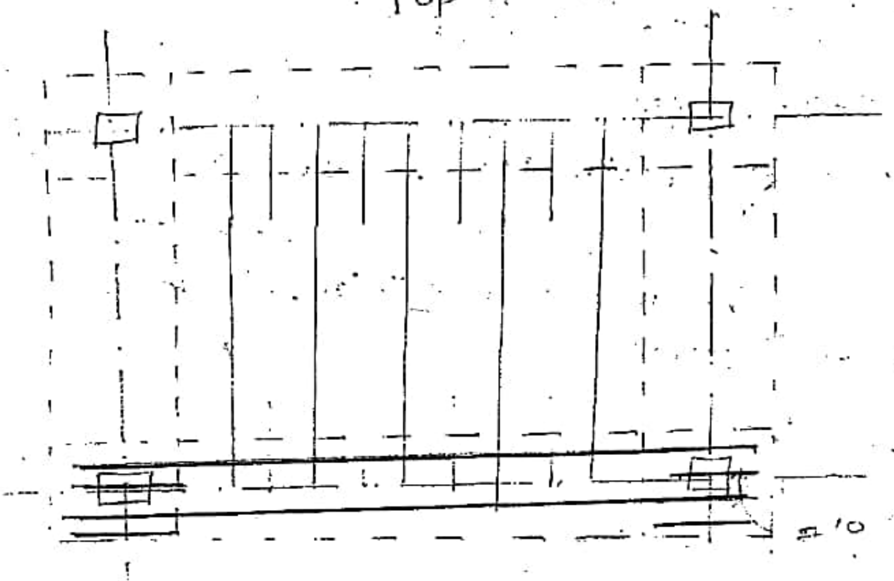
Spacing of links : $\frac{2 \times 78.53}{862.60} = 182 \ mm$

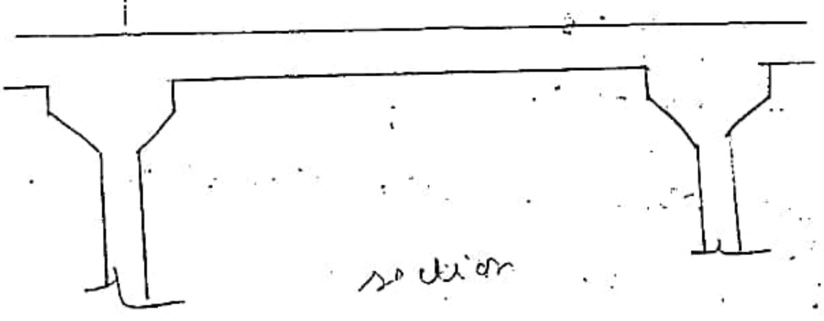
$\therefore$ provided 2 legged 10 mm ϕ vertical links @ 180 mm c/c.



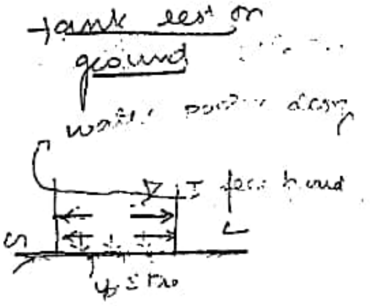
2 legged # 10 — 300 mm c/c.
10mm
12mm.
12 - 6 Nos.
2 legged #10 - 180 mm c/c.
#10 -
12mm.
1m
10mm.

Top plan





section



UNIT-IV.

WATER TANKS

1). IS 3370 - 1965 (working stress)

part - I - G.R

part - II - R.C.C. structure

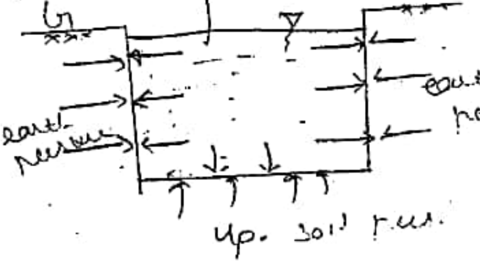
IV - Tables

2. under ground tanks:-

eg. Septic tank.

purification tank

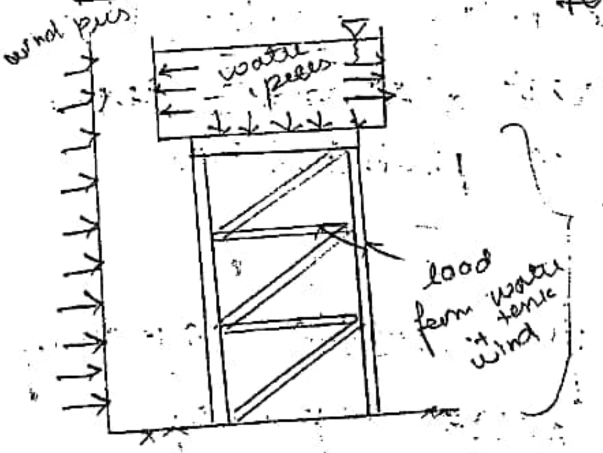
gas moulder



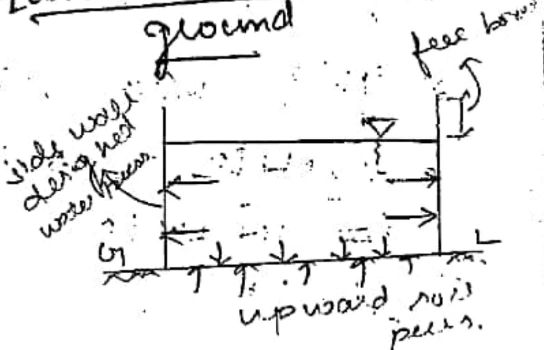
→ side wall designed water press. and earth press

→ bottom designed by water press. and upward soil press.

3. over head tanks: eg water tank



1. tank resist on the ground

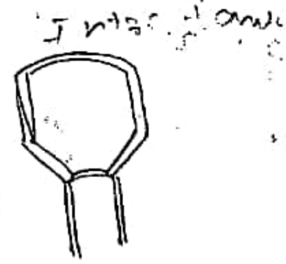


bottom design water press upward soil pres.

from design pt of view the tank classified

as per shape:-

1. * ⊔ tank
2. * O tank
3. * spĥeri cal tank
4. * Intze tank ⟶
5. * conical bottom tank



min grade of conc : 2009 - M25
IS 3370 ; 1965 - M20

→ Limit state method allowing the tank cracks for 0.1 to 0.2mm.

→ But in previous code IS 3370 : 1965 - working stress method not allowing the cracks on tank.

Design Requirement of concrete in Liquid storage:-

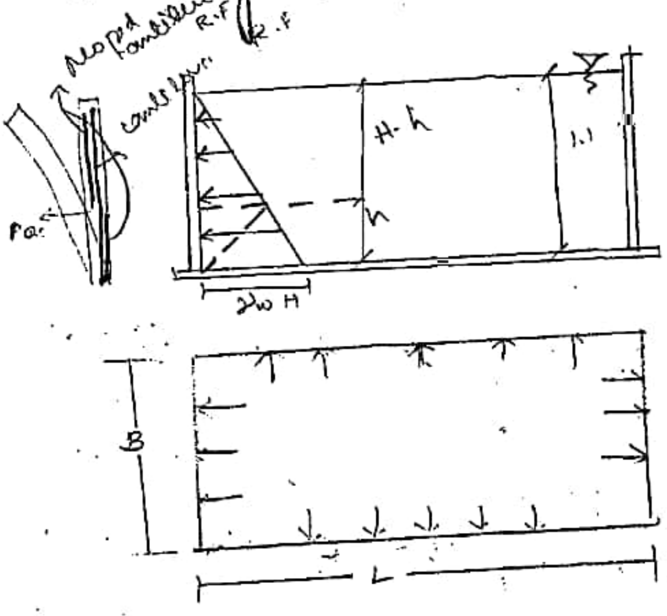
* The concrete should be dense impermeability and high quality.
* concrete mix greater than equal to M20 (i.e) min. grade is M20
* min quantity of cement in concrete mix is ≮ 300 kg/m³

* proper w/c ratio can be used.
* Join used should be made water tight.
* concrete should not crack, tensile stress in concrete should be permissible limit.

* ▭ is economical for smaller capacity
* O is economical for large capacity.

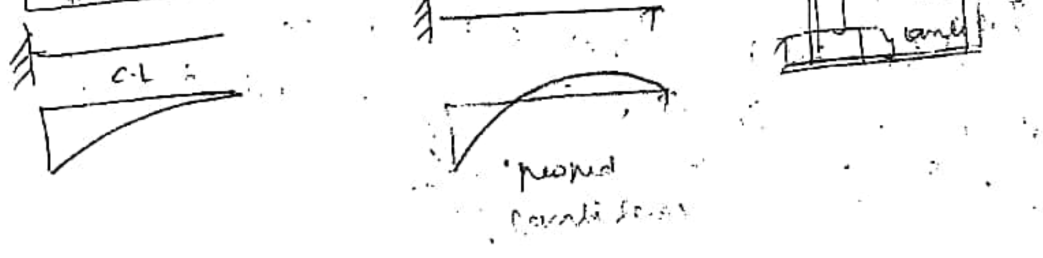
→ ▭ subjected to B.M & direct tension.
→ O subjected to hoop tension & B.M.

Rectangular tank :- (▭) :- $L/B \leq 2$ (slab 2 ways)



* tank walls are designed as continuous frame subjected to pressure varying from zero @ the top to max @ H/4 (or) 1m from base whichever is more.

* The bottom portion H/4 (or) 1m whichever is more is designed as "cantilever"



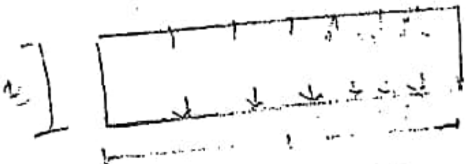
* walls are subjected to direct T & Bending.
(T = tension)

* Bending mt in walls are counted in any elastic method.

* direct tension in long wall $= \dfrac{w(H-h)B}{2}$

* direct tension in short wall $= \dfrac{w(H-h)L}{2}$

⇒ long wall tension due to short wall bending
⇒ short wall tension due to long wall "



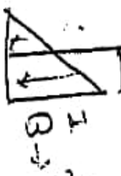
$L/B > 2$

⇒ long wall can be designed as a cantilever
⇒ and the short wall designed as a slab supported on long wall

⇒ The bottom $H/4$ (or) 1 m which ever is more in short wall can be designed as "cantilever" → R.F

long walls:-

⇒ max B.M @ base



$$\boxed{BM = \frac{wH^3}{6}}$$

short wall:-

max B.M @ support:-

$$\boxed{\text{B.M @ support} = \frac{w(H-h)B^2}{12}}$$

$$\boxed{\text{@ mid span or centre, B.M} = \frac{w(H-h)B^2}{16}}$$

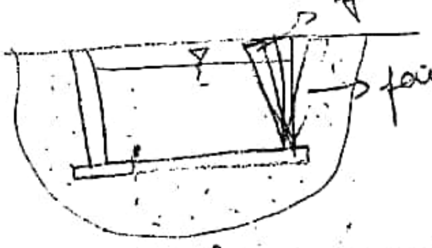
⇒ Bottom portion of a short wall, which is designed as "cantilever".

⇒ cantilever max BM for bottom portion:-

$$\boxed{BM = \frac{wHh^2}{6}}$$

$h = H/4$ (or) 1m which is more

⇒ in addition to BM, short wall / long wall is subjected to direct tension



under ground tank

elastic limit of working stress upto

28/8/12

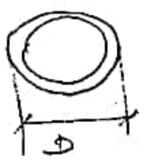
cylindrical Tank:-

side wall flexible base

fixed base

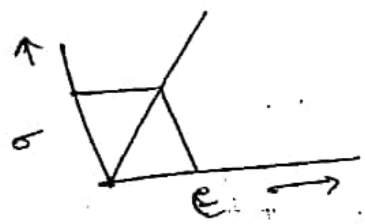
side wall:-

* monolithically constructed
* Hoop tension & BM.
* co-efficient taken from - Design tables IS 3370 - part - IV

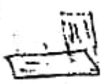


hoop tension $= p \times D/2$

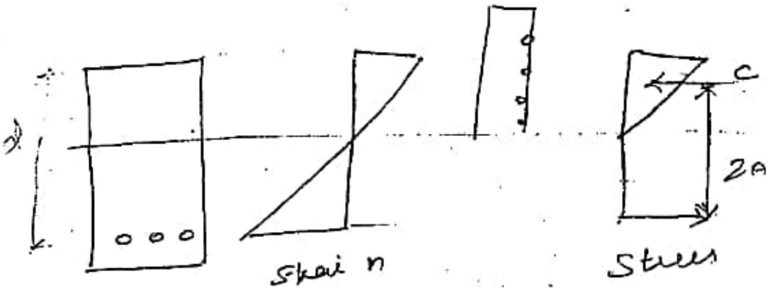
upto elastic limit variation is linear



in flexible base and the wall designed separately



in rigid the base and the steel of the wall should be designed monolithically



IS 3370 - part - II

<u>min. R.F:</u>

. cl. 7.0 pg : 13

0.3% $\Rightarrow$ 100mm Tk of wall

0.2% $\Rightarrow$ 450 mm "

<u>min cover:</u>

25 mm (or) dia of bar, which is greater.

1). design a circular water Tank of capacity 2.5 l akhs the depth of water in the tanker limited 3m with 0.25 m free board the joints of the wall & the base slab are flexible & base slab rest on the ground. use M_{20} & Fe 415 steel.

<u>solution:-</u> (approximate method)

<u>step:</u> <u>Dimensions:-</u>

capacity = 2.50 lakhs lit

$= \frac{2,50,000}{1000}$

$\boxed{V = 250 \ m^3}$

$\left(\frac{\pi D^2}{4}\right) 3 = 250$

$D = 10.3$

$\left(\frac{\pi D^2}{4}\right) 2.75 = 250$

$\boxed{D = 10.76 \simeq 11 m}$

hoop tension = $wh \cdot \frac{D}{2}$

$= 9.81 \times 2.75 \times \frac{11}{2}$ = 148.37

$(A_{st})_{reqd} = \frac{T}{\sigma_{st}}$

pg: 8 - part - II IS 3370

$\boxed{\sigma_{st} = 150 \ kN/m^2}$

$(A_{st})_{req} = \frac{148.37 \times 10^3}{150}$

$\boxed{(A_{st})_{re} = 989.13 \ mm^2}$

use 12 mm φ

$no = \frac{989.13}{\frac{\pi \times 12^2}{4}}$

$= 8.74$

$spacing = \frac{989.13}{8.74}$

$\boxed{spacing = 113.09 \ mm}$

→ provide 12 mm φ @ 110 mm c/c

(A_{st}) provided $= \frac{\frac{\pi \times 12^2}{4}}{110} \times 1000$

$\boxed{(A_{st})\ provd = 1028.16 \ mm^2}$

step 2:-

Thks of wall:-

* Thks of the wall shall not less than the following.

1). 150 mm

2). 30 mm per m depth + 50 mm = 140 mm

(30 mm × 3 + 50) = 140 mm

3). Tensile stress requirement

for M_{20} concrete allowable tensile stress $= 1.2 \ N/mm^2$ (12 kg/cm²)

/ pg: 7 - II /

permissible tensile stress $= \dfrac{F_t}{A_{ct} + m A_{st}}$ (IS 456 pg: 80)

$$\boxed{T.S = \frac{F_t}{A_{ct} + m A_{st}}}$$

$m = \dfrac{280}{3\sigma_{cbc}}$ (IS 456, pg: 80).

for M_{20} concrete

$\sigma_{cbc} = 7\ N/mm^2$ (from table 21 pg: 81)

$$\boxed{m = 13.333}$$

$A_c = b \times t$

$b = 1000$

$$1.2 = \frac{148.37 \times 10^3}{(1000 \times t) + 13.33\,(1028.16)}$$

$\quad A_g$

$$\cancel{1200\,t + 16446.44 = 148.37 \times 10^3}$$

$$\cancel{\boxed{t = 109.93\ mm}}$$

$$T.s = \frac{F_t}{A_g - A_{st} + m\,(A_{st})}$$

$$= \frac{F_t}{A_g + A_{st}(m-1)}$$

$$1.2 = \frac{148.37 \times 10^3}{1000 \times t + 1028.16\,(13.33 - 1)}.$$

$15212.65 + 1200\,t = 148370$

$$\boxed{t = 110.97\ mm}$$

Let us provide over all thks of "150 mm"

30/8/12

Step 3:-

Vertical Reinforcement (distribution steel)

This shall be atleast 0.3% for the cross area

$$= \frac{0.3}{100} \times \overset{b}{1000} \times \overset{D}{150}$$

$$= 450 \text{ mm}^2 \text{ (Main Reinforcement)}$$

for each face $= \frac{450}{2} = 225$

use 8mm dia bars,

$$\text{spacing} = \frac{\frac{\pi \times 8^2}{4}}{225} \times 1000$$

$$\boxed{\text{spacing} = 223.4 \text{ mm}}$$

provide # 8mm ϕ bars @ 220 mm c/c on both faces.

Step 4:-

Design of base slab:-

The base slab will be laid on a 75mm lean mix bed covered with tarfelt since the load gets transfer to ground directly. a nominal thickness of 150mm nearly provided with min reinforcement in both the direction.

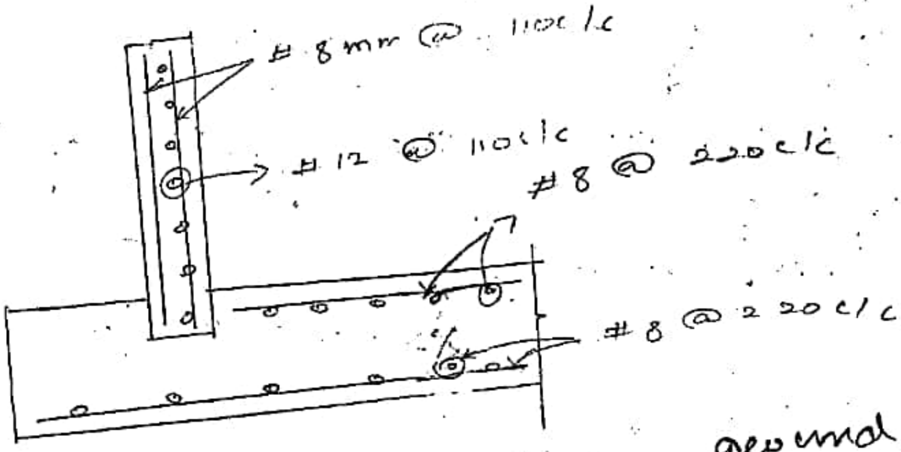
$$\text{min R.F reqd} = \frac{0.3}{100} \times 1000 \times 150$$

$$= 450 \text{ mm}^2 \text{ (M.A)}$$

$$\text{for each face} = \frac{450}{2} = 225 \text{ mm}^2$$

$$\text{spacing} = 223.4 \text{ mm}$$

→ provide # 8 mm ϕ bars @ 220 mm c/c. on



H.W
2) design → flexible circular W.T to rest on ground to store 500000 lit. the depth of tank may be kept as 4m. use M_{25} & Fe415 steel.

overall depth calculate
free board

hoop tension:-
to provide over flow pipes
eff. depth d — free board
wall on base
or construction monolithic

3). design a circular W.T of rigid base rest on ground to store 50000 lit the depth of the tank = 4 m. use M_{25}, Fe415, sketch the wall and base R.F details.
(to avoid leakage problem ... 150 mm)

2). Solution:- (exact method)

$$\text{capacity} = \frac{500000\,\text{lit}}{1000} = 500\,m^3$$

assume a free board of 200mm.

(with out free board considered)

$$500 = \frac{\pi \times d^2}{4} \times 4 \rightarrow \text{eff depth}$$

$$d = 12.6$$

provide $\boxed{d = 13\ m}$

total ht. of the tank $H = 4 + 0.2$

$\boxed{H = 4.2\,m}$

table 9:- part IV pag: 35

Thickness of Tank Wall:-

This is taken as the greater of the following, 150 mm,

$\boxed{\text{Tks} > 150\text{ mm}}$

= 30mm ~~x~~ depth + 50mm

Tks = 176mm

provide a Tks of "180mm"

$$\frac{H^2}{dt} = \frac{4.2^2}{1\cancel{3} \times .18} = \cancel{24.5}\ 7.54$$

from table 9, pg. 35 :- IS 3370 (part IV)

coefficients:-

6	0.514	
7.54		0.56
8	0.575	

* max co-eff for hoop tension for $\frac{H^2}{Dt} = 6$ is 0.514 @ 0.6H depth

* max co-eff for hoop tension for $\frac{H^2}{Dt} = 8$ is 0.575 @ 0.6H depth

max hoop tension = 0.56 x 10 x H x R

$= 0.56 \times 9.81 \times 4.2 \times \frac{13}{2}$

$\boxed{\text{max hoop tension} = 149.97\text{ kN}}$

hoops steel per meter ht :-

$= \frac{\text{hoop tension}}{\text{stress in steel}}$

$= \frac{149.97 \times 10^3}{150}$ ⇒ (taken from table 2, pg 8 IS.3370, part

$\boxed{\text{hoop's steel }/m_{wk} = 999.8\ mm^2}$

use 10mm ϕ bars.

$$\text{spacing} = \frac{\frac{\pi \times 10^2}{4}}{999.8} \times 1000$$

$$\boxed{\text{spacing} = 78.55\ mm}$$

⇒ provide hoop R.F for both faces.

⇒ 10mm dia bars @ 150mm c/c near each face.

Check for tensile stress for concrete:-

$$\text{actual hoop steel } A_{sh} = \frac{\frac{\pi \times 10^2}{4}}{75} \times 1000$$

$$\boxed{(A_{sh})_{pro} = 1047.19\ mm^2}$$

actual Tensile stress $= 149.9 \times 10^3$

$$\boxed{\frac{F_t}{A_c + m A_{st}} = \frac{F_t}{A_g + (m-1) A_{st}}}$$

$$\boxed{A_g = A_g^c + A_{st}}$$

$$m = \frac{280}{3\,\sigma_{cbc}} \Rightarrow \text{(table 21, pg: 81 IS 456)}$$

$$= \frac{280}{3 \times 8.5}$$

$$\boxed{m = 10.98}$$

$$\frac{F_t}{A_g + (m-1) A_{st}} = \frac{149.97 \times 10^3}{(1000 \times 180) + (10.98 - 1)\ 1047.19}$$

$$\boxed{T.S = 0.787\ N/mm^2}$$

Table (1) part II

$\boxed{\text{permissible direct tensile stress} = 1.3\ N/mm^2}$

Table 10:- Pg: 36 :- (part IV). IS - 3370

		(+)ve	-ve
$\frac{H^2}{dt}$	= 6	+ 0.0051	- 0.0187
7.54 $\rightarrow$		0.0047	- 0.0155
$\frac{H^2}{dt}$	= 8	+ 0.0038	- 0.0146

(+ve) B.M = $0.0047 \times 9.81 \times (4.2)^3$

$= 3.4$ kN m (outer R.F)

(-ve) BM = $0.0155 \times 9.81 \times (4.2)^3$

$= 11.26$ kN m (inner R.F)

for M25 & Fe415 steel

$\boxed{m = 10.98}$

$k = \frac{m\,\sigma cbc}{m\,\sigma cbc + \sigma st}$ $\rightarrow$ ~~IS 456~~ - 3370

$= \frac{10.98 \times 8.5}{(10.98 \times 8.5) + 150}$ $\rightarrow$ IS 3370

$\boxed{k = 0.3835}$

$j = 1 - k/3$

$\boxed{j = 0.872}$

$A_{st} = \frac{M}{150 \times j\,d} = \frac{M}{\sigma st \cdot j \cdot d}$

$= \frac{11.265 \times 10^6}{150 \times 0.872 \times 150}$

$\boxed{A_{st} = 572.84 \text{ mm}^2}$

$d = 180 - 25 - \frac{10}{2}$

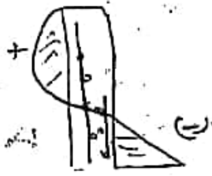
$\boxed{d = 150 \text{ mm}}$

d = 180 - 25 - 10 - $\frac{10}{2}$

=

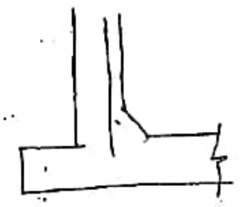
use 10 mm ϕ bars,

$$spacing = \frac{\frac{\pi \times 10^2}{4}}{572.84} \times 1000$$



$\boxed{spacing = 137.1\ mm}$

⇒ provide 10 mm ϕ @ 130 mm c/c on vertically inner face

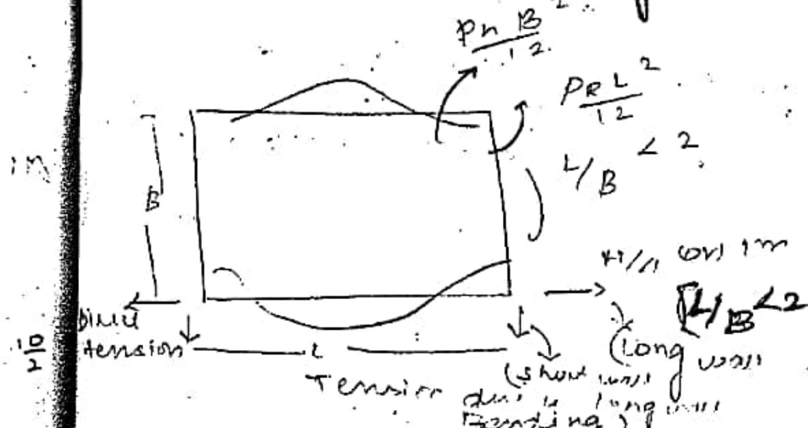


$$\min (A_{st}) = \frac{0.3}{1000} \times 1000 \times 180$$

$\boxed{(A_{st})_{min} = 540\ mm^2}$

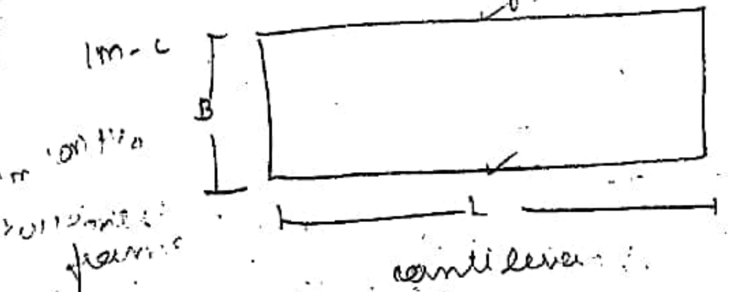
⇒ provide 10 mm ϕ @ 130 mm c/c on vertically on both faces.

30/9/12 Rectangular Tanks



[$L/B < 2$ ⇒ Both the walls are designed as continuous]

Long wall tension due to short wall Bending on ...



[long wall designed as cantilever
$L/B > 2$
short wall designed as continuous]

(water tank Bavi katti — advanced in concrete technology,
(Ramamrutham. have many changes)

Approximate ...

1). An open tank 4 × 3 m × of 2.5 m (d) rest on ground. Design the tank. Use M_{20} & Fe_{415} steel. Approximate method may be used for the analysis. (approximate method)

Solution:-

$$\frac{L}{B} = \frac{4}{3} = 1.33 < 2$$

* Hence the walls will be designed as span in horizontally.

Step 1:-

Design of the wall:-

Tks of the wall shall not be less than ae

1). 150 mm

2). 30 mm per m depth + 50 = 30 × 2.5 + 50 = 125

3). 60 mm per m length of the long er side $= 60 \times 4 = 240$ mm

Tks of wall is "250 mm"

⇒ providing the Tks of short wall = 3.25 m

effective span of long wall $= 4 + \frac{0.25}{2} + \frac{0.25}{2}$

effective span $= 4.25$ m

$\frac{H}{4}$ (or) 1 m $= \frac{2.5}{4}$ (or) 1 m

The bottom 1 m above the base slab can be designed as "cantilever"

water pressure @ H level $= \gamma_w (H-h)$

$= 9.81 (2.5 - 1)$

$= 14.715$ kN/m²

Step 2:- Design of long wall:-

B.M. @ 1m ht @ the corners

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

$$= \frac{14.415 \times 4.25^2}{12}$$

$$\boxed{B.M = 22.149 \text{ KNm}}$$

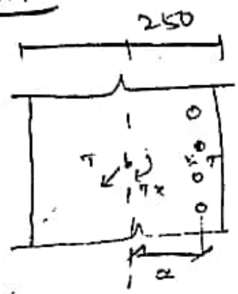
direct pull in wall strip.

$$= 14.15 \times 3/2$$

$$\boxed{\text{direct pull} = 22.0425} \text{ KN}$$

assume use 12 mm ϕ bars with a clear cover of 25 mm

effective cover $= 25 + \frac{12}{2} = 31$ mm



Resultant B.M = M - Tx

$$= 22.149 - \left(22.0425 \times \frac{94}{1000}\right)$$

$$\boxed{B.M_R = 20.074 \text{ KNm}}$$

$$x = \frac{250}{2} - 31$$

$$\boxed{x = 94}$$

→ B.M produces tension on the water face.

