OBJECTIVE

At the end of this course students should have learnt to design Reinforced Concrete Structures such as building frames, Retaining walls and water tanks.

UNIT 1

YIELD LINE THEORY

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 FRAMES12

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

UNIT III RETAINING WALLS12

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 SELECTED TOPIC 12

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

TOTAL HRS:60

TEXT BOOKS:

Sl.No	Title of Book	Author of Book	Publisher	Year Publishing	of
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12

1	R.C.C. Designs Reinforced	Punmia	B.C,	Laxmi Publications	2006
	Concrete Structures	Ashok Jain, Arui	Kumar nK.Jain	Pvt. Ltd., New Delhi	

REFERENCES:

Sl.No	Title of Book	Author of Book	Publisher	Year of Publishing
1	Advanced Reinforced	Varghese.P.C	Prentice Hall of	2012
	Concrete Design		LtdNew Delhi.	
2	Reinforced Concrete	Mallick, D.K. and Gupta A.P	Oxford and IBH Publishing Company, New	2003
3	Reinforced Concrete Structures	Syal, I.C. and Goel, A.K	A.H. Wheelers & Co. Pvt. Ltd.,Chennai	2001
4	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/

KARPAGAM ACADEMY OF HIGHER EDUCATION (Under section 3 of UGC act 1956) COIMBATORE-641021 FACULTY OF ENGINEERING DEPARMENT OF CIVIL ENGINEERING

15BECE702 / DESIGN OF RC STRUCTURES II LECTURE PLAN

Number of credits	: 4
Contact hours	: 5 hours per week
Lecturer	: Mr. V. Johnpaul
Course Type	: Core

S.No	Period/ Hours	Topics	Units	T(Book)	Page No
		UNIT-1 YIELD LINE THEORY	7		
1.	1	Introduction	1	T1	516
2.	1	characteristics, sign conventions, yield line patterns in slab		T1	517-519
3.	1	Upper bound theory-Virtual work method , Equilibrium method		T1	520-528
4.	1	Design of simply supported square slab with uniformly distributed and central concentrated loads by virtual method		T1	521-523
5.	1	Design of fixed supported square slab with uniformly distributed and central concentrated loads by virtual method		T1	523-524
6.		Tutorial -1			
7.	1	Design of simply supported rectangular slab with uniformly distributed and central concentrated loads by equilibrium method		T1	525-527
8.	1	Design of fixed supported rectangular slab with uniformly distributed and central concentrated loads by equilibrium method		T1	533-534
9.	1	Tutorial-2			
Total	9 hours(7+ 2)				
		UNIT-II LIME – BUILDING FRAM	AES		
10	1	Difference between multi-storeyed load bearing and framed structures	2	T1	722
11	1	substitute frames- Elastic analysis for gravity loads		T1	723-725
12	1	Analysis of multi-storey building by substitute framed method –load calculation, formation of substitute frame		T1	726-727
13	1	Fixed end moment and distribution factor calculation		T1	726-728

14	1	Formation of distribution table, to draw		T1	728-729
		B.M and S.F diagram			
15	1	Tutorial -1			
16	1	Approximate analysis of two bay frames		T1	733-735
		upto 3 storeys for wind loads using portal			
		method- column shear and moment at the			
15		end of the column calculation		T 1	
17	1	Calculation of moments at the end of beams,		TI	/35-/36
		beam shear and axial force, to draw B.M			
10	1	and S.F diagram		T 1	
18	1	Analysis of building frame subjected to		11	/36-/3/
		wind loads by cantilever method- location of			
		centroidal axis of column, axial force			
10	1	Calculation		T 1	729 720
19	1	Calculation of B.M and S.F in beam &		11	/38-/39
20	1	Tutorial 2			
20 Total	1 11hours(
Total	1 mouts(
	9+2)				
		UNIT III - RETAINING WALL	S		
21	1	Retaining wall- angle of repose, back fill.	3	T1	671
	1	surcharge, earth pressure	Ũ		0/1
22	1	Types of retaining walls, forces on retaining		T1	671-676
	1	wall			0/1 0/0
23	1	Design of cantilever retaining wall-		T1	680-682
	-	preliminary dimensions, check stability of			
		retaining wall			
24	1	Check for pressure distribution at base,			682-683
		design of stem			
25	1	design of toe slab, design of heel slab		T1	683-685
26	1	design of shear key		T1	685-686
27	1	Design of counter fort retaining wall-		T1	704-706
		preliminary dimensions, check stability of			
		retaining wall, Check for pressure			
		distribution at base			
28	1	design of stem, toe slab and heel slab		T1	706-714
29	1	design of counter fort		T1	714-716
30	1	design of horizontal and vertical stirrups		T1	716-719
31	1	Tutorial-1			
32	1	Stability requirements		T1	676-678
33	1	Description of inclined backfill (no		T1	674
		problems)			
34	1	Tutorial-2			
	14hours(
	12+2)				
		UNIT IV- GORUND WATER TAI	NK		
35	1	Classifications-based on shapes, levels and	4	T2	254
		functions			
36	1	Principles of design-IS code provision-no		T2	254-256
		tension basis			
37,38	2	Design of underground rectangular tank		Τ2	274-282

39	1	Design of rectangular ground level tank		T2	269-272
40	1	Design of circular ground level tank		T2	258-263
41	1	Tutorial -1			
42,43	2	Design of overhead rectangular tank		T2	289-294
44	1	Design of staging for rectangular tank with 4		T2	294-296
		columns only			
45	1	Descriptions of Intze tank (no problems)		T2	296-299
46	1	Tutorial-2			
	12				
	hours(10				
	+2)				
		UNIT V- BRIDGES			
47	1	Types of bridges-IRC loadings	5	T3	4,146-181
48	1	Design of single span slab bridges for class		T3	471-472
		A loading only- preliminary dimensions,			
		load calculation			
49	1	B.M calculation		T3	472-473
50	1	S.F calculation		T3	473-474
51	1	Design of slab		T3	474-475
52	1	Concept of skew slab bridge. (no problems)		T3	526-527
53	1	Tutorial -1			
		PRESTRESSED CONCRETE			
54	1	Materials-Principles of prestressing		T4	1-3
55	1	Different methods and systems		T4	73-84
56	1	Losses due to prestress (no problems), IS		T4	124-139
		code provisions			
57	1	Analysis of stresses for rectangular section		T4	90-96
		only			
58	1	Tutorial -2			
Total	12				
	hours(10				
	+2)				

SUPPORT MATERIALS

	TEXT BOO	DKS
T1	Limit state design of reinforced concrete by Dr.P.C.Punmia	Laxmi publications (p) ltd
T2	Advanced R.C.C. design by Krishna Raju	CBS publishers & distributors, New Delhi
T3	Prestressed concrete by N. Krishna Raju	Tata McGraw Hill Education Private Limited, New Delhi,2012
T4	Bridge engineering by S.Ponnuswamy	McGraw Hill Education Private Limited, 2009
	REFERENCE	BOOKS
R1	"Reinforced Concrete Design" by	Tat Mc Graw Hill Publishing Co, New
	Unnikrishna Pillai& Devados Menon	Delhi,
R2	,"Reinfroced Concrete Limit State Design", by Ashok K Jain	NewChand Brothers,Roorkee
R3	"Concrete Structures" By Vazirani V.N. and Ratwani, N.M	Khanna Publishers

STAFF INCHARGE

Prof.N.Balasundaram M.E, {Ph.d} (HOD/IC Civil Department) Dr.R.Sundrarajan. (DEAN/FOE)(NOC)

RETAINING WALL

Retaining wall - Retains Earth - when level difference exists between two surfaces



A) Gravity wall (h<3m) - Masonry or Stone



(e) basement wall

(f) bridge abutment

F) Bridge abutment [Additional horizontal restraint from bridge deck]

Stability – Overturning and Sliding – Avoided by providing sufficient base width.

Earthpressureandstabilityrequirements:





Pressure, $P \quad CJ_e Z$ Where, Z = depth, J_e = Unit weight

$$\mathbf{C}_{\mathbf{a}} = \frac{1 \quad Sin\mathbf{I}}{1 \quad Sin\mathbf{I}}; \qquad \qquad Cp \quad \frac{1 \quad Sin\mathbf{I}}{1 \quad Sin\mathbf{I}}$$

Always, Cp > Ca. Eg: If $\phi = 30^{\circ}$, Ca = 1/3 and Cp = 3.

In sloped backfill,

$$C_{a} = \frac{{}^{a}CosT}{{}^{\ll}CosT} \frac{\sqrt{Cos}^{2}T}{\sqrt{Cos}^{2}T} \frac{Cos}{Cos}^{2}I} \overset{\circ}{\underset{M}{\mathbb{A}}} CosT ; \qquad Cp \quad \frac{1}{1} \frac{SinI}{SinI}$$

1. Effectofsurchargeonlevelbackfill:



$P_a = P_{a1} + P_{a2}$, where,	
$P_{a1} = C_a.W_s.h = C_a.\gamma_e.h_s.h \&$	[h/2 above heel]
$P_{a2} = Ca. \gamma_{e.}h^2/2$	[h/3 above heel]

<u>Note</u> : Purpose of retaining wall is to retain earth and not water. Therefore, submerged condition should be avoided by providing and maintaining proper drainage facilities [including provision of weep holes].

2. Effectofwaterinthebackfill:





<u>Stability</u> requirements FOS against $\frac{Overtuning}{Sliding} \ge 1.4$

i) Overturning:

 $FOS_{overturning} = \frac{0.9Mr}{Mo} \ge 1.4$

a) For sloping backfill, $M_o = (P_a.CosT) \cdot \frac{h'}{3} = C_a \cdot J_e \cdot \frac{h'^3}{6} \cdot CosT$ $Mr = W(L - X_w) + (P_a.SinT) \cdot L$ b) For level backfill [with surcharge, $M_o = (P_{a1}(h/2) + (P_{a2}(h/3)))$ $Mr = W(L - X_w)$ [as $T = 0^o$]



Sliding is reduced b

Passive resistance due to shear key

$$P_{ps} = C \mathbf{J}_{e} (h_{2}^{2} h_{1}^{2}) / 2$$

 X_{sk} -> Flexural reinforcement from stem is extended straight into shear key near the toe.

Note: For economical design, soil pressure resultant(R) must be in line with front face of wall.

Preliminaryproportioningofcantileverretainingwall:

- 1. The thickness of base slab is h/12 or 8% of the height of wall + surcharge.
- 2. The base thickness of stem should be greater than the thickness of base slab
- 3. The top thickness of stem should not be less than 150mm.
- 4. Clear cover for stem is 50mm and base slab is 75mm
- 5. Minimum length of base slab is given by

$$L_{\min} \quad \stackrel{\$}{\underset{0}{\sim}} \sqrt{\frac{C_a}{3}} \stackrel{\cdot}{\underset{\bullet}{\bullet}} \frac{h'}{D_{\mu}}$$

where, α_R = Coefficient depending on the pressure distribution α_R = 0.5 for rectangular pressure distribution & 0.67 for trapezoidal pr.dist.

6. Minimum length of heel slab is given by

$$X = \frac{\$}{0} \sqrt{\frac{C_a}{3}} h'$$

Notes:

- 1. The critical section for moment is at front face of stem.
- 2. The critical section for shear is at 'd' from face of stem.
- 3. The stem, heel and toe slabs are designed as cantilever slabs for the resultant pressure.
- 4. Temperature and shrinkage reinforcement is provided as 0.12% of cross section along the transverse direction to the main reinforcement and front face of the stem.

1) Determine suitable dimensions of a cantilever retaining wall, which is required to support a bank of earth 4.0m high above ground level on the toe side of the wall. Consider the backfill surface to be inclined at an angle of 15° with the horizontal. Assume good soil for foundation at a depth of 1.25m below ground level with SBC of 160kN/m². Further assume the backfill to comprise of granular soil with unit weight of 16kN/m³ and an angle of shearing resistance of 30° . Assume the coefficient of friction between soil and concrete to be 0.5.

Given: h = 4.0 + 1.25m

$$\theta$$
 = 15°
 ϕ = 30°
 J_e = 16kN/m³
 q_a = 160kN/m²
 μ = 0.5

Minimum depth of foundation (Rankine's),



$$\mathsf{d}_{\mathsf{m}} = \frac{q_a}{\mathsf{J}_e} \underbrace{\$1 \quad \sin \mathbf{I}}_{1} \cdot \underbrace{\$i}^2 = \frac{160}{16} \underbrace{\$1}_{0} \cdot \underbrace{\$1}^2 = 1.11\mathsf{m}$$

Earth pressure coefficient,

$$C_{a} = \frac{{}^{a}CosT}{{}^{a}CosT} \frac{\sqrt{Cos^{2}T} Cos^{2}I}{\sqrt{Cos^{2}T} Cos^{2}I} {}^{a}_{\frac{1}{2}} CosT = 0.373$$
$$Cp \quad \frac{1}{1} \frac{SinI}{SinI} = 3.0$$

Preliminaryproportioning:

Thickness of footing base slab = $0.08h = 0.08 \times 5.25 = 0.42m$ Provide a base thickness of 420mm for base slab.

Assume stem thickness of 450mm at base of stem tapering to 150mm at top of wall.

For economical proportioning of length 'L', assume vertical reaction R at the footing base to be in line with front face of the stem.

$$X = \int_{0}^{\$} \sqrt{\frac{C_{a}}{3_{t}}} h' \sqrt{\frac{0.373}{3}} (5.25 \quad 0.4) = 2.0m \qquad \text{[where 0.4m is assumed as}$$

height above wall]

Assuming a triangular base pressure distribution,

L = 1.5X = 3.0m

Preliminary proportions are shown in figure.

For the assumed proportions, the retaining wall is checked for stability against overturning and sliding.

Force	Force (kN)	Distance from heel	Moment
ID		(m)	(kNm)
W1	16(1.85)(5.25 - 0.42) = 143.0	0.925	132.3
W2	16(1.85)(0.5x0.536) = 7.9 [2tan15°=0.536]	0.617	4.9
W3	25(0.15)(5.25-0.42) = 18.1	1.925	34.8
W4	(25-16)(4.83)(0.5x0.3) = 6.5	1.75	11.4
W5	25(3)(0.42) = 31.5	1.50	47.2
PaSinθ	25.9	0	0
Total	W = 232.9		Mw = 230.6

Stabilityagainstoverturning:

Pa = Active pressure exerted by retained earth on wall [both wall and earth move in same direction]

Pp = Passive pressure exerted by wall on retained earth [both move in opposite direction]

Ca -> same for dry and submerged condition, since ϕ for granular soil does not change significantly.

Force due to active pressure,

 $\begin{array}{l} {\sf P}_{\sf a} = {\sf Ca}, \ \gamma_{\sf e}, {\sf h}'^2/2 \\ {\sf Where}, \qquad {\sf h}' = {\sf h} + {\sf Xtan} \theta \\ = 5250 + 2000 {\sf tan15}^\circ = 5786 {\sf mm} \\ {\sf P}_{\sf a} = 0.373(16)(5.786)^2/2 = 99.9 {\sf kN} \qquad [{\sf per \ m \ length \ of \ wall}] \end{array}$

Therefore, $FOS_{(overturning)} = \frac{0.9Mr}{Mo} \ge 1.4$

Overturning moment, $M_o = (P_a Cos\theta)h'/3 = 96.5(5.786/3) = 186.1kNm$

To find the distance of resultant vertical force from heel,

Distance of resultant vertical force from heel, $X_w = M_w/W = 230.6/232.9 = 0.99m$

 $\begin{array}{ll} \text{Stabilising moment (about toe),} \\ \text{M}_{r} = W(L - X_{w}) + P_{a}\text{Sin}\theta(L) = 232.9(3 - 0.99) + 77.6 \\ = 468.1 \text{kNm} \qquad [\text{per m length of wall}] \end{array}$

 $FOS_{(overturning)} = \frac{0.9Mr}{Mo} = \frac{0.9x(468.1 \quad 77.6)}{186.1} = 2.26 > 1.40$

Soilpressureatfootingbase:

Resultant vertical reaction, R = W = 232.9kN [per m length of wall]





Distance of R from heel, $L_R = (M_w + M_o) / R = (230.6 + 186.1)/232.9 = 1.789m$ Eccentricity, $e = L_R - L/2 = 1.789 - 3/2 = 0.289m < L/6 \rightarrow [0.5m]$

Hence the resultant lies within the middle third of the base, which is desirable.

Maximum pressure as base,

$$q_{\text{max}} = \frac{R}{L} \$_1 = \frac{6e}{L} \cdot \frac{232.9}{3} (1 \ 0.578) \ 122.5kN / m^2 \ q_q \quad \text{[where } q_a = 160 \text{kN/m}^2\text{]}$$
 [Safe].

 $q_{\min} = \frac{R \$_1}{L \And 0} \frac{6e}{L !} \cdot \frac{232.9}{3} (1 \quad 0.578) \quad 32.8kN / m^2 ! 0 \quad [\text{No tension develops}] \quad [\text{Safe}].$

Stabilityagainstsliding:

Sliding force, $P_aCos\theta = 96.5kN$ Resisting force, $F = \mu R = 0.5 \times 232.9 = 116.4kN$

[Ignoring passive pressure on toe side]

 $FOS(Sliding) = \frac{0.9F}{P_a CosT} = \frac{0.9x116.4}{96.5} \quad 1.085 \quad 1.40$ [Not sufficient]

Hence a shear key may be provided.

Assume a shear key 300mm x 300mm at a distance of 1300mm from toe as shown in figure.



design of shear key

 $h_2 = 950 + 300 + 1300 \tan 30^\circ = 2001 \text{mm}$

 $Pp = C_p \cdot \gamma_e \cdot (h_2^2 - h_1^2)/2 = 3 \times 16 \times (2.001^2 - 0.950^2)/2 = 74.11 \text{kN}.$

FOS (Sliding) = $\frac{0.9(116.4 \quad 74.44)}{96.5}$ 1.78 ! 1.4 [SAFE]

Designoftoeslab:

Assuming a clear cover of 75mm and 16mm Φ used,

$$V_{u} = 1.5 \frac{a_{112}}{\sqrt{2}} \frac{81.9^{\circ}}{2} x(1 \quad 0.337) \quad 96.42kN$$

[V_u is design shear at 'd' from face of stem]

 $M_u = 1.5\{(81.9 \times 1^2/2) + (112 - 81.9) \times 1/2 \times 1^2 \times 2/3\}$

= 76.48 kNm/m length Nominal shear stress, $W_{\nu} = \frac{V_{\mu}}{bd} = \frac{96.42 \times 10}{1000 \times 337^2} = 0.286 N / mm^2$

Using M20 concrete, For a W_c 0.29N/mm₂, p_t (required) = 0.2%

[Page 178, SP-16]

$$\mathsf{K} = \frac{M_{u}}{bd^{2}} - \frac{76.48 \times 10^{6}}{1000 \times 337^{2}} = 0.673 N / mm^{2}$$
 [Page 48, SP-16]

For $p_t = 0.2\%$, $A_{st} = 0.2/100 \text{ x} 1000 \text{ x} 337 = 674 \text{ mm}^2 \text{ / m}$

Spacing = $\frac{1000xSx16^2}{674}$ /4 = 298mm Provide 16mm I @ 290mm c/c at bottom of toe slab

$$Ld = \frac{IV_{g}}{4W_{bd}} = \frac{(16).0.87f_{y}}{4W_{bd}} = 16 \times 47 = 752 \text{mm}, \text{ beyond face of stem}.$$

Since length available is 1m, no curtailment is sorted.