$\sigma_{st} = 150$ N/mm² (pg: 8, IS 3370 - part II)

$\sigma_{cbc} = 7$ N/mm² (pg: 81, IS 456)

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

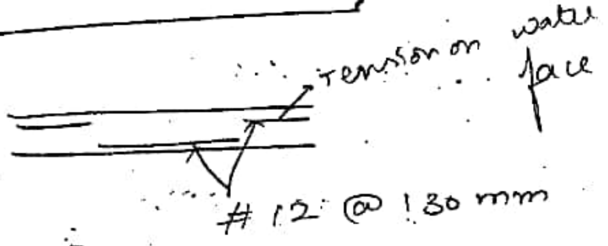
$$\boxed{m = 13.33}$$

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

$$\boxed{k = 0.3835}$$

$j = 1 - k/3$

$$\boxed{j = 0.8721}$$



<u>steel for corner moment:-</u>

steel for B.M $(A_{st})_1 = \dfrac{M}{\sigma_{st} \cdot j d}$

$$= \frac{20.074 \times 10^6}{150 \times 0.872 \times (250 - 31)}$$

$$\boxed{(A_{st})_1 = 700.7 \text{ mm}^2}$$

steel for direct tension,

$$(A_{st2}) = \frac{T}{\sigma_{st}}$$

$$= \frac{22.073 \times 10^3}{150}$$

$$\boxed{(A_{st})_2 = 147.15 \text{ mm}^2}$$

Area of steel $= (A_{st})_1 + (A_{st})_2$

$$= 700.6 + 147.15$$

$$\boxed{(A_{st}) = 847.85 \text{ mm}^2}$$

$$\text{spacing} = \frac{\frac{\pi \times 12^2}{4}}{847.85} \times 1000$$

$$\boxed{\text{spacing} = 133.4 \text{ mm}}$$

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Steel for mid spans-

$$B.M = \frac{W l^2}{16}$$

$$= \frac{14.715 \times 4.25^2}{16}$$

$$\boxed{B.M = 16.61\ kNm}$$

resultant $BM = M - Tx$

$$= 16.61 - \left(22.07 \times \frac{25.794}{1000}\right)$$

$$\boxed{\text{Resultant } B.M = 14.53\ kNm}$$

$\Rightarrow$ Resultant BM process tension on "outer face"

$\sigma_{st} = 190\ N/mm^2$ (pg: 8, IS 3370 part II)

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

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

$$= \frac{280}{3 \times 7}$$

$$\boxed{m = 13.33}$$

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

$$= \frac{13.33 \times 7}{7 \times 13.33 + 190}$$

$$\boxed{k = 0.329}$$

$$j = 1 - k/3$$

$$\boxed{j = 0.89}$$

Scanned with CamScanner

steel for B.M $= \dfrac{M}{\sigma st \times j d}$

$\boxed{Ast_2 = 47.15 \ mm^2}$

$= \dfrac{14.53 \times 10^6}{190 \times 0.89 \times 219}$

$Ast = Ast_1 + Ast_2$

$\boxed{Ast = 539.5 \ mm^2}$

$\boxed{(Ast)\ reqd_1 = 392.35 \ mm^2}$

$(Ast)\ min = \dfrac{0.30}{100} \times 1000 \times 250$

$\boxed{(Ast)\ min = 750 \ mm^2}$ ✓ $> Ast\ (539.5\ mm^2)$

spacing $= \dfrac{\frac{\pi \times 12^2}{4}}{750} \times 1000$

$\boxed{spacing = 150 \ mm}$

→ Let us provide steel @ corner 12 mm ϕ @ 130 mm c/c

→ steel @ mid span 12 mm ϕ @ 130 mm c/c horizontally.

Step 3:- Design of short wall:-

i) steel for corner moment:-

BM @ 1m ht @ the corner

$= \dfrac{wl^2}{12} \Rightarrow \dfrac{14.715 \times 3.25^2}{12}$

$= 12.952$ kNm

Direct pull in the wall strip,

$= \dfrac{14.715 \times 4}{2} = 29.43$ kN

Assume to use 12 mm ϕ bars with clear cover of 25mm.

effective cover $= 25 + \dfrac{12}{2} = 31$ mm.

Resultant BM $= M - Tx$

$= 12.952 - (29.43 \times 0.094)$

$= 10.185$ kNm

BM produce tension on inner side of water face.

$$\sigma_{st} = 150\ N/mm^2$$

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

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

$$= \frac{280}{3 \times 150}$$

$$\boxed{m = 13.33}$$

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

$$\boxed{k = 0.383}$$

$$j = 1 - k/3$$

$$= 1 - 0.383/3$$

$$\boxed{j = 0.872}$$

steel for BM,

$$(A_{st})_1 = \frac{M}{\sigma_{st} \cdot j \cdot d}$$

$$= \frac{10.185 \times 10^6}{150 \times 0.872 \times (250 - 31)}$$

$$\boxed{(A_{st})_1 = 355.55\ mm^2}$$

Direct Tension

$$(A_{st})_2 = \frac{T}{\sigma_{st}} = \frac{29.43 \times 10^3}{150}$$

$$\boxed{(A_{st})_2 = 196.2\ mm^2}$$

Area of steel $= (A_{st})_1 + (A_{st})_2$

$= 551.75\ mm^2$

using 12 mm ϕ bars

... $= 204.979$ mm

ii) Steel for Mid Span :-

$$BM = \frac{wl^2}{16} = \frac{14.715 \times 3.25^2}{16}$$

$$\boxed{BM = 9.714 \ KNm}$$

BM produce the tension in outer face.

$\sigma_{st} = 190 \ N/mm^2$

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

$m = 13.33, \ k = 0.329, \ j = 0.890$

steel for BM,

$$(Ast)_1 = \frac{M}{\sigma_{st} \cdot j \cdot d}$$

Resultant $Mt = M - Tx$

$$= 9.714 - (29.43 \times 0.094)$$

$$\boxed{Mt_R = 6.947 \ KNm}$$

$$(Ast)_1 = \frac{6.947 \times 10^6}{190 \times 0.89 \times (250-31)}$$

$$\boxed{(Ast)_1 = 187.589 \ mm^2}$$

Direct Tension,

$$(Ast)_2 = \frac{T}{\sigma_{st}} = \frac{29.43 \times 10^3}{190}$$

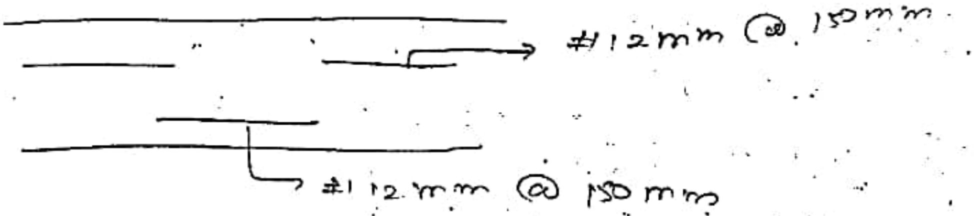
$$\boxed{(Ast)_2 = 154.89 \ mm}$$

Area of steel = $(Ast)_1 + (Ast)_2$ = $\underline{\underline{342.47 \ mm^2}}$

$(Ast)_{min} = \frac{0.3}{100} \times 1000 \times 250 = 750 \ mm^2 > 342.47$

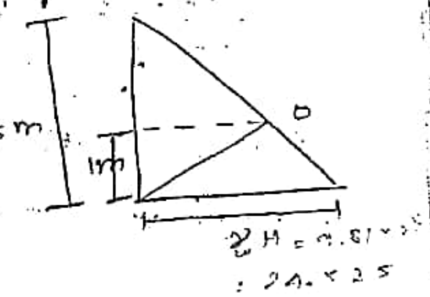
$\Rightarrow$ provide 12 mm ϕ bars @ 150 mm c/c

$$spacing = \frac{\frac{\pi \times 12^2}{4}}{750} \times 1000 = \underline{\underline{150.796}} \ mm$$



Step 4: Design of bottom 1m ht of wall:-

* max press. intensity @ the bottom of the wall,



$= 9.81 \times 2.5 = \underline{24.525}\ kN/m^2$

* max cantilever mt,

$= \frac{1}{2} \times 1 \times 24.52 \times \frac{1}{3}$

assume to use 10mm φ vertical bars

effective depth $= 250 - 25 - \frac{10}{2} - 12$

$= 208\,mm$

$\sigma_{st} = 150\ N/mm^2$

$j = 0.890$

steel for BM $= \frac{M}{\sigma_{st}\, j d} = \frac{4.087 \times 10^6}{150 \times 0.890 \times 208}$

$= 147.183\ mm^2$

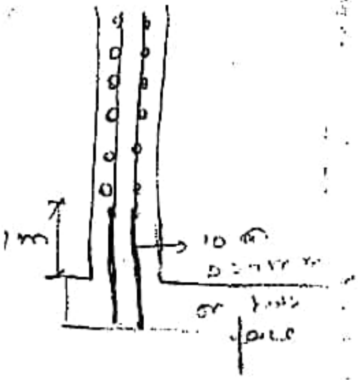
min area of steel $= 0.3\%$ of g.A

$= \frac{0.3}{100} \times 1000 \times 250$

$\boxed{(A_{st})\ min = 750\ mm^2}$

use 10mm φ bars

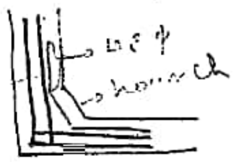
spacing $= \frac{\frac{\pi \times 10^2}{4}}{430} \times 1000$



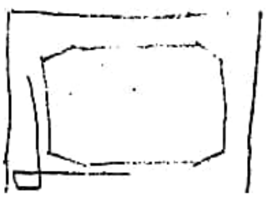
$= 104.71\ mm$

⇒ provide 10mm φ bars @ 200 mm c/c on both the face.

con



providing
① Ties ↑
② development length



providing haunch:-
① depth increases

-ve BM is more some of positive

Step 5:-

Design of base slab:-

It is made be 250 mm thick with a top & bottom mesh of reinforcement.

→ provide 10 mm ϕ @ 200 mm c/c.

H.W Design a open water tank 6×6 m × 3 m depth rest on firm ground Use M20 & Fe415 steel.

2). circular water Tank (approximate method)

Solution:-

capacity = $\frac{500,000}{1000}$ = 500 m^3

assume free board is not provided. 250 mm

$500 = \frac{\pi \times d^2}{4} \times 3.75$

$\boxed{d = 14.92 \text{ m}}$

$\boxed{\text{provide} \; d = 15 \text{ m}}$

hoop tension $= wh \cdot \frac{D}{2}$

$= 9.81 \times 3.75 \times \frac{4.5}{2}$

(FU) $\boxed{T = 82.77 \text{ kN}}$

$(A_{st})_{reqd} = \frac{T}{\sigma_{st}}$

pa: 8 , part-II 18 3370

$\sigma_{st} = 150 \text{ N/m}^2$

$(A_{st})_{reqd} = \frac{82.77 \times 10^3}{150}$

$\boxed{(A_{st})_{reqd} = 551.8 \text{ mm}^2}$

use 12 mm ϕ

no. of bars $= \frac{551.8}{\frac{\pi \times 12^2}{4}}$

$\boxed{\text{no. of bars} = 4.878}$

F: spacing $= \frac{551.8}{4.878}$ $= 113.09$ mm

→ provide 12 mm ϕ 110 mm c/c.

(A_{st}) provide $= \frac{\frac{\pi \times 12^2}{4}}{110} \times 1000$

mm^2

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Step 2:- Thickness of wall:-

Thickness of the wall shall not less than the following,

i). 150 mm

ii). 30 mm per m depth + 50 mm

$(30 \times 4) + 50 = 170\,mm$

iii). Tensile stress requirement,

$= 1.2\ N/mm^2$ (for M_{20} concrete) (pg:7, part-II - IS3370)

$$\text{permissible tensile stress} = \frac{F_t}{A_c + m A_{st}} \quad \text{(IS 456 pg: 80)}$$

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

for M_{20} concrete,

$\sigma_{cbc} = 7\ N/mm^2$ (from table 21, pg 81)

$$m = \frac{280}{3 \times 7}$$

$$\boxed{m = 13.33}$$

$$A_c = b \times t = 1000 \times t$$

$$1.2 = \frac{82.77 \times 10^3}{(1000 \times t) + (13.33-1)(1028.16)}$$

$$15212.65 + 1200\,t = 82.77 \times 10^3$$

$$\boxed{t = 56.29\,mm}$$

Let us to provide over all thickness of 170 mm

Step 3:-

Vertical Reinforcement (distribution steel):-

This shall be atleast 0.3% for the cross area.

$$(A_{st})_{min} = \frac{0.3}{1000} \times 1000 \times 170$$

$$\boxed{(A_{st})_{min} = 510\ mm^2}$$

for each face = $\frac{510}{2}$ = 255

use 8 mm dia bars.

$$\text{spacing} = \frac{\frac{\pi \times 8^2}{4}}{255} \times 1000$$

$$\boxed{\text{spacing} = 197.11\ mm}$$

⇒ provide 8 mm φ bars @ 190 mm c/c on both the faces.

Step 4:-

Design of base slab:-

The base slab will be laid on a 75 mm lean mix bed covered with tar felt, since the load gets transfer to ground directly. A nominal Thk of 170 mm nearly provided with mix. R.F in both the direction.

$$(R.F)\ reqd = \frac{0.3}{100} \times 1000 \times 170$$

$$= 510\ mm^2$$

for each face = 255 mm²

spacing = 197.11 mm

⇒ provide 8 mm φ bars @ 190 mm c/c

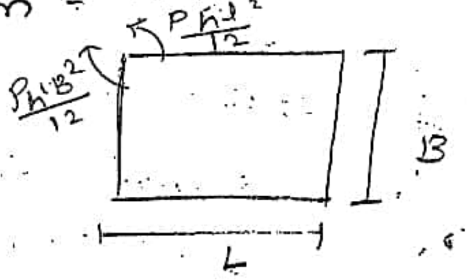
6/9/12 Design a ▭** water tank of size 5m×4m × 3m deep resting on the firm ground. use M_{25} concrete & Fe415 steel.

<u>solution:-</u> (exact method)

$$\frac{L}{B} = \frac{5}{4} = 1.25 < 2$$

Both the wall should be designed as spans in horizontally. H/4 or 1m = 0.75 or 1 from the bottom

here h = 1m



$$P_h = \gamma_w (H-h)$$
$$= 9.81 (3-1)$$
$$= 19.62 \ kN/m^2$$

∴ fixed end moments are

$$= \frac{P_h L^2}{12} = \frac{19.62 \times 5^2}{12} = 40.875 \ kNm \text{ in L.W.}$$

$$\frac{P_h B^2}{12} = \frac{19.62 \times 4^2}{12} = 26.16 \ kNm \text{ in S.W.}$$

Since The thickness of S.W & L.W are maintained same. The distribution factors @ joints are calculated as below.

member	stiffness	Total joint stiffness	$\frac{K}{\Sigma K}$ distribution factor
LW	$\frac{4EI}{l} = \frac{4EI}{5} = 0.8EI$	$EI + 0.8EI = 1.8EI$	$\frac{0.8EI}{1.8EI} = 0.444$
SW	$\frac{4EI}{l} = \frac{4EI}{4} = EI$		$\frac{EI}{1.8EI} = 0.556$

Due to symmetry one balancing will take care of moment distribution as shown below.

SW	0.556	0.444	L.W
	-26.16	+40.87	→ always S.W in (-) sign. L.W in (+) sign (only for 1st step).
	+14.7		
	-14.7	-6.536	
	-8.178 (-14.7 × 0.556)	(-14.7 × 0.444)	
	-34.338	+34.338	
	moment = -34.338		tension on inner

corner face.

Effective Thk reqd $= \sqrt{\dfrac{M}{Qb}}$

working stress ← Q = mt of resistance factor
↓ [Ru max in limit state)

Design constants:-

$\sigma_{cbc} = 8.5\ N/mm^2$

$m = \dfrac{280}{3\sigma_{cbc}} = 10.980$

$\sigma_{st} = 150\ N/mm^2$

$k = \dfrac{m\,\sigma_{cbc}}{m\,\sigma_{cbc} + \sigma_{st}}$

$\boxed{k = 0.383}$

$j = 1 - k/3$

$\boxed{j = 0.872}$

$Q = \frac{1}{2}\,\sigma_{cbc} \cdot k\, j$

$= \frac{1}{2} \times 8.5 \times 0.384 \times 0.872$

$\boxed{Q = 1.419}$

$(d\ eff)\ req = \sqrt{\dfrac{34.338 \times 10^6}{1.419 \times 1000}} = \underline{155.55\ mm}$

The section will be balanced section.
* Section is to be kept sufficiently under reinforced. Hence Let us provide a over all Thks of "230 mm"

Effective thcs $= 230 - \frac{12}{2} - 25 = \underline{199}$ mm

Assume $\phi = 12$ mm

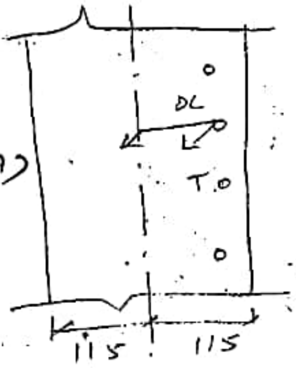
clear cover $= 25$ mm

Design of long wall:- ((corner side)

Direct pull on the long wall $= [T_L] = \frac{P_h B}{2}$

$= 39.24$ KN (for 1 m ht)

Design mt @ the corner $= M - T x$



181 $= 115 - 25 - 12/2$

$= \underline{84}$ mm

$M = 34.338 - (39.24 \times 0.084)$

$= \underline{31.041}$ KNm.

horizontal reinforcement req to resist the B.M,

$Ast_1 = \frac{M}{\sigma st \cdot j d} = \frac{31.041 \times 10^6}{150 \times 0.872 \times 199}$

$\boxed{Ast_1 = 1195.319 \text{ mm}^2}$

Steel for direct tension,

$(Ast)_2 = \frac{T_L}{\sigma st} = \frac{39.24 \times 10^3}{150}$

$\boxed{(Ast)_2 = 261.6 \text{ mm}^2}$

Total (Ast) $= Ast_1 + Ast_2$

$= \underline{1456.919}$ mm^2

Use 12 mm ϕ bars,

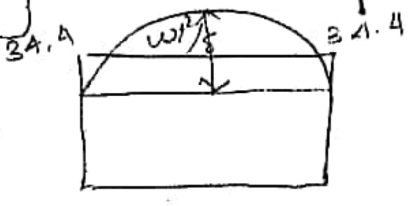
spacing $= 77.62$ mm

Use 16 mm ϕ bars $= 138.004$ mm.

So provide 16 mm ϕ bars @ 130 mm c/c on inner face..

iii. Mid span of long wall:-

$$\boxed{BM = \frac{P_h l^2}{8} - mt \text{ @ corner}}$$



$$= \frac{19.62 \times 5^2}{8} - 34.338$$

$$\boxed{BM = 26.97 \text{ KNm}}$$

Design moment $= M - Tx$

$$= 26.97 - (39.24 \times 0.084)$$

$$\boxed{mt = 23.67 \text{ KNm}}$$

Steel for BM $(Ast)_1 = \frac{M}{\sigma st \cdot jd}$

$$= \frac{23.67 \times 10^6}{150 \times 0.872 \times 199}$$

$$\boxed{(Ast)_1 = 911.45 \text{ mm}^2}$$

Steel for Tension.

$$(Ast)_2 = \frac{T}{\sigma st} = 261.6 \text{ mm}^2$$

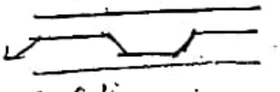
$$\boxed{\text{Total } (Ast) = 1173.05 \text{ mm}^2}$$

use 16 mm φ bars,

$$\boxed{\text{Spacing} = 171.40 \text{ mm}}$$

The middle bars are to be provided @ outer face.

Here also bars may be provided @ 130mm c/c. so that bars may be bond & used.



Design for short wall:-

i) corner side:-

Direct pull on the short wall $= \frac{P_h L}{2}$

$$= \frac{19.62 \times 5}{2}$$

$$\boxed{T = 49.05 \text{ KN}}$$

Design mt @ the corner $= M - Tx$

$$= 34.338 - 49.05(0.084)$$

$$\boxed{mt = 30.218 \text{ KNm}}$$

$$Ast_1 = \frac{M}{\sigma st \cdot jd} = \frac{30.2178 \times 10^6}{150 \times 0.87 \times 199}$$

$$\boxed{(Ast)_1 = 1163.58\ mm^2}$$

Steel for tension,

$$Ast_2 = \frac{T}{\sigma st} = \frac{49.05 \times 10^3}{150} = \underline{327}\ mm^2$$

$$\boxed{\text{Total } Ast = 1490.58\ mm^2}$$

Use 16mm ϕ bars,

$$\boxed{\text{spacing} = 134.88\ mm}$$

So provide 16 mm ϕ bars @ 130 mm c/c.

ii). <u>Mid span:-</u>

$$BM = \frac{P_h B^2}{8} - mt \text{ @ corner side}$$

$$\boxed{BM = 4.902\ KNm}$$

Design $mt = M - Tx$

$$= 4.902 - (89.24 \times 0.082)$$

$$\boxed{mt = 1.305\ KNm}$$

Steel for BM,

$$(Ast)_1 = \frac{M}{\sigma st \cdot jd} = \frac{1.305 \times 10^6}{150 \times 0.872 \times 199}$$

$$\boxed{(Ast_1) = 50.28\ mm^2}$$

Steel for Direct tension,

$$(Ast)_2 = \frac{T}{\sigma st} = \underline{327}\ mm^2$$

$$\boxed{\text{Total } (Ast) = 377.28\ mm^2}$$

$(Ast)_{min} = 690\ mm^2$ ✓

use 16 mmϕ bars, $\boxed{\text{spacing } \underline{291.39}\ mm}$

⇒ So provide 16mm ϕ bars @ $\underline{160}$ mm c/c

use 12 mmϕ bars, $\boxed{\text{spacing} - \underline{163.59}\ mm}$

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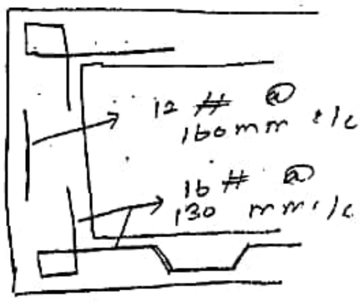
Reinforcement @ vertical

Reinforcement:-



bottom 1m ht

$= \gamma_w H$

$= 9.81 \times 3 = \underline{\underline{29.43}}$ KN/m²

max. cantilever mt $= (\frac{1}{2}\ bh) \times \frac{1}{3}$

$= \frac{1}{2} \times 29.43 \times 1 \times \frac{1}{3}$

$\boxed{mt = 4.905 \text{ kN m}}$

Assume to use 10mm ϕ vertical bars,

$d_{eff} = 230 - 25 - 16 - 10/2$

$\boxed{d_{eff} = 184 \text{ mm}}$

$\sigma_{st} = 150 \text{ N/mm}^2$

steel for BM, $= \dfrac{M}{\sigma_{st}\, j d} = \dfrac{4.905 \times 10^6}{150 \times 0.872 \times 184}$

$\boxed{(A_{st})_1 = 204.27 \text{ mm}^2}$

steel for Tension $= \dfrac{T}{\sigma_{st}}$

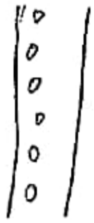
$= \dfrac{49.05 \times 10^3}{150}$

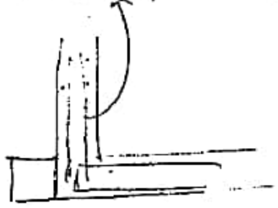
$(A_{st})_2 = 327 \text{ mm}^2$

$\boxed{\text{Total } (A_{st}) = 531.27 \text{ mm}^2}$

$(A_{st})_{min} = 690 \text{ mm}^2$

so provide 12mm ϕ bars @ 160 mm c/c.





1 7/9/12

Design of ~~underground water tank~~

Tank full water pres: in Inside, no earth pres.
condition } In outside the wall will defect outwards.

1) Design an underground W.T of size 3 x 8 x 3 m for the following data. The type of soil - submerged sandy soil with $\gamma' = 16\, kN/m^2$, $\phi = 30^\circ$, W.T can rise upto G.L. Grade of concrete i) for tank M25 (ii) roof slab M20. Grade of steel of Fe 415. Take L.L of roof slab $2 kN/m^2$.

Design of roof slab :-

$L/B = \frac{8}{3} = 2.66 > 2$

one way slab.
slab is designed as one way slab.

$d = \frac{le}{25} = \frac{3000}{25} = 120\, mm$

effective depth required = 120 mm

over all depth = 120 + 25 + 10/2

$\boxed{D\ req = 150\ mm}$

Hence provide over all depth D = 160 mm

effective depth d = 160 − 25 − 10/2

$\boxed{d = 130\, mm}$

clear cover = 25 mm
dia of bars = 10 mm.

load calculation :-

$L.L = 2 kN/m^2$

self weight of slab

$= 25 \times 1 \times 1 \times 0.16$

$S.W = 4\ kN/m^2$

Finishing load = $0.5\ kN/m^2$ (assume)

$\boxed{\text{Total load} = 6.5\ kN/m^2}$

factored load $= 1.5 \times 6.5 \, kN/m^2$

$FL = 9.75 \, kN/m^2$

Factored moment $= \frac{FL \times le^2}{8}$

$= \frac{9.75 \times 3^2}{8}$

$\boxed{Mu = 10.96 \, kNm}$

$Mu \, lim = 0.138 \, f_{ck} \, bd^2$

$= 0.138 \times 20 \times 1000 \times 130^2$

$= 46.64 \, kNm < Mu$

under Reinforced section.

$\boxed{Mu, lim > Mu}$

$Ast = 0.5 \frac{f_{ck}}{f_y} \left(1 - \sqrt{1 - \frac{4.6 \, Mu}{f_{ck} bd^2}} \right) bd$

$= 0.5 \frac{\times 20}{415} \left(1 - \sqrt{1 - \frac{4.6 \times 10.96 \times 10^6}{20 \times 1000 \times 130^2}} \right) 1000 \times 130$

$\boxed{Ast_{req} = 243.05 \, mm^2}$

spacing $= \frac{\frac{\pi \times 10^2}{4}}{243.05} \times 1000$

$= 323.13 \, mm$

uses 10 mm dia bars.

Let us provide 10 mm φ bars @ 300 mm c/c.

distribution bars:-

$$(A_t)_{min} = \frac{0.12}{100} \times 1000 \times 160$$

$$\boxed{A_t\ min = \cancel{192}\ mm^2}$$

$$\text{spacing} = \frac{\frac{\pi \times 8^2}{4}}{\cancel{480}\ 192} \times 1000$$

$$= 261.79\ mm$$

Let us provide 8 mm ϕ bars @ 260 mm c/c.

Step 2: Design of walls:-

$$L/B = 8/3 = 2.67 > 2$$

The long wall will be designed as fully cantilever and short wall will be designed as slab supported by L.W bottom H/4 or 1m is designed as cantilever of short wall.

For M25, $\sigma_{cbc} = 8.5\ N/mm^2$ IS 456 - pg - 81

for Fe415, $\sigma_{st} = 150\ N/mm^2$ (For value of permissible stress is consider to avoid leakage problem).

$$m = \frac{280}{3\sigma_{cbc}} = 10.98,\ k = 0.384,\ j = 0.872,\ Q = 1.419$$

Step 3:- Design of long wall:-

case (i):- when the tank is empty condition:-

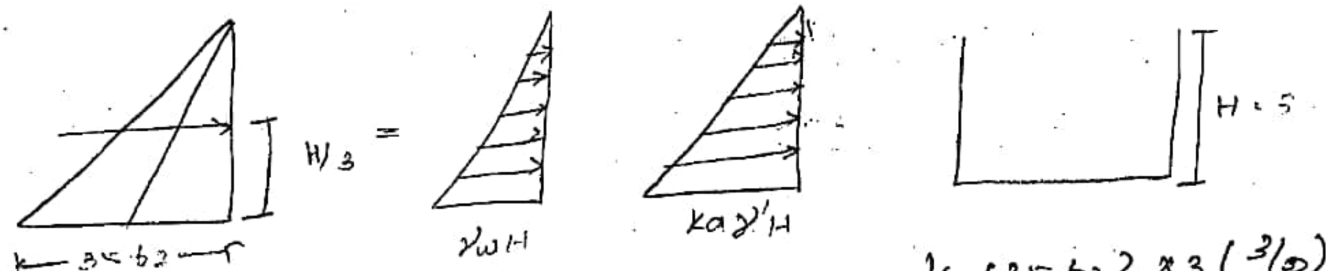
Pressure of saturated soil is acting from outside & no water pressure on Inside.

Total pressure $= k_a \gamma' H + \gamma_w H.$

$$= \tfrac{1}{3} \times (16 - 9.81) \times 3 + (9.81 \times 3)$$

$$\gamma' = 16 - 9.81 = 6.91$$

$$= 35.65\ kN/m^2.$$



Max B.M @ base of L.W = $\frac{1}{2}(35.62) \times 3 (3/3)$

$= \underline{53.43}$ kN m

from packing consideration, the Tcs of wall is determined as follows

pg: 7 IS 3370 part - II - 1965

$f = 1.8$ (Tension due to BT)

$M = fZ$

$53.43 \times 10^6 = 1.8 \times \frac{1000 \times D^2}{6}$

$\boxed{D = 422.01 \text{ mm}}$

provide over all depth D = 430 mm. Assume clear cover of 25 mm

to use 12 mm ϕ bars with

$d = 430 - 25 - 12/2$

$\boxed{d = 399 \text{ mm}}$

$A_{st} = \frac{M}{\sigma_{st} \times j \times d} = \underline{1023.77} \text{ mm}^2$

$(A_{st})_{min} = \frac{0.3}{100} \times 1000 \times 400$

$\boxed{(A_{st})_{min} = 1200 \text{ mm}^2}$

$d_{reqd} = \sqrt{\frac{M_u}{Qb}} = \sqrt{\frac{53.43 \times 10^6}{1.419 \times 1000}}$

$\boxed{d_{reqd} = 194 \text{ mm}}$

provide over all depth $\boxed{D = 250 \text{ mm}}$

use 16 mm ϕ bars with c/c 25 mm.

$d = 250 - 25 - 16/2 = \underline{217} \text{ mm}$

$$A_{st} = \frac{M}{\sigma_{st} \times j \times d} = \frac{53.43 \times 10^6}{150 \times 0.872 \times 217}$$

$$\boxed{A_{st} = 1882.42\ mm^2}$$

$$\text{spacing} = \frac{\frac{\pi (16)^2}{4}}{1882.42} \times 1000$$

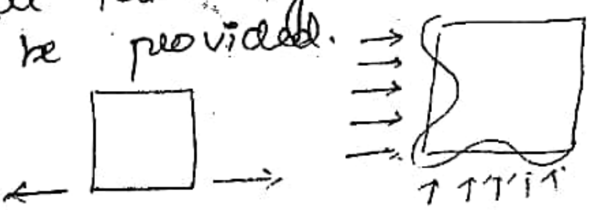
$$\boxed{\text{spacing} = 106.8\ mm}$$

provide 16 mm ϕ bars @ 100 mm c/c vertical near outer face.

taken by distribution steel

18/9/12 * Direct compression:-

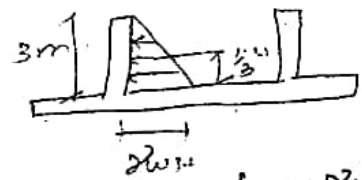
caused in long wall because of earth press. acting on short wall which is generally of little affect which will take by wall 2 distribution steel will be provided.



case 2:-

tank full condition:-

water press. is acting on inside and no earth press. from out side.



mom. cantilever BM due to water press. $= \gamma_w H^3/6$

$= 9.81 \times \frac{3^3}{6}$

$$p_h = \frac{1}{2} \times \gamma_w \times H \cdot H$$

$$BM = \frac{1}{2} \gamma_w \cdot H^2 \cdot H/3 = \frac{1}{6} \gamma_w H^3$$

$$\boxed{BM = 44.145\ kNm} < 53.43\ kNm$$

$\therefore$ Tks provided is OK.

$$(A_{st})_{reqd} = \frac{M}{\sigma_{st} \cdot j \cdot d} = \frac{44.145 \times 10^6}{150 \times 0.872 \times 217}$$

$$\boxed{(A_{st})_{reqd} = 1555.29\ mm^2}$$

use 16 mm ϕ

$$\text{spacing} = \frac{\frac{\pi \times 16^2}{4}}{1555.29} \times 1000 = \underline{129\ mm}$$

→ providing 16 mm ϕ @ 125 mm c/c vertically inner face.

curtailement of Reinforcement:-

$(A_{st})_h$ Reinforcement reqd @ h_1 depth from top. The BM @ any depth is proportional to h_1^3.

$$\therefore \frac{(A_{st})_h}{A_{st}} = \frac{h_1^3}{H^3}$$

$$1/2 = \frac{h_1^3}{3^3}$$

$$\boxed{h_1 = 2.380\ m}$$ from top. (3 - 2.38)

as per Theoritical cut of pt 0.62 m from base

12 times ϕ of the bars or effective depth which ever is less.

d or 12×16

219 192

actual cut of pt = 0.62 m from base + 0.219 m

say 0.62 ≃ 0.65

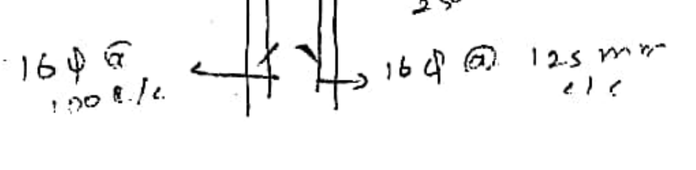
= 0.65 + 0.219

= 0.869 m ≃ 1 m

The alternate bars can be curtail @ 1 m from base it is applicable for both the face.

Main Reinforcement:-

provide 16 mm ϕ @ 200 c/c on (out side) outer face and 16 mm ϕ @ 250 mm c/c @ inner face above 1 m @ from the base.



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Distribution steel - (horizontally)

(part - II pg: 13)

Ths	(A_{st}) min
100 →	0.3
250 →	0.2571
450 →	0.2

$$0.2571 = \frac{0.3-0.2}{350}(450-250)+0.2$$

$$(A_{st})\ min = \frac{0.2571}{100} \times 1000 \times 250$$

$$\boxed{(A_{st})\ min = 642.857\ mm}$$

* (8 mm φ bars used.

$$spacing = \frac{\frac{\pi \times 8^2}{4}}{642.857} \times 1000$$

$$\boxed{Spacing = 78.23\ mm}$$

use 10 mm φ bars

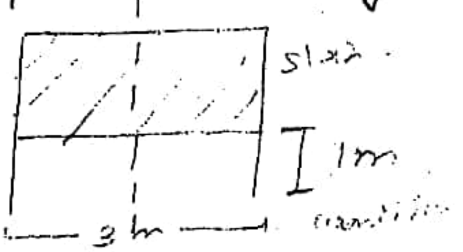
$$spacing = \frac{\frac{\pi \times 10^2}{4}}{642.857} \times 1000$$

$$\boxed{Spacing = 122.2\ mm}$$

⇒ provide 10 mm φ bars "240 mm" c/c on each face.

Direct tension in long wall:-

Direct tension is caused in long wall because of water pressure acting on short wall which act as slab supported on long wall



⇒ direct tension @ 1m above base

$$= \frac{9.81 \times (3-1) \times 3}{2}$$

$$\boxed{T = 29.43\ KN}$$

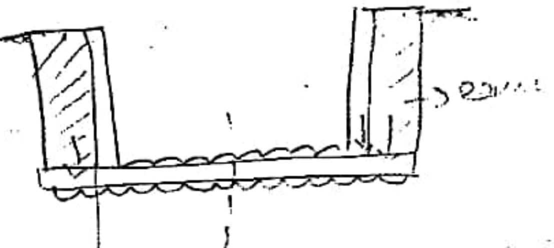
$$(A_{st})\ reqd\ @\ 1m\ ht. = \frac{T}{\sigma_{st}} = \frac{29.43 \times 10^3}{150}$$

$\boxed{(A_{st})\ reqd = 196.2\ mm^2}$ < (A_{st}) min

Distribution steel will take care the direct tension. Therefore "no additional steel is reqd".

Design of short wall:-

few: empty condition and design base slab.



2019/12 Domes ⇒ (wind load & seismic load cal. is difficult)

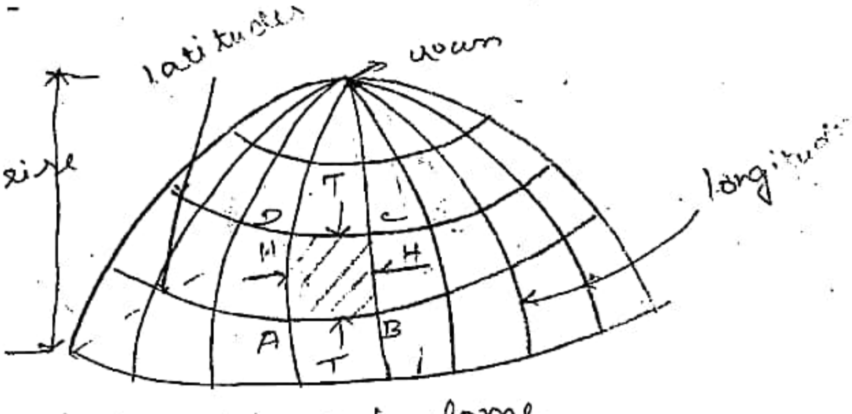
disad:-

* form work will be costly

Stress in domes:-

A frustum of a spherical dome is size shown in fig.

The dome may be Thought of the circular ring ~~of~~ continuously reducing the dia. placed one above the other.



* The direct compression acting along the meridien is called "meridien Thrust" denoted (T).

* The element ABCD of a figure as a tendency to fall inwards which is prevented by the wedge side element. and This line gives the hoops compression. (H) in each ring which will hold. each ring in stable form.

DL ✓
LL ✓
W.L / se → equivalent udl – 1.5 to 2 kN/m²

Scanned with CamScanner

- min Ths of dome 75mm".
- practical Ths "75-150 mm".
- Reinforcement is designed for "Tension"
- min R.F along latitude & longitude shall be "0.3%".
- square mesh is provided near crown to avoide congestion. of
* Rise to span ratio is "1/6 to 1/4" (or) 1/9 R
- at the free edge of the dome ring beam is provided to supported "loads from dome".
- ring beam is supported on either column, or wall
- ring beam is supported as column shall be designed as vertical loads torsion
- The R.F of dome are designed as "pure tension". therefor lap length "2Ld (or) 30φ" (pg: 45)

Design eqnt (or) Design formulae for spherical dome.

* Dome - designed for meridinal stress and hoop stress.

* max meridinal Thrust per m length ...

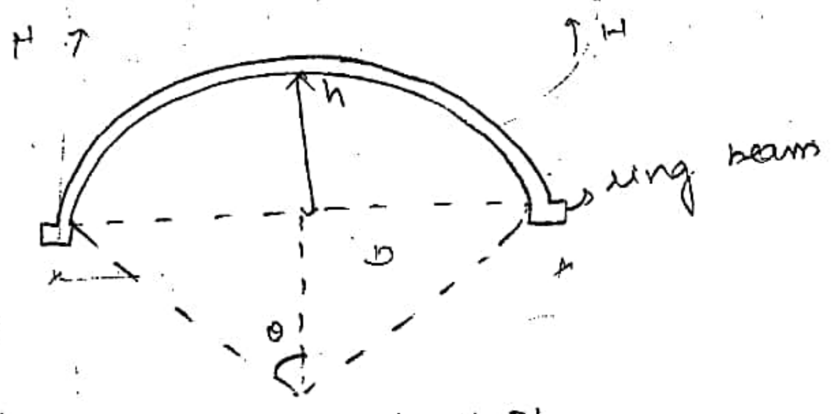
$$(T) = \frac{wR}{(1+\cos\theta)}$$

* circumferential force,

$$H = wR\left(\cos\theta - \frac{1}{1+\cos\theta}\right)$$

R ⇒ Radius of the dome.

* spherical dome of radius are the small unit area of surface.



dome subjected to UDL

* @ crown θ = 0,
* @ crown $\boxed{\theta = 0,\ T = 0,\ H = wR/2}$
* hoop's stress goes on decrease with θ upto some value. to get a circle of 0, hoop's stress $\boxed{H = 0}$
  To get a
  $\boxed{\theta = 51.8^\circ}$
* for θ = 51.8° 'H' will be -ve. Therefore tensile stress are developed called "hoop tension"
* The ring beam is designed for tension

$= T_1 \cos\theta \times D/2$

D ⇒ dia of dome @ base

* The cross sectional area of the ring beam is dete. by limiting the tensile stress in the ring beam.

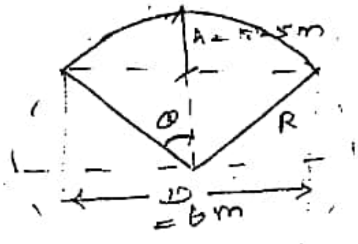
$T.s = \frac{T}{A_c + m A_{st}}$ (pg: 3)

① A Reinforced concrete dome of 6m base dia with rise of 1.25m is to be designed for a water tank. The U.D live load including on finish on dome may be taken as 2kN/m². adopting M20 & Fe415 steel.

Solution:-

$D = 6m$

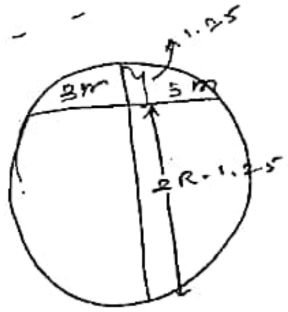
$h = 1.25\,m$



Step 1:-

Dimensions of dome:-

R ⇒ Radius of dome.



$\sin\theta = \dfrac{3}{4.225} = 45^\circ 14' 23$

$\cos\theta = \dfrac{4.225 - 1.25}{4.225}$

$\boxed{\cos\theta = 0.7041}$

$3 \times 3 = 1.25\,(2R - 1.25)$

$\boxed{R = 4.225}$

assume the Thk of the dome $= \underline{100\,mm}$

Step 2:-

Load Calculation:-

Self wt of dome $= 0.1 \times 25$

$= 2.5\ kN/m^2$

L.L & finish $= 2\ kN/m^2$

$\boxed{T.L = 4.5\ kN/m^2}$

Step 3:-

Stress Calculation:-

meridinal Thrust $T_1 = \dfrac{wR}{1+\cos\theta}$

$= \dfrac{4.5 \times 4.225}{1 + 0.7041}$

$\boxed{T_1 = 11.1569\ kN/m}$ (or) $11.1569\ N/mm$

Meridinal comp. stress $= \dfrac{11.157}{100 \to thk}\ N/mm^2$

$= 0.1116\ N/mm^2$

$H = wR\left(\cos\theta - \dfrac{1}{1+\cos\theta}\right)$ / unit Length

$= 4.5 \times 4.225\left(0.7041 - \dfrac{1}{1+0.7041}\right)$

$\boxed{H = 2.2298\ kN/m}$

C, hoop stress = $\frac{2.229 \cdot 8}{100}$

circumferencial hoopstress = $\underline{0.0222}3$ N/mm

<u>Step 4:- Ast calculation:-</u>

The stress are very small.

∴ Therefore provide nominal Reinforcement of 0.3% gross area

$$(Ast) = \frac{0.3}{100} \times 1000 \times 100$$

$$\boxed{Ast = 300\ mm^2}$$

use 8mm φ dia

$$\text{spacing} = \frac{\frac{\pi \times 8^2}{4}}{300} \times 1000$$

$$\boxed{\text{spacing} = 167.552\ mm}$$

provide 8 mm φ of 160 mm c/c both <u>meridinally</u> and <u>circumferencially</u>.

<u>step 5:-</u>

<u>Design of ring beam:-</u>

Meridinal thrust per metre length of beam at the base) (T_1) = 11.57 KN/m

(horizontal component = $T \cos\theta$ = 11.57 × 0.7071
= 7.8556)

Total hoop tension acting per metre length of the ring beam = $T_1 \cos\theta \cdot D/2$
= 7.8556 × 6/2

$$F_t = \underline{23.56}7\ kN$$

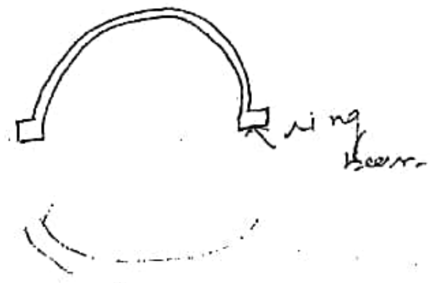
$$\text{Area of steel} = \frac{\text{Hoop tension}}{\text{stress }(\sigma_{st})}$$

$$= \frac{23.567 \times 10^3}{150}$$ → As per IS 3370

$$\boxed{(A_{st}) = 157.113 \ mm^2}$$

for M_{20} concrete, modular ratio $m = \frac{280}{3\sigma_{cbc}}$

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



$$m = \frac{280}{3 \times 7}$$

$$\boxed{m = 13.33}$$

use 10 mm ϕ bars

no. of bars $= \frac{157.113}{\frac{\pi \times 10^2}{4}} = 2.0004$

→ provide 4 nos of 10 mm ϕ bars.

$$(A_{st})_{pro} = \frac{4 \times \pi \times 10^2}{4}$$

$$\boxed{(A_{st})_{pro} = 314.15 \ mm^2}$$

Tensile stress in the concrete should not exceed $\not>$ 2.8 N/mm² $\boxed{\text{(from pg: 80 IS 456)}}$

$A_g = A_c + A_{st}$

$A_c = A_g - A_{st}$

$$\text{Tensile stress} = \frac{F_t}{A_c + m A_{st}}$$

$$= \frac{F_t}{A_g + (m-1) A_{st}}$$

$$2.8 = \frac{23.567 \times 10^3}{bD + (13.33 - 1)\, 314.15}$$

assume width $b = 150$ mm

$150 \times 2.8 \times D = 15721.28$

$420 \ D = 4576.54$

$D = 11.6985$

∴ provide ring beam of 150 mm × 150 mm.

provide min shear reinforcement using HYSD 2legged stirrups of 8mm ϕ / or 6mm ϕ mild steel bars also.

$$\frac{A_{sv}}{b S_v} \geq \frac{0.4}{0.87 f_y}$$

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

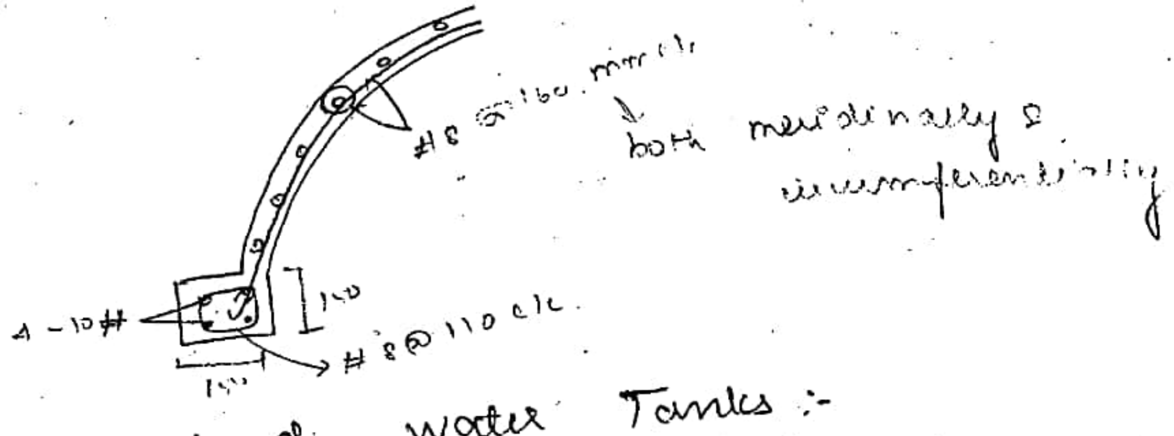
$$= \frac{\frac{\pi \times 8^2 \times 2}{4} \times 0.87 \times 230}{0.4 \times 150}$$

$$\boxed{S_v = 335.27 \text{ mm}}$$

min spacing $0.75\, d = 0.75 \times 150$

$$\boxed{\text{spacing} = 112.5 \text{ mm}} < 335.27$$

so provide 8mm ϕ 2 legged 110 mm c/c



Over head water Tanks :-

Bracing providing in the over head W.T

Reasons :-

(The span of the column is to long column effect)

They subject to long column effect by

1. To avoide the long column effect by reducing the effective length of column.

2. To take care the lateral force like wind & seismic force

(long column will be designed axial load & B.M).

23/9/12

UNIT- IV

Building Frames.

2 cycle moment distribution method / substitution method - earlier perf'd to analyse the no. of storey of the Building frames. (In this method the above & below floor effect it always les.)

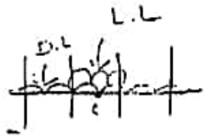
Substitute frame

1/10/12

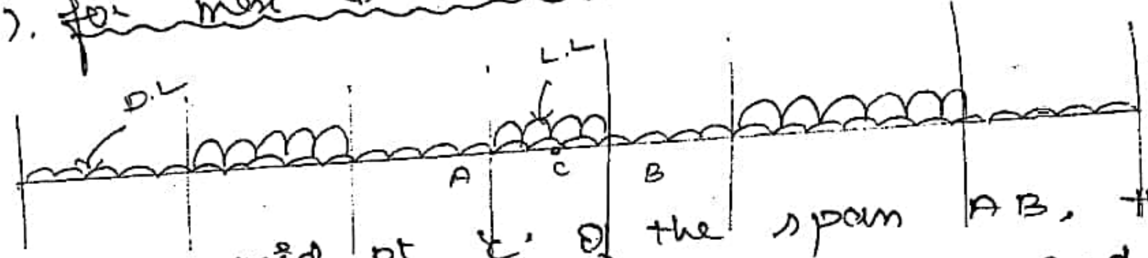
* whatever be the no. of storey it is customery (and permissible) to analyse only a part of a frame, termed as "substitute frame".

* It is based on the assumption # the mts in one floor hv negligible mt of the floor above and below it.

Max BM in Beam:-



i). for max +ve BM @ midspan of C:-



@ the mid pt 'C' of the span AB, the loads should be placed on the span and on alternate spans as shown in fig.

ii). for max -ve BM @ midspan of C:-

@ the mid pt 'c' of the span AB the span AB should be unloaded while load should be placed on span adjacent to the span and the consideration & on the alternate spans.

13
CHAPTER

Water Tanks

13.1 INTRODUCTION

To meet the daily requirement of water by industries, campuses, localities, towns and cities various types of R.C. water tanks are used. Such tanks may be in general, classified as:

(i) Tanks resting on ground,
(ii) Under ground tanks, and
(iii) Elevated tanks.

The tanks may have circular or rectangular sections. Tanks resting on ground and underground tanks have flat bottom slab while elevated water tanks may have flat bottom or conical bottom.

Apart from strength requirement, another essential requirement in the design of water tank is imperviousness. To make water tanks impervious, wider cracks should be avoided in the concrete, which may be achieved by

(i) Use richer concrete mix, say M25 or M30.
(ii) Give a minimum clear cover of 25 mm.
(iii) Provide smaller diameter bars at closer intervals.
(iv) Keep the tensile stresses in concrete low.
(v) Follow good construction practices like thorough mixing good compaction and good curing.

13.2 DESIGN REQUIREMENT

IS: 3370 is the Indian code of practice for concrete structures for the storage of liquids. This was adopted in December 1967. It incorporated two amendments in 1997 and the same is reaffirmed in 1999. The code is available in the following four parts:

Part I : General requirements

Part II : Reinforced concrete structures

Part III : Prestressed concrete structures, and

Part IV : Design tables.

To avoid leakage problems, limit state method of design should not be used in water tanks. IS 456–2000 is silent about permissible stresses in direct tension. Hence from IS: 3370 (Re affirmed in 1999) it is obvious that earlier version of IS: 456 guide lines should be used, which is based on working stress method. Permissible stresses for concrete and steel are as shown in Tables 13.1 and 13.2.

Table 13.1: Permissible Stresses in Concrete

Grade of Concrete	Permissible Stress in Tension in N/mm²		Permissible Stress in Shear in N/mm²
	Direct	Bending	
M20	1.2	1.7	1.7
M25	1.3	1.8	1.9
M30	1.5	2.0	2.2
M35	1.6	2.2	2.5
M40	1.7	2.4	2.7

Types of Stress	Permissible Stress in N/mm²	
	Mild Steel	HYSD Bars
1. Direct tensile stress	115	150
2. Tensile stress in bending		
(i) On liquid retaining face	115	150
(ii) On face away from liquid if it is less than 225 mm	115	150
(iii) On face away from liquid, if it is ≥ 225 mm	125	190
3. Tensile stress in shear reinforcement		
(i) For members less than 225 mm thick	115	150
(ii) For members ≥ 225 mm thick	125	175
4. Compressive stress in columns subjected to direct load	125	175

Minimum Reinforcement

For thickness upto 100 mm, minimum percentage of reinforcement should be 0.3. For thicknesses from 100 mm to 450 mm it may be reduced linearly to 0.2 per cent. Hence

$$p_{min} = 0.3 \text{ upto } 100 \text{ mm thick sections}$$

$$= 0.3 - 0.1\frac{t-100}{450-100} \quad \text{for} \quad t = 100 \text{ mm to } 450 \text{ mm}$$

Minimum reinforcement should be ensured in both directions.

If thickness of section is more than 225 mm, layers of bars are required near both face, however it is enough if total steel meets the minimum requirement.

13.3 METHODS OF ANALYSIS

The behaviour of walls of water tank is more complex. They need sophisticated methods of analysis. For cylindrical tank, bending theory of cylinders with different edge conditions is required. For rectangular tanks, plate theory with appropriate boundary conditions at the four edges give better results. The continuity with adjacent walls and with top and bottom slabs also influence the values of moments and shears. One can think of finite element analysis to get the good result. IS: 3370 (Part IV) reaffirmed in 1999 gives the design tables to pick up moment and shear coefficients for the design of cylindrical as well as rectangular walls. Use of sophisticated analysis makes the design more economical.

However there are approximate methods of analysis, commonly used in the design. In the approximate method, it is assumed that in case of circular tanks bottom 1/3rd height or 1m, whichever is greater, is predominantly under cantilever action where as in case of rectangular tanks bottom 1/4 height or 1 m, whichever is greater is mainly under cantilever action. Rest of the wall is resisting water pressure by forces developed in horizontal directions. Approximate method is always on safer side and hence design is uneconomical. However it has the following advantages:

(i) It is simple

(ii) It gives feel of the structural behaviour.

Hence designer or site engineer can always avoid disasters of mistakes of draftman or those due to confusion of sign conventions in the analysis.

We may have lot of sophisticated methods of analysis to assess the design forces, but it is necessary for engineers to develop feel of structural behaviour. Hence the approximation methods of analysis should be learnt by engineering students. In this book designs are carried out after using approximate methods for the analysis.

13.4 DESIGN OF CIRCULAR TANKS RESTING ON GROUND

Circular tanks can have flexible base or rigid base. Fig. 13.1 shows typical circular tanks. In case of flexible joints, the wall is free to move outward when internal water pressure is applied and hence, the wall is subjected to hoop forces 'T' only.

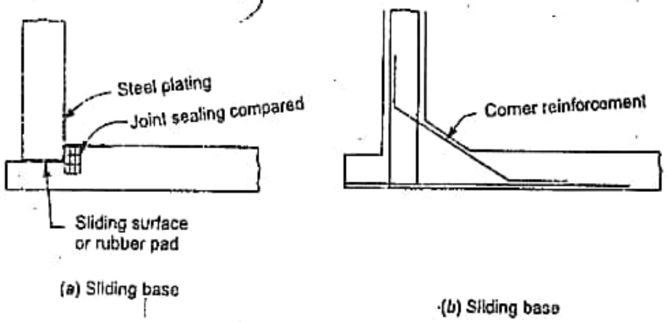


Fig. 13.1 *Typical circular tanks*

$$\boxed{T = \gamma H \frac{D}{2}} \qquad ...(13.1)$$

where γ = Unit weight of water

H = Height of tank and

D = Diameter of circular tank.

The reinforcement for hoop forces is to be given in horizontal directions. In vertical direction only minimum steel is to be provided.