Designofheelslab:

 $V_{u} = 1.5 \frac{82.54}{2} \frac{128.6^{\circ}}{2} x(1.55 \quad 0.337) \quad 128.06 kN$

 $M_u = 1.5\{(82.54 \times 1.55^2/2) + (128.6 - 82.54) \times 1/2 \times 1.55^2 \times 2/3\}$

= 203.96 kNm/m length Nominal shear stress, $\mathbb{W}_{v} = \frac{V_{u}}{bd} = \frac{128.06 \times 10^{3}}{1000 \times 337} = 0.38 N / mm^{2}$

Using M20 concrete,

For a W_c 0.39N/mm₂, p_t (required) = 0.3% [Page 178, SP-16]

$$\mathsf{K} = \frac{M_u}{bd^2} - \frac{203.96 \times 10^6}{1000 \times 337^2} = 1.8N / mm^2$$
 [Page 48, SP-16]

For $p_t = 0.565\%$, $A_{st} = 0.565/100 \times 1000 \times 337 = 1904.05 \text{ mm}^2 / \text{m}$

Spacing = $\frac{1000xSx16^2}{1904.05} = 105.61$ mm Provide 16mmI @ 100mm c/c at bottom of toe slab

$$L_{d} = \frac{IV_{s}}{4W_{bd}} = \frac{(16).0.87 f_{y}}{4W_{bd}} = 16 \times 47 = 752 \text{mm}, \text{ beyond face of stem.}$$

Since length available is 1.55m, no curtailment is sorted.

Designofverticalstem:

Height of cantilever above base = 5250 - 420 = 4830mm

Assume a clear cover of 50mm and 16mmΦ bar,

 $d_{at \ base} = 450 - 50 - 16/2 = 392 \text{mm}$

 $M_{u} = 1.5(C_{a}.\gamma_{e}.h^{3}/6) = 1.5(1/3)(16 \times 4.92^{3}/6) = 150.24 \text{kNm}.$ $K = \frac{Mu}{bd^{2}} = \frac{150.24 \times 10^{\frac{6}{2}}}{1000 \times 4.92^{2}} = 1 \text{N/mm}^{2}.$

 $p_t = 0.3\%$, $A_{st} = 0.295/100 \times 1000 \times 392 = 1200$ mm2.

Spacing = 1000 x 201/1200 = 160mm Provide 16mm I @ 160mm c/c in the stem, extending into the shear key upto 47Φ =

752mm.

<u>Checkforshear</u>: [at 'd' from base]

 $V_{u} \text{ (stem)} = 1.5(C_{a}.\gamma_{e}.Z^{2}/2) = 1.5(1/3 \times 16 \times (4.83 - 0.392)^{2}/2) = 53.83 \text{kN}$ $W_{v} \frac{V_{u}}{bd} \frac{53.83 \times 10^{3}}{1000 \times 392} = 0.135 \text{N} / \text{mm}^{2} \quad W_{c} \text{ [where, } \zeta_{c} = 0.39 \text{N}/\text{mm}^{2} \text{ for } p_{t} = 0.3\%))$

Hence, SAFE.

Curtailmentofbars:

Curtailments of bars in stem are done in two stages:

At $1/3^{rd}$ and $2/3^{rd}$ height of the stem above base.

Temperatureandshrinkagereinforcement:

 $A_{st} = 0.12/100 \times 10 \times 450 = 540 \text{mm}^2$

For I 1/3rd height, provide 2/3rd of bar near front face (exposed to weather) and 1/3rd near rear face.

For II $1/3^{rd}$ height, provide 1/2 the above.

For III 1/3rd height, provide 1/3rd of I case.

Provide nominal bars of 10mm @ 300mm c/c vertically near front face.



Detailing of cantilever wall

DESIGNOFCOUNTERFORTRETAININGWALL



(c) counterfort wall

Preliminaryproportioningofcounterfortretainingwall:

- 1. Thickness of heel slab and stem = 5% of Height of wall
- 2. Thickness of toe slab [buttress not provided] = 8% of Height of wall
- 3. Thickness of counterfort = 6% of height of wall
- 4. In no case thickness of any component be less than 300mm.
- 5. Spacing of counterforts = $1/3^{rd}$ to 1/2 of Height of wall
- 6. Each panel of stem and heel slab are designed as two way slab with one edge free

(one way continuous slab).

- 7. The toe slab is designed as
 - a. Cantilever slab when buttress is not provided
 - b. One way continuous slab, when buttress is provided
- 8. Counterfort is a triangular shaped structure designed similar to a T-Beam as vertical cantilever with varying depth (stem acts as flange). The main reinforcement is along the sloping side. Stirrups are provided in the counterfort to secure them firmly with the stem. Additional ties are provided to securely tie the counterfort to the heel slab.

1) Design a suitable counterfort retaining wall to support a leveled backfill of height 7.5m above ground level on the toe side. Assume good soil for the foundation at a depth of 1.5m below ground level. The SBC of soil is 170kN/m^2 with unit weight as 16kN/m^3 . The angle of internal friction is $\varphi = 30^\circ$. The coefficient of friction between the soil and concrete is 0.5. Use M25 concrete and Fe415 steel.



Stabilityconditions:

Earth pressure calculations:

Force	Force (kN)	Distance from heel	Moment
ID		(m)	(kNm)
W1	16(7.5+1.5-0.5)(2.5) = 340	(3 - 0.5)/2 = 1.25	425
W2	25(0.5)(9-0.5) = 106.25	0.5/2 + 2.5 = 2.75	292.18
W3	25(0.5)(3) = 37.5	1.5	56.25
W4	25(1.5)(0.72) = 27	1.5/2 + 3 = 3.75	101.25
Total	W = 510.75		Mw = 874.69

Xw = 874.69/510.75 = 1.713m

FOS (overturning) = $0.9M_r/M_o$

Where, $Mo = P_a \cdot h/3 = C_a \cdot \gamma_e \cdot h^3/6 = 0.33 \times 16 \times 93/6 = 647.35 \text{kNm}.$

Mr = (L - Xw).W = 510.75(4.5 - 1.713) = 1423.6kNm.

FOS (overturning) = 1.98 > 1.4

Hence, section is safe against overturning.

Sliding:

FOS (sliding) = $0.9(\mu R)/P_aCos\theta$

F = µR = 0.5 x 510.75 = 255.375kN

 $P_a = C_a \cdot \gamma_e \cdot h^2 / 2 = 215.784$



Base pressure calculation:

 $q_{\max} = \frac{R \$}{L \$} \frac{6e}{L !} \cdot \frac{510.75}{4.5} (1 - \frac{6x0.73}{4.5}) = 223.97 kN / m^2 ! q_q \quad \text{[where } q_a = 170 \text{kN/m}^2\text{]} \quad \text{[UnSafe]}.$ $q_{\min} = \frac{R \$}{L \$} \frac{6e}{L !} \cdot \frac{510.75}{4.5} (1 - \frac{6x0.73}{4.5}) = 3.027 kN / m^2 ! 0 \quad \text{[No tension develops]} \quad \text{[Safe]}.$

Where, $L_R = (M + M_o)/R$, $e = L_R - L/2$, where,

 $L_R = (874.688 + 647.352)/510.75 = 2.98m$ & e = L_R − L/2 = 2.98 − (4.5/2) = 0.73 < L/6 → (0.75m)

Since the maximum earth pressure is greater than SBC of soil, the length of base slab has to be increased preferably along the toe side. Increase the toe slab by 0.5m in length.

∑W = 510.75 + 0.5 x 25 x 0.72 = 519.75kN



Additional load due to increase in toe slab by 0.5m is,

Moment = 0.5/2 + 4.5 = 4.75m

 $\Sigma M = 874.69 + 42.75 = 917.438$ kNm.

 $L_R = (Mo + M) / R = (917.438 + 647.352)/519.75 = 3.011m$

e = L_R - L/2 = 3.011 - (5/2) = 0.511m < L/6 → (0.833m)

$$q_{\text{max}} = \frac{R_{\$}}{L_{0}} \frac{6e}{L_{t}} \cdot \frac{519.75}{5} (1 - \frac{6x0.511}{5}) - 166.32kN/m^{2} - q_{q}$$
 [where q_a = 170kN/m²] [Safe].

 $q_{\min} = \frac{R}{L} \begin{cases} 1 & \frac{6e}{L} & \frac{519.75}{5} \\ L & f & 5 \end{cases} (1 & \frac{60.511}{5}) = 41.58 kN/m^2 ! 0 \qquad [\text{No tension develops}] \qquad [\text{Safe}].$

FOS $_{(Sliding)} = 0.9(\mu R)/P_a = 0.9(0.5 \times 519.75)/215.784 = 1.08 < 1.4.$ Hence the section is not safe against sliding. Shear key is provided to resist sliding.

Assume shear key of size 300 x 300mm.



$$\begin{split} P_{ps} &= C_p.\gamma_{e.}(h2^2-h1^2)/2 = 164.5 kN/m\\ FOS_{(sliding)} &= 0.9(\mu R + P_{ps})/P_a = 1.77 > 1.4 \end{split}$$

[where, h1 = 1.2m, h2 = 1.2 + 0.3 + 1.39 = 2.88m]

Hence, section is safe in sliding with shear key 300 x 300mm.



Net soil pressures acting on base slab

DesignofToeSlab:

Effective cover = 75 + 20/2 = 85mm

Toe slab is designed similar to cantilever slab with maximum moment at front face of the stem and maximum shear at'd' from front face of stem.

d = 720 - 85 = 635mm. M = 80.38x2²/2 + ½ x 2 x 49.94 x 2/3 x 2 = 160.76 + 122.76 = 227.35kNm. SF at 0.635m, = 49.94/2 x 0.635 = 15.606kN Area of trapezoid = ½.h.(a + b) = ½(2 - 0.635)(130.32 + 95.98) = 154.44kN Factored SF = 231.66kN; Factored Moment = 341.02kNm. K = M_u/bd² → A_{st} = 1551.25mm² → Spacing = 1000a_{st}/A_{st} → 16mm @125mmc/c. Transverse reinforcement: = 0.12% of c/s = 0.12/100 x 1000 x 720 = 864mm²

Provide 10mm @100mm c/c.

Designofheelslab:

The heel slab is designed as an one way continuous slab with moment $wl^2/12$ at the support and $wl^2/16$ at the midspan. The maximum shear at the support is w(l/2 - d).

The maximum pressure at the heel slab is considered for the design.

Moment at the support, $M_{sup} = wl^2/12 = 106.92 \times 2.5^2/12 = 55.688 kNm$.

Moment at the midspan, $M_{mid} = wl^2/16 = 41.76kNm$

The maximum pressure acting on the heel slab is taken as 'w' for which the Ast required at

midspan and support are found.

Factored M_{sup} = 83.53kNm \rightarrow Ast = 570.7mm²

Factored $M_{mid} = 62.64 \text{kNm} \rightarrow \text{Ast} = 425.4 \text{mm}^2$

Using 16mm ϕ bar, Spacing = 1000ast / Ast = 110.02mm \rightarrow Provide 16mm @ 110mm c/c

At midspan, spacing = 156.72mm → Provide 16mm @ 150mm c/c

Transverse reinforcement = 0.12% of c/s = 0.12/100 x 1000 x 500 = 600 mm^2 For 8mm bar, Spacing = 83.775mm \rightarrow Provide 8mm @80mm c/c.

Checkforshear:

Maximum shear = w (l/2 - d) = 107 (2.5/2 - 0.415) = 89.345kN

Factored shear force = 134.0175kN

 $\zeta v = 0.33 \text{N/mm}^2$, $\zeta c = 0.29 \text{N/mm}^2$, $\zeta \text{cmax} = 3.1 \text{N/mm}^2$

Depth has to be increased.

Designofstem:

The stem is also designed as one way continuous slab with support moment $wl^2/12$ and midspan moment $wl^2/16$. For the negative moment at the support, reinforcement is provided at the rear side and for positive moment at midspan, reinforcement is provided at front face of the stem.

The maximum moment varies from a base intensity of $K_a.\gamma_e.h=1/3x16x(9-0.5)=$ 45.33kN/m

 $M_{sup} = wl^2/12 = 1.5 \times 45.33 \times 3.54^2/12 = 71kNm$

 $M_{mid} = wl^2/16 = 1.5 \times 45.33 \times 3.54^2/16 = 53.26 kNm$ Effective depth = 500 - (50 + 20/2) = 440mm

 A_{st} at support = 1058mm², For 16mm ϕ , Spacing = 190mm. Provide 16mm @ 190mm c/c

 A_{st} at midspan =718mm², For 16mm ϕ , Spacing = 280mm. Provide 16mm@280mm c/c

Max. SF = w (I/2 - d) = 60.29kN, Factored SF = 90.44kN

Transverse reinforcement = 0.12% of c/s \rightarrow 8mm @ 80mmc/c $\zeta_v = 0.188$ N/mm², $\zeta_c = 0.65$ N/mm², $\zeta_{cmax} = 3.1$ N/mm² Safe in Shear.

DesignofCounterfort:

The counterfort is designed as a cantilever beam whose depth is equal to the length of the heel slab at the base and reduces to the thickness of the stem at the top. Maximum

moment at the base of counterfort, $M_{max} = K_a \cdot \gamma_e \cdot h^3 / 6 \times L_e$

Where, $L_e \rightarrow c/c$ distance from counterfort

M_{max} = 1932.5kNm, Factored Mmax = 2898.75kNm

 $A_{st} = 2755.5 \text{mm}^2$, Assume 25mm ϕ bar, No. of bars required = 2755.5/491.5 = 5.61 ~ 6

The main reinforcement is provided along the slanting face of the counterfort.

Curtailmentofreinforcement:

Not all the 6 bars need to be taken to the free end. Three bars are taken straight to the entire span of the beam. One bar is cut at a distance of,

 $\frac{n-1}{n} = \frac{h1^2}{8.5^2}$, where n is the total number of bars and h1 is the distance from top.

When n = 6, h1 = 7.75m [from bottom]

The second part is cut at a distance of,

 $\frac{n-2}{n} = \frac{h2^2}{8.5^2}$, h2 = 6.94m [from bottom]

The third part is cut at a distance of,

 $\frac{n-3}{n} = \frac{h3^2}{8.5^2}$, h3 = 6.01m [from bottom]

Vertical ties and horizontal ties are provided to connect the counterfort with the vertical

stem and the heel slab.

Designofhorizontalties:

Closed stirrups are provided to the vertical stem and the counterfort. Considering 1m strip,

the tension resisted by reinforcement is given by lateral pressure on the wall multiplied by

contributing area.

 $T = C_a.\gamma_e.h x h$, where, $A_{st} = \frac{T}{0.87 fy}$

 $T = 1/3 \times 16 \times (9 - 0.5) \times 3.54 = 160.48$ kN

Factored force, $T = 1.5 \times 160.48$ kN

 $A_{st} = 666.72 \text{mm}^2$. For 10mm ϕ , Spacing = 110mm.

Provide 10mm@110mm c/c closed stirrups as horizontal ties.

Designofverticalties:

The vertical stirrup connects the counterfort and the heel slab. Considering 1m strip, the

tensile force is the product of the average downward pressure and the spacing between the counterforts. T = Avg(43.56 & 107) x Le = 266.49 kN

 $\label{eq:Factored} \begin{array}{l} Factored \ T=399.74 \ kN\\ Ast=1107.15 mm^2. \ \ For \ 10 mm \ \phi, \ Spacing=70.93 mm. \end{array}$

Provide 10mm @ 70mm c/c.





Reinforcement details of stem and counterfort



Section through counterfort showing counterfort reinforcement



Further, water tanks are classified based on their positions as,

- 1. Resting on ground
 - a. Flexible or free base
 - b. Hinged
 - c. Fixed
- 2. Elevated or overhead
- 3. Underground



Components of water tank:

- 1. Side Walls [Rectangular or cylindrical]
- 2. Base slab
- 3. Cover slab or dome
- 4. Staging [Overhead] \rightarrow Columns, Beams, Bracings

Types of Joints:

- i) Flexible base
- ii) Hinged base
- iii) Fixed base



Water pressure distribution:

1. On side walls, it acts linearly varying from 0 at top to 'wh' at bottiom.

2. On base slab, uniform pressure acts with intensity 'wh'

Permissible stresses:

The method of design adopted for design of water tank is working stress method. The permissible stress values in concrete and steel are given in Tables 21 and 22 of IS456-2000, as follows:

For M20 $\rightarrow \sigma_{cbc}$ = 7 N/mm², σ_t = 5 N/mm² For Fe250 steel, σ st = 130 - 140 N/mm² For Fe415 steel, σ st = 230 - 240 N/mm²

	Grade of Concrete					
Stress (N/mm ²)	M-15	M-20	M-25	M-30	M-35	M-40
Direct Tension	1.1	1.2	1.3	1.5	1.6	1.7
Bending Tension	1.5	1.7	1.8	2.0	2.2	2.4

Permissible Concrete Stresses in calculations relating to Resistance to Cracking in Water retaining Structures (IS: 3370-Part-II-1965)

Permissible Stresses in Steel reinforcement for Strength Calculations in Water retaining Structures (IS: 3370-Part-II-1965)

	Stress (N/mm²)	Plain Mild Steel bars	HYSD bars
	Tensile stresses in members Under direct tension	115	150
	Tensile stresses in members In Bending		
a)	On liquid retaining face of members	115	150
b)	On face away from liquid for members less than 225 mm thick	115	150
c)	On face away from liquid for members 225 mm or more in thickness	125	190
d)	Compressive stresses in columns subjected to direct load	125	175

For liquid retaining structures, the stress values are further reduced and given in IS3370

Part II [Reinforced concrete structures].

Concrete Grade	Direct Tension	Bending Tension
M15	1.1	1.5
M20	1.2	1.7
M25	1.3	1.8
M30	1.5	2.0
M35	1.6	2.2
M40	1.7	2.4

As per clause B - 2.1.1, the tensile stress is given by,

$$\sigma = \frac{Ft}{A_c \quad mA_{st}} \quad \text{where, } m \rightarrow \text{modular ratio} = 280 / 3 \sigma_{cbc} = E_s/E_c$$

 σcbc is the permissible compressive stress.

Permissible stress in steel is given in IS3370 Part II as,

<u>Tensilestress</u>: Fe250 \rightarrow 115 N/mm², Fe415 \rightarrow 150 N/mm²

Bendingstress:

	< 225mm		>	225mm
	<u>OnFace</u>	<u>Onfaceaway</u>	<u>OnFace</u>	<u>Onfaceaway</u>
Fe250	115	115	115	125
Fe415	150	150	190	175

Reinforcement requirements: [As per IS3370]

- 1. Minimum $A_{st}\xspace$ is 0.3% for 100mm section and 0.2% for 450mm section.
- If thickness exceeds 200mm, the reinforcement is provided in double layers. A minimum cover of 25mm is provided along the liquid retained face and the cover is increased by 12mm (37mm) if the wall is subjected to aggressive soil or liquid.

HOOP TENSION AND BENDING MOMENTS IN CYLINDRICAL TANK WALLS



Hoop Tension and Bending Moments in Cylindrical Tank Walls

CYLINDRICAL TANK WITH FLEXIBLE BASE:

The wall is designed for hoop stress, for which circumferential horizontal steel is provided. Minimum reinforcement is provided along the vertical direction

1) Design a circular water tank with flexible base for a capacity of 4 lakh litres with the tank having a depth of 4m, including a free base of 200mm. Use M20 concrete and Fe415 steel. Area of water tank = Volume / Height

$$= 400 / 4 = 100m^2$$
 [As Vol. = 4 Lakh Litres = 400 m³]

 $\Pi.d^2/4 = 100 \text{ m}^2 \Rightarrow d = 11.28 \text{m}$

Provide a diameter of 11.5m

The height of water to be retained = 3.8m

The wall is subjected to hoop tension acting along the circumferential direction. The hoop tension per metre height is given by,

Hoop tension =
$$\frac{J.h.D}{2}$$
 = (9.81 x 3.8 x 11.5)/2 = 214.34 kN

Permissible stress in tension as per IS3370 for Fe415 steel is 150 N/mm².

 A_{st} required = 214.34 x 10³ / 150 = 1428.9 mm² Spacing = 1000.ast / Ast, → 16mm @ 140mm c/c

Thickness of tank is adopted based on the tensile stress concrete can take.

$$\sigma_{ct} = \frac{Ft}{A_c \quad mA_{st}}$$
Ac = 1000t, F = 214.34kN, m = 280 / 3 σ_{cbc} = 280 / (3 x 7) = 13.33 ≈ 13
 σ_{ct} = 1.2 = $\frac{214.34}{1000t \quad (13.33x1428.9)}$

→ t = 160mm

Minimum thickness as per empirical formula is,

$$t_{min} = (30h + 50)mm$$
 where $h \rightarrow m$

= 30 x 3.8 + 50 = 164mm

Provide a thickness of 170mm.

Minimum reinforcement is provided as vertical steel.

Minimum Ast is 0.3% for 100mm section and 0.2% for 450mm section.

Therefore, for 170mm thickness, Ast required is 0.28% of c/s

 $A_{st} = (0.28/100) \times 1000 \times 170 = 476 \text{mm}^2$

Provide 8mm @ 100mm c/c

Curtailmentofreinforcement:

At 2m height,

Hoop tension = $\frac{J.h.D}{2}$ = (9.81 x 1.8 x 11.5) / 2 = 101.5335 kN Ast = 101.535 x 103 / 150 = 676.89 mm² Provide 10mm @ 110mm c/c (or) 16mm @ 290mm c/c

Designofbaseslab:

Since the base slab rests directly on the ground, a nominal thickness of 150mm is provided and a minimum reinforcement of 0.3% of c/s is provided along both ways and along both the faces.

0.3% of c/s \rightarrow 0.3/100 x 150 x 1000 = 450 mm², Required 8mm @110 mm c/c

Provide 8mm @220 mm c/c on both faces.

Below the base slab, a layer of lean concrete mix M20 is provided for 75mm thickness with a layer of tar felt.



Designofdome: - Roof covering for cylindrical water tanks

[A dome acts as a roof covering for cylindrical water tanks. The dome slab is cast between the ring beams provided at the top edge of the side walls of the water tank. The dome is designed for meridonial thrust and hoop force]. 2) In the above problem, design a spherical dome having a central rise of one fifth the diameter.

Height = $1/5 \times 11.5 = 2.3 \text{m}$



Details of Dome of Circular Water Tank

Radius of curvature of the dome [R]

$$R^2 = (R - 2.3)^2 + 5.75^2$$

 $\cos\theta = 6.04 / 8.33 = 0.724$

The dome is subjected to meridonial thrust and hoop force, for which the permissible stress should be within permissible compressive strength of concrete.

$$\sigma_c = 5 \text{ N/mm}^2$$

Assume thickness of dome as 100mm

Meridonial thrust, $T = \frac{WR}{1 CosT}$, where 'w' is the loading of dome slab, self weight of slab and any live load acting on it.

Self weight of slab = 0.1×25 = 2.5 kN/m^2 Live load = 2kN/m^2 W = 4.5 kN/m^2

$$\mathsf{T} = \frac{4.5x8.3}{1 \quad 0.74} = 21.46 \mathsf{kN}$$

Meridonial thrust = T / (c/s area) = 21.46×10^3 / (100 x 1000) = 0.21 N/mm² < 5 N/mm² The hoop stress developed in dome slab is given by,

Hoop stress =
$$\frac{WR}{t} \mathop{\otimes}_{0}^{\&} CosT$$
 $\frac{1}{1 CosT} \mathop{:}_{1}^{:} = \frac{4.5x8.3}{100} \mathop{\otimes}_{0}^{\&} 0.74 \frac{1}{1 0.74} \mathop{:}_{1}^{:} = 0.06 \text{ N/mm}^2 < 5 \text{ N/mm}^2$

The provided 100mm section is sufficient. Minimum reinforcement of 0.3% of c/s is provided both ways.

0.3% of c/s = 0.3/100 x 1000 x 100 =
$$300 \text{ mm}^2$$
 $\rightarrow 8 \text{mm} @ 160 \text{ mm} \text{ c/c}$

Design of ring beam:

The horizontal component of meridonial thrust acts on the ring beam. The horizontal thrust, $A_{st} = H_{L \ Comp} / \sigma_s$ where, $H_{L \ Comp} = T.Cos\theta \ x \ D/2 = 21.46 \ x \ 10^3 \ x \ 0.74 \ x \ 5.75 = 91.3 kN$ $A_{st} = 91.3 \ x \ 103 / 150 = 608 \ mm^2$

Provide (2# 16mm
$$\phi$$
 + 2# 12mm ϕ) \rightarrow (626mm²)

Size of ring beam is obtained based on tensile stress relation,

$$\sigma_{\rm ct} = \frac{F_t}{A_c \quad (m \quad 1)A_{st}}$$

Here ring beam is not subjected to water load. So, permissible tensile stress for M20 concrete is 2.8 N/mm² (Annex B-3.11) of IS456-2000.

$$\sigma_{ct} = 2.8 \text{ N/mm}^2$$

$$\Rightarrow 2.8 = \frac{F_t}{A_c \quad 13.3x608} \Rightarrow A_{st} = 608 \text{ mm}^2$$

$$F = T \cos\theta \times D/2 = 21.46 \times 0.74 \times 5.75 = 91.31 \times 10^{3} N$$

→
$$A_c = 25314$$
 mm².

Provide 150 x 150 mm size of ring beam with (2# $16mm\phi + 2# 12mm\phi$) bars. If any tensile stress is developed, the steel will take the tensile stress.



Sectional elevation

DESIGN OF CYLINDRICAL WATER TANK: [With fixed base]

For cylindrical tanks fixed at the base, bending moment and hoop tension are developed, whose values are based on non-dimensional parameter, $\frac{H^2}{Dt}^2$. The vertical reinforcement is provided for the bending moment developed and the transverse reinforcement is provided for hoop tension developed.

The coefficients of the non-dimensional parameter $\frac{H^2}{Dt}^2$ are given in Tables 9 (HT) and 10 (BM).

Maximum hoop tension = Coefficient x W.H.D/2

Bending moment = Coefficient $x w.H^3$

Where, $R \rightarrow Radius$ of tank, $D \rightarrow Diameter$ of tank

3) In the above problem, design a water tank for fixed base condition. Permissible stresses are, for Fe415, $\sigma_{st} = 150 \text{N/mm}^2$, as per IS3370 – Part II and for M20, $\sigma_{ct} = 1.2 \text{ N/mm}^2$. H = 4m, D = 11.5m

t = 30h + 50 = 170mm

$$\frac{H^2}{Dt} = \frac{4^2}{11.5x0.17} = 8.184$$

From Table 9 of IS3370 – Part IV, the coefficient for maximum hoop tension is taken and from Table 10 of IS3370 – Part IV, the coefficient of bending moment is taken.

$$\frac{H^2}{Dt} \rightarrow 8, \text{ Hoop Tension Coeff.} \rightarrow 0.575 @ 0.6H$$

$$\frac{H^2}{Dt} \rightarrow 10, \text{ Hoop Tension Coeff.} \rightarrow 0.608 @ 0.6H$$

$$\Rightarrow \frac{H^2}{Dt} \rightarrow 8.18, \text{ Hoop Tension Coeff.} \rightarrow 0.578 @ 0.6H$$
Therefore, T = Coeff. X wHR = 0.578 x 9.81 x 4 x 11.5/2 = 130.41 kN
$$\frac{H^2}{Dt} \rightarrow 8, \text{ BM Coeff.} \rightarrow -0.0146$$

$$\frac{H^2}{Dt} \rightarrow 10, \text{ BM Coeff.} \rightarrow -0.0122$$

$$\Rightarrow \frac{H^2}{Dt} \rightarrow 8.18, \text{ BM Coeff.} \rightarrow 0.01438$$
Max. BM = -0.01438 x 9.81 x 4^3 = -9.031 kNm

The vertical reinforcement is provided for the above moment and transverse steel is provided for hoop tension.
For hoop tension as,

 $A_{st} = HT/150 = 130.41 \times 10^3 / 150 = 869.33 \text{ mm}^3$.

Provide 12mm @ 130mm c/c along the transverse direction.

For bending moment,

 $A_{st} = \frac{M}{V_{st} \cdot j.d}$ Where, for M20, j = 0.84, R = 1.16, $\sigma_{cbc} = 7 \text{ N/mm}^2$ $A_{st} = \frac{9.03}{150x0.84x129} = 555 \text{ mm}^2$ $d = D - \phi_{HT} - \phi_{BM}/2 - C_C$ = 170 - 12 - 8/2 - 25 = 129 mm $d = \sqrt{\frac{M}{R.b}} = \sqrt{\frac{9.2x10^{-6}}{1.16x1000}} = 88.23 \text{ mm} < d \text{ [129 mm]}$

Min. depth required = 88.23mm

0.3% of c/s (vertical steel) = 0.28/100 x 1000 x 170 = 476 mm²

Provide 8mm @ 100mm c/c

The maximum of Ast for BM at Ast {maximum} is provided as vertical reinforcement [555mm²].

Provide 8mm @ 90mm c/c

Provide 10mm @ 130mm c/c as vertical reinforcement. If the reinforcement is provided as double layers on both the faces, the spacing is doubled.

Provide 10mm @ 260mm c/c as vertical reinforcement along the two faces with transverse hook reinforcement at 12mm @ 260mm c/c along both the faces.

Designofbaseslab:

Provide a nominal thickness of 150mm for base slab with 0.3% distribution steel [8mm @ 200mm c/c, along both faces, both ways].

Also provide a haunch of size 160mm at the junction of the wall and the base slab.





When L / B < 2, cantilever moment generates at the base and maximum bending action takes place along the continuous edges of the side walls in both short and long direction. When L / B > 2, only cantilever action takes place in long wall. Whereas, a short wall is designed for horizontal bending and cantilever action.

[Cylindrical \rightarrow Hoop Tension \rightarrow Transverse reinforcement]

WhenL/B<2,

i) Cantilever moment at base = w.H.h² / 6



- ii) For horizontal bending of the walls, the maximum moment is found from,
 - a) Fixed end moments,

PL²/12 & PB²/12

b) Positive moment at midspan,

 $M_f = PL^2/8$ & $M_f = PB^2/8$

Where, M_f is the final moment at the supports (junction between long wall and short wall). The maximum value is taken for design. Here,

 $P \rightarrow$ water pressure given as p = w(H - h)

For the maximum moment, the area of steel along the longer direction and shorter direction are found from the relation,

$$A_{stL} = \frac{M P_L x}{V_{st} \cdot j.d} \quad \frac{P}{V_{st}}; \qquad \qquad A_{stB} = \frac{M P_B x}{V_{st} \cdot j.d} \quad \frac{P_B}{V_{st}}$$

Where, $P_L \& P_B$ are tension in long and short walls

p - Pressure exerted by water

Tension in the walls is given by, $P_L = p \times B/2 \& P_B = p$

1. Design a rectangular tank of size $4m \times 6m$ with height 3m. The tank rests on firm ground. Use M20 concrete and Fe415 steel. Take design constants j = 0.853 & R = 1.32.

Pressure exerted by water, p = w (H - h)

Where, $h = 1m \text{ or } H/4 \rightarrow 1m \text{ or } 0.75m = 1m \text{ [greater]}$

P = 9.81 (3 - 1) = 19.62 kN/m2

To find the final moment at the junction of long wall and short wall based on the fixed end moment and distribution factor, the moment distribution is done.

Joint A:

D.F. =
$$\frac{I_1/L_1}{I_1/L_1 - I_2/L_2}$$
.

The stiffness along the long wall and short wall are the same $(I_1 \ I_2)$, since uniform thickness of wall is adopted along long wall and short wall.

AB
D.F.
$$\frac{I_1/L_1}{I_1/L_1 I_2/L_2}$$
. $\frac{I_2/L_2}{I_1/L_1 I_2/L_2}$.
 $\frac{1/6}{1/6 1/4}$. $\frac{1/4}{1/6 1/4}$.

Short wall is stiffer than the long wall. $M_{FAB} = pL^2 / 12 = 19.62 \times 6^2 / 12 = 58.86 \text{ kNm} (3p)$ $M_{FAD} = pB^2 / 12 = 19.62 \times 4^2 / 12 = 26.16 \text{ kNm} (1.33p)$ Joint A,

	AB	AD	
FEM	Зр	-1.33p	
DF	0.4	0.6	
BM	(-1.67 x 0.4)	(-1.67 x 0.6)	
	= -0.67p	= -1.002p	
СО	-	-	
M _f	+ 2.33p	- 2.33p	
Mf = 2.33p =	= 2.33 x 19.62 = 45	5.72kNm	
Moment at midspan,			

Long wall:

$$\frac{pL^2}{8}$$
 $M_f = 42.57$ kNm,

Short wall:

$$\frac{pB^2}{8}$$
 $M_f = -6.48$ kNm,

The reinforcement is provided for maximum moment generated.

Therefore, maximum moment generated in the water tank is

45.72kNm.

$$A_{stL} = \frac{M}{V_{st} \cdot j.d} \frac{P}{V_{st}}; \qquad A_{stB} = \frac{M}{V_{st} \cdot j.d} \frac{P_{B}}{V_{st}}$$

$$P_{L} = p \times B / 2 = 19.62 \times 4/2 = 39.24 \text{ kN}$$

$$P_{B} = p \times L / 2 = 19.62 \times 6/2 = 58.86 \text{ kN}$$

$$d_{req} = \sqrt{\frac{M}{R.b}} = \sqrt{\frac{45.72 \times 10^{6}}{1.32 \times 1000}} = 186 \text{mm}$$

Provide d = 190mm, D = 220mm Provide effective depth of 190mm and effective cover of

Area of steel required for Tension in the wall along the longer side and shorter side,

$$A_{stL} = \frac{45.72 \times 10^6 \quad (39.24 \times 80 \times 10^3)}{150 \times 0.853 \times 190} \quad \frac{39.24 \times 10^3}{150} = 2013 \text{mm}^2$$

$$A_{stB} = \frac{45.72x10^{\circ} (58.86x80x10^{\circ})}{150x0.853x180} \quad \frac{58.86x10^{\circ}}{150} =$$

2079mm²

Provide 20mm ϕ bar, spacing required along horizontal direction is 150mm

Provide 20mm @ 300mm c/c along both faces in the horizontal direction, along short wall and long wall.





The vertical cantilever moment for a height of 'h' is

$$=$$
 w.H.h² / 6 [L / B < 2]

 $= 9.81 \times 3 \times 1^2 / 6 = 4.905 \text{ kNm}$

$$A_{st} = \frac{M}{V_{st} \cdot j.d} = 201.76 \text{mm}^2$$

Min. Ast = 0.3% of c/s

= 0.3/100 x 1000 x 220

 $= 660 \text{ mm}^2$

Spacing is provided for the maximum of the above two [66mm²]

Required, 10mm @ 110mm c/c

Provide, 10mm @ 220mm c/c as vertical reinforcement along both the faces. For the base slab, provide a nominal thickness of 150mm and minimum A_{st} of 0.3% of c/s.

 $A_{st} = 0.3/100 \text{ x} 1000 \text{ x} 150 = 450 \text{ mm}^2$

Spacing of 10mm bars required = 170mm

Provide 10mm @ 300 mm c/c along both faces, both ways.

Provide 75mm lean mix with a layer of tar felt which acts as a water bar, provided between the tank and lean mix concrete.

2) Design a water tank of size $4m \times 9m$ with height 3m. Use M20 concrete and Fe415 steel. The design constants are j = 0.853 and R = 1.32.

Since L/B > 2, the tank behaves such that the long wall acts as a cantilever member with moment w.H 3 /6 and short wall is subjected to both cantilever moment and horizontal bending moment.

Longwall:

Cantilever moment at base = $w.H^{3}/6$

Where, h = 1m or $3/4m = 1m = 9.81 \times 3^3 / 6 = 44.145 \text{ kNm}$

Shortwall:

Cantilever moment = w.H. $h^{2}/12$ = 9.81 x 3 x $1^{2}/2$ = 14.715 kNm

Horizontal bending moment = $pB^2/16 = 19.62 \times 4^2 / 16 = 19.62$ kNm (Where, p = w (H - h) = 9.81 (3 - 1) = 19.62kN/m)

Maximum of the three moments = 44.145kNm

$$A_{stL} = \frac{M}{V_{st}} \frac{P_L x}{P_L x} = \frac{P}{V_{st}}$$
, where, $P_L = p \ge B/2 = 19.62 \ge 4/2 = 39.24$ kN and $P_B = p = 19.62$ kN

$$d_{req} = \sqrt{\frac{M}{R.b}} = \sqrt{\frac{44.145 \times 10^6}{1.32 \times 1000}} = 182.87 \text{mm}$$

Provide d = 190mm, D = 220mm Provide effective depth of 190mm and effective cover of 30mm.

x = D/2 - Eff. Cover

= 220 / 2 - 30 = 80mm

Area of steel required for Tension in the wall along the longer side,

$$A_{stL} = \frac{44.145x10^{6} (39.24x80x10^{3})}{150x0.853x190} \quad \frac{39.24x10^{3}}{150} = 1948.35 \text{mm}^{2}$$

Provide 20mm φ bar, spacing required along horizontal

direction is 160mm. Provide 20mm @ 300mm c/c along both faces in the horizontal direction, along short wall and long wall.

Along the shorter side,

Along vertical direction,

 $A_{stB} = \frac{14.72x10^6 \quad (19.62x80x10^3)}{150x0.853x180} \quad \frac{19.62x10^3}{150} = 690.45 \text{mm}^2$

Required - 10mm @ 220mm c/c on both faces.

Along horizontal direction,

 $A_{stB} = \frac{19.62x10^6 \quad (19.62x80x10^3)}{150x0.853x180} \quad \frac{19.62x10^3}{150} = 897.5 \text{mm}^2$

Required - 10mm @160mm c/c on both faces.

Distribution steel:

100mm \rightarrow 0.3% 450mm \rightarrow 0.2% 220mm \rightarrow 0.26%

Min $A_{st} = 0.26\%$ of c/s = 0.26 / 100 x 1000 x 220 = 572mm²

Min. Required - 10mm @ 260mm c/c, which is less than the above two values.



Baseslab:

Since base slab is resting directly on the ground, nominal thickness of 150mm is provided and minimum reinforcement of 0.3% of cross-section is provided both ways along both faces.

 A_{st} required = 0.3% of c/s = 465mm².

Provide 8mm @ 200mm c/c.

Below the base slab, a layer of lean concrete mix is provided with 75mm thick tar felt layer between them.