In case of rigid joint, lower portion is having predominantly cantilever action while upper portion is mainly in hoop tension. Fig. 13.2 gives the approximated load diagram for the two actions. If 'h' is the height BD, then cantilever moment at base

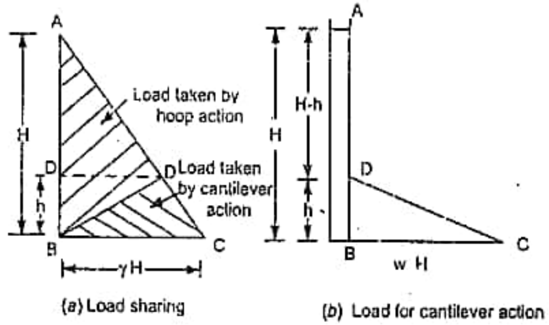


Fig. 13.2

$$= \gamma(H - h)\frac{h}{3} \qquad ...(13.2)$$

and maximum hoop tension at D

$$T = \gamma(H - h)\frac{D}{2}$$

For circular tanks 'h' may be taken as $\frac{H}{3}$ or 1 m whichever is more.

Examples 13.1 and 13.2 illustrate the method of design.

13.5 DESIGN CONSTANTS

Referring to Fig. 13.3, depth of neutral axis is 'nd' where

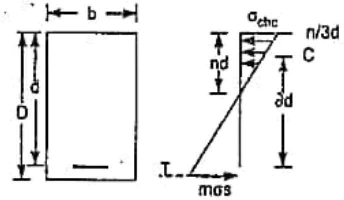


Fig. 13.3

$$n = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} \qquad ...(13.3)$$

Lever arm is 'jd' where

$$j = 1 - \frac{n}{3} \qquad ...(13.4)$$

and moment of resistance is given by

$$M = kbd^2$$

where $$k = \frac{1}{2}\sigma_{cbc}' jk \qquad ...(13.5)$$

The equivalent concrete section

$$A = A_c + mA_{st} = A_g + (m-1)A_{st} \qquad ...(13.6)$$

In the above expressions,

σ_{cbc} = Permissible compressive stress in concrete in bending

σ_{st} = Permissible stress in steel

and $\boxed{m = \text{Modular ration} = \frac{E_s}{E_c}}$

$= \frac{280}{3\sigma_{cbc}}$, which is usually rounded of to nearest full number ...(13.7)

A_c = Area of concrete

A_{st} = Area of steel

A_g = Gross area of cross-section.

Free Board

In all water tanks a free board of about 200 mm is to be given; in other words depth of water tanks in kept 200 mm more than the required depth for the full capacity. However for the design depth of water is taken as the total depth only since occasionally a stagnant water upto full height may be stored.

Example 13.1: *Design a circular water tank with flexible base resting on the ground to store 50,000 litres of water. The depth of tank may be kept 4 m. Use M25 concrete and Fe-415 steel.*

Solution:

Capacity of tank = 50,000 litres = 50 m^3

Depth of tank = 4 m

∴ If D is the diameter, then

$$\frac{\pi}{4}D^2 \times 4 = 50$$

$$D = 3.989 \text{ m}$$

Provide 4 m diameter

Free board = 200 mm

∴ Total height of tank $H = 4 + 0.2 = 4.2$ m

Unit weight of water $\gamma = 9.8$ kN/m^3

Permissible tensile stress in Fe-415 steel = 150 N/mm^2

Permissible tensile stress in concrete = 1.3 N/mm^2

Maximum hoop tension

$$T = \gamma H \frac{D}{2} = 9.8 \times 4.2 \times \frac{4}{2} = 82.32 \text{ kN/per meter height at base}$$

$$\sigma_s = 150 \text{ N/mm}^2$$

∴ Area of steel required for taking hoop tension

$$A_{sh} = \frac{82.32 \times 1000}{150} = 548.8 \text{ mm}^2 \rightarrow \frac{T}{\sigma_{st}}$$

Using 12 mm bars, spacing

$$\frac{\frac{\pi}{4} \times 12^2}{548.8} \times 1000 = 206 \text{ mm}$$

Provide 12 mm bars at 200 mm c/c.

A_{sh} provided $= \frac{\pi}{4} \times 12^2 \times \frac{1000}{200} = 565.5$ mm^2 per metre height.

Increase the spacing to 300 mm at a height 1.5 m from base.

Thickness of Wall

Maximum hoop tension T = 82.32 kN

Permissible stress in tension = 1.3 N/mm^2

Modular ratio for M25 concrete

$$m = \frac{280}{3 \times \sigma_{cbc}} = \frac{280}{3 \times 8.5} = 11$$

If 't' is the thickness of wall, equivalent area of concrete per metre height

$$= 1000\,t + (m-1)A_{sh}$$

Hence

$$\therefore \quad \sigma_c = \frac{T}{1000t + (m-1)A_{sh}}$$

$$1.3 = \frac{82.32 \times 1000}{1000t + (11-1) \times 565.5}$$

or $t = 57.66$ mm

Provide $t = 100$ mm

Vertical Steel

Only minimum reinforcement is required.

∴ A_{st} minimum $= \frac{0.3}{100} \times 100 \times 1000 = 300$ mm^2

Using 8 mm bars,

$$s = \frac{\frac{\pi}{4} \times 8^2}{300} \times 1000 = 167 \text{ mm}$$

Provide 8 mm bars at 150 mm c/c.

Base Slab

The base slab will be laid on a 75 mm lean mix bed covered with tarfelt. Since the load gets transferred to ground directly, a nominal thickness of 150 mm may be provided with minimum reinforcement in both direction.

$\therefore A_{st}$ minimum $= \dfrac{0.3}{100} \times 150 \times 1000 = 450 \text{ mm}^2$

Providing half the reinforcement near each face

$$A_{st} = 225 \text{ mm}^2$$

Using 8 mm bars,

$$s = \frac{\frac{\pi}{4} \times 8^2}{225} \times 1000 = 223 \text{ mm}$$

Provide 8 mm bars at 220 mm c/c on both faces in both directions.

Fig. 13.4 shows the details of reinforcement.

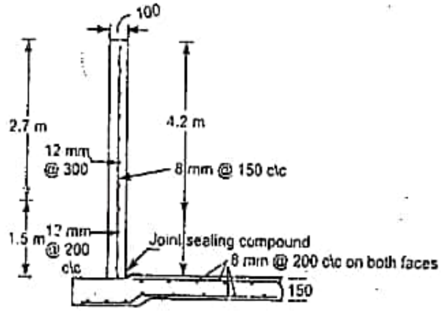


Fig. 13.4

②

Example 13.2: *Design the water tank for the data given in example 13.1 assuming that the joint between wall and base slab is rigid. Approximate method may be used for the analysis.*

Solution:

Dimensions of the tank: Diameter D = 4 m

Total height H = 4.2 m

Mix used: M25. Steel to be used Fe-415

$\therefore$ $\sigma_{cbc} = 8.5 \text{ N/m}^2$ and $\sigma_{st} = 150 \text{ N/mm}^2$

$\therefore$ Modular ratio $= \dfrac{280}{3 \times 8.5} = 11$

Design constants are

$$k = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = \frac{11 \times 8.5}{11 \times 8.5 + 150} = 0.384$$

$$j = 1 - \frac{k}{3} = 1 - \frac{0.384}{3} = 0.872$$

$$Q = \frac{1}{2}\sigma_{cbc} \times j \times k = \frac{1}{2} \times 8.5 \times 0.872 \times 0.384 = 1.428$$

$\frac{1}{2}\sigma_{cbc} \, j \, k$

Design for Cantilever Action

The height 'h' above base upto which cantilever action exist is given by

$$h = \frac{H}{3} \text{ or } 1 \text{ m whichever is more}$$

$\therefore$ $h = \dfrac{4.2}{3} = 1.4$ m

$\boxed{\text{Cantilever moment} \quad = \frac{1}{2}\gamma H \times h \times \frac{h}{3}}$

h γH

$$= \frac{1}{2} \times 9.8 \times 4.2 \times 1.4 \times \frac{1.4}{3} = 13.446 \text{ kN-m}$$

$\therefore$ Depth of balance section

H γ_w

$$d = \sqrt{\frac{M}{k \times b}} = \sqrt{\frac{13.446 \times 10^6}{1.428 \times 1000}} = 97.3 \text{ mm}$$

To keep the section sufficiently under reinforced.

150 (cm) / 30 / dph + 50

C.C = 30 mm

Let $d = \dfrac{4}{3} \times 97.3 = 129.7$ mm

10.1

Let us keep d = 130 mm and total thickness 165 mm. (Note: Minimum thickness of 150 mm is normally kept to avoid leakage problems).

$$\boxed{A_{st} = \frac{M}{\sigma_{st} jd}} = \frac{13.446 \times 10^6}{150 \times 0.872 \times 130} = 790.8 \text{ mm}^2$$

Using 10 mm bars,

$$s = \frac{\frac{\pi}{4} \times 10^2}{790.8} \times 1000 = 99.32$$

Provide 10 mm bars at 95 mm c.c. near inner face, keeping clear cover of 30 mm.

Hence let us provide 10 mm bars at 95 mm c.c. at base and curtail alternate bars at a height of 1.4 m, so that a spacing of 190 mm is available in top 2.8 m height.

Design of Section for Hoop Action

For this reinforcement is to be provided in horizontal direction. Max hoop tension is to be considered at height h =1.4 m in this case. Hoop tension is given by

$$\boxed{T = \gamma_{w}(H - h) \times D/2}$$

$$= 9.8(4.2 - 1.4) \times \frac{4}{2} = 54.88 \text{ kN}$$

$$\therefore \quad A_{sh} = \frac{54.88 \times 1000}{150} = 365.8 \text{ mm}^2 = \frac{T}{\sigma_{st}}$$

Using 10 mm bars, spacing

$$s = \frac{\frac{\pi}{4} \times 10^2}{365.8} \times 1000 = 214 \text{ mm}$$

Provide 10 mm bars @ 200 mm c/c.

Check for tensile stress in concrete:

Actual $$A_{sh} = \frac{\frac{\pi}{4} \times 10^2}{200} \times 1000 = 392.6 \text{ mm}^2$$

$$\therefore \quad \sigma_{ct} = \frac{T}{\text{Equivalent concrete area}} = \frac{F_t}{A_c + mA_{st}} \quad \frac{F_t}{A_g + (m-1)A_{st}}$$

$$= \frac{54.88 \times 1000}{165 \times 1000 + (11 - 1) \times 392.6} = 1.10 \text{ N/mm}^2$$

Permissible σ_{ct} for M25 concrete = 1.3 N/mm²

Hence safe.

For bottom 1.4 m above base the spacing of 100 mm may be maintained. In the remaining portion it may be raised to 300 mm c/c.

Distribution Steel (In vertical direction)

Minimum steel required $= \frac{0.3}{100} \times 165 \times 1000 = 495 \text{ mm}^2$.

$\therefore$ Vertical steel for cantilever action serves this purpose also.

Base Slab

Provide nominal thickness of 150 mm with nominal reinforcement of 8 mm bars at 220 mm c/c in both direction.

Provide 150 mm × 150 mm haunches at junction. To ensure the rigidity of connection, provide junction reinforcement of 8 mm bars at 220 mm c/c. It takes care of development length required for cantilever steel.

Fig. 13.5 shows the details of reinforcement.

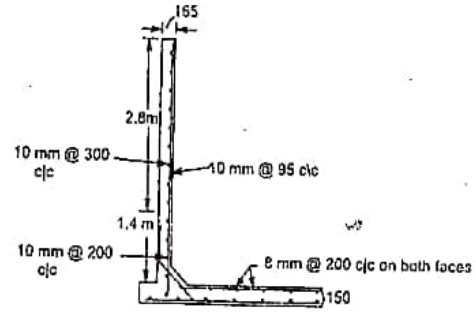


Fig. 13.5

13.6 RECTANGULAR TANKS RESTING ON GROUND

Consider the design of rectangular water tank of size L × B × H, where

L — Length of tank

B — Breadth of tank

H — Total height of tank

In the approximate methods such tanks are divided into two categories:

(*i*) Tanks with L/B < 2

(*ii*) Tanks with L/B ≥ 2

(*i*) **Design of tanks with L/B < 2:** Similar to design of circular tanks, here also lower part is assumed to have predominantly cantilever action and upper portion to have resistance by horizontal action. The load taken by the two actions is shown in Fig. 13.6(*a*), where D is a point at a height

$$h = H/4 \text{ or } 1 \text{ m, whichever is more} \qquad \text{...(13.8)}$$

Hence maximum cantilever moment on the wall

$$= \frac{1}{2}\gamma H h . \frac{h}{3} \qquad \text{...(13.9)}$$

For horizontal action, maximum pressure is at (H – h) m below top (at D). Hence $p_h = \gamma(H - h)$ as shown in Fig. 13.6 (*b*).

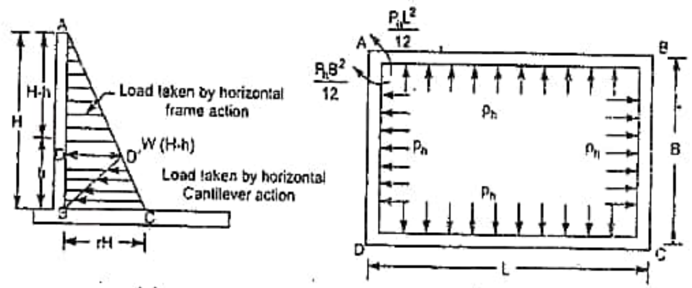


Fig. 13.6 *Pressure Diagram*

This is resisted by closed frame action. The frame being symmetric, analysis is simple. The fixed ·nd moment at A are

$$p_h \frac{B^2}{12} \text{ and } \frac{p_h L^2}{12} \qquad \ldots(13.10)$$

Using moment distribution the moments may be balanced.

Since long wall supports short wall and short wall supports long wall, horizontal tension develops i: the walls. At D, horizontal forces developed are

$$T_L = \gamma(H-h)\frac{B}{2} \qquad \ldots(13.8(a))$$

a: [

$$T_B = \gamma(H-h)\frac{L}{2} \qquad \ldots(13.8(b))$$

The effect of horizontal tensile forces is to reduce the net moment in walls to an extend Tx, where x : distance of tensile reinforcement from the centre of wall. Thus final horizontal design moment is

$$= M - Tx \qquad \ldots(13.8)$$

'he bending moment reduces towards top above 'h'. Hence spacing may be increased towards the to; However minimum reinforcement requirement should not be violated.

may be noted that, near corners bending tension is on inner face and near centre it is on outer sid

: i) **Design of tanks with L/B ≥ 2:** In such cases, long walls behave like cantilevers of height H. Thi·kness of walls may be decided on the basis of cantilever moments in long wall. The horizontal stee required in the long wall is to resist direct tension $T_L = \gamma(H-h)\frac{B}{2}$. Designer will find this requ rement is automatically satisfied by providing minimum reinforcement of 0.3 per cent.

L wer portion of short wall of height 'h' is resisting the load by cantilever action and top H – h resi· the load by horizontal frame action, as discussed in case (i). Hence cantilever moment in short wall is $\gamma(H-h)\frac{h^2}{6}$.

D: to horizontal frame action, bending moment may be taken equal to $\frac{\gamma(H-h)B^2}{16}$, both at ends and c ntre. At ends, tension is on inner face and at centre it is on outer face. Though long walls are redo ninantly resisting the load by cantilever action, end 1 m may be considered as supported by hon ·alls

$$T_B = \gamma(H-h) \times 1$$

Th: will reduce the design moment by $T_B x$, where x is the distance of reinforcement from centre f sec: on. The reinforcement is calculated for bending and direct tension separately and total steel is ovi: d in horizontal direction.

The lesign procedure is illustrated with two examples below:

Example 13.3: *Design a rectangular water tank of size 5 m × 4 m × 3 m deep resting on firm ground. Use M25 concrete and mild steel.*

Solution:

Size of tank = 5 m × 4 m × 3 m, deep

Grade of concrete M25

$$\therefore \quad \sigma_{cbc} = 8.5 \text{ N/m}^2$$

$$m = \frac{280}{3 \times 8.5} = 11$$

$$\sigma_{st} = 115 \text{ N/mm}^2$$

∴ **Design constants are**

$$n = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = \frac{11 \times 8.5}{11 \times 8.5 + 115} = 0.448$$

$$j = 1 - \frac{n}{3} = 1 - \frac{0.448}{3} = 0.850$$

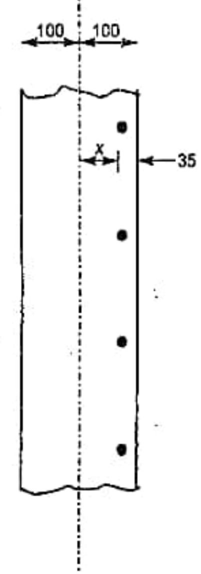
$$k = \frac{1}{2}\sigma_{cbc} j n = \frac{1}{2} \times 8.5 \times 0.850 \times 0.448 = 1.619$$

In this problem,

$$\frac{L}{B} = \frac{5}{4} = 1.25 < 2$$

Hence both long and short wall resist the load by cantilever action for height $h = 1$ m and by horizontal action resist the load in the top H-h = 3 – 1 = 2 m. In such water tanks, moment due to horizontal action is considerable and it governs the selection of thickness of walls. Hence horizontal frame action is first considered.

Horizontal Frame Action



The critical section is at a height $h = \frac{H}{4}$ or 1 m whichever is more. Hence in this case $h = 1$ m

$$p_h = \gamma(H-h) = 9.8(3-1) = 19.6 \text{ kN/m}^2$$

∴ Fixed end moments are

$$= \frac{p_h L^2}{12} = \frac{19.6 \times 5^2}{12} = 40.833 \text{ kN-m, in long wall}$$

and

$$= \frac{p_h B^2}{12} = \frac{19.6 \times 4^2}{12} = 26.133 \text{ kN-m, in short walls}$$

Since thickness of short and long walls are maintained same, distribution factors at joints are as shown below:

Member	Stiffness	Total joint stiffness	Distribution factor
Short wall	$\frac{4EI}{4} = EI$	1.8EI	0.556
Long wall	$\frac{4EI}{5} = 0.8EI$		0.444

Due to symmetry one balancing will take care of moment distribution as shown in table below:

Short wall	0.556	0.444	Long wall
	−26.133	40.883	
	− 8.20	−6.55	
	34.333	34.333	

∴ Corner moment = 34.333 kN-m, tension outside.

Effective thickness required for balance section is

$$d = \sqrt{\frac{M}{kb}} = \sqrt{\frac{34.333 \times 10^6}{1.619 \times 1000}} = 146 \text{ mm}$$

Section is to be kept sufficiently under reinforced. Hence let us keep overall thickness of 200 mm with effective cover of 35 mm.

∴ $d = 200 - 35 = 165$ mm

Direct pull on long and short walls are given by

$$T_L = p_h \times \frac{B}{2} = 19.6 \times \frac{4}{2} = 39.2 \text{ kN}$$

and $$T_B = p_h \times \frac{L}{2} = 19.6 \times \frac{5}{2} = 49 \text{ kN}$$

Eccentricity of reinforcement from centre of wall

$$x = \frac{200}{2} - 35 = 65 \text{ mm}$$

∴ Design moment at corner

$$= M - Tx$$
$$= 34.333 - 39.2 \times 0.065 = 31.785 \text{ kN-m}$$

Hence at corner, horizontal reinforcement required for bending resistance

$$A_{st1} = \frac{31.785 \times 10^6}{\sigma_{st} j d} = \frac{31.785 \times 10^6}{115 \times 0.850 \times 165} = 1970 \text{ mm}^2$$

for direct tension

$$A_{st2} = \frac{39.2 \times 1000}{115} = 341 \text{ mm}^2$$

∴ Total $A_{st} = 2311$ mm^2

Using 20 mm bars, spacing required is

$$s = \frac{\frac{\pi}{4} \times 20^2}{2311} \times 1000 = 136 \text{ mm}$$

Provide 20 mm bars at 130 mm c/c. It is to be provided on water face.

Reinforcement at middle of long walls:

$$\text{Bending moment} = \frac{p_h L^2}{8} - \text{Moment at corner}$$

$$= 19.6 \times \frac{5^2}{8} - 34.333 = 26.917 \text{ kN-m}$$

Design moment $= M - T_L x$

$$= 26.917 - 39.2 \times 0.065 = 24.369 \text{ kN-m}$$

∴ $$A_{st1} = \frac{24.369 \times 10^6}{115 \times 0.850 \times 165} = 1511 \text{ mm}^2$$

∴ $$A_{st2} = \frac{39.2 \times 1000}{115} = 341 \text{ mm}^2$$

∴ $$A_{st} = A_{st1} + A_{st2} = 1852 \text{ mm}^2$$

Using 20 mm bars, spacing

$$s = \frac{\frac{\pi}{4} \times 20^2}{1852} \times 1000 = 169 \text{ mm}$$

These bars are to be provided at outer face.

Here also bars may be provided at 130 mm c/c, so that bars may be bent and used.

Reinforcement for Short Wall

$$M = 34.333 - T_B x = 34.333 - 49 \times 0.065$$
$$= 31.148 \text{ kN-m}$$

∴ $$A_{st1} = \frac{31.148 \times 10^6}{115 \times 0.850 \times 165} = 1931 \text{ mm}^2$$

$$A_{st2} = \frac{49 \times 1000}{115} = 426 \text{ mm}^2$$

Total $A_{st} = 2357$ mm^2

Using 20 mm bars, spacing $= \frac{\frac{\pi}{4} \times 20^2}{2357} \times 1000 = 133$ mm

Provide 20 mm bars at 130 mm c/c.

Bending moment at centre of wall

$$= \gamma(H-h)\frac{B^2}{8} - \text{Moment at ends}$$

$$= 9.8(3-1)\times\frac{4^2}{8} - 34.333 = 4.867 \text{ kN-m}$$

It is quite small. It is taken care by minimum reinforcement. Bend alternate bars provided for end moment at a distance $\frac{B}{4} = 1$m from each end and continue remaining half throughout. Hence at centre wall reinforcement consist of 20 mm bars at 260 mm c/c.

Reinforcement in Vertical Direction

Cantilever moment $= \gamma H\frac{h^2}{6} = 9.8\times 3\times\frac{1^2}{6} = 4.9$ kN-m

$$\therefore \quad A_{st} = \frac{M}{\sigma_{st}\, j d} = \frac{4.9\times 10^6}{115\times 0.850\times 165} = 304 \text{ mm}^2$$

Minimum reinforcement $= \frac{0.3}{100}\times 200\times 1000 = 600$ mm^2

Provide 304 mm^2 area on each face so that required distribution steel is also available.

Using 10 mm bars, $s = \dfrac{\frac{\pi}{4}\times 10^2}{304}\times 1000 = 258$ mm

Provide 10 mm bars at 250 mm c/c on both faces.

Note: Inside bars should not be bent as shown in Fig. 13.7 (*a*).

To avoid bursting of concrete due to resultant force they should be bent as shown in Fig.13.7(*b*)

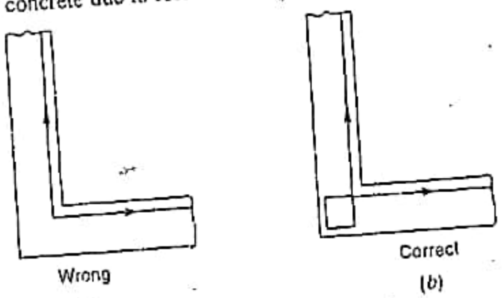


Fig. 13.7

Base Slab

Provide nominal base slab of thickness 150 mm with 8 mm bars at 220 mm c/c in both direction at top and bottom of slab. A lean concrete bed of 100 mm may be provided on which bottom slab can rest.

Details of reinforcement are shown in Fig. 13.8.

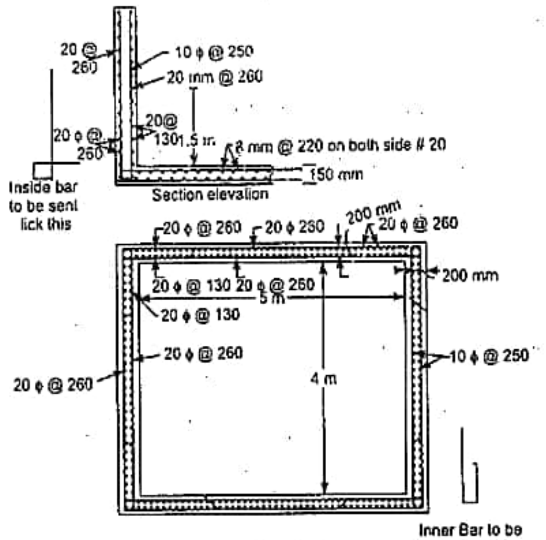


Fig. 13.8

Example 13.4: *Design an open rectangular tank of size 3 m × 8 m × 3 m deep resting on a firm ground. Use M25 grade concrete and Fe 415 steel. Approximate method may be used for the analysis.*

Solution:

Size of the tank 3 m × 8 m × 3 m deep

Grade of concrete: M25, Grade of steel Fe-415

$$\therefore \quad \sigma_{cbc} = 8.5 \text{ N/mm}^2 \text{ and } \sigma_{st} = 150 \text{ N/mm}^2$$

Modular ratio $m = \frac{280}{3\times 8.5} = 11$

$$n = \frac{m\sigma_{cbc}}{m\sigma_{cbc}+\sigma_{st}} = \frac{11\times 8.5}{11\times 8.5+150} = 0.384$$

$$j = 1-\frac{n}{3} = 1-\frac{0.384}{3} = 0.872$$

$$k = \frac{1}{2}\sigma_{cbc}\, n\, j = \frac{1}{2}\times 8.5\times 0.384\times 0.872 = 1.423$$

Now, $\frac{L}{B} = \frac{8}{3} > 2$

Hence long wall predominantly acts as cantilever of height H = 3 m

Design of Long Wall

$$M = \frac{\gamma H^3}{6} = 9.8 \times \frac{3^3}{8} = 44.1 \text{ kN-m}$$

Equating moment of resistance to bending moment, for balanced section we get

$$kbd^2 = M$$

or

$$d = \sqrt{\frac{M}{kb}} = \sqrt{\frac{44.1 \times 10^6}{1.423 \times 1000}} = 176 \text{ mm}$$

Provide 220 mm total thickness with effective cover 35 mm. Hence $d = 220 - 35 = 185$ mm

∴ Reinforcement for cantilever action (vertical on water side)

$$\therefore \quad A_{st} = \frac{M}{\sigma_{st}\, jd} = \frac{44.1 \times 10^6}{150 \times 0.872 \times 185} = 1823 \text{ mm}^2$$

$\frac{h^3}{H^3} = 1/2$ $1/3$ H.
$h = (1/2)$

Using 16 mm bars, spacing is

$$s = \frac{\frac{\pi}{4} \times 16^2}{1823} \times 1000 = 119 \text{ mm}$$

Provide 16 mm bars at 110 mm c/c in vertical direction near innerface of the tank.

Curtailment of bars: Since moment is given γh^3 at any depth 'h' below top,

$\frac{A_{st}}{A_{sth}} = \frac{h^3}{H^3}$ or h = 2.38 m from top or 0.62 m from base. However the above value is only theoretical. As per code requirement actual curtailment should be at

$$\boxed{= 0.62 + 12 \times \text{diameter of bar}}$$

$$= 0.62 + 12 \times \frac{16}{1000} = 0.812 \text{ m}$$

Hence curtail alternate bars at 0.9 m from base.

Reinforcement in Long Wall in Horizontal Direction

Direct tensile force transferred by short wall on long wall

$$T_L = \gamma(H - h)\frac{B}{2} = 9.8(3-1) \times \frac{3}{2} = 29.4 \text{ kN}$$

∴ Horizontal reinforcement required

$$= \frac{29.4 \times 1000}{150} = 196 \text{ mm}^2, \text{ too small}$$

Minimum reinforcement to be provided

$$= \frac{0.3}{100} \times 220 \times 1000 = 660 \text{ mm}^2$$

Hence 330 mm^2 area may be provided on each face. Using 8 mm bars,

$$s = \frac{\frac{\pi}{4} \times 8^2}{330} \times 1000 = 152 \text{ mm}$$

∴ Provide 8 mm bars at 150 mm c/c near each face in horizontal direction.

Design of Short Wall

Reinforcement in vertical direction:

$$M = \frac{\gamma H h^2}{6} = \frac{9.8 \times 3 \times 1^2}{6} = 4.9 \text{ kN-m}$$

$$A_{st} = \frac{4.9 \times 10^6}{150 \times 0.87^2 \times 185} = 202 \text{ mm}^2$$

Too small. Provide minimum reinforcement of 8 mm bars at 150 mm c/c near each face.