DESIGNOFUNDERGROUNDWATERTANK

Design of underground water tank is similar to that of tanks resting on grounds (for rectangular water tanks based on L/B ratio), where additional moment if any due to the earth pressure on the side walls need to be considered. If the soil is submerged, pressure exerted by water is also considered. Thus the side walls are checked for the two critical conditions,

i) No earth pressure with pressure from water inside, $p = \gamma_w(H - h)$

ii) Earth pressure exerted on wall under tank empty condition, $p = H/3.(\gamma - \gamma_w) + \gamma_w H$ The tank has to be checked for uplift water pressure for which frictional resistance should be sufficient. Design an underground water tank of size 12m x 5m with height 4m. The density of soil is 16kN/m³ and coefficient of friction between soil and concrete is 0.15. The soil is saturated. Here, L / B = 12 / 5 = 2.4 > 2

The tank walls are checked for two critical conditions,

i) No earth pressure with pressure from water inside, $p = \gamma_w(H - h)$

$$= 10 (4-1) = 30 \text{kN/m}^2$$

&

ii) Earth pressure exerted on wall under tank empty condition, p =H/3.(γ - γ_w)+ γ_w H = 48kN/m²

Where,
$$h = 1m \text{ or } (H/4 = 1m)$$

16

Therefore, the maximum pressure is used in finding out bending moment on the wall. Long wall,

Cantilever moment =
$$\frac{J_w H^3}{6}$$
 = 10 x 4³/6 = 106.67kNm

Short wall,

Cantilever moment =
$$\frac{J_w Hh^2}{2}$$
 = 20kNm
Horizontal bending moment = $\frac{pB^2}{16}$ = $\frac{30x5^2}{16}$ = 46.875 kNm
= $\frac{48x5^2}{16}$ = 75 kNm

Longwall:

$$A_{stL} = \frac{M P_L x}{V_{st}. j.d} \frac{P}{V_{st}}$$
, where, $P_L = p \times B/2 = 48 \times 2.5 = 120 \text{ kN}$

$$P_{B} = p = 48 \text{ kN}$$

$$\begin{split} &\mathsf{d}_{\mathsf{req}} = \sqrt{\frac{M}{R.b}} = \sqrt{\frac{106.67\,x10^6}{1.32\,x1000}} = 284.27 \text{mm} \\ &\mathsf{Provide} \; \mathsf{d} = 290 \text{mm} \\ &\mathsf{Provide}, \; \mathsf{D} = 290 + 30 = 320 \text{mm} \\ &\mathsf{x} = 320 \ / \ 2 - 30 = 130 \text{mm} \\ &\mathsf{A}_{\mathsf{st}(\mathsf{VL})} = 2454.35 + 800 = 3254.35 \text{ mm}^2 \\ &\mathsf{A}_{\mathsf{st}(\mathsf{HL})} = 0.237\% \; \mathsf{of} \; \mathsf{c/s} \\ & (\mathsf{For} \; 100 \text{mm} \quad 0.3\% \\ & \mathsf{For} \; 450 \text{mm} \quad 0.2\% \\ & \mathsf{For} \; 320 \text{mm} \quad 0.237\%) \\ &\mathsf{A}_{\mathsf{st}(\mathsf{HL})} = 0.237 \; \mathsf{x} \; 1000 \; \mathsf{x} \; 320 = 758.4 \; \mathsf{mm}^2 \end{split}$$



Provide 10mm @ 200mm c/c <u>Shortwall</u>: $A_{st(VL)} = 370.8 + 320 = 690.8 mm^2 < A_{stmin}$ Provide Astmin, 10mm @ 200mm c/c $A_{st(HL)} = 2173.09 mm^2$ Provide 16mm @ 180mm c/c

Checkforuplift:

The tank is checked for uplift pressure. When the uplift pressure exceeds the downward load due to the self weight of the tank and the frictional resistance required. The base slab is projected all around thereby increasing the downward load. The pressure of submerged earth and water at the bottom of the base slab for 1m length of the wall is found. The frictional resistance of the tank is found by multiplying the coefficient of friction between soil and concrete with the pressure exerted.

Assume the thickness of the base slab as 400mm and provide a projection of 300mm all around the water tank.



Downward load due to self weight of tank:

Long wall = $2 \times 12.64 \times 0.32 \times 4 \times 25$ = 808.36 kNShort wall = $2 \times 5 \times 0.32 \times 4 \times 25$ = 320 kNBase slab = $(12.64 + 0.6).(5.64 + 0.6) \times 0.4 \times 25 = 826.176 \text{ kN}$ Total = 1955.136 kNWeight of earth retained over projection, Long wall = $2 \times (12.54 + 0.6) \times 4 \times 0.3 \times 16 = 508.42 \text{ kN}$ Short wall = $2 \times 5.64 \times 4 \times 0.3 \times 16$ = 216.576 kN

= <u>724.996</u> kN

Total load = 2680.132 kN

Frictional force required = 3635.174 - 2680.132 = 955 kN

Pressure exerted by water at a depth of 4.4m,

$$p = H / 3.(\gamma - \gamma_w) + \gamma_w.H$$
$$= \frac{4.4}{3} (16 - 10) + 10 \times 4.4 = 52.8 \text{ kN/m}^2$$

Considering 1m length of wall, the force exerted is = $\frac{1}{2} \times H \times p$

= ½ x 4.4 x 52.8 = 116.16kN/m

The frictional resistance per metre length of wall = μ .F = 0.15 x 116.16 = 17.424 kN/m For the entire perimeter of the wall, the frictional resistance offered,

= 2 x (12.64 + 5.64) x 17.42 = 637 kN < 955 kN

Hence the downward load due to self weight and cantilever projection of bse slab is insufficient against the uplift force. The length of projection has to be increased. Increase the base projection to 0.7m, all around.

Downward load due to self weight of tank:

Long wall = $2 \times 12.64 \times 0.32 \times 4 \times 12.64 \times 10.32 \times 10^{-10}$	25	= 808.36 kN
Short wall = $2 \times 5 \times 0.32 \times 4 \times 25$		= 320 kN
Base slab = (12.64 + 1.4).(5.64 +	1.4) x 0.4 x 25	= 988.416kN
	Total	= <u>2116.78</u> kN

Weight of earth retained over projection,

Long wall = 2 x (12.54 + 1.4) x 4 x 0.3 x 16 = 1257.98 kN Short wall = 2 x 5.64 x 4 x 0.7 x 16 = 505.34 kN = $\frac{1763.32}{1763.32}$ kN

Total load = **3880.1 kN**

Frictional force required = 4349.03 - 3880 = 469.03 kN < 637 kN

(Frictional force offered by walls)

 $p = 52.8 \text{ kN/m}^2$.

The water tank is safe against uplift.

The base is design as a continuous slab, supported between two short walls, for the self weight and weight of side walls.

Designofbaseslab:

Loading on the base slab includes the self weight of base slab and weight of side walls.

Considering 1m strip,

Weight of base slab = $7.04 \times 0.4 \times 25 = 70.4 \text{ kN/m}$

Weight of walls = $2 \times 4 \times 0.32 \times 25 = 64 \text{ kN/m}$





Reaction at the support = 134.4 x 7.04 / 2 = 473.09 kN B.M. at the support = w.l²/2 = 134.4 x 0.86² / 2 = 49.7 kNm B.M. at centre = 134.4 x 3.522 / 2 - 473.09 (2.66) = -425.8kNm [sagging moment] A_{st} required = $\frac{M}{V_{st}.j.d}$ $\frac{425.8x10^6}{150x0.853x365}$ 9118.2mm²

[D = 400mm, d = 400 – 35 = 365mm]

Provide 25mmø in two layers [both faces].

Provide 25mm @ 100mm c/c on both faces.

YIELD LINE THEORY

<u>Yieldlines</u> – Typical crack patterns – generated when ultimate moment is reached Characteristics of yield lines are,

- i) Yield lines are straight
- ii) Yield lines end at supporting edges of slab
- iii) Yield lines passes through intersection of axis of rotation of adjacent slab elements
- iv) Axis of rotation lies along lines of supports and passes over columns



Assumptions:

The following are the assumptions of the yield line analysis of reinforced concrete slabs.

1. The steel reinforcement is fully yielded along the yield lines at collapse. Rotation following yield is at constant moment.

2. The slab deforms plastically at collapse and is separated into segments by the yield lines. The individual segments of the slab behave elastically.

3. The elastic deformations are neglected and plastic deformations are only considered. The entire deformations, therefore, take place only along the yield lines. The individual segments of the slab remain plane even in the collapse condition.

4. The bending and twisting moments are uniformly distributed along the yield lines. The maximum values of the moments depend on the capacities of the section based on the amount of reinforcement provided in the section.

5. The yield lines are straight lines as they are the lines of intersection between two planes.

Rulesofyieldlines:

The two terms, positive and negative yield lines, are used in the analysis to designate the yield lines for positive bending moments having tension at the bottom and negative bending moments having tension at the top of the slab, respectively.

The following are the guidelines for predicting the yield lines and axes of rotation:

1. Yield lines between two intersecting planes are straight lines.

2. Positive yield line will be at the mid-span of one-way simply supported slabs.

3. Negative yield lines will occur at the supports in addition to the positive yield lines at the mid-span of one-way continuous slabs.

4. Yield lines will occur under point loads and they will be radiating outward from the point of application of the point loads.

5. Yield line between two slab segments should pass through the point of intersection of the axes of rotation of the adjacent slab segments.

6. Yield lines should end at the boundary of the slab or at another yield line.

7. Yield lines represent the axes of rotation.

8. Supported edges of the slab will also act as axes of rotation. However, the fixed supports provide constant resistance to rotation having negative yield lines at the supported edges. On the other hand, axes of rotation at the simply supported edges will not provide any resistance to rotation of the segment.

9. Axis of rotation will pass over any column support, if provided, whose orientation will depend on other considerations.

Upper and Lower Bound Theorems

According to the general theory of structural plasticity, the collapse load of a structure lies in between the upper bound and lower bound of the true collapse load. Therefore, the solution employing the theory of plasticity should ensure that lower and upper bounds converge to the unique and correct values of the collapse load.

The statements of the two theorems applied to slabs are given below:

(A) Lower bound theorem: The lower bound of the true collapse load is that external load for which a distribution of moments can be found satisfying the requirements of equilibrium and boundary conditions so that the moments at any location do not exceed the yield moment.

(B) Upper bound theorem: The upper bound of the true collapse load is that external load for which the internal work done by the slab for a small increment of displacement, assuming that moment at every plastic hinge is equal to the yield moment and satisfying the boundary conditions, is equal to the external work done by that external load for the same amount of small increment of displacement.

Thus, the collapse load satisfying the lower bound theorem is always lower than or equal to the true collapse load. On the other hand, the collapse load satisfying the upper bound theorem is always higher than or equal to the true collapse load.

The yield line analysis is an upper bound method in which the predicted failure load of a slab for given moment of resistance (capacity) may be higher than the true value. Thus, the solution of the upper bound method (yield line analysis) may result into unsafe design if the lowest mechanism could not be chosen. However, it has been observed that the prediction of the most probable true mechanism in slab is not difficult. Thus, the solution is safe and adequate in most of the cases. However, it is always desirable to employ a lower bound method, which is totally safe from the design point of view.

Yieldmoments:

 $m = M_u = 0.87 \text{ x } f_y \text{ x } A_{st} (d - 0.42x_u)$





Methods of yield line analysis:

- 1. Virtual work method
- 2. Equilibrium method (Equilibrium of individual elements of slab along yield line)

<u>Virtualworkmethod</u> – Applied load causing virtual displacement is equal to internal work done or energy dissipated in rotation along the yield lines.

I) Isotropicallyreinforced-squareslab-simplysupported-udl

External work done = $w.\delta$

Where, w \rightarrow load, $\delta \rightarrow$ virtual displacement

Internal work done = $M.\theta = \Sigma m.L.\theta$

 $m \rightarrow Ultimate moment / unit length$

 $L \rightarrow$ Length of yield line

 $M \rightarrow$ Total moment produced along all the yield lines



[Opposite of isotropically reinforced is orthotropically reinforced]

Centre point is the place where the first ultimate moment is reached and the crack originates at this point as, $1/\sqrt{2}$

Tan θ = 1 / (L/√2) = √2 /L
1
Σ(M.θ)_{ac} = Σ(m.L.θ)_{ac} = m. √2 L.
$$\frac{2\sqrt{2}}{L}$$
 = 4m [$\frac{2\sqrt{2}}{\sqrt{2}}$ → since defl. Is on two sides]

Work done by 'bd' is same as 'ac'

Total internal work done = $\Sigma M.\theta$ = 8m

For a virtual displacement of 1 at centre (i.e) CG of each triangular deflects 1/3

L

$$\Sigma W.\delta = [(1/2 \times L \times L/2 \times w) \times 1/3] \times 4 = \frac{wL^2}{3}, \text{ where, } w \rightarrow \text{ udl on slab}$$
$$\Sigma M.\theta = \Sigma W.\delta \rightarrow 8m = \frac{wl^2}{3} \rightarrow m = \frac{wl^2}{24}$$

II) Isotropicallyreinforced-squareslab-fixedonalledges-udl



External work done = $w.\delta$

Where, w \rightarrow load, $\delta \rightarrow$ virtual displacement

Internal work done = $\Sigma M.\theta = \Sigma m.L.\theta = m.\sqrt{2} L.\frac{2\sqrt{2}}{L} = 4m$

Work done by 'bd' is same as 'ac'

Total internal work done = $\Sigma M.\theta$ = 8m

Internal work done by negative yield line (ab, bc, cd, de) for,

 $\Theta = 1 \ / \ L/2 = 2/L, \qquad \Sigma M.\theta = 4 \ [w \ x \ L \ x \ 2/L] = 8m$

Total internal work done = 16m

External work done,

 $\Sigma W.\delta = [(1/2 \times L \times L/2 \times w) \times 1/3] \times 4 = wL^2 / 3$ $\Sigma M.\theta = \Sigma W.\delta$ $16m = wL^2 / 3 \implies m = wL^2 / 48$ M \rightarrow moment per metre length along the yield line



III) Orthotropically reir

For element A,

$$\Theta_{x} = \frac{2}{DL} \qquad \& \qquad \Theta_{y} = 0$$

$$M_{x} = mL$$

$$(M = 0, \pm M, 0, 1) = \frac{2m}{2}$$

$$(\mathsf{M}_{\mathsf{x}}.\ \Theta_{\mathsf{x}} + \mathsf{M}_{\mathsf{y}}.\ \Theta_{\mathsf{y}}]_{\mathsf{A}} = \frac{1}{\mathsf{D}}$$

For element D,

Internal work done,

$$\Theta_{x} = 0, \ \Theta_{y} = 1/(\beta L), \ M_{y} = \alpha . L. \mu m$$

$$\Sigma M. \theta = 2 x \underset{\neg}{\overset{a}{\times} \frac{2m}{D}} \frac{DPm^{o}}{E} \underset{\frac{1}{4}}{\overset{w}{\times}}$$

External work done = $\Sigma w.\delta$

$$\mathsf{m} = \frac{w \mathsf{D}^2 L^2}{24} \left[\sqrt{(3 \mathsf{P} \mathsf{D}^2)} \mathsf{D} \sqrt{\mathsf{P}} \right]^2$$

IV) Orthotropically reinforced – rectangular slab – fixed along long edges - simply supported along short edges – udl

$$\underset{\text{m}}{\text{m}} = \frac{w L^{\frac{2}{x}} \$ \tan^2 \mathbf{I}}{24 \text{ } 0 \text{ } P \text{ } 1}, \quad \text{where, } \tan \mathbf{I} = \$ 1.5P \quad \frac{P^2}{4D^2}; \quad \$ \frac{P}{2}; \quad \ast \frac{P}{2}; \quad \ast$$

 $\mu \rightarrow$ Coefficient of orthotrophy [Ratio between reinforcement provided along shorter direction and longer direction]

V) Orthotropically reinforced – rectangular slab – all four edges fixed – udl

$$\mathsf{m} = \frac{w_u L_y^2}{48} \underbrace{\$ \tan^2 \mathbf{I}}_{0} \cdot \underbrace{\mathsf{m}}_{\mathbf{P}} \cdot \underbrace{\mathsf{m}}_{\mathbf{i}}$$

1. Design a square slab fixed along all four edges, which is of side 5m. The slab has to support a service load of $4kN/m^2$. Use M20 concrete and Fe415 steel. As per IS 456-2000,

l/d = (0.8 x 35) = 28 → 5000 / 28 = d

→ d = 178.6mm = 180mm

Provide D = 200mm

Loading on slab:

Self weight	= 0.2 x 25	$= 5 \text{ kN/m}^2$
Live load		$= 4 \text{ kN/m}^2$
Floor finish		$= 1 \text{ kN/m}^2$
	Total	$= 10 \text{ kN/m}^2$
Factored load (w _u) :	= 1.5 x 10	$= 15 \text{ kN/m}^2$

By yield line theory, $m = \frac{wL^2}{48} = 7.8125 kNm$

Limiting moment, $M_{ulim} = 0.138.f_{ck}.b.d^2 = 0.138 \ x \ 20 \ x \ 1000 \ x \ 180^2 = 89.424 \ x \ 10^6 \ Nmm$ Mu < Mulim

$$\mathbf{K} = \frac{M_{\underline{u}}}{bd^2} = 0.241$$

 $A_{st} = 122 mm^2$

Provide 8mm @ 300mm c/c

1b) In the above problem, design the slab using IS456 coefficient method.

 $I_{y} / I_{x} = 1$

Four edges are discontinuous

 $\alpha_x = \alpha_y = 0.056$ $M_x = \alpha_x.w.l_x^2 = 0.056 \times 15 \times 5^2 = 21 \text{ kNm}$ $M_y = \alpha_y.w.l_x^2 = 0.056 \times 15 \times 5^2 = 21 \text{ kNm}$ → Required spacing of 8mm bar is 140mm. Provide 8mm @ 140mm c/c.

2. Design a square slab of size 5m, simply supported along its four edges and subjected to a live load of $4kN/m^2$.

 $m \quad \frac{wL^2}{24}$

As per IS 456-2000,

 $I/d = (0.8 \times 35) = 28 \Rightarrow 5000 / 28 = d$ ⇒ d = 178.6mm = 180mm
Provide D = 200mm
Loading on slab:
Self weight = 0.2 × 25 = 5 kN/m²
Live load = 4 kN/m²
Floor finish = 1 kN/m²
Total = 10 kN/m²
Factored load (w_u) = 1.5 × 10 = 15 kN/m²
By yield line theory, $m \frac{wL^2}{24}$ 15.625kNm

Limiting moment, $M_{ulim} = 0.138.f_{ck}b.d^2 = 0.138 \ x \ 20 \ x \ 1000 \ x \ 180^2 = 89.424 \ x \ 10^6 \ Nmm$

Mu < Mulim

$$A_{st} = 247.59 \text{mm}^2$$

Provide 8mm @ 200mm c/c

3. Design a rectangular slab of size $4m \times 6m$ simply supported along all its edges, subjected to a live load of $4kN/m^2$. The coefficient of orthotrophy is 0.7. Use M20 and Fe415.

For four edges simply supported condition,

$$\mathsf{m} = \frac{w.\mathrm{D}^2 L^2}{24} \left[\sqrt{(3 \mathrm{PD}^2 \mathrm{D} \sqrt{\mathrm{P}})^2} \right]^2$$

Assume span/depth = 28 → Eff.depth = 4000/28 = 142.86mm

Provide d = 150mm, D = 170mm

Loading on slab:

Self weight= 0.17 x 25= 4.25 kN/m²Live load= 4 kN/m²Floor finish= 0.75 kN/m²

Total =
$$\underline{9kN/m^2}$$

Factored load (w_u) = 1.5 x 9 = 13.5 kN/m²

m =
$$\frac{w.D^2 L^2}{24} \left[\sqrt{(3 PD^2)} D \sqrt{P} \right]^2 = 8.8209 \times 1.602 = 14.13 \text{ kNm}$$

Limiting moment, $M_{ulim} = 0.138.f_{ck}.b.d^2 = 62.1 \text{ kNm}$

Mu < Mulim. The section is under-reinforced.