Reinforcement in Horizontal Direction

Water pressure at h =1 m above base

$$p_h = 9.8 \times (3 - 1) = 19.6 \text{ kN/m}^2$$

∴ Bending moment at ends may be taken as $\frac{p_h B^2}{12} = \frac{19.6 \times 3^2}{12} = 14.7$ kN-m

Actual tension due to 1 m length of long wall,

$$\boxed{T_B = \gamma(H - h) \times 1 =} 9.8(3 - 1) \times 1 = 19.6 \text{ kN}$$

$$\therefore \quad A_{st1} = \frac{14.7 \times 10^6}{150 \times 0.872 \times 185} = 607 \text{ mm}^2$$

$$A_{st2} = \frac{19.6 \times 1000}{150} = 130 \text{ mm}^2$$

$$A_{st} = 607 + 130 = 737 \text{ mm}^2$$

Using 12 mm bars spacing,

$$s = \frac{\frac{\pi}{4} \times 12^2}{737} \times 1000 = 153 \text{ mm}$$

∴ Provide 12 mm bars at 150 mm c/c near inner face at ends.

In the middle portion.

$$M = \gamma(H-h)\frac{B^2}{24} = 9.8 \times 2 \times \frac{3^2}{24} = 7.35 \text{ kN·m}$$

$$\therefore \quad A_{st1} = \frac{7.35 \times 10^6}{150 \times 0.872 \times 185} = 303.5 \text{ mm}^2$$

$$\left(\text{Note : It is } \frac{1}{2} \text{ of } A_{st} \text{ required at end}\right)$$

Hence provide 12 mm bars at 300 c/c.

Base Slab

Provide nominal base slab. Reinforcement details are shown in Fig. 13.9.

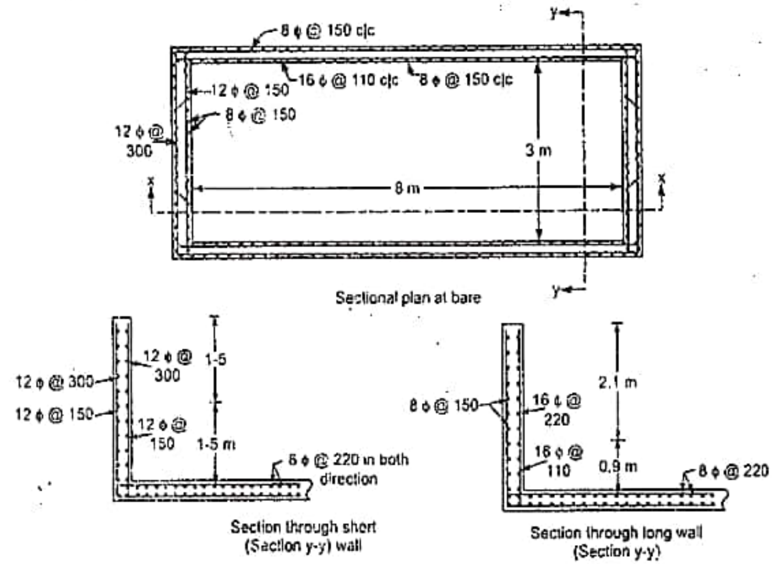


Fig. 13.9

13.7 UNDERGROUND TANKS

In water supply system to towns underground water tanks are used to store water received from mains. The tanks may be circular or rectangular. For larger capacities circular tanks are preferable, since for the same capacity they consume less material. As the cost of shuttering for circular tanks per unit area is large, rectangular tank work out cheaper for small capacities. Under ground tanks are to be designed to sustain the following two cases:

Case (*i*) Tank full and no earthfill.

Case (*ii*) Tank empty and active earth pressure acting from outside.

Design for case (*i*) is same as explained for tanks resting on ground. In case (*ii*), external pressure depends upon the type of back fill.

(*a*) Active earth pressure due to dry soil or wet cohesionless soil

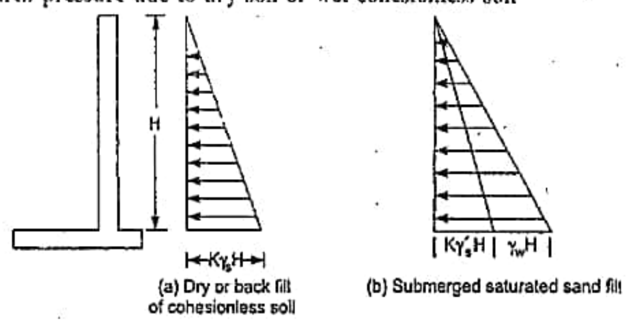


Fig. 13.10

The active earth pressure acting on wall varies linearly (Ref. Fig. 13.10) and its maximum value is

$$p_h = K\gamma_s H \qquad ...(13.13)$$

where, K — Rankines coefficient of earth pressure

γ_s — Unit weight of soil

H — Total depth of tank.

Rankines coefficient of earth pressure is given by

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

where, ϕ is angle of repose.

(*b*) If back fill is saturated sandy soil

It consist of active earth pressure due to saturated backfill (γ'_s) and due to water pressure from outside. This situation for water table upto top of tank is also shown in Fig. 13.10. In this case maximum pressure from outside is

$$p_h = K\gamma'_s H + \gamma_w H \qquad ...(13.17)$$

where γ'_s — Unit weight of saturated sandy soil

and γ_w — 9.8 kN/m^3 is unit weight of water.

Designer has to adjust the reinforcements judiciously to take care of both loading cases discussed above. Apart from designing the walls bottom slab also needs the design. It is designed for uplift ~~pressure from saturated soil~~ below. The tank should not get lifted due to this uplift pressure. Hence bottom slab is projected beyond the walls so that weight of soil on this projected portion helps in adding downward load to resist upward water pressure.

Underground tanks need ~~roof slab~~ to keep water clean. Hence the designer must design the roof slab, which is similar to design of slabs in buildings.

The example below illustrate the design procedure for a rectangular tank. On the same lines design of circular tanks also may be taken up noting that circular sections are subjected to ~~hoop~~ forces in horizontal plane where as rectangular sections are subjected to continuous frame action.

Example 13.5: *Design an underground water tank of size 3 m × 8 m × 3 m for the following data:*

Type of soil: Submerged sandy soil, with

$$\gamma_s = 16 \text{ kN/m}^3, \quad \phi = 30°$$

Water table can rise upto ground level.

Grade of concrete

(i) For tank : M25

(ii) For roof slab : M20

Grade of steel : Fe-415

Unit weight of water $= 9.8 \text{ kN/m}^3$

Live load on roof slab $= 2 \text{ kN/m}^2$

Solution:

Design of Roof Slab

Size 3 m × 8 m

$$\frac{L}{B} > 2$$

Hence may be designed as one way slab. It may be designed by working stress method with higher permissible stresses (230 N/mm² for Fe-415 steel or 140 N/mm² for mild steel) since there is no leakage problem for this element. One can use limit state method also. Concrete of grade M20 is preferred from the consideration of economy.

Let $\quad d = \frac{l_x}{25} = \frac{3000}{25} = 120 \text{ mm}$

Let us select $d = 120$ mm and overall depth D = 150 mm

Using M20 concrete and Fe-415 steel, the slab will be designed

Self weight $= 0.15 \times 1 \times 1 \times 25 = 3.75 \text{ kN/m}^2$

Live load $= 2.0 \text{ kN/m}^2$

Finishing load $= 0.5 \text{ kN/m}^2$

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

$$M = 6.25 \times \frac{3^2}{8} = 7.03 \text{ kN-m}$$

$$M_u = 1.5 \times 7.03 = 10.55 \text{ kN-m}$$

$$M_{u\,lim} = 0.38 f_{ck} bd^2$$
$$= 0.138 \times 20 \times 1000 \times 120^2$$
$$= 39.444 \times 10^6 \text{ N-mm}$$
$$= 39.444 \text{ kN-m} > M_u$$

Hence under reinforced section. Equating moment to moment of resistance,

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

$$A_{st} = \frac{0.5 f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 M_u}{bd^2 f_{ck}}}\right) bd.$$

$$10.55 \times 10^6 = 0.87 \times 415 \times A_{st} \times 120\left(1 - \frac{A_{st}}{1000 \times 120} \times \frac{415}{20}\right)$$

i.e., $\quad 243.5 = A_{st}\left(1 - \frac{A_{st}}{5783.13}\right)$

or $\quad A_{st}^2 - 5783.13\, A_{st} + 243.5 \times 5783.13 = 0$

$$A_{st} = 254.7 \text{ mm}^2$$

Minimum to be provided $= \frac{0.12}{100} \times 1000 \times 120 = 144 \text{ mm}^2$

Using 10 mm bars,

$$s = \frac{\frac{\pi}{4} \times 10^2}{254.7} \times 1000 = 308 \text{ mm}^2$$

Provide 10 mm bars at 300 mm c/c

Distribution steel $= 0.12\% = 144 \text{ mm}^2$

Using 8 mm bars,

$$s = \frac{\frac{\pi}{4} \times 8^2}{144} \times 1000 = 349 \text{ mm}^2$$

Provide 8 mm bars at 300 mm c/c.

Design of Walls

These are to be designed with working stress method with lower values of permissible stresses to avoid leakage problem. Using M25 concrete and Fe-415 steel,

$\therefore \quad \sigma_{cbc} = 8.5 \text{ kN/m}^2 \qquad \therefore m = 11 \qquad \sigma_{st} = 150 \text{ N/mm}^2$

$$n = \frac{m\sigma_{cbc}}{m\sigma_{cbc} + \sigma_{st}} = \frac{11 \times 8.5}{11 \times 8.5 + 150} = 0.384$$

$$j = 1 - \frac{n}{3} = 1 - \frac{0.384}{3} = 0.872$$

$$K = \frac{1}{2}\sigma_{cbc}\, n\, j = \frac{1}{2} \times 8.5 \times 0.384 \times 0.872 = 1.423$$

In such tanks usually cantilever moment, when tank is empty, governs the choice of thickness. Hence let us first consider the design of long wall

(*a*) **When tank is empty**

$$p_h = K\gamma'_s H + \gamma_w H$$

where $\quad K = \frac{1 - \sin\phi}{1 + \sin\phi} = \frac{1 - \sin 30°}{1 + \sin 30°} = \frac{1}{3}$

$$\gamma'_s = \gamma_s - \gamma_w = 16 - 9.8 = 6.2 \text{ kN/m}^2$$

$$\gamma_w = 9.8 \text{ kN/m}^3$$

$$\therefore \quad p_h = \frac{1}{3} \times 6.2 \times 3 + 9.8 \times 3 = 35.6 \text{ kN/m}^2$$

$$\boxed{M = \frac{1}{2} \times 35.6 \times H \times \frac{H}{3}} = \frac{1}{2} \times 35.6 \times 3 \times \frac{3}{3} = 53.4 \text{ kN-m}$$

∴ Depth of balanced section

$$= \sqrt{\frac{53.4 \times 10^6}{1.423 \times 1000}} = 193 \text{ mm}$$

Provide d = 195 mm and D = 195 + 35 = 230 mm

$$\therefore \quad \frac{M}{\sigma_{st} j . d} \quad A_{st} = \frac{53.4 \times 10^6}{150 \times 0.872 \times 195} = 2094 \text{ mm}^2$$

Using 20 mm bars, spacing required is

$$= \frac{\frac{\pi}{4} \times 20^2}{2094} \times 1000 \approx 150 \text{ mm}$$

Provide 20 mm bars at 150 mm c/c near outer face of the wall.

Alternate bars may be curtailed where bending moment is half that at base *i.e.*, at a depth

$$\frac{h^3}{H^3} = \frac{1}{2}; \qquad h = \left(\frac{1}{2}\right)^{\frac{1}{3}} 3 = 2.38 \text{ m}$$

i.e., at a height 3 - 2.38 = 0.62 m from base.

The above value is theoretical value. As per code requirement add 12 × diameter of bars to above value. Hence the bars are to be curtailed at a height = 620 + 12 × 20 = 860 mm from base. Hence curtail alternate bars at 0.9 m from base.

(*b*) **When tank is full and no earth pressure**

$$p_h = \gamma_w H = 9.8 \times 3 = 29.4 \text{ kN/m}^2$$

$$\therefore \text{ Hence cantilever moment } \boxed{M = \frac{1}{2} \times 29.4 \times 3 \times \frac{3}{3}} = 44.1 \text{ kN-m}$$

$$\therefore \quad A_{st} = \frac{44.1 \times 10^6}{150 \times 0.872 \times 195} = 1729 \text{ mm}^2$$

Using 16 mm bars.

$$s = \frac{\frac{\pi}{4} \times 16^2}{1729} \times 1000 = 116 \text{ mm}$$

Provide 16 mm bars at 110 mm c/c on inner face in vertical direction.

Horizontal Bars In Long Walls

Since long wall is predominantly acting as a cantilever, distribution steel is provided and checked for axial tension when tank is full without earth pressure from outside.

Since thickness of wall is more than 225 mm, minimum percentage of steel to be provided is

$$= 0.3 - 0.1 \frac{230 - 100}{450 - 100} = 0.263$$

$$\therefore \quad A_{st} = \frac{0.263}{100} \times 230 \times 1000 = 604 \text{ mm}^2$$

Steel required on each face = 302 mm²

Using 8 m bars, spacing required

$$= \frac{\frac{\pi}{4} \times 8^2}{302} \times 1000 = 166 \text{ mm}$$

Provide 8 mm bars at 160 mm c/c. They hold the vertical steel provided for cantilever action due to the two loading cases considered.

Check for Direct Tension

$$T_L = \gamma_w (H - h) \frac{B}{2} = 9.8(3 - 1) \times \frac{3}{2} = 29.4 \text{ kN}$$

∴ Area of steel required

$$= \frac{29.4 \times 1000}{150} = 196 \text{ mm}^2 < 604 \text{ mm}^2$$

∴ Distribution steel takes care of this tensile force.

Design of Short Wall

Design of lower portion for cantilever action (Vertical reinforcement)

$$h = \frac{H}{4} \text{ or } 1 \text{ m whichever is more}$$

$$= 1 \text{ m, in this problem}$$

When tank is empty and outside sandy soil is saturated.

$$p_h = 35.6 \text{ kN/m}^2$$

$$M = \frac{1}{2} \times 35.6 \times 1 \times \frac{1}{3} = 5.933 \text{ kN-m}$$

$$A_{st} = \frac{M}{\sigma_{st} j d} = \frac{5.933 \times 10^6}{150 \times 0.872 \times 195} = 232 \text{ mm}^2$$

Direct compression due to load on 1 m wide long wall

$$p = 35.6(3 - 1) \times 1 = 71.2 \text{ kN}$$

Concrete alone can resist it.

When the tank is full and no earth fill:

$$p_h = 9.8 \times 3 = 19.4 \text{ kN/m}^2$$

$$M = \frac{1}{2} \times 29.4 \times 1 \times \frac{1}{3} = 4.833 \text{ kN-m, quite small}$$

Provide minimum reinforcement in vertical direction, which is 8 mm bar at 160 mm c/c as found earlier. It is to be provided near both faces.

Design of top $H - h = 3 - 1 = 2$ m portion

$$\boxed{p_h = k\gamma'_s (H - h) + \gamma_w (H - h)}$$

$$= \frac{1}{3} \times (16 - 9.8)(3 - 1) + 9.8(3 - 1) = 23.73 \text{ kN/m}^2$$

Moment at support $\boxed{= \frac{23.73 \times 3^2}{12}} = 17.8$ kN-m

$$A_{st} = \frac{17.8 \times 10^6}{150 \times 0.872 \times 195} = 698 \text{ mm}^2$$

At mid span bending moment is half of 17.8 kN-m

Hence $A_{st} = 399$ mm^2

At support, using 10 mm bars spacing required is

$$s = \frac{\frac{\pi}{4} \times 10^2}{698} \times 1000 = 112 \text{ mm}$$

Provide 10 mm bars at 110 mm c/c (near outer face).

At middle portion alternate bars may be bent inside.

Bottom Slab

Assuming thickness of bottom slab ≈ 0.2 m,

$$H = 3 + 0.2 = 3.2 \text{ m}$$

$\therefore$ Upward pressure when sandy soil is saturated

$$= 9.8 \times 3.2 = 31.36 \text{ kN/m}^2$$

The bottom slab is to be projected beyond walls of tank so that soil over it helps in avoiding floatation of tank. Hence first the required project is to be determined. Let it be x metres as shown in Fig. 13.11.

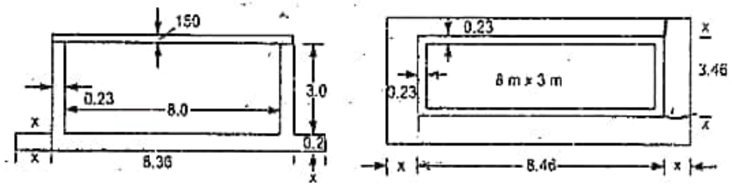


Fig. 13.11

Downward Loads

(a) Weight of top slab = $0.15 (8 + 2 \times 0.23)(3 + 2 \times 0.23) \times 25 = 109.77$ kN

(b) Weight of long walls = $2 \times 0.23 (8 + 2 \times 0.23) \times 3 \times 25 = 291.9$ kN

(c) Weight of short walls = $2 \times 0.23 \times 3 \times 3 \times 25 = 103.5$ kN

(d) Weight of bottom slab = $(8.46 + x)(3.46 + x) \times 0.2 \times 25 = 146.4 + 59.6x + 5x^2$

(e) Weight of soil on the projection of bottom slab

$$= [(8.46 + 2x)(3.46 + 2x) - 8.46 \times 3.46] \times 3 \times 16$$

$$= (23.84\,x + 4x^2) \times 48 = 1144.32\,x + 192\,x^2$$

Uplift force on bottom slab

$$= 31.36\,(8.46 + 2x)(3.46 + 2x)$$

$$= 917.96 + 747.62x + 125.44x^2$$

Equating upward force to total downward force, minimum x required can be obtained.

$$917.96 + 747.62x + 125.44x^2 = 109.77 + 291.9 + 103.5 + 146.4 + 59.6x + 5x^2 + 1144.32x + 192\,x^2$$

$$71.56x^2 + 456.3\,x - 266.39 = 0$$

$$x = \frac{-456.3 + \sqrt{456.3^2 + 4 \times 71.56 \times 266.39}}{2 \times 71.56} = 0.538 \text{ m}$$

Hence provide a projection of 0.6 m all around. The base slab is to be designed as one way slab.

The loads acting on this slab is shown in Fig. 13.12.

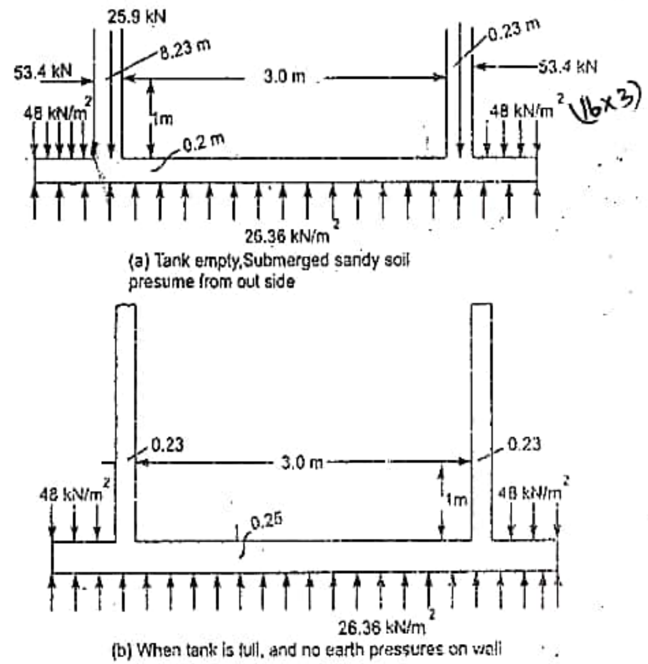


Fig. 13.12

Self weight of slab directly get transferred to soil. Hence upward pressure to be considered for bending moment calculation is

$$= 31.36 - 0.2 \times 1 \times 1 \times 25 = 26.36 \text{ kN/m}^2$$

Weight of soil on projected portion,

$$= 16 \times 3 = 48 \text{ kN/m}^2$$

Reaction on wall $= \boxed{\frac{1}{2} p_h \times 3} = \frac{1}{2} \times 35.6 \times 3 = 53.4$ kN acting at $\frac{3}{3} + 0.2 = 1.2$ m

∴ Cantilever moment at the face of the wall

$$= 26.36 \times \frac{0.6^2}{2} + 53.4 \times 1.2 - \frac{48 \times 0.6^2}{2} = 60.18 \text{ kN-m}$$

Moment at Centre of Slab

Load transferred by wall per meter length of base slab

$$= \text{weight of 1m long wall} + \frac{1}{2} \text{ weight of roof slab per meter length}$$

B L h L Tk

$$= 0.23 \times 1 \times 3 \times 25 + \frac{1}{2} \times (3.0 + 2 \times 0.23) \times 0.2 \times 25 = 25.9 \text{ kN}$$

∴ Moment at centre of slab

$$= 26.36 \times \frac{(3.46+1.2)}{2} + 53.4 \times 1.2 - 48 \times 0.6 \times \left(\frac{3.46}{2} + \frac{0.6}{2}\right) - 25.9\left(1.5 + \frac{0.23}{2}\right)$$

$= 25.2$ kN-m, producing tension at bottom.

Moment at centre of slab is critical when tank is full and there is no outside pressure. In this case weight of water directly gets transferred to soil without carrying flexure. Water pressure acting at $\frac{H}{3} = 1$ m from the base is

$$P = \frac{1}{2} \times (9.8 \times 3) \times \frac{3}{2} = 44.1 \text{ kN}$$

∴ Moment at centre of slab (see Fig. 13.12 *b*)

$$= 26.36 \times \frac{(3.46+1.2)}{2} - 44.1 \times 1.2 - 48 \times 0.6 \times \frac{3.46+0.6}{2}$$

$= -50.23$ kN-m

$= 50.23$ kN, carrying tension at top

∴ Thickness of slab required for balance section

$$= \sqrt{\frac{60.18 \times 10^6}{1.423 \times 1000}} = 205 \text{ mm}$$

Provide $d = 215$ mm and D = 250 mm

$$A_{st} = \frac{60.18 \times 10^6}{150 \times 0.872 \times 215} = 2139 \text{ mm}^2$$

Using 16 mm bars,

$$s = \frac{\frac{\pi}{4} \times 16^2}{2139} \times 1000 = 94 \text{ mm}$$

Provide 16 mm bars at 90 mm c/c near bottom face for the cantilever moment.

In the middle portion, reinforcement required at top is

$$A_{st} = \frac{50.23 \times 10^6}{150 \times 0.872 \times 215} = 1786 \text{ mm}^2$$

Continue cantilever reinforcement throughout *i.e.*, 16 mm bars at 90 mm c/c.

At bottom

$$A_{st} = \frac{25.2 \times 10^6}{150 \times 0.872 \times 215} = 896 \text{ mm}^2$$

Using 26 mm bars, spacing required is

$$s = \frac{\frac{\pi}{4} \times 12^2}{896} \times 1000 = 126 \text{ mm}$$

Provide 12 mm bars at 120 mm c/c.

Distribution Steel

% of steel $= 0.3 - \frac{250 - 225}{450 - 100} = 0.229$

∴ $A_{st} = \frac{0.229}{100} \times 250 \times 1000 = 571 \text{ mm}^2$

A_{st} on each face $= \frac{571}{2} = 286 \text{ mm}^2$

Using 8 mm bars

$$s = \frac{\frac{\pi}{4} \times 8^2}{286} \times 1000 = 175 \text{ mm}$$

Provide 8 mm bars at 170 mm c/c in longitudinal direction near both faces.

Reinforcement detail is shown in Fig. 13.13.

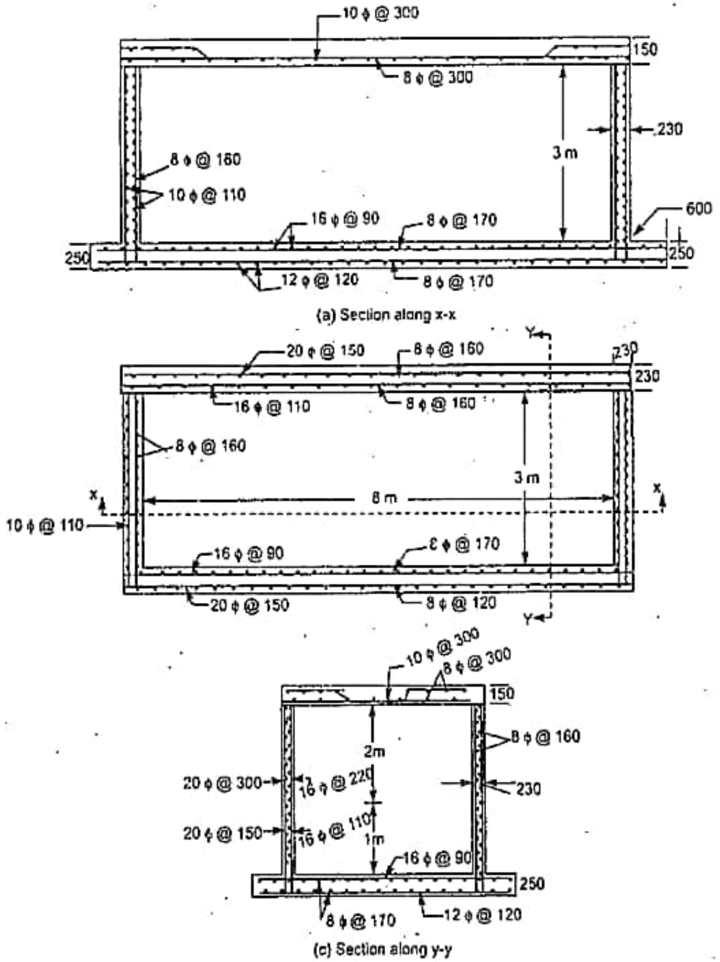


Fig. 13.13

13.8 OVERHEAD WATER TANKS

Various overhead tanks being used may be classified as

(a) Rectangular over head tanks,

(b) Circular overhead tanks, and

(c) Intz type tanks.

Rectangular overhead tanks are used for smaller capacities only, say 50,000 liters to 75, 000 liters. For larger capacities they become uneconomical. Circler overhead tanks are used to store water upto 7,50,000 liters. Their diameter varies from 5 to 15 m while height varies from 3 to 4.5 m. Intz tanks are used to store large quantity of water. Intz tanks of capacity one million liters are commonly used in water supplies in cities.

All overhead water tanks need top slab cover and also staging to support them. When top slab is provided, the top edge of tank wall may be treated as hinged. Walls are always monolithic with base slab. Hence walls may be treated as having edges fixed at base, and hinged at top.

In case of circular tanks, dome is preferred to top flat slab. Many times bottom flat slab is replaced by dome.

The exact analysis of over head tanks is not simple since all structural elements (top slab, walls bottom slab and beam supporting bottom slab) are built monolithic. The continuity analysis is required The attempt of Jai Krishna and O.P. Jain (Ref.1) for continuity analysis is note worthy. However since now a days finite Element Analysis packages are available one can think of using them to get better results. Approximate analysis based on assumed boundary conditions and membrane theories may be practiced, provided detailing is made to take care of edge disturbances in the form of edge moments.

In this book designs are made by approximate methods.

13.9 RECTANGULAR OVER HEAD WATER TANKS

Top slab may be designed by limit state method or by working stress method in which permissible stress in mild steel = 190 N/mm^2 and for Fe-415, σ_{st} = 230 N/mm^2. Live load on tank may be taken as 2 kN/m^2.

The walls may be designed by approximate method as discussed in this chapter earlier or one can make use of moment shear coefficients given in IS: 3370 (Part IV) (reaffirmed in 1999).

Base slab is heavily loaded when tank is full. Hence it is designed for the water pressure when tank is full, taking edges as fixed. The base slab is supported along its edges by wall or beams and some time additional beams may be there in the middle also. Beams are supported by columns of the staging

For the design of tank and base slab working stress method with reduced values of permissible stresses in steel should be used, since in these elements crack widths are to be kept minimum to avoid leakage problem.

13.10 CIRCULAR OVER HEAD WATER TANKS

As stated earlier circular water tanks are preferred upto 750,000 litres capacity. They are usually provided with dome as top cover. The investigations of author[2] for optimum design of such dome has shown that the rise of spherical dome may be kept as $\frac{1}{7}$th of diameter. Referring to Fig. 13.14

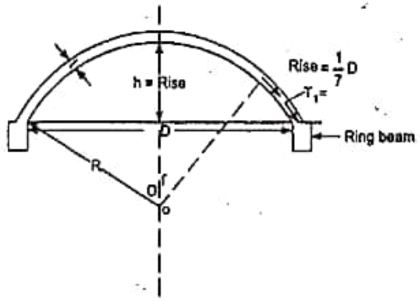


Fig. 13.14

D = Diameter of Dome

R = Radius of curvature dome

h = Rise of dome, $\frac{1}{7}$th D.

t = Thickness of dome usually minimum of 75 mm and maximum of 100 mm. The load on the dome is self weight plus live load. Self weight may be found by assuming thickness of 75 mm. Live load may be assumed about 1.5 kN/m^2. Finishing load may be added to get total load acting per unit area of surface. If 'w' is load on the surface per unit area, membrane theory of shells give the following expression:

Meridional thrust $\quad T_1 = \dfrac{wR}{1+\cos\phi}$, per unit length

Circumferential force $\quad T_2 = wR\left(\cos\varphi - \dfrac{1}{1+\cos\varphi}\right)$, per unit length

Maximum values of above forces occur when $\phi = \theta$, *i.e.*, at junction with top ring beam.

The reinforcement is provided in meridional and circumferential directions.

The dome rests on top ring beam. Top ring beam is subjected to load from meridional thrust T_1. Hence hoop tension in top ring beam is given by

$$T_1 \frac{\cos\theta D}{2}$$

Tensile stress in concrete should not exceed the values given in Table 13.1 for direct tension. Based on this, size of ring beam may be determined.

Cylindrical wall may be designed for cantilever action in lower potion and hoop action in upper potion. Because of continuity with slab, the lower edge cannot be treaded as fixed. Along with the slab cylindrical wall also rotates. This results into decrease in cantilever moment and increase in the depth of tank in hoop action. The exact analysis involves cylindrical shell analysis of wall and plate analysis of slab to find the rotations and ensure same values to get continuity. As indicated earlier one can think of finite element analysis of the tank. In approximate method one can design lower potion h (H/3 or 1 m, whichever is more) for cantilever action and consider entire depth H for hoop action.

Hence maximum tension in wall

$$= \frac{\gamma H D}{2} \text{ and cantilever moment is } \frac{1}{2}\gamma H h.\frac{h}{2}.$$

Reinforcement is to be provided for the above forces on appropriate side. Provide minimum reinforcement on the other side. To ensure the continuity in actual structure bars on inner face of tank should be looped at corner and anchorage length is ensured.

Design of base slab depends on how it is supported. Fig. 13.15 shows different methods of supporting base slab.

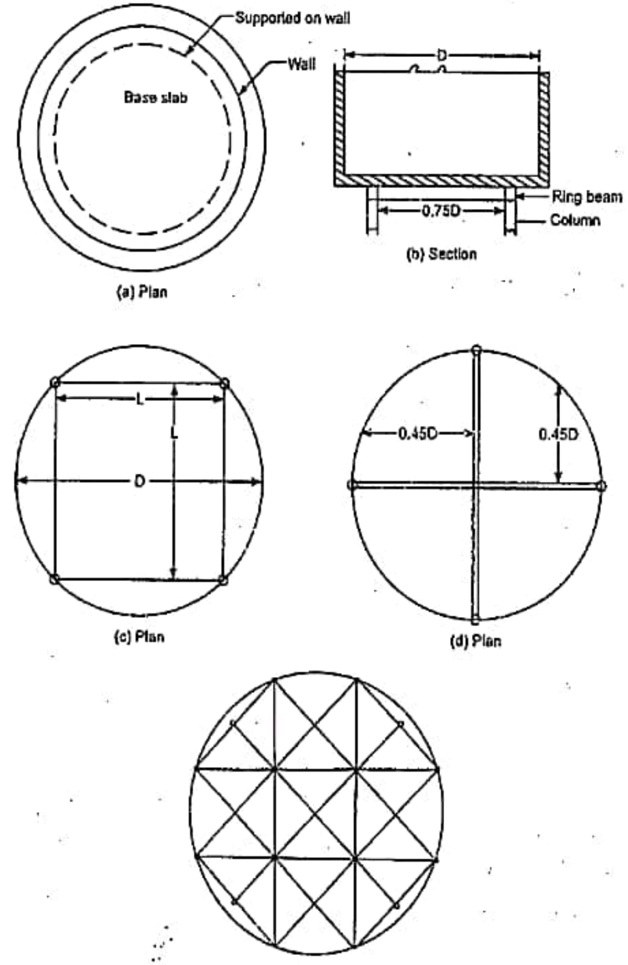


Fig. 13.15

(*a*) **Tank supported on wall :** In smaller diameter tank required at lesser heights, circular walls may be built to support the tank along its periphery. In such cases flat bottom slab is designed as a circular plate subjected to water pressure and self weight. The end condition may be assumed as simply supported. From circular plate theory we know radial and circumferential moments are given by the expressions

$$M_r = \frac{q}{16}(3+\mu)\left(a^2 - r^2\right)$$

and $$M_\theta = q\frac{a^2}{16}(3+\mu) - \frac{r^2}{16}(1+3\mu)$$

where q = Load per unit area = γ_w H + self weight

a = Radius of bottom slab

μ = Poissons ratio

r = Radial distance where values are required.

Radial and circumferential reinforcement may be designed. Tank wall is provided with arbitrarily steel to act as beam to support its own weight. The additional steel is provided at top and bottom of tank wall.

(*b*) **Tank supported on ring beam:** In case of larger tanks it is economical to support circular base slab with a ring beam of diameter 0.75 of the diameter of tank as shown in Fig. 13.14 (*b*). The ring beam is supported by a number of columns spaced at regular intervals. If ϕ is the angle subtended by the arc between any two consecutive columns at the centre of ring beam, then

$$\varphi = \frac{360}{n}$$

where n is number of columns supporting ring beam.

If 'w' is the load per unit run of beam, the shear force at support is

$$F_v = \frac{wR\phi}{2}$$

Let support moment $= k\,w\,R^2\,\phi$

Mid-span moment $= k'\,w\,R^2\,\phi$

and

Maximum torsional moment $= k''\,w\,R^2\,\phi$

α = The angle at which maximum twisting moment occurs.

Then structural analysis (Ref. Structural Analysis, Vol. II by the author) gives the following values of k, k', k'' and α for various number of columns used to support ring beam.

Table 13.1 Coefficient for Bending Moment, Torsional Moments and Location of Point of Maximum Torsion in Ring Beams

No. of Column Supports	ϕ (Degrees)	k	k'	k''	α for Maximum Torsion
4	90	0.137	0.070	0.021	19.25
5	72	0.108	0.054	0.150	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	36	0.045	0.017	0.002	6.25

It is to be noted that the section at which torque is maximum bending moment is zero and at support there is no torsional moment.

In such case, slab may be analyzed by plate theory. Fig. 13.16 shows load on slab which consists of total weight of dome, top ring beam and wall transferred at the edge of base slab and uniformly distributed load of γ_w H plus self weight. The slab is resting on ring beam of radius b. The total load on ring beam from slab may be found which consists of total load on slab. Let it be W. Then slab is analyzed as

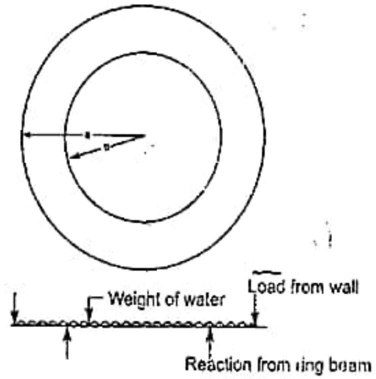


Fig. 13.16

(*i*) Circular slab simply supported at outer periphery by walls and subjected to γ_w H plus self weight.

(*ii*) Circular slab simply supported at outer periphery by walls and subjected to total ring load W at a concentric circle of radius b.

In plate theory the expressions for moment for the above two cases are given below:

Case I:

$$M_r = \frac{q}{16}(3+\mu)\left(a^2 - r^2\right)$$

$$M_\theta = \frac{qa^2}{16}(3+\mu) - \frac{qr^2}{16}(1+3\mu)$$

as given earlier. For concrete value of μ is small and many designer takes it as zero. Then,

$$M_r = \frac{3q}{16}(a^2 - r^2)$$

and
$$M_\theta = \frac{3qa^2}{16} - \frac{qr^2}{16}$$

For case II:

For $r < b$

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

For $r > b$

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

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

Note that in this case $M_r = 0$ at $r = a$.

(*c*) **Slab supported on four beams as shown in Fig. 13.15 (*c*)**

(*i*) The slab between the beams has size

$L = 2a\sin 45° = \sqrt{2}\,a.$

The square slab of size $\sqrt{2}\,a \times \sqrt{2}\,a$ is designed as a two way slab with corners held down. Sufficient reinforcement is provided in the beams to take up negative moment.

(*ii*) If W is the total load of water and self weight of slab, each beam carries a load of $\frac{W}{4}$. The load is triangular in shape with maximum ordinate at mid span of beam. Hence maximum moment in the beam is $\frac{W}{4} \times \frac{L}{6} = \frac{WL}{24}$. The beam is designed as a T-beam.

(*iii*) The tank wall needs additional reinforcement so as to act as a beam to support own weight. This is achieved by arbitrarily providing additional horizontal (hoop) reinforcement at top and bottom of tank wall.

(*d*) **Slab supported by two crossed beams (Fig. 13.15 *d*):** In this case slab may be designed as a two way reinforced slab of span equal to 0.45 times the diameter.

The beam is designed to carry half the total load. The span of the beam is equal to diameter of the slab and it acts as a T-beam.

The tank wall acts as a curved beam and hence needs additional steel at top and bottom of tank.

(*e*) **Slab supported on a number of beams. (Fig. 13.15 *e*):** If the base slab diameter is 12 to 15 m slab needs support from several beams. Common arrangement of beams is shown in Fig. (13.15. *e*). Each panel of slab between the beams is designed as continuous slab. The beams are designed as continuous beams subjected to triangular loading. The tank wall is provided with additional steel to act as a beam.

Example 13.6: *Design a flat bottom circular elevated water tank of diameter 10 m and total height 4 m which is to be supported by ring beam of 7.5 m diameter. The ring beam is to be supported by six columns equally placed. Use M25 concrete and Fe-415 steel. Design the following components of water tank*

(a) Top dome
(b) Top ring beam
(c) Cylindrical wall
(d) Bottom slab
(e) Bottom ring beam.

Solution:

Diameter of tank = 10 m $\quad\therefore$ Radius $a = 5$ m

H = 4 m.

Diameter of bottom ring beam = 7.5 m $\quad\therefore$ Radius $b = 3.75$ m

Concrete Mix: M 25 $\quad$ Steel: Fe-415

Design of Top Dome

Referring to Fig. 13.14,

D = 10 m $\quad$ Rise = $h = \frac{1}{7}$ D, say $h = 1.5$ m.

R = radius of dome

Then $\quad (2R - h)h = \left(\frac{D}{2}\right)^2$

$\therefore \quad (2R - 1.5)\,1.5 = 5^2$

$$R = \frac{5^2 + 1.5^2}{2 \times 1.5} = 9.083 \text{ m.}$$

$\therefore$ Semi central angle $\quad \theta = \cos^{-1}\left(\frac{R-h}{R}\right) = \cos^{-1}\frac{9.083 - 1.5}{9.083} = 33.4°.$

Assuming thickness of dome 75 mm,

Self weight of dome $= 0.075 \times 1 \times 1 \times 25 = 1.875$ kN/m²

Live load $= 1.5$ kN/m²

Finishing load $= 0.5$ kN/m²

Total $\quad w = 3.875$ kN/m²

$\therefore$ Max. meridional thrust $T_1 = \frac{wR}{1+\cos\theta} = \frac{3.825 \times 9.083}{1 + \cos 33.4} = 19.18$ kN/m.

Maximum circumferential force

$$T_2 = wR\left(\cos\theta - \frac{1}{1+\cos\theta}\right)$$

$$= 3.875 \times 9.083\left(\cos 33.4 - \frac{1}{1+\cos 33.4}\right)$$

$$= 10.202 \text{ kN/m.}$$

$\therefore$ Maximum stress $= \frac{19.18 \times 1000}{1000 \times 75} = 0.256 \text{ N/mm}^2$

Permissible stress in M25 concrete in compression = 6 N/mm^2. Hence safe.

$\therefore$ Provide only nominal reinforcement of 8 mm dia at 180 mm c/c in both circumferential and meridional directions.

Design of Top Ring Beam

Hoop Tension $= T_1 \cos\theta \frac{D}{2} = 19.18 \cos 33.4 \times \frac{10}{2} = 80.062$ kN.

$\therefore$ $A_{st} = \frac{80.062 \times 1000}{150} = 533 \text{ mm}^2$

Provide 6 bars of 12 mm.

A_{st} provide $= 6 \times \frac{\pi}{4} \times 12^2 = 678 \text{ mm}^2$

m = modular ratio $= \frac{280}{3 \times 8.5} = 11$

$\therefore$ Area of concrete required is given by

$$\frac{80.062 \times 1000}{A_c + 11 \times 678} = 1.3$$

$\therefore$ $A_c = 54122 \text{ mm}^2$.

Provide 250 mm $\times$ 300 mm top ring beam with 6 bars of 12 mm main reinforcement. Nominal stirrups of 6 mm at 225 mm c/c are to be provided in the beam.

Design of Tank Wall

Depth of water tank = 4 m

and diameter of water tank = 10 m

$\therefore$ Maximum hoop tension in the wall

$$= \frac{\gamma h D}{2} = 9.8 \times 4 \times \frac{10}{2} = 196 \text{ kN/m}$$

$\therefore$ $A_{st} = \frac{196 \times 1000}{150} = 1306 \text{ mm}^2$

A_{st} on each face $= \frac{1306}{2} = 653$

Using 12 mm bars spacing required is

$$\frac{\frac{\pi}{4} \times 12^2}{653} \times 1000 = 173 \text{ mm.}$$

Provide 12 mm bars at 170 mm c/c near base, on each face. It may be gradually increased to 30[0] mm spacing at $\frac{173 \times 4}{300} = 2.3$ m below the top. In the top 2.3 m maintain 300 mm spacing.

$\therefore$ A_{st} provided at base $= \frac{\frac{\pi}{4} \times 12^2}{170} \times 1000 = 665 \text{ mm}^2$

Let thickness of wall be t. Then to keep direct compression in wall within limiting value

$$\frac{196 \times 1000}{1000t + 11 \times 665} = 1.3$$

$$t = 188.7 \text{ mm.}$$

Provide 200 mm thickness.

Vertical Steel

Bottom $\frac{4}{3} = 1.333$ m is under cantilever moment

Cantilever moment $= \frac{\gamma H h^2}{6} = \frac{9.8 \times 4 \times 1.333^2}{6} = 11.61$ kN-m

For M25 concrete and Fe-415 steel.

$\sigma_{cbc} = 8.5 \text{ N/mm}^2$ $m = 11$ $\sigma_{st} = 150$

$\therefore$ $n = 0.384$ $j = 0.872$ and $K = 1.423$.

Effective depth $d = 200 - 35 = 165$

$\therefore$ $A_{st} = \frac{11.61 \times 10^6}{150 \times 0.872 \times 165} = 538 \text{ mm}^2$

Using 10 mm bars $s = \frac{\frac{\pi}{4} \times 10^2}{538} \times 1000 = 145$ mm.

Minimum steel to be provided in vertical direction

$$A_{st}, \min = \frac{0.3}{100} \times 200 \times 1000 = 600 \text{ mm}^2$$

∴ Minimum steel on each face = 300 mm^2

Using 10 mm bars $\quad s = \dfrac{\frac{\pi}{4}\times 10^2}{300}\times 1000 = 261$ mm.

Hence provide 10 mm bars at 130 mm c/c in the lower 1.3 m on inner face. Curtail alternate bars. On outer face provide 10 mm bars at 260 mm c/c.

Design of Base Slab

Total load from dome $= T_1 \sin\theta \times 2\pi \dfrac{D}{2}$

$= 19.18 \sin 33.4 \times 2\pi \times 5 = 331.7$ kN

Weight of ring beam $= 0.25 \times 0.30 \times 2\pi \times 5 \times 25 = 58.90$ kN

Weight of wall $= 0.20 \times (4 - 0.3) \times 2\pi \times 5.2 \times 25 = 604.4$ kN

Total weight $= 995$ kN

Weight of water $= \gamma H\pi \dfrac{\pi D^2}{4} = 9.8 \times 4 \times \pi \times \dfrac{10^2}{4}$ 3078.8 kN

On edge of slab

Self-weight of slab: Assuming slab thickness

$t = \dfrac{D}{35} = 0.29$ m, say 300 mm.

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

∴ Total self-weight (Note, total slab diameter = 10 + 2 × 0.2 = 10.4 m)

$= 7.5 \times \dfrac{\pi}{4} \times 10.4^2 = 637.1$ kN.

Finishing load $= 0.6 \times \dfrac{\pi}{4} \times 10^2 = 47.1$ kN.

∴ Total downward load $= 995 + 3078.8 + 637.1 + 47.1$

$= 4758$ kN

∴ Total upward force from ring beam = 4758 kN.

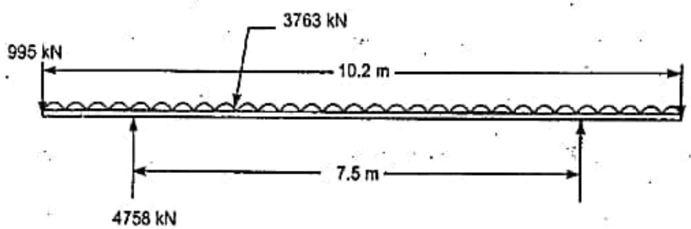


Fig. 13.17 *Load on Bank slab*

Fig.13.17 shows free body diagram of base slab in which total loads are indicated.

Now the slab may be treated as freely supported by walls and subjected to

(*i*) Uniformly distributed downward load of $q = \dfrac{3763}{\frac{\pi}{4}\times 10.2^2} = 46.05$ kN/m^2.

(*ii*) Upward ring load of W = 4758 kN.

For case (*i*) loading

$$M_r = \frac{3q}{16}\left(a^2 - r^2\right) \text{ and } M_\theta = \frac{3qa^2}{16} - \frac{qr^2}{16}$$

where $a = \dfrac{10.2}{2} = 5.1$ m

∴ Moments at critical points are as listed below:

r in m	0	1.875	3.75	5.1
M_r in kN-m	224.6	194.2	103.2	0
M_θ in kN-m	224.6	214.5	184.1	149.7

In Case II,

For $r < 3.75$

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

For $r > 3.75$

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

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

Noting that $a = 5.1$ m and $b = 3.75$,

Moment at critical points are as listed below:

(Note: W upward. Hence may be taken as –ve)

r in m	0	1.875	3.75	5.1
M_r in kN-m	–201.4	–201.4	–201.4	0
M_θ in kN-m	–201.4	–201.4	–201.4	–99.23

∴ Net moment in the slab is as given below:

r in m	0	1.875	3.75	5.1
M_r in kN-m	+23.2	–7.2	–98.2	0
M_θ in kN-m	23.2	13.1	–17.3	50.47

Design moment = 98.2 kN-m

$$d = \sqrt{\frac{98.2 \times 10^6}{1.423 \times 1000}} = 262.7 \text{ mm}$$

∴ Provide d = 265 mm and t = 300 mm

$$A_{st} = \frac{97.2 \times 10^6}{150 \times 0.872 \times 265} = 2804 \text{ mm.}$$

Using 25 mm bars

$$s = \frac{\frac{\pi}{4} \times 25^2}{2804} \times 1000 = 175 \text{ mm.}$$

∴ Provide 25 mm bars at 175 mm c/c. This is required at top of the slab in radial direction. At the edges

$$M_\theta = 50.47 \text{ kN-m, hogging, } d = 265 - 25 = 240 \text{ mm}$$

$$A_{st} = \frac{50.47 \times 10^6}{150 \times 0.872 \times 240} = 1654 \text{ mm}^2$$

Using 20 mm bars

$$s = \frac{\frac{\pi}{4} \times 20^2}{1654} \times 1000 = 189 \text{ mm.}$$

Provide 20 mm bars at 175 mm c/c at top of slab in circumferential direction at the outer edges of slab.

In the central portion of about 1.8 m, sagging moment exist. Maximum sagging moment is 25.5 kN-m

$$A_{st} = \frac{25.5 \times 10^6}{150 \times 0.872 \times 240} = 812 \text{ mm}^2$$

Provide a mesh of 20 mm bars at 300 mm c/c in two mutually perpendicular directions near bottom face of the slab. Size of this mesh may be kept 2 m × 2 m since actual length required is 12 × ϕ more than required.

Design of Bottom Ring Beam

Radius = 3.75 m

Total load on it from slab = 4758 kN

$$\therefore \text{Load per meter run} = \frac{4758}{2 \times \pi \times 3.75} = 202 \text{ kN}$$

Since it is subjected to torsion, let as use wider beam, say 350 mm wide. Taking depth of beam approximately $\frac{1}{15}$th of diameter, D = 600 mm.

∴ Self-weight of beam = 350 × 0.600 × 25 = 5.25 kN/m

With finishing, say 6 kN/m. Then load on ring beam W = 202 + 6 = 208 kN/m.

Number of columns supporting beam n = 6.

$$\therefore \quad \phi = \frac{360}{6} = 60° = \frac{\pi}{3} \text{ radians}$$

$$\therefore \text{ Maximum shear at support } = \frac{W R \varphi}{2} = \frac{208 \times 3.75 \times \frac{\pi}{3}}{6} = 408 \text{ kN.}$$

$$\text{Support moment} = k\,W\,R^2\,\varphi = 0.089 \times 208 \times 3.75^2 \times \frac{\pi}{3} = 272.6 \text{ kN-m.}$$

$$\text{Mid-span moment} = k'\,W\,R^2\,\varphi = 0.045 \times 207 \times 3.75^2 \times \frac{\pi}{3} = 137.8 \text{ kN-m.}$$

$$\text{Maximum torsional moment} = k''\,W\,R^2\,\varphi = 0.009 \times 207 \times 3.75^2 \times \frac{\pi}{3} = 27.68 \text{ kN-m.}$$

It occurs at α = 12.75° with radius joining the column position.

Let us use limit state method for design and make use of SP 16 for design.

Keeping effective cover of 50 mm. d = 550 m

$$\frac{d'}{d} = 0.1$$

$$\frac{M_u}{bd^2} = \frac{1.5 \times 272.6 \times 10^6}{350 \times 550^2} = 3.86$$

∴ Referring to Table 51 in SP – 16,

$$p_t = 1.333 \quad \text{and} \quad p_c = 0.146$$

$$\therefore \quad A_{st} = \frac{1.333 \times 350 \times 550}{100} = 2566 \text{ mm}^2 \qquad A_{sc} = \frac{0.146}{100} \times 350 \times 550 = 281 \text{ mm}^2.$$

Provide 8 bars of 20 mm as tensile steel and 2 bars of 20 mm as compression steel.

A_{st} provided = 2875 mm², at top near support

A_{sc} provided = 628 mm², at bottom near support

At mid-span moment is almost half of that at support. Hence provide 4 bars at mid span as tensile reinforcement.

Check for torsion at $\alpha = 12.75° = 0.2225$ radians

$\therefore$ It's distance from support $= 3.75 \times 0.2225 = 0.835$ m

Torsional moment $T = 27.6$ kN-m

$\therefore$ $T_u = 1.5 \times 27.6$ kN-m.

Bending moment $= 272.6 - 208 \times \dfrac{0.835^2}{2} = 200$ kN-m.

$M_u = 300$ kN-m.

$\therefore$ $M_e = 300 + 1.5 \times 27.6 \times \dfrac{1 + \frac{600}{350}}{1.7} = 366.1$ kN-m.

At support $M_u = 1.5 \times 272.6 = 408.9$ kN-m.

Hence the reinforcement provided at support may be continued to take care of this section also.

Shear reinforcement:

$V = 408$ kN

$V_u = 609$ kN

$\therefore$ $\tau_v = \dfrac{609 \times 1000}{350 \times 550} = 3.16$ N/mm^2

> 3.1 N/mm^2

$\therefore$ Increase the section. Let $b = 400$ mm.

A_{st} provide 2875 mm^2.

$p = \dfrac{2875 \times 100}{400 \times 550} = 1.309$ N/mm^2

$\therefore$ $\tau_c = 0.70$ N/mm^2

$V_{us} = V_u - \tau_c bd = 609 \times 1000 - 0.70 \times 400 \times 550 = 455000$ N

Using 2 legged 12 mm stirrup

$$s_v = \frac{0.87 f_y A_{sv} d}{V_{us}} = \frac{0.87 \times 415 \times 2 \times \frac{\pi}{4} \times 12^2 \times 550}{455000} = 98.7 \text{ mm.}$$

Provide 12 mm 2 legged stirrups at 95 mm c/c near support. Shear reduces by W = 208 kN per metre length. Hence increase the spacing to 160 mm after 1 m *i.e.*, provide 2 legged 12 mm stirrups at 160 mm c/c in the middle half potion.

Side face reinforcement $= \dfrac{0.1}{100} \times 400550 = 229.8$ mm^2.

Provide one bar of 16 mm at mid-depth on both faces. Reinforcement details are shown in Fig. 13.18.

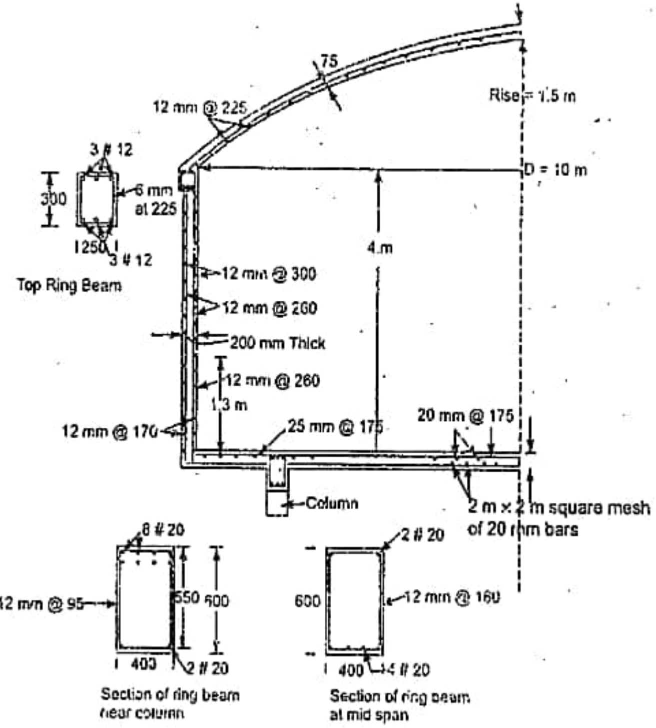


Fig. 13.18 *Details of reinforcement*

13.11 INTZ TANK

For larger capacity of over head tanks, flat bottom circular tanks become uneconomical, since thickness of slab required increase considerably. In such cases Intz tanks are more economical. A typical intz tank is shown in Fig. 13.19.

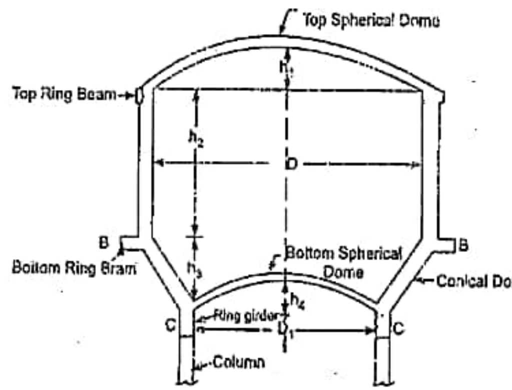


Fig. 13.19 *Typical intz tank*

UNIT-3 SELECTED TOPICS

→ Staircases

→ Flat slabs

→ principles of design of mat foundation box culvert and road bridges.

STAIRCASES:

Staircases are generally provided connecting successive floors of a building & in small buildings. They are the only means of access between the floors.

The staircase comprises of flight of ~~staircas~~ generally with one or more intermediate landings provided between the floor levels.

The structural components of a flight of stairs comprises of the following elements

1. Tread: The horizontal portion of a step where the foot rests is referred as tread. The tread is usually 250 to 300mm wide depending upon the type of building.
2. Riser: Riser is the vertical distance ~~of the step~~ adjacent tread (or) vertical projection of the step, generally in the range of 150 to 190mm depending upon the type of building.
3. Going: Going is the horizontal projection (plan) of an inclined

flight of steps between first & last riser. A flight comprises of two landings & one going with 10 to 12 steps.

1. Width of stairs varies in the range of 1 to 1.5m with a minimum value of 850 mm or not less than 850mm. public buildings should be provided with larger widths to facilitate free passage of users and prevent overcrowding.