 $K = Mu/b.d^2 \rightarrow A_{st} = 277 mm^2$ Minimum $A_{st} = 0.12\%$ of c/s = $204mm^2$ A_{st} along longer span = $\mu x A_{st} = 0.7 x 237 = 194 \text{mm}^2 < 204 \text{ mm}^2$ Provide 8mm @ 240mm c/c Checkforshear (factored): $V_u = \frac{W_u l_x}{2} = 13.5 \text{ x } 4 / 2 = 27 \text{ kN}$ $K.\zeta_v = \frac{V_u}{bd} = \frac{27 \times 10^{-3}}{1000 \times 150} = 0.18 \text{ N/mm}^2$ $\zeta_{\rm c}$ for [100A_{st}/bd = (100 x 277)/(1000 x 150) = 0.185N/mm²] $0.15 \rightarrow 0.28$ $0.25 \rightarrow 0.36$ 0.185 → 0.308 As per CI.42.4 of IS456-2000, $K \rightarrow 1.3$ for d = 150 mm $K \rightarrow 1$ for d = 300 mm $K \rightarrow ((1.126 + 0.133) = 1.26)$ for d = 170mm

K. $\zeta_c = 1.26 \text{ x } 0.308 = 0.388 \text{ N/mm}^2 > \zeta_v (0.18 \text{ N/mm}^2)$ Section is safe in shear.

4. In the above problem, design the slab if all the supports are fixed.For four edges simply supported condition,

$$m = \frac{w \cdot L_{y}^{2}}{48} \left[\frac{\tan^{2} \mathbf{I}}{P}\right],$$

where, $\tan \varphi = \frac{\$}{0} 1.5P \quad \frac{P^{2}}{4D^{2}} \cdot \frac{P}{2D}$
 $\mu = 0.7, \ \alpha = 0.66, \ \tan \varphi = 0.8, \ \varphi = 38.65^{\circ}$

Assume span/depth = $28 \rightarrow \text{Eff.depth} = 4000/28 = 142.86\text{mm}$ Provide d = 150mm, D = 170mm Loading on slab:

Self weight = 0.17×25 = 4.25 kN/m^2 Live load = 4 kN/m^2 Floor finish = 0.75 kN/m^2

Total $= 9 \text{ kN/m}^2$ Factored load (w_u) = 1.5 x 9 = 13.5 kN/m² m = $\frac{w.L_{y}^{2}}{48} [\frac{\tan^{2} \mathbf{I}}{\mathbf{p}}] = \frac{13.5x6^{2}}{48} [\frac{0.801}{0.7}]^{2} = 9.28 \text{ kNm}$ Limiting moment, $M_{ulim} = 0.138.f_{ck}.b.d^2 = 62.1 \text{ kNm}$ The section is under-reinforced. $M_u < M_{ulim}$. $K = Mu/b.d^2 \rightarrow A_{st} = 175 mm^2$ Minimum $A_{st} = 0.12\%$ of c/s = $204mm^2$ A_{st} along longer span = $\mu x A_{st} = 0.7 x 175 = 122.5 mm^2 < 204 mm^2$ Provide 8mm @ 240mm c/c bothways [Min. Ast] Checkforshear (factored): {No change} $V_u = \frac{w_u l_x}{2} = 13.5 \text{ x } 4 / 2 = 27 \text{ kN}$ K. $\zeta_v = \frac{V_u}{bd} = \frac{27 \times 10^{-3}}{1000 \times 150} = 0.18 \text{ N/mm}^2$ $\zeta_{\rm c}$ for [100A_{st}/bd = (100 x 277)/(1000 x 150) = 0.185N/mm²] 0.15 → 0.28 0.25 → 0.36 0.185 → 0.308 As per CI.42.4 of IS456-2000, $K \rightarrow 1.3$ for d = 150mm $K \rightarrow 1$ for d = 300mm $K \rightarrow ((1.126 + 0.133) = 1.26)$ for d = 170 mm

K. ζ_c = 1.26 x 0.308 = 0.388 N/mm² > ζ_v (0.18 N/mm²) Section is safe in shear.

5. Solve the above problem if two long edges are fixed.

$$\mathsf{m} = \frac{w \, \mu^2_x \, \$. \tan^2 \mathbf{I}}{24} \frac{1}{0} \frac{1}{\mathbf{P}} \frac{1}{1}$$

Provide 8mm @ 240mm bothways.

Triangularslab:

I) Isotropically reinforced – Equilateral triangular slab – simply supported along all edges – udl

$$\mathsf{M} = \frac{w_u L^2}{72}$$

II) Isotropically reinforced – Equilateral triangular slab – simply supported along two adjacent edges – udl

$$\mathsf{M} = \frac{w_u \mathsf{D}L^2}{6} \operatorname{Sin}^2 \frac{\mathsf{E}}{2}$$

III) Isotropically reinforced – Right angled triangular slab – simply supported along all edges – udl

$$\mathsf{M} = \frac{w_u \mathsf{D} L^2}{6}$$

IV) Isotropically reinforced – Circular slab – simply supported along edges – udl
 Failure takes place by formation of infinite number of positive yield lines running radially
 from centre to circumference, forming a flat cone at collapse.

$$\mathsf{M} = \frac{w_u r^2}{6}$$

6. Design an equilateral triangular slab of side 5m, isotropically reinforced and is simply supported along its edges. The slab is subjected to a superimposed load of 3kN/m2. Use M20 concrete and Fe415 steel.

Assume span/depth = 28 → Eff.depth = 5000/28 = 178.57mm

Provide d = 180mm, D = 200mm

Loading on slab:

Self weight = 0.2×25 = 5 kN/m^2 Live load = 3 kN/m^2 Floor finish = 1 kN/m^2 Total = 9kN/m^2

Factored load $(w_u) = 1.5 \times 9 = 13.5 \text{ kN/m}^2$

m =
$$\frac{w L^2}{72}$$
 = 4.688 kNm

Limiting moment, $M_{ulim} = 0.138.f_{ck}.b.d^2 = 89.42$ kNm Mu < Mulim. The section is under-reinforced.

K = Mu/b.d² → A_{st} = 72.754mm² Minimum A_{st} = 0.12% of c/s = 240mm² A_{st} < min Ast [240 mm²] Provide 8mm @ 200mm c/c <u>Checkforshear</u> (factored): $V_u = \frac{w_u J_x}{2} = 13.5 \times 5 / 2 = 33.75 \text{ kN}$ $K.\zeta_v = \frac{V_u}{bd} = \frac{27 \times 10 =^3}{1000 \times 150} = 0.1875 \text{ N/mm}^2$ ζ_c for [100A_{st}/bd = (100 x 277)/(1000 x 150) = 0.185N/mm²] 0.13 → 0.28 As per Cl.42.4 of IS456-2000, K → 1.3 for d = 150mm K → ((1.04 + 0.2) = 1.24) for d = 180mm

K. $\zeta_c = 1.24 \text{ x } 0.28 = 0.3472 \text{ N/mm}^2 > \zeta_v (0.18 \text{ N/mm}^2)$ Section is safe in shear.

MISCELLANEOUS STRUCTURES [STAIRCASE, FLAT SLAB, RC WALL]

DESIGN OF STAIRCASE

GEOMETRICAL TYPES OF STAIRCASE:





A typical flight in a staircase



Special support conditions for longitudinally spanning stair slabs

- Based on loading and support conditions:
 - Spanning along transverse direction

Cantilever staircase

Slab supported between stringer beams



(b) slab doubly cantilevered from a central spine beam



• Spanning along longitudinal direction



Typical examples of stair slabs spanning longitudinally

Loading on staircase:

Dead load:

- Self weight of slab
- Self weight of step
- Tread finish [0.6 1 kN/m²]

Live load:

For overcrowding \rightarrow 5 kN/m² No overcrowding \rightarrow 3 kN/m²

For independent cantilever state, the following live load condition is also checked:



STAIRCASE SPANNING TRANSVERSELY

1) A straight staircase is made of structurally independent tread slab cantilevered from a RC wall. Given, the riser is 150mm and tread is 300mm with width of flight 1.5m. Design a typical cantilever tread slab. Apply live load for overcrowding. Use M20 concrete and Fe250 steel.

Loading on the staircase (0.3m width)

Dead load: Self weight of tread slab = $25 \times 0.15 \times 0.3 = 1.125 \text{ kN/m}$ Floor finish (0.6 kN/m2) = $0.6 \times 0.3 = 0.18 \text{ kN/m}$ Total = 1.305 kN/mDead load moment, M_D = $1.305 \times 1.5^2 / 2 = 1.468 \text{ kNm}$ Live load: Maximum of,

i) Overcrowding \rightarrow 5 kN/m² L.L. = 5 x 0.3 = 1.5 kN/m M_L = wl²/2 = 1.5 x 1.5² / 2 = 1.69 kNm

1.5m

ii)

M_L = 1.3 x 1.5 = 1.95 kNm

The maximum of the above two values is $M_L = 1.95 \text{kNm}$

Total moment = 1.468 + 1.95 = 3.42 kNm

Factored moment = 5.13 kNm

Effective depth = 150 - (20 + 10/2) = 125 mm

[Slab cover \rightarrow 15mm to 20 mm]

$$M_{u} = 0.87.f_{y}.A_{st} \stackrel{\$}{\longrightarrow} d = 0.42 \stackrel{\$}{\longrightarrow} \frac{0.87.f}{0} \stackrel{A}{\longrightarrow} \stackrel{st}{\longrightarrow} \qquad \text{[OR]} \qquad \mathsf{K} = \frac{M_{u}}{bd^{2}} \Rightarrow \mathsf{Take } \mathsf{p_{t} from SP16}$$

5.13 x
$$10^6 = 65.25 \text{ x } 10^3 \text{.A}_{st} - 9.19 \text{.A}_{st}^2$$

 $\Rightarrow A_{st} = 202 \text{ mm}^2$

No. of bars =
$$\frac{A_{st}}{a_{st}} = \frac{202}{S \cdot 10^2 / 4} = 3$$

3 nos. of 10mm ϕ are provided at the top.

Distribution steel:

MS \rightarrow 0.15% of c/s \rightarrow 0.15/100 x 300 x 150 = 67.5mm²

Spacing = 1000 x $\frac{a_{st}}{A_{st}}$ = 744.67 mm

Provide 8mm @ 300 mm c/c

Check for shear:

$$\zeta_v = V_u / b.d$$

Shear force due to dead load = w.l = $1.305 \times 1.5 = 1.958 \text{ kN}$ Shear force due to live load,

i) 1.5 x 1.5 = 2.25 kN

ii) 1.3 kN

 $V_u = 2.25 + 1.958 = 4.208 \text{ kN}$

Factored Vu = 6.312 kN

As per Cl.40.2.1.1, IS456 – 2000, ζ_c value is modified when thickness is less than 300mm Upto a depth of 150mm, \rightarrow K = 1.3

When depth is > 300mm, \rightarrow K = 1

Development length = $\frac{\mathbf{I}.\mathbf{V}_s}{4\mathbf{W}_{bd}}$ = 453.125 mm

Provide a development length of 450mm

Providing a 90° bent, length required = 450 - 80 = 370mm

$$\zeta_{v} = \frac{V_{u}}{b.d} = 0.168 \text{ N/mm}^{2}$$
For $p_{t} = \frac{100A_{s}}{b.d} = 0.54\%$,
 $\zeta_{c} = 1.3 \times 0.4928 = 0.6406 \text{ N/mm}^{2}$
 $\zeta_{v} < \zeta_{c}$

Hence, safe.



2) Design a waist slab type staircase comprising of a straight flight of steps supported between two stringer beams along the two sides. Assume an effective span 1.5m, a riser of 150mm and tread of 270mm. Assume a live load of 3kN/m². Use M20 concrete and Fe250 steel. Assume mild exposure condition.



S - Stringer beam

The staircase is spanning along the transverse direction.

The main reinforcement should be provided along the transverse direction and distribution steel is provided at the top.

Inclined length of one step = $\sqrt{R^2 T^2}$ = 308.87 mm ~ 309mm

The loading on the slab is found for an inclined width of 309mm, which is later converted for 1m length.

Assume $l/d = 30 \rightarrow 1500/d = 30 \rightarrow d = 50mm$ Assume d = 60mm D = 60 + cover + Bar dia./2 = 60 + 20 + 10/2 = 85mm

Loading on slab over each tread width:

Self weight of slab	= 0.309 x 25 x 0.085	= 0.657 kN/m
Self weight of step	= ½ x 0.15 x 0.27 x 25	= 0.50625 kN/m
Tread finish	= 0.27 x 0.6	= 0.162 kN/m
Live load (3kN/m ₂)	= 0.27 x 3	= 0.81 kN/m
	Total	= <u>2.135kN/m</u>

The load 2.135 kN/m acts vertically downwards. The load acting along the inclined slab is the cosine value of the above.

→ 2.135 x cosθ = 2.135 (270/309) = 1.866 kN/m

The distributed load for 1m step along the inclined slab is $1.86 \times 1/0.309 = 6.02 \text{ kN/m}$ Factored load = 9.03 kN/m

 $M = wl^2/8 = 9.03 \times 1.5^2 / 8 = 2.54 \text{ kNm}$

$$2.54x10^{6} \quad 0.87.250.A_{st} \stackrel{\$}{\sim} 60 \quad 0.42^{\$} \frac{0.87x250xA_{st}}{0.36x20x1000^{11}}$$

→ $A_{st} = 205.72 \text{ mm}^2/\text{m}$

Provide 8mm, spacing required is 240mm c/c

Spacing < [300mm and 3d = 180mm]

Provide 8mm @ 180mm c/c

Distribution steel:

MS \rightarrow 0.15% of c/s

 $= 0.15/100 \times 1000 \times 60 = 90 \text{mm}^2$

Provide 6mm ϕ , spacing required = 314.15mm < (5d = 300mm)

Provide 6mm @ 300mm c/c

STAIRCASE SPANNING LONGITUDINALLY

1) Design the staircase slab shown in figure. The stairs are simply supported on beams provided at the first riser and at the edge of upper landing. Assume a floor finish of 0.8kN/m² and a live load of 5 kN/m². Use M20 concrete and Fe415 steel. Assume mild exposure condition.

L = 3 + 1.5 + (0.15 + 0.15) = 4.8m

Assume L/d as 20,

4.8/d = 20 → d = 240mm

D = 240 + 20 + 10/2 = 265mm

Loading on going slab:

Length of the inclination of one step is $\sqrt{R^2 T^2}$,

Where, R = 150mm, T = 300mm

L = 335.41mm

Self weight of waist slab	= 25 x (0.265 x (0.3354/0.3)	$= 7.4 \text{ kN/m}^2$
Self weight of step	= 25 x ½ x 0.15	= 1.875 kN/m ²
Floor finish		$= 0.8 \text{ kN/m}^2$
Live load		$= 5 \text{ kN/m}^2$
	Total	$= 15 \text{ kN/m}^2$

Loading on going slab:

Self weight of slab	= 25 x 0.265	$= 6.625 \text{ kN/m}^2$
Floor finish		$= 0.8 \text{ kN/m}^2$
Live load		$= 5 \text{ kN/m}^2$
	Total	= 12.425 kN/m ²

Considering 1m strip,

The staircase slab is idealized as given below:



 $\begin{aligned} \mathsf{R}_{\mathsf{A}} \ge 4.8 - (15 \ge 3.45 \ge 3.075) - (12.425 \ge 0.675 \ge 1.35) &= 0 \\ \mathsf{R}_{\mathsf{A}} &= 35.69 \ \mathsf{kN}, \qquad \mathsf{R}_{\mathsf{B}} &= 33.11 \ \mathsf{kN} \end{aligned}$



2) Design a dog legged staircase having a waist slab for an office building for the following data:

i) Height between floor = 3.2m

ii) Riser = 160mm

iii) Tread = 270mm

iv) Width of flight is equal to the landing width = 1.25m

 $LL = 5 \text{ kN/m}^2$, FF = 0.6 N/mm²

Assume the stairs to be supported on 230mm thick masonry walls at the outer edges of the landing parallel to the risers. Use M20 concrete and Fe415 steel.

Note : Based on riser, number of steps is found. Based on tread, length of staircase is found.

No. of steps = 3.2/0.16 = 20

10 numbers of steps are used for first flight and other 10 to the second flight.

Loading on going:

Self weight of waist slab	= 25 x 0.283 x (0.31385/0.270)	= 8.22 kN/m ²
Self weight of step	= 25 x ½ x 0.16	$= 2 \text{ kN/m}^2$
Tread finish		$= 0.6 \text{ kN/m}^2$
Live load		$= 5 \text{ kN/m}^2$
	Total	= <u>15.82</u> kN/m ²

Loading on landing slab:

Self weight of slab = $25 \times 0.283 = 7.075 \text{ kN/m}^2$

I/d = 20 → 5.16/d = 20

→ d = 258mm

D = 258 + 20 + 10/2 = 283mm

Length of inclination of one step,

R = 160mm, T = 270mm, L = 313.85mm

Self weight of slab = $25 \times 0.283 = 7.075 \text{ kN/m}^2$ FF = 0.6 kN/m^2

LL



1.215m
$R_A \times 5.16 - (12.675 \times 1.365 \times 4.4775) - (15.82 \times 2.43 \times 2.58) - (12.675 \times 1.365 \times 0.6875) = 0$ $R_A = 36.54 \text{ kN}$ $R_{B} = 36.51 \text{ kN}$ Maximum moment at centre = 36.5 x 2.58 - (12.675 x 1.365 x (0.6825 x 1.215) - (15.82 x $1.215^{2}/2) = 49.66$ kNm Factored moment = 74.49 kNm $M_{u} = 0.87.f_{y}.A_{st}...d = 0.42: \underbrace{\overset{\$}{0}}_{0} \underbrace{0.87.f_{y}A_{st}}_{0} \cdots \underbrace{0}_{0} \underbrace{0.36.f_{ck}}_{0} b \underbrace{\overset{\$}{1}}_{1} \underbrace{0}_{1}$ b = 1000mm, d = 258mm 74.49 x 10^6 = 93.15 x 10^3 A_{st} - 7.4 A_{st}² $A_{st} = 868.99 \text{ mm}^2$ Provide 12mm φ @ 130mm c/c Distribution steel: 0.15% of c/s = $0.15/100 \times 1000 \times 283 = 424.5 \text{ mm}^2$ 8mm @ 110mm c/c Check for shear: $\zeta_v = V_u/b.d$ Maximum shear force = $[36.5 - (12.675 \times 0.258)] \times 1.5 = 49.84$ kN $\zeta_v = 0.193 \text{ N/mm}^2$ ζ_c: Pt = 100.Ast/b.d = 0.336% For pt = 0.25% $\rightarrow 0.36$ For pt = 0.5% $\rightarrow 0.48$ For pt = 0.336%→ 0.40 $\zeta_{\rm c} = 0.40 \text{ N/mm}^2$ Modification factor, For D = 150mm, \rightarrow 1.3 For D = 300 mm, \rightarrow 1 For D = 283mm, → 1.03 $\zeta_{c \text{ modified}} = 1.03 \text{ x } 0.40 = 0.412 \text{ N/mm}^2$ $\zeta_v < \zeta_c$ Safe in shear.



TREAD RISER STAIRCASE

Actual analysis \rightarrow Theory of plates

I/d limited to 25

Support conditions:

- i) Transverse direction Stair is spanning along transverse direction
- ii) Longitudinal direction
- iii) Stair slab spanning longitudinally and landing slab supported transversely

In Tread – Riser stair span by depth ratio is taken as 25 and the loading on the folded slab comprising the tread and riser is idealized as a simply supported slab with loading on landing slab and going similar to a waist like slab. The loading on folded slab includes,

- i) self weight of tread riser slab
- ii) floor finish
- iii) live load \rightarrow 5 kN/m² (overcrowded), 3 kN/m² (No overcrowding)

Note:

For staircase spanning longitudinally where the landing is supported along the transverse direction only. While finding the effective length along the longitudinal direction only half the length of the landing slab is considered. There is no change in the loading of going slab. But the loading on landing slab is half (waist type and tread-riser type). The landing slab is designed separately for the full load on landing plus half the loading from going slab.