Types of staircases:

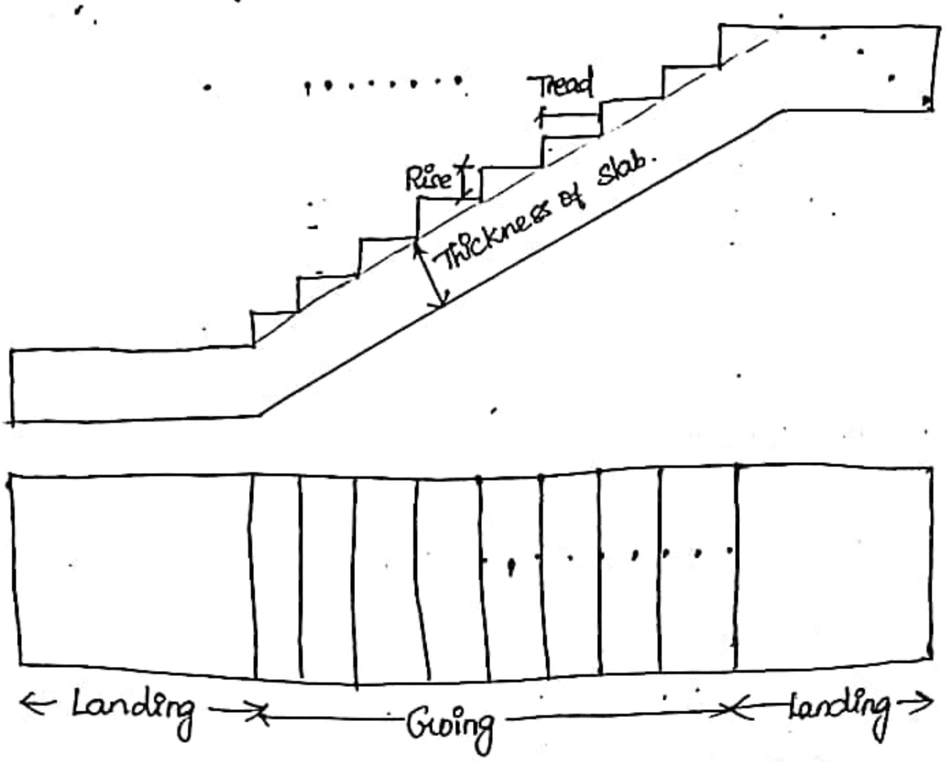
1. Dog-legged staircases: It is the most common type of staircase used in all types of buildings. It comprises of two adjacent flights running parallel with a landing slab at midheight. Where space is at a premium, It is generally adopted resulting in economical utilisation of available space.
2. Open-well staircase: It is generally adopted in public buildings where large spaces are available. This type of staircase consists of smaller flights and provides better accessibility, comfort & ventilation due to open well at the centre.
3. Tread-Riser staircase: It is very popular due to its aesthetic appearance & comprising only the horizontal & vertical slabs in the form of a folded plate.
4. Isolated cantilever staircase: It comprises only the horizontal tread slab projecting from a wall or inclined beam serving as a fixed end with open risers.
5. Spiral staircase: In congested locations, where space available is small, spiral stairs are ideally suited. It comprises a

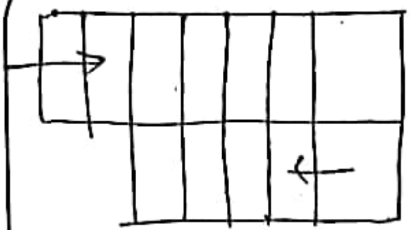
Central post with precast slab tread anchored to the central column. It is not userfriendly due to the reduced tread width near the post and is suitable only for single person to use the staircase at a time.

6. Helicoidal staircase: It is generally used in the entrance foyer of cinema theatres & shopping malls to connect the ground & first floors.

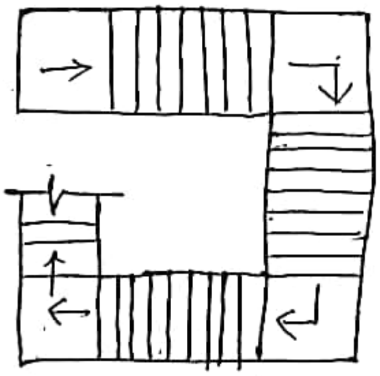
7. Quarter turn staircase: It is used in domestic houses where floor heights are limited to 3m.

8. Single flight staircase: It is used in cellars where the height between floors is small.

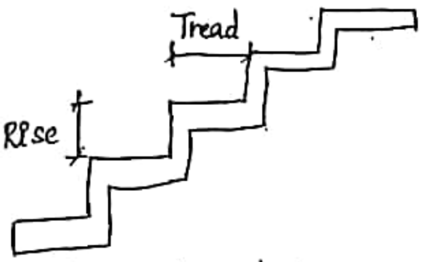




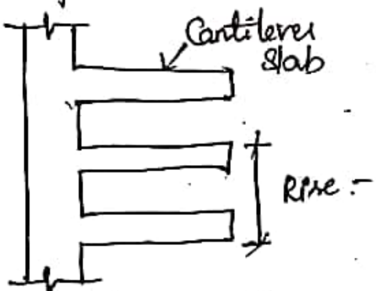
Dog-legged staircase.



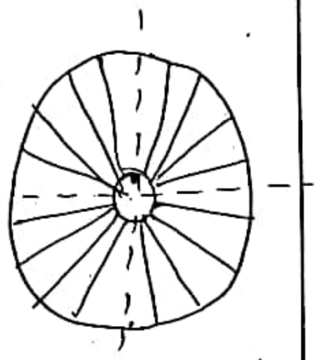
Open well staircase



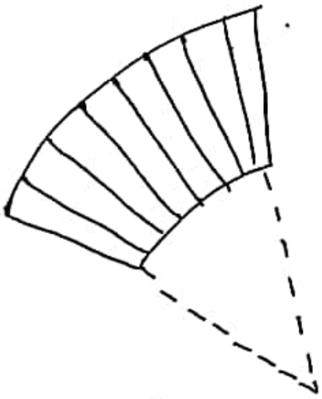
Tread-Rise staircase.



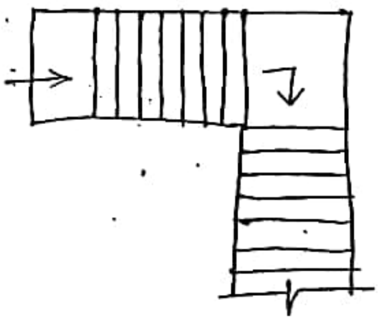
Isolated cantilever staircase.



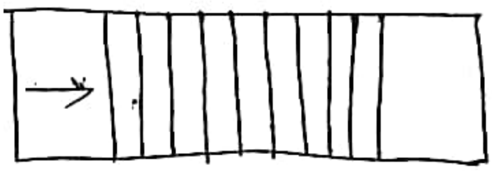
spiral staircase



Helicoidal staircase.



Quarter turn staircase.



Single flight staircase.

Loads on staircase:

The various types of loads to be considered in the design of staircase are:

(a) Dead load which include the self weight of ~~staircase~~ waist slab, tread and riser including self wt of finishes.

b) Live load are considered as specified in IS 875-1987(part II)

Residential buildings - 2 to 3 kN/m^2

public buildings - 5 kN/m^2.

Thickness of waist slab $= \frac{L}{20}$ for simply supported

$= \frac{L}{25}$ for continuous.

Dead load of slab on horizontal span, $w = \frac{w_s\sqrt{R^2+T^2}}{T}$.

Problem:

1. Design a flight of stairs for a school building spanning between landing beam to suit the following data:

No. of steps = 12

Tread = 300 mm

Riser = 160 mm.

Width of landing beam = 400 mm.

M25 grade & Fe415 steel.

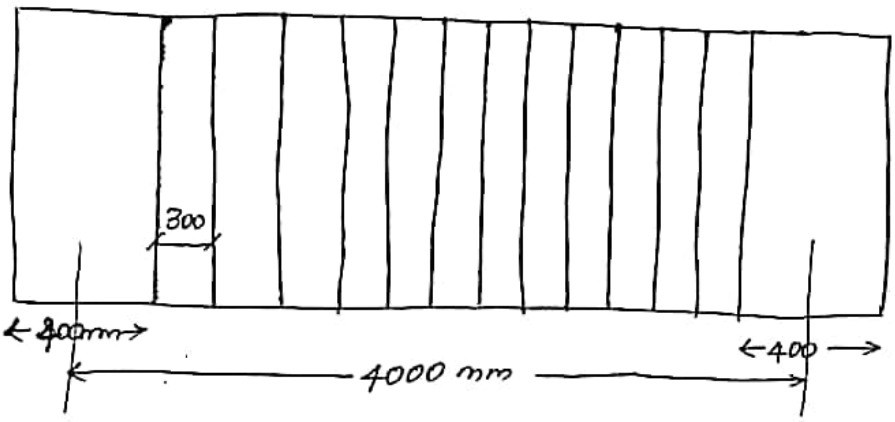
Given data:

Soln:

Step 1: Effective span:

→ C/c distance btwn landing beam.

$$\text{Eff. span} = (12 \times 300) + 400 = 4000 \text{ mm}$$



$$\text{Thickness of waist slab} = \frac{\text{span}}{20} = \frac{4000}{20} = 200 \text{ mm}.$$

$$D = 200 \text{ mm}.$$

cover = 20 mm, $d = 200 - 20 = 180$ mm.

Step 2: Load calculation:

Dead load of slab, $w_s = 0.2 \times 1 \times 25 = 5$ KN/m.

Dead load of slab on horizontal span, $w = \frac{w_s\sqrt{R^2+T^2}}{T}$

$$= \frac{5\sqrt{160^2+300^2}}{300}$$

$$= 5.67 \text{ KN/m}$$

Dead load of one step = $\frac{1}{2} \times 0.16 \times 0.3 \times 25 = 0.6$ KN/m

Dead load of steps/m length = $\frac{0.6 \times 1000}{300} = 2$ KN/m.

Load due to finishes = 0.5 KN/m.

Total DL = $5.67 + 0.5 + 2$

= 8.27 KN/m.

LL = $5 \times 1 = 5$ KN/m.

Total Load = $8.17 + 5 = 13.17$ KN/m.

Factored load = 13.17×1.5

$W_u = 19.76$ KN/m

Step 3: Determination of BM:

$$M_u = \frac{W_u L^2}{8} = \frac{19.76 \times 4^2}{8}$$

$= 39.52$ KNm.

Step 4: Check for depth:

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

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}} = \sqrt{\frac{39.52 \times 10^6}{0.138 \times 25 \times 1000}}$$

$= 107.03$ mm $<$ 180 mm.

Step 5: Reinforcement Details:

Main Reinf: $M_u = 0.87 f_y A_{st} d \left[1 - \frac{f_y A_{st}}{b d f_{ck}}\right]$

$$39.52 \times 10^6 = 0.87 \times 415 \times A_{st} \times 180 \left[1 - \frac{415 A_{st}}{1000 \times 180 \times 25}\right]$$

$$608.1 = A_{st} - (9.22 \times 10^{-5}) A_{st}^2$$

$$A_{st} = 646.65 \ mm^2.$$

12mm ϕ bars $\Rightarrow a_{st} = 113.1 \ mm^2$.

$$\text{Spacing} = \frac{113.1}{646.65} \times 1000 = 174.89 \simeq 170 \ mm.$$

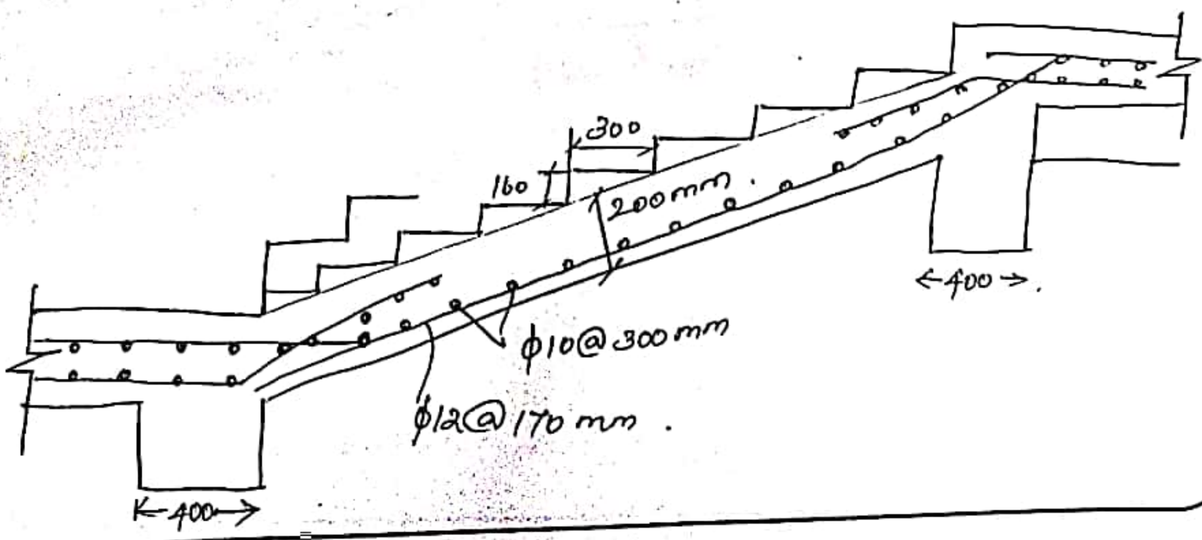
Distribution Reinf: $A_{st} = 0.12\% \ bD$

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

$$= 240 \ mm^2.$$

10mm ϕ bars $\Rightarrow a_{st} = 78.54 \ mm^2$.

$$\text{Spacing} = \frac{78.54}{240} = 327.25 \simeq 300 \ mm$$



2. Design a dog-legged staircase having a waist slab for an office building for the following data:

Height btwn floor = 3.2 m

Riser = 160 mm

Tread = 270 mm.

Width of landing = 1.25 m.

LL = 5 kN/m².

FL = 0.6 kN/m².

Assume the stairs to be supported on 230 mm tk masonry walls at the outer edge of landing llel to the riser. Use M20 concrete & Fe415 grade steel.

Given data:

Soln:

Note: Based on riser, the no. of steps is found. Based on tread, the length of staircase is found.

Step 1: Eff. span:

$$\text{No. of steps} = \frac{3.2}{0.16} = 20.$$

So, provide 2 flights of 10 steps each.

$$\text{Eff. span} = \frac{230}{2} + 1250 + (9 \times 270) + 1250 + \frac{230}{2}$$

$$= 5160\text{ mm} = 5.16\text{ m}.$$

Thickness of waist slab = $\frac{\text{Span}}{20}$ $\left[\frac{l}{d} = 20\right]$

$$D = \frac{5.16}{20} = 0.258 = 258\ mm \simeq 260\ mm$$

Cover = 20 mm

$d = 260 - 20 = 240$ mm.

Step 2: Load calculation:

Load on going:

$$w_s = 0.26 \times 1 \times 25 = 6.5\ kN/m$$

$$w = \frac{w_s\sqrt{R^2+T^2}}{T} = \frac{6.5\sqrt{160^2+270^2}}{270}$$

$$= 7.56\ kN/m.$$

DL of one step = $\frac{1}{2} \times 0.16 \times 0.27 \times 25$

$$= 0.54\ kN/m$$

DL of step per m = $\frac{0.54 \times 1000}{270}$

$$= 2\ kN/m$$

Floor finish = 0.6 kN/m.

LL = 5 kN/m.

Total Load = $5 + 0.6 + 2 + 7.56$.

$$= 15.16\ kN/m$$

Load on Landing slab:

Self wt of slab $= 0.26 \times 1 \times 25$

$= 6.5\ kN/m$

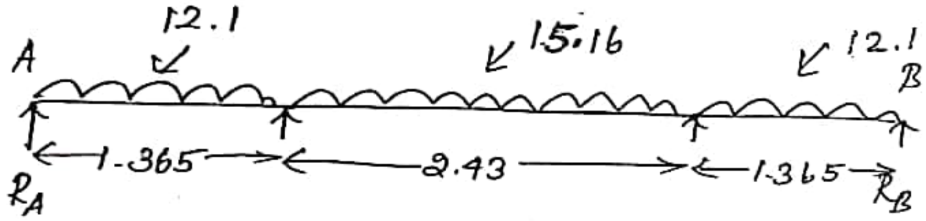
$FL = 0.6\ kN/m.$

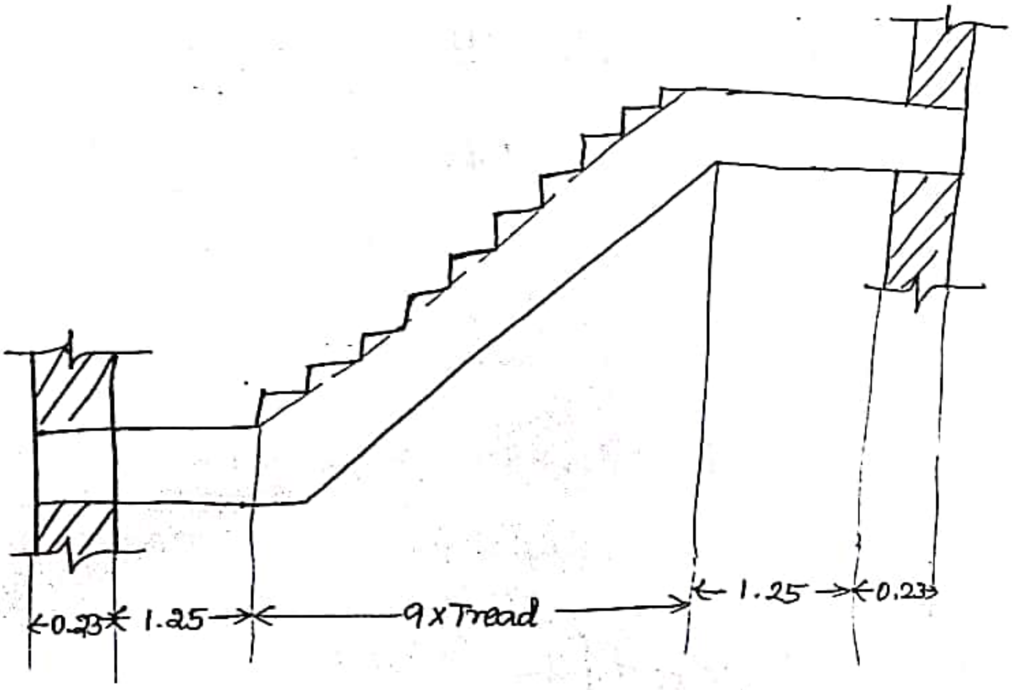
$LL = 5\ kN/m.$

Total load $= 12.1\ kN/m.$

~~Span Calculation~~.

Step 3: Determination of BM:





$$R_A + R_B = (12.1 \times 1.365 \times 2) + (15.16 \times 2.43)$$

$$R_A + R_B = 69.87$$

Taking moment abt B,

$$(R_A \times 5.16) = (12.1 \times 1.365)\left(\frac{1.365}{2} + 2.43 + 1.365\right)$$
73.95

$$+ (15.16 \times 2.43)\left(\frac{2.43}{2} + 1.365\right)$$
95.04

$$+ (12.1 \times 1.365)\left(\frac{1.365}{2}\right)$$
11.27

$R_A = 34.94$ kN.
$R_B = 34.94$ kN. } symmetric.

Max moment will occur at centre:

$$= 34.94\left(1.365 + \frac{2.43}{2}\right) - 12.6 \times 1.865\left(\frac{1.365}{2} + \frac{2.43}{2}\right)$$
90.14 ... 32.64

$$- 15.74 \times \frac{2.43}{2} \times \frac{2.43/2}{2}.$$
11.62

$= 45.89$ kNm

Factored Moment $= 1.5 \times 45.89 = 68.84$ kNm.

Step 4: A_{st}.

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

$$68.84 \times 10^6 = 0.87 \times 415 \times A_{st} \times 240\left[1 - \frac{415 A_{st}}{20 \times 1000 \times 240}\right]$$

$$794.44 = A_{st} - (8.64 \times 10^{-5}) A_{st}^2$$

$A_{st} = 858.05$ mm²

$12mm\ \phi \Rightarrow a_{st} = \frac{\pi}{4} \times 12^2 = 113.1\ mm^2.$

$$\text{Spacing} = \frac{113.1}{858.05} \times 1000 = 131.81\ mm \simeq 130mm$$

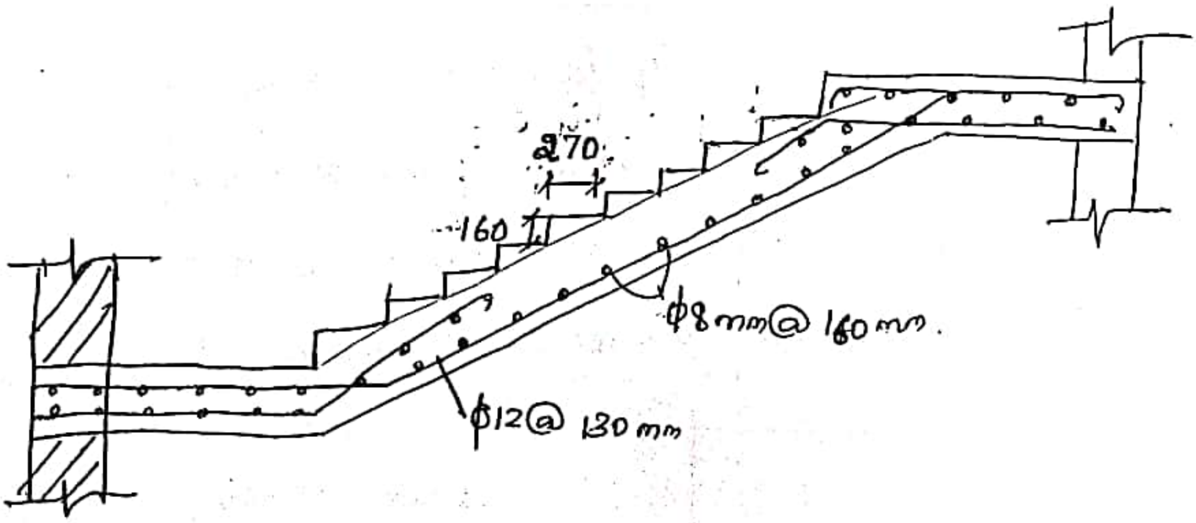
Distribution Reinf $\Rightarrow A_{st} = 0.12\%\ bD$

$$= \frac{0.12}{100} \times 1000 \times 260$$

$$= 312\ mm^2.$$

$8mm\ \phi \Rightarrow a_{st} = 50.27\ mm^2.$

$$\text{Spacing} = \frac{50.27}{312} \times 1000 = 161.12 \simeq 160\ mm$$

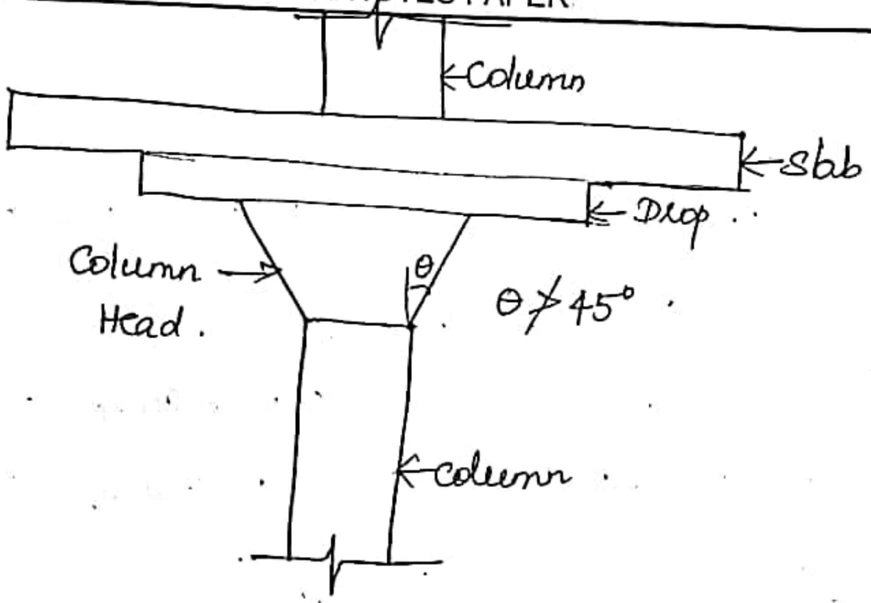


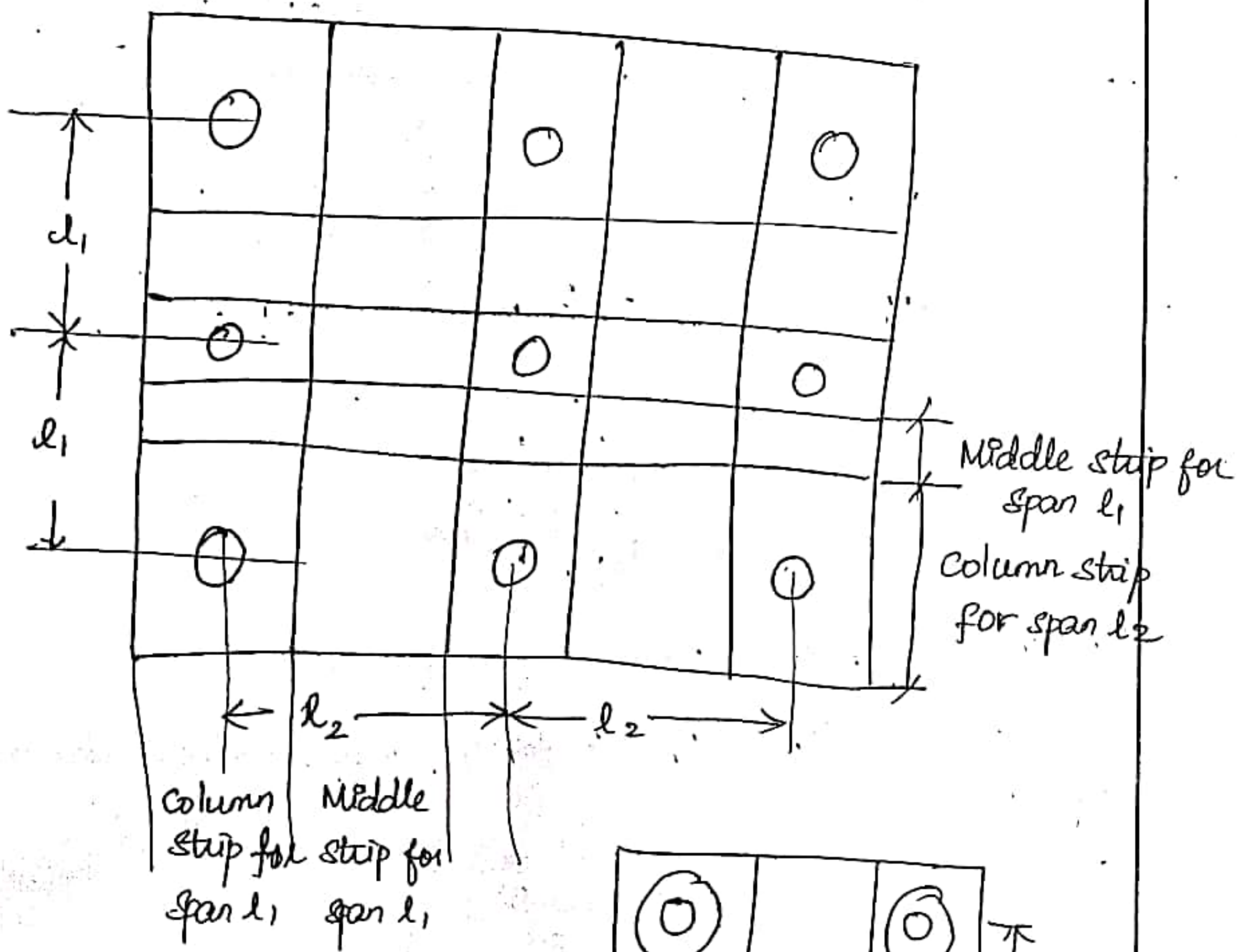
FLAT SLAB :

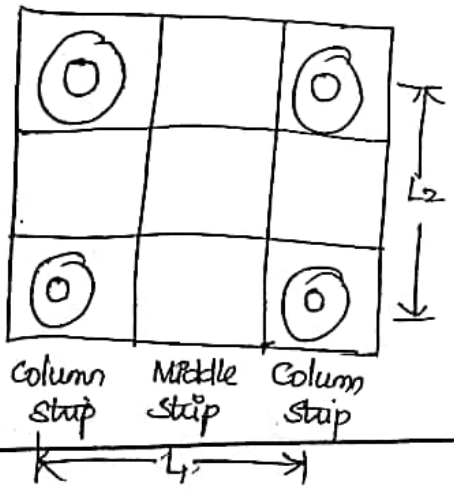
→ A flat slab is a reinforced concrete slab with or without drops. They are generally supported without beams by columns with or without column heads.

Definition of flat slab Terms :

1. **Drop**: It is the thickened part of the slab over the column
2. **Column Head**: It is the widened ~~pa~~ area at the top of the column to provide additional support to the slab.. It is also called column capital.
3. **panel**: It is the portion of slab bounded on each of its four sides, that is bounded by the centre line of column or centre lines of adjacent spans.
4. **Column strip**: It is a design strip having a width of $0.25\,l_2$ (but not greater than $0.25\,l_1$) on each side of column centre line. Here l_1 is the span in the direction of moments is determined and l_2 is the transverse span, measured c/c of supports.
5. **Middle strip**: It is a design strip bounded on each of its opposite sides by column strip.


Column
slab
Drop
Column Head.
θ
θ ≯ 45°
column

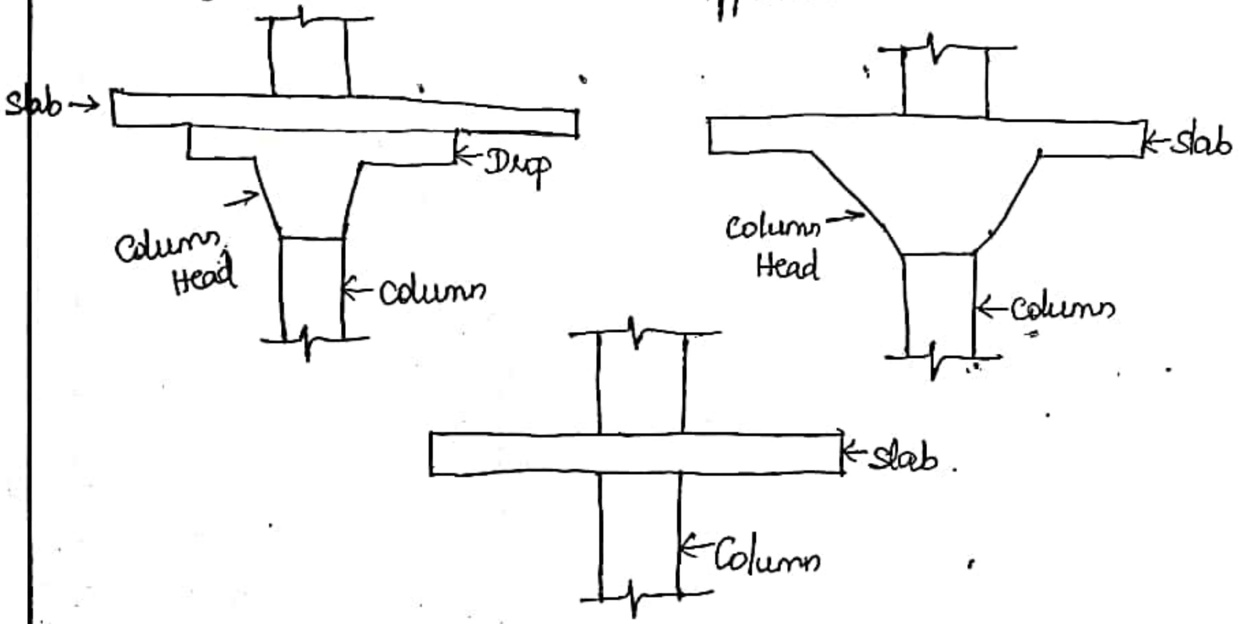

l1
l1
Middle strip for span l1
Column strip for span l2
l2
l2
Column strip for span l1
Middle strip for span l1


L2
Column strip
Middle strip
Column strip
L1

Types of flat slab:

1. Slabs with drop and column with column head
2. Slabs without drop and column with column head
3. Slabs without drop and column without column head

Drop and column head assist in resisting the high shear forces and negative moments at the supports.



Thickness of Flat slab: [cl. 31.2.1, IS 456, pg. 53]

→ It is controlled by span to eff. depth ratio based on deflection limits.

Drop: [cl. 31.2.2, IS 456] pg.no: 53]

The drop panel is formed by increasing the thickness of slab in the vicinity of the supporting column. It is provided to reduce the shear stress around the column supports. Since, the col moments in the CS are higher than in MS, drops helps to reduce the steel requirement to resist -ve moments at supports.

Column Head: [Cl. 31.2.3, IS 456 : 2000] pg. 53].

Where column heads are provided, that portion of a column head which lies within the largest right circular cone that has a vertex angle of 90° & can be included entirely within the outlines of the column & column head, shall be considered for design purposes..

Design Method:

1. Direct Design Method [Cl. 31.4.1, IS 456]
2. Equivalent Frame Method [Cl 31.5, IS 456]

Direct Design Method:

The Direct design method facilitates the computation of +ve & -ve design moments under design loads at critical sections in the slab using moment coefficient.

Limitations: [Cl. 31.4.1, IS 456, pg.no : 54]

Total Design Moments for a span:

$$M_0 = \frac{W l_n}{8}$$

where W = design load on area $l_2 l_n = w l_2 l_n$

l_n = clear span extending from face to face of column.

Interior span:

Distribution of design moment, Negative design moment = 0.65

[Cl. 31.4.3.2, pg. 55] positive design moment = 0.35

LECTURE NOTES PAPER

Types of Moment	Column strip	Middle strip.
-ve Moment	$(0.65 \times 0.75) M_0$ 49% M_0	0.15 85%
+ve Moment	$(0.35 \times 0.60) M_0$ 21% M_0	15%

Exterior span: [cl. 31.4.3.3, pg. 55, IS 456]

Interior -ve design moment $= 0.75 - \dfrac{0.10}{1+\frac{1}{\alpha_c}}$

Interior +ve design moment $= 0.63 - \dfrac{0.28}{1+\frac{1}{\alpha_c}}$

Exterior -ve design moment $= \dfrac{0.65}{1+\frac{1}{\alpha_c}}$

α_c can be calculated based on $\frac{LL}{DL}$ & $\frac{l_2}{l_1}$ ratio, refering to (Table 17, cl. 31.4.6, pg. 56)

As per cl. 31.5.5, Distribution of bending moment across the panel width is given by: [cl. 31.5.5.1 to cl. 31.5.5.4].

Column strip: -ve moment at interior support = 75%

-ve moment at exterior support = 75%

+ve moment for each span = 60%.

Middle strip: Remaining moment will be taken by it.

-ve moment = 25%

+ve moment = 40%

1. Design the interior panel of a flat slab with drops for an office floor to suit the following data:

Size of office floor = 20m × 20m

Size of panels = 5m × 5m

Loading class = 4 kN/m²

M20 grade & Fe415 steel.

Soln:

Step 1: Dimensions of slab.

$$\text{Thickness of slab} = \frac{\text{Span}}{40} = \frac{5000}{40} = 125\,mm.$$

Adopt thickness of slab in middle strip = 150 mm.

Thickness of slab at drop = 150 + 50 = 200 mm.

Column Head diameter = $D \not> 0.25\,l$

$$\Rightarrow D = 0.25 \times 5 = 1.25\,m.$$

length of drop = $l/3 = \frac{5}{3} = 1.66\ m$.

Adopt drop width = ~~B/10~~ 2.5m.

Column strip = Drop width = 2.5 m.

Middle strip width = 2.5m.

Step 2: Load Calculation

Self-wt of slab = $0.15 \times 1 \times 25 = 3.75$ kN/m² in middle

= $0.2 \times 25 = 5$ kN/m² in column strip.

$LL = 4\ kN/m^2$

Finishes $= 1\ kN/m^2$

Working load $= 10\ kN/m^2$

Factored load $= 15\ kN/m^2$.

Step 3: BM.

$$M_0 = \frac{W l_n}{8}$$

l_n = span − column head diameter

$= 5 - 1.25 = 3.75m > 0.65 l_1$

$W = W_u l_2 l_n$

$= 15 \times 5 \times 3.75 = 281.25\ kN$

$$M_0 = \frac{281.25 \times 3.75}{8} = 131.83\ kN.m.$$

Column Strip Moments:

Negative BM = 49% of M_0 = 0.49×131.83

$= 64.59\ kNm.$

Positive BM = 21% of M_0 = 0.21×131.83

$= 27.68\ kNm$

Middle Strip Moments:

Negative BM = 15% of M_0 = 0.15×131.83

$= 19.77\ kNm$

Positive BM = 15% of M_0 = 0.15×131.83

$= 19.77\ kNm.$

Step 4: Check for thickness of slab:

Column strip:
$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}}$$
$$= \sqrt{\frac{64.59 \times 10^6}{0.138 \times 20 \times 2500}}$$
$$= 96.75 < 200 \text{ mm}$$

Middle strip:
$$d = \sqrt{\frac{19.77 \times 10^6}{0.138 \times 20 \times 2500}}$$
$$= 53.53 < 150 \text{ mm}$$

Step 5: check for shear stress:

Shear stress is checked near the column head at section (D+d). Total load on circular area with (D+d) as diameter is given by:

$$W_1 = \frac{\pi}{4}(D+d)^2 W_u = \frac{\pi}{4} \times (1.25+0.17)^2 \times 15$$
$$= 23.75 \text{ KN}.$$

$$SF = \text{Total load} - \text{Load on circular area}$$
$$= (15 \times 5 \times 5) - 23.75 = 351.25 \text{ KN}.$$

Shear force/m width of perimeter $= V_u = \frac{351.25 \times 10^3}{1.25+0.17}$
$$= 247.36 \text{ KN/m}$$

$$\tau_v = \frac{V_u}{bd} = \frac{247.36 \times 10^3}{2500 \times 170}$$
$$= 0.58 \text{ N/mm}^2$$

31.6.3.1, IS 456 : 2000,

Permissible shear stress $= K_s \tau_c$

$$K_s = 0.5 + \beta_c$$

$$\beta_c = \frac{5}{5} = \frac{l_1}{l_2} = 1$$

$$K_s = 0.5 + 1 = 1.5 \not> 1.0$$

$$K_s = 1.0$$

$$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{20}$$
$$= 1.12 \ N/mm^2$$

$$K_s \tau_c = 1 \times 1.12 = 1.12 \ N/mm^2$$

$\tau_v < K_s \tau_c \Rightarrow$ Safe.

Step 6: Reinforcement Details.

(a) Column strip.

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

$$64.59 \times 10^6 = 0.87 \times 415 \times A_{st} \times 170 \left[1 - \frac{A_{st} \times 415}{2500 \times 170 \times 20}\right]$$

$$1052.32 = A_{st} - (4.882 \times 10^{-5}) A_{st}^2$$

$A_{st} = 1112.77 \ mm^2$. $\frac{A_{st}}{m} = \frac{1112.77}{2.5}$

$= 445.108$

ϕ 12mm $\Rightarrow a_{st} = 113.1 \ mm^2$

$$\text{Spacing} = \frac{113.1 \times 1000}{\cancel{1112.77}} = \cancel{101.63 \simeq 100\ mm}$$

$$= 254.09 \simeq 250 \ mm$$

(+ve BM) $\Rightarrow$ $27.68\times10^{6} = 0.87\times415\times A_{st}\times170\left(1-\frac{A_{st}\times415}{2500\times170\times20}\right)$

$$450.97 = A_{st} - (4.882\times10^{-5})A_{st}^2$$

$$A_{st} = 461.36\ mm^2$$

$$\frac{A_{st}}{m} = \frac{461.36}{2.5} = 184.54\ mm^2/m$$

$$\phi\ 10mm \Rightarrow a_{st} = \frac{\pi}{4}\times10^2 = 78.54\ mm^2$$

(b) Middle strip. $\quad$ Spacing $= \frac{a_{st}}{A_{st}}\times1000 = 425.59 \simeq 300\ mm$.

positive & negative moment.

$$19.77\times10^{6} = 0.87\times415\times A_{st}\times120\left[1-\frac{415A_{st}}{2500\times120\times20}\right]$$

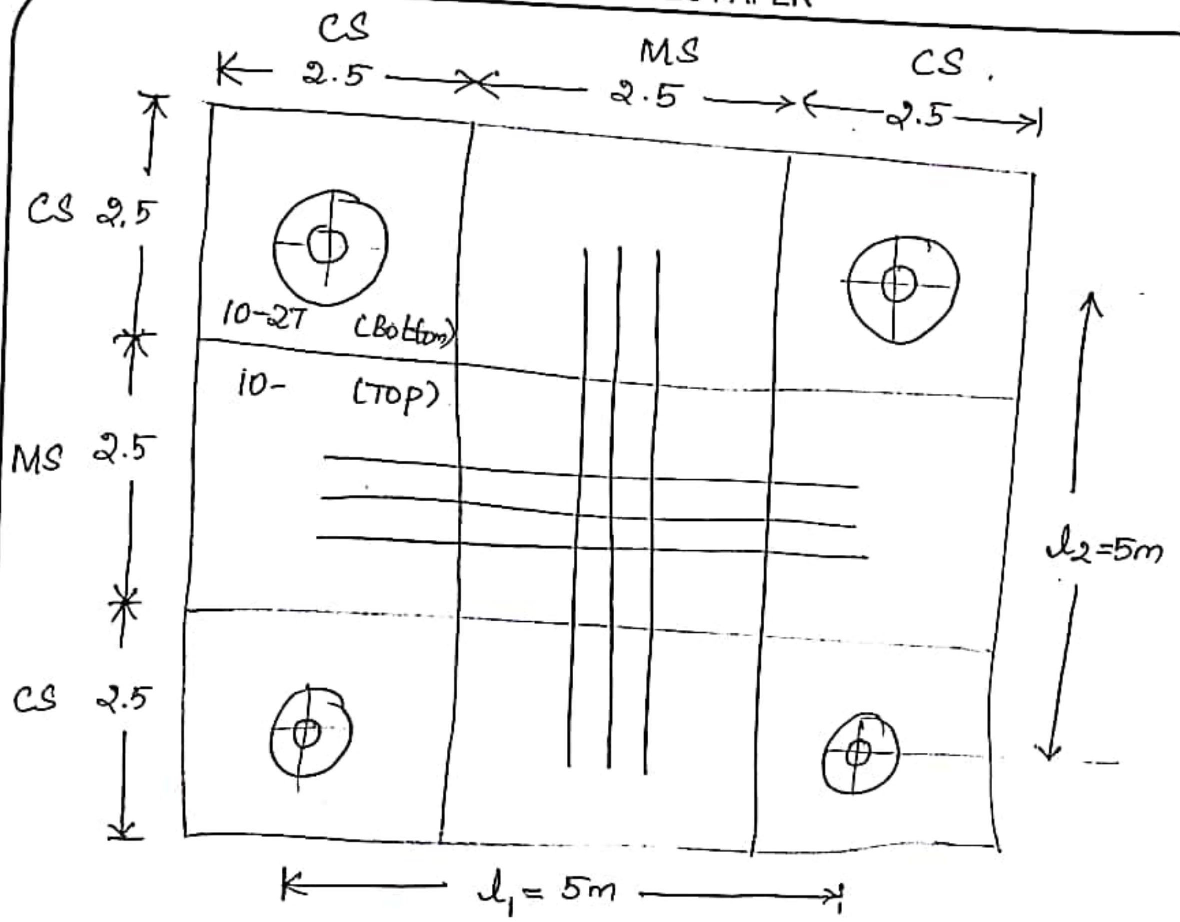
$$456.31 = A_{st} - (6.92\times10^{-5})A_{st}^2$$

$$A_{st} = 471.71\ mm^2$$

$$\frac{A_{st}}{m} = \frac{471.71}{2.5} = 188.68\ mm^2/m$$

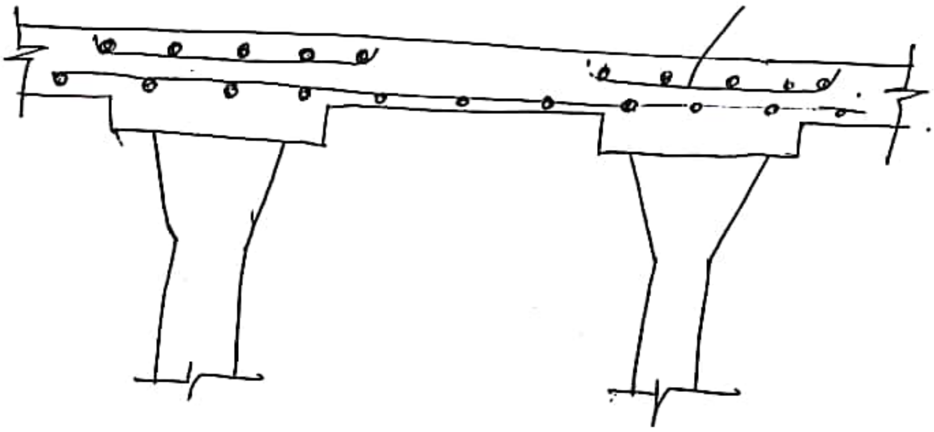
$$10mm\ \phi \Rightarrow a_{st} = \frac{\pi}{4}\times10^2 = 78.54\ mm^2$$

$$\text{spacing} = \frac{a_{st}}{A_{st}}\times1000 = 416.26 \simeq 300\ mm\ c/c.$$



Positive moment = ~~Top~~ Bottom

Negative moment = ~~Bottom~~ Top.



LECTURE NOTES PAPER

Exterior Panel:

Design the exterior panel of a flat slab with drops for an office floor to suit the following data:

Size of office floor: 20m x 20m.

Size of panel: 5m x 5m

$LL = 4\ kN/m^2$

Soln:

Step 1: Dimensions of flat slab

Step 2: Load calculation.

Step 3: Determination of BM:

$$M_0 = 131.83\ kNm.$$

Table 17, IS4

$\frac{L_2}{L_1} = \frac{5}{5} = 1$

$\frac{LL}{DL} = \frac{4}{5} = 0.8$

$\alpha_c = 0.7$

,1.4.3.3

Interior -ve moment $= \left[0.75 - \frac{0.10}{1 + 1/\alpha_c}\right] M_0$

$= \left[0.75 - \frac{0.10}{1 + \frac{1}{0.7}}\right] 131.83$

$= 93.44\ kNm$

Interior +ve moment $= \left[0.63 - \frac{0.28}{1 + 1/0.7}\right] 131.83$

$= 67.85\ kNm$

Exterior -ve moment $= \left[\frac{0.65}{1 + \frac{1}{\alpha_c}}\right] 131.83 = 35.28\ kNm$

31.5.5. Distribution of moments.

Column strip: 31.5.5.1, IS 456.

Interior.

⇒ -ve moment = 0.75 × 93.44 = 70.08 KNm

Exterior -ve moment = 0.75 × 35.28 = 26.46 KNm

Interior +ve moment = 0.60 × 67.85 Cl. 31.5.5.3.
= 40.71 KNm

Middle strip: cl. 31.5.5.4, IS 456

Interior -ve moment = 0.25 × 93.44 = 23.36 KNm

Exterior +ve moment = 0.40 × 67.85 = 27.14 KNm.

Step 4: check for depth

Step 5: Reinforcement details.

PRINCIPLES OF DESIGN OF MAT FOUNDATION:

* A mat or raft is a thick reinforced concrete slab which supports all the load bearing walls and column loads of a structure.
* A mat is required when the loads are heavy and the soil is very weak and it is more economical than individual footing when the total base area required for the individual footings exceeds about one half of the area covered by the structure.
* A mat is preferred to individual footings when the soil mass has very erratic properties and contains lenses of compressible soils. In such cases, it would be difficult to control the differential settlements if individual footings are provided.
* The mat spans over weak patches of the soil and thus the differential settlements are considerably reduced.
* Like all other shallow foundations, a mat must be safe against shear failure and the settlement should be within the allowable limits.
* As the width of a raft is very large, the pressure bulb is quite deep. Thus, the loose soil pockets under raft may be more evenly distributed. This results in a smaller differential settlement than the individual footings.
* It is assumed that differential settlement of 19mm would occur in a raft when the maximum settlement is twice that in the individual footing. Thus, maximum settlement of 50mm can

be permitted when the differential settlement allowed is 19mm

(a) Rafts on Cohesionless soils:

* The bearing capacity of a foundation on cohesionless soils depends upon the width. As the width is very large, the bearing capacity is very high, and therefore, the shear failure generally doesnot occur.
* Accordingly, the safe settlement pressure governs the design not the bearing capacity.
* But, for very loose sands ($N < 5$), Bearing capacity governs the design.
* Settlement depends upon the depth of the soil stratum. If a firm stratum exists at a shallow depth below the raft, the settlements are small. However, if the sand deposit extends to a great depth, the settlement would be large. The allowable soil pressure can be found using the following equations.
* The safe bearing capacity can be determined as:

$$q_{ns} = 0.22 N^2 B W_\gamma + 0.67 (100 + N^2) D_f W_q.$$

* The safe settlement pressure for a settlement of 25mm is given by,

$$q_{np} = 17.5 (N-3) W_\gamma$$

Where B = Smaller dimension of raft (m)

D_f = Depth of foundation

W_q, W_γ = water table correction factors.

The smaller of the two values is the allowable soil pressure.

Teny's equation for the safe settlement (q_{np}) is conservative, using Bowle's equation for the settlement of 25 mm.

$$q_{np} = 12.2\, N \left(\frac{B+0.3}{B}\right)^2 R_d W_\gamma$$

Where R_d = depth factor = $1 + 0.33\left(\frac{D_f}{B}\right)$

In general, $q_{np} = 12.2\, N \left(\frac{B+0.3}{B}\right)^2 R_d W_\gamma \left(\frac{S}{25}\right)$

Where S = allowable settlement

* As the width of a raft is very large,

$$\frac{B+0.3}{B} \approx 1.0$$

$$\therefore q_{np} = 12.2\, N R_d W_\gamma \left(\frac{S}{25}\right)$$

Taking $R_d = 1.00$ & $S = 50$ mm

$$q_{np} = 24.4\, N W_\gamma \ \ kN/m^2 \quad \text{——— (1)}$$

* In case of rafts, as the width B is very large and the pressure bulb is deep, the water table generally affects the safe settlement pressure.

Taking $W_\gamma = 0.5$, $q_{np} = 12.2\, N$ kN/m^2 ——— (2)

The above equations are applicable for $5 \leq N \leq 50$.

* If the value of N after correction is less than 5, the sand is too loose for a raft foundation. The sand should be either compacted or a deep foundation

such as pile foundation should be provided.

* For value of N greater than 50, the above equations gives unconservative results.

* According to IS 6403, the safe settlement pressure for a settlement of 65mm, is given by:

$$q_{np} = 25.4 (N-3) W_\gamma$$

Taking $W_\gamma = 0.5$, $q_{np} = 12.7 (N-3)$

* As the raft foundations are generally used below basements, the foundations are not backfilled.

Thus, $$\frac{Q}{A} = q_{na} + \gamma D_f$$

Where Q = Superimposed load

A = Area of the raft

D_f = Depth of foundation.

(b) <u>Rafts on clay</u>:

* The net ultimate bearing capacity is determined using skempton's equation:

$$q_{nu} = 5(1+0.2(D_f/B))(1+0.2(B/L))C_u$$

Where C_u = undrained cohesion.

* The safe net bearing capacity can be obtained as:

$$q_{ns} = q_{nu}/F$$

* Under normal loading conditions, factor of safety should not be smaller than 3. IS 6403 recommends a minimum factor of safety of 2.5.

* Incase of rafts on clay, as the safe bearing capacity is independent of its size, it generally governs the design.

* Incase of rafts, the pressure bulb extends to a much greater depth than that for an isolated footing.

* The settlements of rafts on normally consolidated clays are usually very large. However, incase of over-consolidated clays, the settlements are small. The settlements are calculated due to the net increase in pressure, given by.

$$q_n = \left(\frac{Q}{A}\right) - \gamma D_f \quad \text{—— (3)}$$

* If soil stratum extends to a depth greater than about twice the width of the mat, the load on the mat would tend to act as a point load for the soil at large depth and the settlement would be the same whatever be the type of foundation.

* If the settlements are large, deep foundations such as piles or drilled caissons, would be more suitable.

* The factor of safety against bearing capacity failure can be written as:

$$F = \frac{q_{nu}}{q_n}$$

* The settlement of a mat foundation can be reduced by decreasing the net increase in pressure & by increasing D_f. For no increase of net pressure, Equation 3 gives: $\gamma D_f = Q/A$

$$D_f = \frac{Q}{\gamma A} \quad \text{--- (4)}$$

* A foundation that satisfies equation 4 is known as fully compensated or floating foundation.

DESIGN PRINCIPLES OF ROAD BRIDGES:

* Reinforced concrete bridges are generally used for highway bridges, their use for rail road bridges is limited
* An Rcc bridge usually consists of T-beams supporting continuous slabs. The beams are supported on intermediate piers and end abutments.
* Solid slab, T-beam, continuous girder, cantilever and arch bridge are the different types of bridges.
* Dead load, live load, impact effect, centrifugal force, wind load, longitudinal forces and seismic forces are the types of loading acting on the bridge.

* In case of live load for road bridges, three classes of IRC loading are used.
  - → IRC class AA loading
  - → IRC class A loading
  - → IRC class B loading.
* IRC class AA loading is to be adopted within certain limits in certain existing (or) complicated industrial areas.
* IRC class A loading is to be adopted on all roads in which prominent bridge and culverts are constucted.
* IRC class B loading is normally adopted for temporary structures. Structures with the timber are to be regarded as temporary structures.
* The impact effect is calculated as, $I = \frac{4.5}{6+L}$ where L is the span in metres.
* centrifugal force is calculated by $C = \frac{WV^2}{127R}$ where w is live load, V = design speed of vehicle & R = Radius of curvature
* Deck slab bridges is economical upto 8m span whereas girder (or) T-beam bridge is economical for spans between 10 m to 20 m.

GENERAL DESIGN REQUIREMENTS:

* Size of the bars: The maximum size of reinforcements shall be 45mm ϕ and the diameter of longitudinal reinforcing bars in columns shall not be less than 12mm. The diameter of bars including transverse ties, stirrups and all the secondary reinforcement shall not be less than 6mm

* Distance between bars: The horizontal distance between two parallel reinforcing bars shall not be less than the greatest of the following dimensions:

(a) Diameter of the bar if the diameters are equal.

(b) Diameter of the largest bar - if the diameters are unequal.

The vertical distance between two main reinforcing bars shall be 12mm or the maximum size of the bar whichever is greater.

* Distribution Reinforcement: The distribution reinforcement shall be equal to 0.3 times the moment due to concentrated live load plus 0.2 times the moment due to other loads such as dead load, shrinkage, temperature, etc

* Shear stress: shear stress should not exceed

(a) 4 times the permissible shear stress.

(b) 2½ times the permissible shear stress.

* permissible stresses: It has been specified in IRC code provision

* Increase in permissible stresses:

(I) When the effect of temperature, shrinkage and creep is taken into consideration, the permissible stresses may be exceeded by 15%.

(II) When the effect of wind forces is taken into consideration, permissible stresses may be exceeded by 25%.

(III) When the effect of seismic forces is considered, the permissible stresses may be exceeded by 50%.

DESIGN PRINCIPLES OF BOX CULVERT:

* A box culvert is used where a small drain crosses a high embankment of a road or a railway or a canal especially when the bearing capacity of the soil is low.

* A box culvert is a continuous rigid frame of rectangular section in which the abutment and the top and bottom slabs are cast monolithic.

* In case of high embankments, an ordinary culvert will

require very heavy abutments which will be expensive and will transfer heavy loads to the foundations, while RC box culvert will be cheaper.

* A box culvert will be subjected to soil load from outside and water load from inside. The vertical walls are subjected to earth pressure from outside and water pressure from inside.

* Similarly, the bottom slab will be subjected to soil pressure from outside and water pressure from inside. The top slab will however be subjected to embankment weight and traffic loads

* A box culvert is therefore designed for two condition

(I) The box culvert will be dry from inside and the side walls will be subjected to earth pressure from outside.

(II) Water in the box culvert will be subjected to earth pressure from outside and water pressure from inside.

* The analysis is usually done using moment-distribution method.

* For the purpose of design, one metre length of the box culvert is considered, The analysis is done for the following cases:

(I) Live load, Dead load and earth pressure acting with no water pressure from inside.

(II) Live load and dead load on top and earth pressure acting from outside and water pressure acting from inside.

(III) When the sides of the culvert doesnot carry live load and the culvert is full of water.

* From these three cases, take the maximum values of moment which is been calculated using moment distribution method and draw the reinforcement details for three moment. Calculate Ast and reinforcement required.