1) Design a dog legged staircase having a tread-riser slab for an office building for the following data:

- i) Height between floor = 3.2m
- ii) Riser = 160mm
- iii) Tread = 270mm
- iv) Width of flight is equal to the landing width = 1.25m
- $LL = 5 \text{ kN/m}^2$, FF = 0.6 N/mm²

Assume the stairs to be supported on 230mm thick masonry walls supported only on two edges perpendicular to the risers. Use M20 concrete and Fe415 steel.

The length of the landing slab is halved while finding the effective length along the longitudinal direction since the staircase is supported only on the landing slab along the transverse direction.

Effective length = 2.43 + 1.25 = 3.68mAssume l/d = 25, 3.68/d = 25→ d = 147.2 mm ~ 150mm D = 150 + (20 + 10/2) = 175mm Loading on the going and landing slab: [Folder Tread Riser] Self weight of tread riser = $25 \times (0.27 + 0.16) \times 0.175/0.27 \times 1 = 6.97 \text{ kN/m2}$ Floor finish = 0.6 kN/m2Live load = 5 kN/m2Total = 12.57 kN/m2 Considering 1m strip, w = 12.57 kN/m Loading on landing slab: Self weight of slab = $25 \times 0.175 = 4.375 \text{ kN/m}^2$ Floor finish $= 0.6 \text{ kN/m}^2$ Live load $= 5 \text{ kN/m}^2$ Total $= 9.975 \text{ kN/m}^2$ Considering 1m strip, w = 9.975 kN/m 50% of load on landing slab is considered along the longitudinal direction. Along the longitudinal direction, the loading is, $R_A \times 3.68 - (4.99 \times 3.3675) - (12.57 \times 1.84 \times 2.43) - (4.99 \times 0.3125 \times 0.625) = 0$ → R_A = 18.39 kN → R_B = 18.39 kN Moment at centre, (i.e. 1.84m), $M_{max} = 18.39 \times 1.84 - (4.99 \times 0.625 \times 1.5275) - (12.57 \times 1.215 \times 0.675)$ = 19.79 kNm Factored moment = 29.69 kNm For b = 1000mm, d = 150mm, $K = M_u/b.d^2$ \rightarrow A_{st} = 598.36 mm² / m Provide 12mm ϕ , spacing required = 189mm Provide 12mm @ 180mm c/c [Main bar as cross links on riser and tread] Distribution steel: 0.12% of c/s = 0.12/100 x 1000 x 175 = 210 mm² Provide 8mm @ 230mm c/c [Dist. bar along the width of stair]

Check for shear: $\zeta_v = V_u/b.d$, $V_u = [18.39 - (4.99 \times 0.15)] \times 1.5 = 26.46$ kN $\zeta_{\rm v} = 0.1764 \ {\rm N/mm^2}$ ζ_c : $100A_{st}/b.d. = 0.3989\%$ For pt = $0.25 \rightarrow$ 0.36 For pt = $0.5 \rightarrow$ 0.48 For pt = $0.39 \rightarrow$ (0.1584 + 0.2688) = 0.427Modification factor (K): For D = 150 mm \rightarrow 1.3 For D = 300mm \rightarrow 1 For D = 175mm 1.08 + 0.17 = 1.25 \rightarrow $\zeta_{c \text{ modified}} = 0.534 \text{ N/mm}^2$ $\zeta_v < \zeta_c$ Hence safe in shear.

Design of landing slab:

The landing slab is designed as a simply supported slab which includes the load directly acting on the landing and 50% of the load acting on the going slab.

The loading on the landing is,

i) Directly on landing

= 9.98 kN/m

ii) 50% of load on going slab = $(12.57 \times 2.43)/2$ = 15.27 kN/m

w = 25.25 kN/m



Providing 12mm ϕ bar, spacing required = 173.9mm

Provide 12mm @ 170mm c/c

Distribution steel:

0.12% of c/s

Provide 8mm @ 230mm c/c

3 numbers of 8mm bars are provided between the cross links as distribution bars. A nominal reinforcement of 10mm @ 200mm c/c is provided at the top of landing slab. Note : Shear in tread riser slab is negligible. Check for shear is not required.

DESIGN OF REINFORCED CONCRETE WALL

- Compression member
- In case where beam is not provided and load from the slab is heavy
- When the masonry wall thickness is restricted
- Classified as
 - plain concrete wall, when rein. < 0.4%
 - $\circ~$ Reinforced concrete wall, when rein. > 0.4%

Load from slab is transferred as axial load to wall. When depth is large, it is called RC wall. Design is similar to a RC column, breadth equal to thickness of wall and depth equal to 1m.

- Axially loaded wall
- Axially loaded with uniaxial bending

General conditions:

Classification of concrete walls:

- 1. Plain concrete wall
- 2. Reinforced concrete wall

In plain concrete wall, the reinforcement provided is less than 0.4% of c/s. In reinforced concrete wall, the percentage of steel provided is greater than 0.4% and is designed similar to reinforced concrete columns. Slenderness ratio is equal to

Least of (l/t or h/t), where,

 $I \rightarrow$ effective length of wall, $h \rightarrow$ effective height of wall, $t \rightarrow$ thickness of wall If $\lambda < 12$, the wall is short and if $\lambda > 12$, the wall is slender.

Braced and Unbraced:

Braced : When cross walls are provided for the walls such that they can take lateral load and 2.5% of vertical load, then the wall is braced. Otherwise, the wall is known as unbraced wall.

Note: Other walls under special cases are,

- i) Cantilever wall
- ii) Shear walls To take lateral loads [Take care of flexure developed due to lateral loading on the structure, depth is provided along the transverse direction]

Guidelines for RC wall:

- 1. The limiting slenderness (λ_{lim}) if any for unbraced wall is 30 and for braced wall is 45.
- 2. For short braced RC wall ($\lambda < 12$), $P_u = 0.4.f_{ck}.A_c + 0.67.f_y.A_{st}$
- For short unbraced RC wall, along with the above axial load Pu, the moment due to minimum eccentricity is checked for e_{min} = t/20 or 20mm, where, M = P x e.
 For the above axial load and moment, the RC wall is designed similar to RC column subjected to axial load and uniaxial moment.
- 4. Slender braced wall ($\lambda < 45$):

The additional moment due to additional eccentricity as per Table 1 of SP16 is considered. Where the additional eccentricity,

$$e_{a} = \frac{L_{e}^{2}}{2000.t}$$
, $M_{a} = P \times e_{a}$

The additional moment due to eccentricity is added with the moment on the column and moment on the wall. The wall is designed for axial load with uniaxial moment.

- 5. For slender unbraced wall [λ limited to 30]: Similar procedure as in case 4 is adopted.
- 6. Detailing of reinforcement [IS456 Guidelines]:
 - a. For plain concrete wall, minimum vertical steel is 0.12% for HYSD bars and 0.15% for mild steel bar
 - b. For RC wall, minimum vertical reinforcement is 0.4% of c/s
 - c. In plain concrete wall, transverse steel is not required
 - d. In RC wall, transverse steel is not required (not less than 0.4%)
 - e. Maximum spacing of bars is 450mm or 3t, whichever lesser
 - f. The thickness of wall in no case should be less than 100mm
 - g. If thickness is greater than 200mm, double grid reinforcement is provided along both the faces.
- 7. BS 8110 guidelines:
 - a. Horizontal reinforcement same as IS456
 - b. Vertical reinforcement not to be greater than 4%
 - c. When compression steel is greater than 2% of vertical reinforcement, horizontal reinforcement of 0.25% for HYSD bars or 0.3% of MS bars are provided. [As per IS456, it is 0.2% for HYSD bars and 0.3% for mild steel bars].

- d. The diameter of transverse bars (horizontal) should not be less than 6mm or $\phi_L/4$.
- 8. Links are provided when the compression steel is greater than 2%. Horizontal links are provided for thickness less than 220mm. Diagonal links are provided when thickness is greater than 220mm. The spacing of links should be less than 2t and diameter of links not less than 6mm or $\varphi_L/4$.

The support conditions:

1. Both ends fixed (Restrained against rotation and displacement)	\rightarrow I _{eff} = 0.65I _o - 0.75I _o
2. Both ends hinged	\rightarrow I _{eff} = I _o
3. One end fixed and other end free	\rightarrow I _{eff} = 2I _o
4. One end fixed and other end hinged	\rightarrow I _{eff} = I _o / $\sqrt{2}$

1) Design a reinforced concrete wall 3m height, 4m length between cross walls. The wall is 100mm thick and carries a factored load of 600 kN/m length. Use M20 concrete and Fe415 steel.

Since cross walls are provided, the wall is braced.

$$\lambda = h/t \text{ or } l/t$$

- = 3000/120 or 4000/120
- = 30 or 40

Assume both ends fixed (restrained against rotation and displacement)

 L_{eff} = 0.75 I_o where, I_o is least of length and height

= 0.75 x 3 = 2.25m

 $\lambda = 2250/100 = 22.5 > 12$

 $\lambda_{\text{lim}} = 45 > 22.5$ $[\lambda < \lambda_{\text{lim}}]$

Accidental minimum eccentricity due to,

e_{min} = t/20 or 20mm = 5 or 20mm

Therefore, moment due to accidental eccentricity of 20mm is considered.

Additional eccentricity due to slenderness,

 $e_{a} = \frac{L_{e}^{2}}{2000t} = \frac{2.25 \times 1000^{2}}{2000 \times 100} = 25.3125 \text{mm}$

Total eccentricity = $e_{min} + e_a = 20 + 25.3 = 45.3 mm$

Moment $M_u = P_u x 45.3 = 27.2 x 10^6 Nmm$

For the axial load and moment, RC wall is designed similar to a RC column subjected to axial load and uniaxial moment.

$$\frac{P_u}{f_{ck}bt} = \frac{600x10}{20x100x1000}^3 = 0.3$$
$$\frac{M_u}{f_{ck}bt^2} = 0.136$$

From SP16 chart, for Reinforcement along two sides, Fe415 steel, d'/D = 0.1, Referring Chart 32,

$$\frac{p}{f_{ck}}$$
 0.07 \Rightarrow p = 0.07 x 20 = 1.4%

Area of steel = $1.4/100 \times 100 \times 1000 = 1400 \text{ mm}^2$

Provide 16mm @ 140mm c/c as vertical compression bar

Horizontal - Provide a nominal transverse reinforcement of 0.4% of c/s

 $Ast = 0.4/100 \times 1000 \times 100 = 400 \text{ mm}^2$

Provide 8mm @ 120mm c/c

Since vertical reinforcement is less than 2%, no horizontal links are required.

2) A reinforced concrete wall of height 5m is restrained in position and direction carrying a factored load of 600 kN and factored moment of 25kNm at right angles to the plane of the wall. Use M20 concrete and Fe415 steel. Design the wall.

Eccentricity for the given moment is, $e = \frac{M_{u}}{P_{u}} = \frac{25x10^{-6}}{600x10^{3}} = 41.67 \text{mm}$

The eccentricity is compared with e_{min} . The larger of the two is added with additional eccentricity due to slenderness, if any.

Assume I/d of 22. [Generally assume I/d from 20 – 25]

d = 5000/22 = 227.27

Assume a thickness of 225mm

 $L_{eff} = 0.75 \text{ x} 5 = 3.75 \text{m}$

 $\lambda_{act} = 3750/225 = 16.67 > 12$

The given wall is slender.

 $e_{min} = t/20 \text{ or } 20mm$

= 225/20 or 20mm

= 11.25mm or 20mm < 41.67mm

Additional eccentricity due to slenderness,

$$e_a = \frac{L_e^2}{2000t} = \frac{3750}{2000x225} = 31.25$$
mm

Total eccentricity = $e_{min} + e_a = 41.67 + 31.25 = 72.92mm$ Moment $M_u = P_u \times 72.92 = 600 \times 1000 \times 72.92 = 43.75 \times 10^6 Nmm$ For the axial load and moment, RC wall is designed similar to a RC column subjected to axial load and uniaxial moment.

$$\frac{P_u}{f_{ck}bt} = \frac{600x10=}{20x100x225}^3 = 0.13$$

$$\frac{M_u}{f_{ck}bt^2} = 0.0432$$

From SP16 chart, for Reinforcement along two sides, Fe415 steel, d'/D = 0.1, Referring Chart 32,

$$\frac{-p}{f_{ck}} = 0$$
 No reinforcement is required.

But minimum reinforcement of vertical compression steel of 0.4% is provided.

Area of steel = $0.4/100 \times 225 \times 1000 = 900 \text{ mm}^2$

Provide 12mm @ 120mm c/c as vertical compression bar

Since thickness of wall is 225mm, reinforcement is provided on both faces of the wall.

Therefore, provide 12mm @ 250mm c/c < 3t and 450mm

Horizontal - Provide a nominal transverse reinforcement of 0.4% of c/s on both faces.

 $Ast = 0.4/100 \times 1000 \times 100 = 400 \text{ mm}^2$

Provide 8mm @ 120mm c/c

Since vertical reinforcement is less than 2%, no horizontal links are required.

3) In the above problem, design the wall for factored axial load of 1000kN and factored moment of 50kNm.

 $\begin{array}{ll} P_u = 1000 \text{kN}, & M_u = 50 \text{kNm} \\ e = 50 \ x \ 10^6 / 1000 \ x \ 10^3 = 50 \text{mm} \\ \text{I/d} &= 22 \quad [\text{Generally I/d taken from } 20 - 25] \\ d &= 5000 / 22 = 227.27 \text{mm} \\ \text{Adopt a thickness of } 225 \text{mm}. \\ \text{I}_{eff} = 0.75 \ x \ 5000 = 3750 \text{mm} \\ \lambda_{act} = 3750 / 225 = 16.67 > 12 \\ \lambda_{min} = 45 > 16.67 \\ \text{Wall is slender.} \end{array}$

 $e_{min} = t/20 \text{ or } 20mm$

= 225/20 or 20mm

= 11.25mm or 20mm < 41.67mm

Additional eccentricity due to slenderness,

$$e_{a} = \frac{L_{e}^{2}}{2000t} = \frac{3750}{2000x225}^{2} = 31.25$$
mm

Total eccentricity = e_{min} + e_a = 50 + 31.25 = 81.25mm

Moment $M_u = P_u \ x \ 81.25 = 1000 \ x \ 1000 \ x \ 81.25 = 81.25 \ x \ 10^6 \ Nmm$

For the axial load and moment, RC wall is designed similar to a RC column subjected to axial load and uniaxial moment.

$$\frac{P_u}{f_{ck}bt} = \frac{1000 \times 10^{-3}}{20 \times 100 \times 225} = 0.22$$
$$\frac{M_u}{f_{ck}bt^2} = 0.0802$$

From SP16 chart, for Reinforcement along two sides, Fe415 steel, d'/D = 0.1, Referring Chart 32,

$$\frac{p}{f_{ck}}$$
 0.03 \rightarrow p = 0.03 x 20 = 0.6%

Area of steel = $0.6/100 \times 225 \times 1000 = 1350 \text{ mm}^2$

Provide 16mm @ 140mm c/c as vertical compression bar

Horizontal - Provide a nominal transverse reinforcement of 0.4% of c/s

Area of steel = $0.4/100 \times 225 \times 1000 = 900 \text{ mm}^2$

Since thickness of wall is 225mm, reinforcement is provided on both faces of the wall.

Therefore, provide 12mm @ 250mm c/c < 3t and 450mm

Since vertical reinforcement is less than 2%, no horizontal links are required.

DESIGN OF FLAT SLAB

- a) Interior panel
- b) Exterior panel

Various components of flat slab:

- i) Without drop and head
- ii) With drop and without head
- iii) With drop and head

Column strip : It is the design strip having a width of $I_2/4$, where I_2 is the span transverse to I_1 . I_2 – longer span, moment is considered along the span I1

Middle strip : It is the design strip bounded by a column strip on its opposite sides

Proportioning of flat slabs:

As per cl.31 of IS456-2000, the span by depth ratio of two way slab is applicable for flat slabs and the values can be (I/d)modified by 0.9 for flat slabs with drops.

Take \rightarrow I/d as 32 for HYSD bars

As per ACI – The drop thickness should not be less than 100mm or (Thickness of slab)/4.

While calculating span by depth ratio, longer span is used.

The thickness of slab should not be less than 125mm.

The purpose of column drop is to reduce the shear stress and also reduce the reinforcement in the column strip.

The increase in column diameter at the head flaring of column head takes care of punching shear developed at a distance of d/2 all around the junction between the slab and column head.

Two methods of design are available for flat slabs:

- 1. Direct design method
- 2. Equivalent frame method

Direct design method: (CI.31.4.1, IS456-2000)

Requirements for direct design methods are,

- 1. There must be atleast three continuous spans in each direction
- 2. The panels should be rectangular with $ly/lx = l_2/l_1$ ratio < 2

- 3. The columns must not offset by more than 10% of the span from either of the successive columns
- 4. Successive span length in each direction must not differ by more than one third of longer span.
- 5. Design live load must not exceed 3 times the designed dead load

Design procedure:

As per Cl.31.4.2.2, IS456-2000, the total moment for a span bounded by columns laterally is $M_o = WI_o/2$, where M_o is the sum of positive and negative moment in each direction. W is the total design load covered on an area L_2L_1

 $W = w \times L_2 \times L_n$

This moment is distributed for the column strip and middle strip

Moment distribution for Interior Panel:

	Column strip	Middle strip
Negative moment (65%)	65 x 0.75 = 49%	65 x 0.2 = 15%
Positive moment (35%)	35 x 0.6 = 21%	35 x 0.4= 15%

$$M_{o} = \frac{W}{L_{\underline{o}}}$$

Also L_o should not be less than 0.65 times of $L_1[L_n > 0.65L_1)$

1) Design a flat slab system (interior panel) to suit the following data:

Size of the floor = $20 \times 30m$

Column interval = 5m c/c

Live load on slab = $5kN/m^2$

Materials used are Fe415 HYSD bars and M20 concrete



Proportioning of flat slab:

Assume I/d as 32, → d = 5000/32 → d = 156.25mm

d = 175mm (assume), D = 175 + 20 + 10/2 = 200mm

As per ACI code, the thickness of drop > 100mm and > (Thickness of slab)/4

Therefore, 100mm or 200/4=50mm

Provide a column drop of 100mm

Overall depth of slab at drop = 200 + 100 = 300mm

Length of the drop > L/3 = 5/3 = 1.67m

Provide length of drop as 2.5m. For the panel, 1.25m is the contribution of drop.

Column head = L/4 = 5/4 = 1.25m

$$L_1 = L_2 = 5m$$

$$L_n = L_2 - D = 5 - 1.25 = 3.75m$$

As per code,
$$M_o = \frac{WL_o}{8}$$



Loading on slab: (Average thickness = (300 + 200)/2 = 250mm) Self weight of slab = 25×0.25 = 6.25 kN/m^2 Live load = 5 kN/m^2 Floor finish = 0.75 kN/m^2 Total = 12 kN/m^2

Factored load = $1.5 \times 12 = 18 \text{ kN/m}^2$

 $W = w_u \times L_2 \times L_n = 18 \times 5 \times 3.75 = 337.5 \text{ kN}$

Total moment on slab panel = (337.5 x 3.75)/8 = 158.203 kNm

Distribution of moment:

	Column strip	Middle strip
Negative moment (65%)	65 x 0.75 = 49%	65 x 0.2 = 15%
	0.49 x 158.2 = 77.52kNm	0.15 x 158.2 = 23.73kNm
Positive moment (35%)	35 x 0.6 = 21%	35 x 0.4= 15%
	0.21 x 158.2 = 33.22 kNm	0.15 x 158.2 = 23.73kNm

Check for depth adopted:

Column strip:

$$\begin{split} M_u &= 0.138.f_{ck}.b.d^2 & b = 2.5m \\ 77.5 \ x \ 10^6 &= 0.138 \ x \ 20 \ x \ 2.5 \ x \ 1000 \ x \ d^2 \end{split}$$

→ d = 105.98mm ~ 106mm < 275mm

Middle strip:

$$M_u = 0.138.f_{ck}.b.d^2$$
 $b = 2.5m$

 $23.73 \times 10^6 = 0.138 \times 20 \times 2.5 \times 1000 \times d^2$

→ d = 58.68mm ~ 59mm < 175mm

Check for punching shear:

The slab is checked for punching shear at a distance of d/2 all around the face of the column head. The load on the slab panel excluding the circular area of diameter (D + d) is the punching shear force.



Critical section for shear in a flat slab

=
$$18 \times 5 \times 5 - (\pi(D + d)^2/4) \times w_n$$

= 417.12 kN

Shear force along the perimeter of the circular area = $\frac{ShearForce}{S(D \ d)}$ = 87.06 kN

Nominal shear stress: (b = 1m) $\varsigma_v = \frac{V_u}{h.D} = \frac{87.06x10^3}{1000x275} = 0.317 \text{ N/mm}^2$ Design shear stress: $\varsigma_c = K.\varsigma_c$ ' Where, $K = (0.5 + \beta) \le 1$ = 1.5 ≤ 1 ς_{c} ' = 0.25. $\sqrt{f_{ck}}$ = 1.118 N/mm² $\varsigma_c = 1 \times 1.118 = 1.118 \text{ N/mm}^2$ $\zeta_v < \zeta_c$ Safe in shear. **Reinforcement:** Column strip: (b=2.5m), (d = 275mm)Negative moment = 77.5×10^6 Nmm $M_{u} = 0.87.f_{y}.A_{st}...d = 0.42 \stackrel{\$}{...} \frac{0.87.f}{0} \frac{A}{0.36.f_{ck}} \stackrel{st}{...} \stackrel{st}{...}$ [OR] $K = \frac{M_u}{hd^2} \rightarrow \text{Take } p_t \text{ from SP16}$ 77.6 x 10^6 = 99.29 x 10^3 .A_{st} - 3.04.A_{st}² → $A_{st} = 800.16 \text{ mm}^2$ Required 10mm @ 240mm c/c Min A_{st}: 0.12% of c/s = 0.12/100 x 1000 x 275 = 825 mm² Provide 10mm @ 230mm c/c Positive moment = 33.2 kNm $A_{st} = 337.876 \text{ mm}^2$ Provide 8mm @ 370mm c/c Min. steel: Provide 10mm @ 230mm c/c Middle strip: (b = 2.5m), (d = 175mm)Negative and positive moment: 23.7 kNm $A_{st} = 382.6 \text{ mm}^2$

 $A_{st min.} = (0.12/100 \text{ x } 1000 \text{ x } 2500 \text{ x } 175) = 525 \text{ mm}^2$



FLAT SLAB [EXTERIOR PANEL](CI31.4.3.3, IS456-2000)Stiffness of slab and column = $\frac{4EI}{L}$,where, I = bd^3/12 (or) $\pi d^4/64$,E = 5000 $\sqrt{f_{ck}}$ α_c is checked with α_c min given in Table 17 of IS456.From Cl.31.4.3.3, the interior and

Interior negative design moment is,

0.75 $\frac{0.10}{1 \frac{1}{D_c}}$ where, $\alpha_c = \frac{\mathbf{I} K_c}{K_s}$

exterior negative moments and the positive moments are found.

Interior positive design moment is,

$$\begin{array}{c} 0.63 \quad \frac{0.28}{1 \quad \frac{1}{D_c}} \end{array}$$

Exterior negative design moment is,

 $\frac{0.65}{1 \frac{1}{D_c}}$

The distribution of interior negative moment for column strip and middle strip is in the ratio 3:1 (0.75 : 0.25)

The exterior negative moment is fully taken by the column strip. The distribution of positive moment in column strip and middle strip is in the ratio 1.5 : 1 (0.6 : 0.4).

Design an exterior panel of a flat slab floor system of size 24m x 24m, divided into panels 6m x 6m size. The live load on the slab is 5 kN/m^2 and the columns at top and bottom are at diameter 400mm. Height of each storey is 3m. Use M20 concrete and Fe415 steel.

I/d = 32 → d = 6000/32 = 187.5 mm Length of drop ≥ 3m Length of drop = Column strip = 3m Assume effective depth, d = 175mm, D = 200mm As per ACI, Assume a drop of 100mm Depth of slab at the drop is 300mm Diameter of column head = I/4 = 6/4 = 1.5m Loading on slab:

 $= 6.25 \text{ kN/m}^2$ Self weight of slab = $(0.2 + 0.3)/2 \times 25 = 0.25 \times 25$ $= 5 \text{ kN/m}^2$ Live load $= 0.75 \text{ kN/m}^2$ Floor finish $= 12 \text{ kN/m}^2$

Total

Factored load

To find the value of $\alpha_c = -\frac{1}{K_c}$

as per Cl.3.4.6.,

 α_c = flexural stiffness of column and slab

 K_{s}

 ΣK_c = summation of flexural stiffness of columns above and below

 ΣK_s = summation of flexural stiffness of slab

$$\begin{split} \Sigma \mathsf{K}_{c} &= 2 \frac{\$}{9} \frac{4EI}{L} \stackrel{\cdot}{_{1}} = 2 \frac{\$}{9} \frac{4xExI}{L_{c}} \stackrel{\cdot}{_{1}} = \frac{2x4x22.3606x10^{3}x1.25x10^{9}}{3000} \\ \text{Where, I} &= \pi d^{4}/64 = \pi \times 400^{4}/64, \qquad \mathsf{E} = 5000 \sqrt{f_{ck}} = 22.3606 \times 10^{3} \\ \Sigma \mathsf{K}_{s} &= \frac{4xEx6000x(250)^{3}}{12x6000} = 5.208 \times 10^{6}\mathsf{E} \\ \mathfrak{a}_{c} &= 0.644 \\ \text{From Table 17 of IS456-2000} \\ \mathfrak{a}_{c \min} &= L_{2}/L_{1} = 6/6 = 1 \\ L_{L} / D_{L} &= \frac{5}{(6.25 \ 0.75)} = 0.71 \sim 1 \\ \mathfrak{a}_{c \min} &= 0.7 \\ \mathfrak{a}_{c \min} &= 0.7 \\ \mathfrak{a}_{c \min} &= 0.7 \end{split}$$

Total moment on slab = $\frac{W.L_n}{8}$ = 273.375 kNm

$$W = w_u \times L_2 \times L_n = 18 \times 6 \times 4.5 = 486 \text{ kN}$$

 $L_n = 6 - 1.5 = 4.5m$

As per Cl.31.4.3.3 of IS456-2000,

Exterior negative design moment is,

 $\frac{0.65}{1 \quad \frac{1}{D_c}} \quad x \text{ M}_o = \frac{73.168}{1 \text{ M}_c} \text{ kNm} \qquad \text{ where, } \alpha_c = 0.7$

Interior negative design moment is,

0.75
$$\frac{0.10}{1 \quad \frac{1}{D_c}} \times M_o$$
 where, $\alpha_c = \frac{\mathbf{I} K_c}{K_s}$

= 193.775 kNm

For column strip (75%),

For middle strip (25%),

Interior positive design moment is,

0.63
$$\frac{0.28}{1 D_c} \times M_0$$

= 140.708 kNm

For column strip (60%),

= 0.6 x 140.708 = <u>84.43</u> kNm

For middle strip (40%),

Check for depth:

 $M_{ulim} = 0.138 \text{ fck.b.d}^2$

→ 145.331 x 106 = 0.138 x 20 x 3 x d²

→ d_{cs} = 132.484 mm < 275 mm

$$M_{ms} = 82.4 \text{ kNm}$$

→ d_{ms} = 82.4 mm < 175 mm

Check for punching shear:

$$\begin{aligned} \text{SF} &= \text{TL} - (\text{Load on circular area}) \\ &= 18 \times 6 \times 6 - [\pi (1.775)^2 / 4] \times 18 \\ &= 648 - 44.54 = 603.45 \text{ kN} \end{aligned} \qquad \begin{bmatrix} \text{W}_{\text{n}} &= 18 \end{bmatrix} \\ & [\text{D} + \text{d} &= 1.5 + 0.275 = 1.775\text{m}] \end{aligned}$$

Shear force/m along the perimeter of the circular area = $\frac{SF}{S(D - d)}$ = 108.216 kN/m

Nominal shear stress = $\zeta_v = \frac{V_u}{b.d} = \frac{108.216 \times 10^3}{1000 \times 275} = 0.394$ N/mm2 Design shear stress: $\zeta_c = K.\zeta_c$, where, $K = (0.5 + \beta) \le 1$ $= (0.5 + 6/6) \le 1$ $= 1.5 \le 1 \implies K = 1$ $\zeta_c = 0.25 \sqrt{f_{ck}} = 1.118$ N/mm² $\zeta_v < \zeta_c$

Section is safe in shear.

Ast for exterior negative moment (73.168 kNm), b = 3000mm, d = 275mm,

73.168 x 10^6 = 99.28 x 10^3 .A_{st} - 2.53.A_{st}²

→ $A_{st} = 751.373 \text{ mm}^2$

Required 10mm @ 310mm c/c

Min A_{st}: 0.12% of c/s = 0.12/100 x 3000 x 275 = 990 mm²

Provide 10mm @ 230mm c/c

Similarly the reinforcement required in CS and MS for -ve and +ve moments are found and listed below:

Location	A _{st} Req.	Min. A _{st}	A _{st} Provided	Rein. Provided
Extve Mom. CS	751	990	990	10 @ 230 c/c
Int. –ve Mom. CS	1522	990	1522	10 @ 150 c/c
Int. –ve Mom. MS	791	630	791	10 @ 290 c/c
+ve Mom. CS	869	990	990	10 @ 230 c/c
+ve Mom. MS	925	630	925	10 @ 250 c/c



DESIGN OF BRICK MASONRY

[IS1905-1987] - Reaffirmed 1998Brick
(Comp. strength \rightarrow 3.5 to 40 N/mm²)
Clay
Clay
Calcium Silicate (Cement, FAL-G)

Used for,

- 1. External and internal bearing walls
- 2. Load bearing piers and columns
- 3. Paritition walls
- 4. Brick masonry foundations
- 5. Floorings and Pavings

Advantages of brick masonry:

Attractive appearance, economical light weight, durable, strength, fire resistance, sound insulation, low thermal conductivity, minimum maintenance.

Classificationofbricks:[Basedonshapes]

1. Solid bricks - Perforations or holes not greater than 25% of volume

- 2. Perforated bricks Perforation is greater than 25% of volume. Advantages:of perforated bricks are high thermal insulation and light weight. Water absorption should not be greater than 15% after 24 hours of insertion and compressive strength not less than 7N/mm².
- 3. Hollow blocks Holes greater than 20% and sizes of holes greater than 20mm.
- 4. Cellular bricks Holes greater than 20% and closed at one end

5. Ornamental bricks – Bricks used in corbels, cornices, etc.

Size of bricks: [As per IS1077] Standard size – 19 x 9 x 9 cm Modular brick – 20 x 10 x 10 cm

The average compressive strength of brick unit as per IS3495 (Part I) - 1976 is,

3.5 - 40N/mm².

Testsonbricks:

1. Water absorption: Brick units immersed in water for 24 hours has,

i) upto 12.5 N/mm² strength and water absorption should not be greater than 20%

ii) for higher classes, water absorption should not be greater than 15%

2. Efflorescence: Leaching of water soluble salts (white coloured) under efflorescence.

Test for efflorescence is done as per IS3495 (Part III) – 1976. The brick is kept in a dish with water height as 25mm and the time for water absorption and evaporation is noted. This value is compared with the same dish with 25mm water height kept for evaporation alone. Based on the code, efflorescence in brick is reported as nil, slight, moderate, heavy and serious.

3. Hardness: For the brick to be hard, it should create no impression by finger nail.

4. Soundness: When two bricks are struck, it should not break and produce a clear ringing sound.

5. Compressive test: 3.5 - 40 N/mm²

6. Flexure test: Rarely done

Classification of brick based on structure and usage:

- 1. Solid wall
- 2. Cavity wall
- 3. Faced wall
- 4. Veneered wall

Based on loading, walls are classified as,

- 1. Axially loaded walls [Load applied at centre t/2]
- 2. Eccentrically loaded walls
- 3. Laterally loaded walls [Loading applied at sides]

Design procedure:

Slenderness (Least of $l_e/t \& h_e/t$) : ($\lambda_{max} = 27$)

(λ = 60 for RC columns, λ = 45 & 30 for braced and unbraced RC walls)

1. Actual stress on the brick masonry wall is found based on the load from slab and self weight of wall.

2. The permissible compressive stress for masonry based on the type of mortar and compressive strength of brick unit is taken from Table 8, IS1905-1987. This table is valid for slenderness ratio $\lambda \le 6$ and eccentricity e = 0.

3. Corrections are applied for slenderness ratio, eccentricity (if any), shape and size of brick unit. Shape modification factor and cross sectional area of masonry (area reduction factor). 4. Slenderness ratio is found as the least of l_e/t or h_e/t , where, (l_e = Effective length and h_e = effective height).

 Effective length is found from Table 5, IS1905 – 1987 and effective height is found from

 Table4,IS1905-1987.

 Table4-Effectiveheight:

 Supportcondition

 Effectiveheight

Fixed – Fixed	0.75H
Fixed – Hinged	0.85H
Hinged – Hinged	Н
Fixed – Free	1.5H

The permissible value of λ is 27 (λ_{max}) for cement mortar (OPC & PPC), given in Table 7, IS1905-1987.

5. Eccentricity of loading is determined (for axial loading e = 0). Eccentricities for various other cases are to be checked as per Appendix B of IS1905-1987.

6. For the permissible stress adopted, shape modification factor is found based on height to width ratio of each brick unit given in Table 10, IS1905-1987.

7. Area reduction factor is applied for elements having cross section less than $0.2m^2$. The area modification factor, k = 0.7 + 1.5 A

8. After applying modification factors, the actual stress is verified with a modified permissible stress, $\sigma_{act} < \sigma_{per}$

The permissible stress (strength of the wall) depends upon the following factors:

- i) Compressive strength of masonry unit
- ii) Compressive strength of mortar used
- iii) Slenderness ratio of the wall
- iv) Eccentricity in loading
- v) Shape and size of brick unit
- vi) Cross sectional area of masonry

1. Design an interior cross wall for a two storeyed building to carry 100mm thick RC slab with 3m storey height. The wall is unstiffened and supports 2.65m wide slab. Loading on the slab is given as below:

- i) Live load on floor slab = 2 kN/m^2
- ii) Live load on roof slab = 1.5 kN/m^2
- iii) Floor finish = 0.2 kN/m^2
- iv) Roof finish = 1.96 kN/m^2



Assume the compressive strength of brick as 10N/mm² and mortar type as M1.

The loading on the wall includes the load from slab (LL + DL) and self weight of the wall. Assuming the wall thickness as 100mm and size of each masonry unit as 200 x 100 x 90mm,

Loading on slab:

Live load:

on floor slab = 2 kN/m^2 on roof slab = 1.5 kN/m^2

Dead load:

Floor finish = 0.2 kN/m^2

Roof finish = 1.96 kN/m^2

Self weight of slabs = $2 \times 0.1 \times 25 = 5 \text{ kN/m}^2$

Load from slab = 10.66 kN/m^2

For 2.65m length of slab,

Load from slab = $10.66 \times 2.658 = 28.36 \text{ kN/m}$ Self weight of masonry = $2 \times 0.1 \times 20 \times 3 = 12 \text{ kN/m}$ Total = 40.36 kN/m

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength

10 N/mm² is taken from Table 8, IS 1905 – 1987.

Permissible stress = 0.96 N/mm^2

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per CI.5.4.

Stress reduction factor (K_{st})

Slenderness ratio (Least of le/t & he/t)

From Table 4,

 $h_e = 0.75 H = 0.75 x 3 = 2.25m$ [Both ends fixed]

 $h_e/t = 2.25 / 0.1 = 22.5 < 27$

Therefore, the stress reduction factor from Table 10 for $\lambda = 22.5$ and no eccentricity condition is,

For 22 \rightarrow 0.56 (e = 0) For 24 \rightarrow 0.51 For 22.5 \rightarrow 0.55 $K_{st} = 0.55$ Area reduction factor (K_A)

[Cl.5.4.1.2, IS1905-1987]



A = 0.1 x 1 = $0.1m^2 < 0.2 m^2$ K_A = 0.7 + (1.5 x 0.1) = <u>0.85</u>

Shape modification factor (K_{sh})

[Cl. 5.4.1.3, IS1905-1987]

Ksh for block of size $200 \times 100 \times 90$ mm laid along 100mm side, from Table 10 for Height to Width ratio of 90×100 ,

$$\frac{Height}{Width} = \frac{90}{100} = 0.9$$

For $0.75x_0 \rightarrow 1$ For $1x_0 \rightarrow 1.1$ For $0.9x_0 \rightarrow 1.06$ $K_{sh} = 1.06$ $\sigma_{per \ modified} = K_{st}.K_A.K_{sh}. \sigma_{per}$ $= 0.55 \times 0.85 \times 1.06 \times 0.96 = 0.48 \ N/mm^2$

 $\sigma_{\text{act /m}} = \frac{40.36x10^3}{100x1000} = 0.4036 \text{ N/mm}^2 < \sigma_{\text{per}} [0.48 \text{ N/mm}^2]$

Hence the adopted thickness of 100mm with M1 mortar and masonry unit with compressive strength 10N/mm² is safe in carrying the load from slab.

2. In the above problem, design the wall if it is continuous and stiffened by cross wall of 100mm thickness and length of the wall being 3.6m.



Here, $S_p = 3.6m$

Loading on the masonry wall = 40.36 kN/m

Actual stress = $\frac{40.36x1000}{100x1000}$ = 0.4036 N/mm²

 σ_{per} for M1 mortar and masonry unit of compressive strength 10 N/mm² with 100mm thickness.

Permissible stress = 0.96 N/mm^2 [From Table 8, IS1905 - 1987]

Slenderness ratio, $\lambda \rightarrow$ Least of H_e/t & L_e/t

 $H_e = 0.75 H = 0.75 x 3 = 2.25 m$

 $L_e = 0.8L = 0.8 \times 3.7 = 2.96m$ [From Table 5, IS1905 - 1987]

For the cross walls provided, stiffening coefficients are found from Table 6, IS1905 – 1987.

 $t_p \rightarrow Thickness of pier$

[for cross walls] $t_p = 3tw$ cl.4.6.3, IS1905-1987 $S_{D} = 3.7m$, $[S_p \rightarrow c/c \text{ spacing of pier}]$ $t_{w} = 0.1m$ $[t_p \rightarrow thickness of pier]$ $[t_w \rightarrow thickness of wall]$ $t_p = 3tw = 0.3m$ $[w_p \rightarrow width of pier]$ $w_{p} = 0.1m$ cl.4.5.3, IS1905-1987 $\frac{t_p}{t_w} = \frac{0.3}{0.1} = 3,$ $\frac{S_p}{w_p} = \frac{3.7}{0.1} = 37$ From Table 6, for $\frac{S_p}{w_p} = 37$, $\frac{t_p}{t_w} = 3$ $S_e = 1$ Thickness of wall = $1 \times 0.1 = 0.1 \text{m}$ [Considering stiffness] $\lambda = 2.25 / 0.1 = 22.5 < 27$ The stress reduction factor (K_{st}) for Table no.10 for $\lambda = 22.5$ with no eccentricity (e=0) condition, For $\lambda = 22$ 0.56 For $\lambda = 24$ 0.51 For $\lambda = 22.5$ 0.55 $K_{st} = 0.55$ Area reduction factor for area = $0.1 \times 1 = 0.1 \text{m}^2 < 0.2 \text{m}^2$, $KA = 0.7 + (1.5 \times 0.1) = 0.85$ Shape modification factor: [CI.5.4.1.3] Ksh for block size of 200 x 100 x 90mm laid along 100mm side from Table 10 for height to

 $\frac{Height}{Width} = \frac{90}{100} = 0.9$ For H_t/W = 0.75 1 For H_t/W = 1 1.1 For H_t/W = 0.9 1.06 K_{sh} = 1.06 $\sigma_{per \ modified} = K_{st}.K_{A}.K_{sh}. \sigma_{per}$ = 0.55 x 0.85 x 1.06 x 0.96 = 0.48 N/mm² > σ_{act}

width ratio of 90 x 100mm,

Provided masonry wall of thickness <u>100mm</u> with <u>M1</u> mortar and compressive strength of each unit <u>10N/mm²</u> is safe.

3. Design an interior cross wall for a two storeyed building to carry 100mm thick RC slab. Check the safety of the wall if the wall is continuous and cross wall is available on only one side and the storey height is 3.5m. The wall supports 2.65m wide slabs on both sides. Loading on the slab is given as below:

- i) Live load on floor slab = 2 kN/m^2
- ii) Live load on roof slab = 1.5 kN/m^2
- iii) Floor finish = 0.2 kN/m^2
- iv) Roof finish = 1.96 kN/m^2



Assume the compressive strength of brick as 10N/mm² and mortar type as M1. Loading on slab:

Live load:

on floor slab = 2 kN/m^2

on roof slab = 1.5 kN/m^2

Dead load:

Floor finish = 0.2 kN/m^2

Roof finish = 1.96 kN/m^2

Self weight of slabs = $2 \times 0.1 \times 25 = 5 \text{ kN/m}^2$

Load from slab = 10.66 kN/m^2

For 2.65m length of slab,

Load from slab = 10.66 x 2.658 = 28.36 kN/m

Self weight of masonry = $2 \times 0.1 \times 20 \times 3.5 = 14 \text{ kN/m}$

Total = 42.36kN/m

 $\sigma_{act \ \text{/m}} = \frac{42.36 x 10^3}{100 x 1000} = 0.4236 \text{ N/mm}^2$

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength

10N/mm² is taken from Table 8, IS 1905 - 1987.

Permissible stress = 0.96 N/mm^2

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per CI.5.4.

Stress reduction factor (K_{st})

Slenderness ratio (Least of Ie/t & he/t)

From Table 4,

Effective height, $h_e = 0.75 \text{ H} = 0.75 \text{ x} 3.5 = 2.625 \text{m}$ [Both ends fixed]Effective length,le = 1.5 L = 1.5 x 3.65 = 5.475 m[One end fixed, other end free] $h_e/t = 2.625 / 0.1 = 26.25 < 27$

Therefore, the stress reduction factor from Table 10 for λ = 26.25 and no eccentricity condition is,

For $26 \rightarrow 0.45$ (e = 0) For $27 \rightarrow 0.43$ For $26.25 \rightarrow 0.3375 + 0.1075 = 0.445$ <u> $K_{st} = 0.445$ </u>

Area reduction factor (K_A) [Cl.5.4.1.

[Cl.5.4.1.2, IS1905-1987]



A = 0.1 x 1 = $0.1m^2 < 0.2 m^2$ <u>K_A</u> = 0.7 + (1.5 x 0.1) = <u>0.85</u>

Shape modification factor (K_{sh}) [Cl. 5.4.1.3, IS1905-1987]

 K_{sh} for block of size 200 x 100 x 90 mm laid along 100mm side, from Table 10 for Height to Width ratio of 90 x 100,

```
\frac{Height}{Width} = \frac{90}{100} = 0.9
For 0.75x_o \rightarrow 1
For 1x_o \rightarrow 1.1
For 0.9x_o \rightarrow 1.06
```

$$\begin{split} \sigma_{\text{per modified}} &= K_{\text{st}}.K_{\text{A}}.K_{\text{sh}}. \ \sigma_{\text{per}} \\ &= 0.445 \ x \ 0.85 \ x \ 1.06 \ x \ 0.96 = 0.385 \ \text{N/mm}^2 \\ &< \sigma_{\text{act /m}} \ [0.4236 \ \text{N/mm}^2] \end{split}$$

Hence the adopted thickness of 100mm with M1 mortar and masonry unit with compressive strength 10N/mm² is **not** safe in carrying the load from slab. The thickness of wall is increased to 200mm.

Load from slab = 10.66 kN/m^2

For 2.65m length of slab,

Load from slab = 10.66 x 2.658 = 28.36 kN/m

Self weight of masonry = $2 \times 0.2 \times 20 \times 3.5 = 28 \text{ kN/m}$

Total = 56.36kN/m

Loading on masonry wall = 56.36 kN/m

Actual stress $\sigma_{act} = \frac{56.36 \times 1000}{200 \times 1000} = 0.2818 \text{ N/mm}^2$

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength

10 N/mm² is taken from Table 8, IS 1905 - 1987.

Permissible stress $\sigma_{per} = 0.96 \text{ N/mm}^2$

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per CI.5.4.

Stress reduction factor (K_{st})

Slenderness ratio (Least of I_e/t & h_e/t)

From Table 4,

Effective height, $h_e = 0.75 \text{ H} = 0.75 \text{ x} 3.5 = 2.625 \text{m}$ [Both ends fixed] Effective length, le = 1.5 L = 1.5 x 3.65 = 5.475 m [One end fixed, other end free] $h_e/t = 2.625 / 0.2 = 13.125 < 27$ Therefore, the stress reduction factor from Table 10 for $\lambda = 22.5$ and no eccentricity condition is, For $12 \rightarrow 0.84$ (e = 0) For $14 \rightarrow 0.78$ For $13.125 \rightarrow 0.3675 + 0.439 = 0.806$ $K_{st} = 0.806$
$$\begin{split} A &= 0.2 \times 1 = 0.2 m^2 \\ \underline{K_{A}} &= \underline{1} \\ \hline \underline{Shape \ modification \ factor \ (K_{sh})} \\ K_{sh} \ for \ block \ of \ size \ 200 \times 100 \times 90 \ mm \ laid \ along \ 100 mm \ side, \ from \ Table \ 10 \ for \ Height \ to \\ \hline Width \ ratio \ of \ 90 \times 100, \\ \underline{Height} \\ Width \ = \ \underline{90} \\ 100 \ = \ 0.9 \\ \hline For \ 0.75x_o \ ightarrow \ 1.1 \\ \hline For \ 1x_o \ ightarrow \ 1.1 \\ \hline For \ 0.9x_o \ ightarrow \ 1.06 \\ \underline{K_{sh} = 1.06} \\ \sigma_{per \ modified} \ = \ K_{st}.K_A.K_{sh}. \ \sigma_{per} \\ &= \ 0.806 \times 1 \times 1.06 \times 0.96 \ = \ 0.82 \ N/mm^2 \\ &> \sigma_{act \ /m} \ [0.2818 \ N/mm^2] \end{split}$$

Hence the adopted thickness of 200mm with M1 mortar and masonry unit with compressive strength 10N/mm² is safe in carrying the load from slab.

4. Design the interior wall of a single storey building shown in figure. The height of the ceiling is 3.5m and the load from slab including self weight is 30kN/m².

Load from slab = 30×3.65 = 109.5 kN/mSelf weight of wall = $0.15 \times 3.5 \times 1 \times 20$ = 10.5 kN/mTotal = 120 kN/m



150*x*1000

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength 10N/mm² is taken from Table 8, IS 1905 – 1987.

Permissible stress $\sigma_{per} = 0.96 \text{ N/mm}^2$

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per CI.5.4.

Stress reduction factor (K_{st})

Slenderness ratio (Least of l_e/t & h_e/t) From Table 4, Effective height, $h_e = 0.75 \text{ H} = 0.75 \text{ x} 3.5 = 2.625 \text{m}$ [Both ends fixed] Effective length, le = 1 L = 1.0 x 8.15 = 8.15 m $h_e/t = 2.625 / 0.2 = 13.125 < 27$

For the cross walls provided, stiffening coefficients are found from Table 6, IS1905 - 1987.

t _p = 3tw	[for cross walls]	cl.4.6.3, IS1905-1987
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- $$\begin{split} S_p &= 8.15 m, \qquad [S_p \rightarrow c/c \text{ spacing of pier/wall}] \\ t_w &= 0.15 m \qquad [t_p \rightarrow thickness of pier/wall] \end{split}$$
- $t_p = 3tw = 0.45m$ [$t_w \rightarrow$ thickness of wall]

 $w_p = 0.15m$ [$w_p \rightarrow$ width of pier/wall] cl.4.5.3, IS1905-1987

 $\frac{t_p}{t_w} = \frac{0.45}{0.15} = 3, \qquad \qquad \frac{S_p}{w_p} = \frac{8.15}{0.15} = 54.3$

From Table 6, for $\frac{S_p}{w_p} = 54.3$,

$$\frac{t}{t} = 3$$

$$S_e = 1$$

Thickness of wall = $1 \times 0.2 = 0.2m$

[Considering stiffness]

 $\lambda = 2.625 / 0.2 = 17.5 < 27$

Therefore, the stress reduction factor from Table 10 for λ = 17.5 and no eccentricity condition is,

For $16 \rightarrow 0.73$ (e = 0) For $18 \rightarrow 0.67$ For $13.125 \rightarrow 0.1825 + 0.5025 = 0.685$ $K_{st} = 0.685$



For $0.9x_0 \rightarrow 1.06$

 $K_{sh} = 1.06$

$$\begin{split} \sigma_{per \ modified} & = K_{st}.K_{A}.K_{sh}. \ \sigma_{per} \\ & = 0.685 \ x \ 0.925 \ x \ 1.06 \ x \ 0.96 = 0.647 \ N/mm^2 \\ & < \sigma_{act \ /m} \ \ [0.8 \ N/mm^2] \end{split}$$

Hence the adopted M1 mortar and masonry unit with compressive strength 10N/mm² is **not** sufficient in carrying the load.

Increase the strength of brick unit and mortar as,

<u>H1</u> mortar and masonry unit compressive strength $15N/mm^2$

 $\sigma_{per} = 1.31 \text{ N/mm}^2$

 $\sigma_{\text{per modified}} = 0.88 \text{ N/mm}^2$

Therefore, the interior wall of 150mm thickness is safe with H1 mortar and brick units of compressive strength 15 N/mm^2 .

wall is unstiffened[no need to find effective length] at the ends.

Assume a thickness of wall of 300mm

Actual stress = $\frac{20x1000}{300x1000}$ = 0.066N/mm² σ_{per} = 0.96 N/mm² $H_e = 0.75 \text{ H} = 0.75 \text{ x} 4 = 3\text{m}$ $\lambda = \frac{3}{0.3} = 10$ $K_{st} = 0.89$ For A = 0.3 x 1 = 0.3 m2, KA = 1 $K_{sh} = 1.06$ $\sigma_{per modified}$ = 0.27 N/mm² > σ_{act} [0.066 N/mm²] Hence, safe.

6. Design the wall in the GF level for the loading condition as shown in figure.

Loading on brick wall:

12 kN/m Load from slab = 12 + 10 + 10= 32 kN/mWeight of wall (self wt.) = $3 \times 2 \times 0.1 \times 3 \times 20 = 36 \text{ kN/m}$ 3m Total = 68 kN/mActual stress = $\frac{68x1000}{2x100x1000}$ = 0.34 N/mm² 10 kN/m Use M1 mortar and brick of compressive strength 10 N/mm². 3m $\sigma_{per} = 0.96 \text{ N/mm}^2$ [From Table 8, IS1905 - 1987] 10 kN/m $(\lambda \leq 6)$ $h_{eff} = 0.75 \text{ x} \text{ h} = 0.75 \text{ x} 3 = 2.25 \text{ m}$ 3m $t_e = 2/3(t_w + t_w) = 2/3(0.1 + 0.1) = 0.133m$ $\lambda = \frac{2.25}{t} = \frac{2.25}{0.133} = 16.875$ From Table 8, IS1905 - 1987, $\lambda \rightarrow 16$, 0.75 $\lambda \rightarrow 18$, 0.67 $\lambda \rightarrow 16.875$, (0.421875 + 0.293125 = 0.715) Area of wall (each leaf) = $0.1 \times 1 = 0.1 \text{m}^2 < 0.2 \text{ m}^2$ $K_{A} = 0.85$ $K_{sh} = 1.06$
$\sigma_{\text{per (modified)}} = K_{\text{st}} x K_{\text{A}} x K_{\text{sh}} x \sigma_{\text{per}}$

= 0.704 x 0.85 x 1.06 x 1.96 = 0.61 N/mm² > σ_{act}

Therefore, the cavity wall is safe with M1 mortar and masonry unit of compressive strength 10N/mm².

2. Design a cavity wall of overall thickness 250mm and thickness of each leaf 100mm for a

three storeyed building. The wall is stiffened by intersecting walls 200mm thick at 3600mm c/c. The ceiling height is 3m and the loading from roof is 16 kN/m. The loading from each floor is 12.5kN/m. Load from roof = 16kN/mLoad from floor = 12.5 + 12.5 kN/mWall load [3x0.2x20] = 36 kN/mTotal = 77kN/m

Actual stress = $\sigma_{ac} = \frac{77 \times 10^3}{200 \times 1000} = 0.39 \text{ N/mm}^2$

 σ_{per} :

Assume M1 mortar and brick of compressive strength 10 N/mm².

From Table 7, IS1905 – 1987,

 $\sigma_{per} = 0.96 \text{ N/mm}^2 \quad (\lambda \leq 6)$

Effective height = $h_{eff} = 0.75 \times 3 = 2.25m$

Effective length = I_{eff} = 0.8 I = 0.8 x 3600 = 2880mm = 2.88m

Stiffening Coefficient:

Since cross wall is available along one leaf, Sc for

 $\frac{S_{p}}{w_{p}} = \frac{3.6}{0.2} = 18, \qquad \frac{t_{p}}{t_{w}} = \frac{3t_{w}}{t_{w}} = \frac{0.3}{0.1} = 3$

From Table 8, IS1905 - 1987,

 $\begin{array}{ll} \rightarrow 15, & 1.2 \\ \rightarrow 20, & 1 \end{array}$

→ 18, 0.48 + 0.6 = 1.08

Effective thickness of cavity wall, $t_e = 2/3 (1.08(0.1) + 0.1) = 0.139m$

$$\lambda = \frac{2.25}{0.139}$$
 16.18
 $\lambda \to 16$, 0.73

$$\begin{split} \lambda & \Rightarrow 20, & 0.67 \\ \lambda & \Rightarrow 16.18, & 0.664 + 0.0603 = 0.7243 \\ K_{st} &= 0.72 \\ \text{Area} &= 0.1 \text{ x } 1 = 0.1 \text{m}^2 < 0.2 \text{ m}^2 \\ K_A &= 0.85 \\ K_{sh} &= 1.06 \\ \sigma_{\text{per (modified)}} &= 0.62 \text{ N/mm}^2 > \sigma_{\text{act}} [0.39 \text{ N/mm}^2] \end{split}$$

3. Design a masonry column to carry a load of 150kN. The height of the column is 2000mm. The column is restrained against translation (hinged) only.

Assume a column of size 400 x 400mm.

Use M1 mortar and brick of compressive strength 10 N/mm².

Actual stress =
$$\sigma_{act} = \frac{150x10^3}{400x400} = 0.94 \text{ N/mm}^2$$

 $\sigma_{\text{per:}}$

$$\begin{split} h_{eff} &= h[\text{Table 4, IS1905-1987}] \\ \lambda &= 2000/400 = 5 < 6 \\ \text{There is no need for Stress reduction factor (K_{st} = 1)} \\ \text{From Table 7, } \sigma_{per} &= 0.96 \text{ N/mm}^2 \\ A_{st} &= 0.4 \text{ x } 0.4 = 0.16\text{m}^2 < 0.2\text{m}^2 \\ K_A &= 0.7 + (1.5 \text{ x } 0.4 \text{ x } 0.4) = 0.94 \\ K_{sh} &= 1.06 \\ \sigma_{per (modified)} &= 0.94 \text{ x } 0.96 \text{ x } 1 \text{ x } 1.06 = 0.98 \text{ N/mm}^2. \\ \sigma_{act} < \sigma_{per} \end{split}$$

Therefore, the masonry column of size 400 x 400mm with M1 mortar and brick unit of compressive strength $10N/mm^2$ is safe to carry a load of 150kN.

Note: Boundary condition is assumed if not given.

4. Design an interior wall of a single storeyed workshop building of height 5.4m supporting a



Since there is an increase in width at the pier, the actual stress is found for the wall length of 3.6m (One bay).

C/s area of one bay = $(3.6 \times 0.2) + 4(0.1 \times 0.1) = 0.76 \text{ m}^2$ Loading per bay (for 3.6m length) = 45 x 3.6 = 162 kN $= 0.76 \times 5.4 \times 20$ = 82.08 kN Load from brick wall Total = 244.08 kN Actual stress = $\sigma_{act} = \frac{244.08 \times 10^3}{0.76 \times 10^6} = 0.321 \text{ N/mm}^2$ $\sigma_{per:}$ h_{eff} = 0.75h = 4.05m [Table 4, IS1905-1987] demise $leff = 0.8l = 0.8 \times 3.6 = 2.88m$ $\lambda = 2.88 < 6$ There is no need for Stress reduction factor $(K_{st} = 1)$ From Table 7, $\sigma_{per} = 0.96 \text{ N/mm}^2$ $A_{st} = 0.4 \times 0.4 = 0.16m^2 < 0.2m^2$ $K_A = 0.7 + (1.5 \times 0.4 \times 0.4) = 0.94$ $K_{sh} = 1.06$ $\sigma_{\text{per (modified)}} = 0.94 \times 0.96 \times 1 \times 1.06 = 0.98 \text{ N/mm}^2$.

 $\sigma_{act} < \sigma_{per}$

Therefore, the masonry column of size 400 x 400mm with M1 mortar and brick unit of compressive strength $10N/mm^2$ is safe to carry a load of 150kN.

Stiffening coefficient,

$$\begin{array}{lll} \frac{S_p}{w_p} & \frac{3.6}{0.2} = 18, & \frac{t_p}{t_w} & \frac{0.4}{0.2} & 2 \\ (\text{tp} \rightarrow \text{depth of pier (0.4)}) \\ \text{Sc} = 1.04 \\ \text{t}_{\text{eff}} = 1.04 \times 0.2 = 0.208 \text{m} \\ \lambda \text{ is the least of H}_{\text{eff}} \text{ and L}_{\text{eff}} \\ \lambda = \frac{2.88}{0.208} = 13.85 \\ \text{Kst} = 0.785 \\ \text{Area reduction coefficient} = \frac{0.76}{3.6} = 0.211 > 0.2 \\ \text{K}_{\text{A}} = 1 \qquad [\text{Cl.5.4.1.2, IS1905} - 1987] \\ \text{Ksh} = 1.06 \end{array}$$

 $\sigma_{\text{per (modified)}} = 0.785 \text{ x } 0.96 \text{ x } 1 \text{ x } 1.06 = 0.79 \text{ N/mm}^2 > \sigma_{\text{act}}$

Inference : Hence the brick wall is safe with M1 mortar and brick of compressive strength 10N/mm².

5. Design a brick masonry column of height 3m, tied effectively, fixed at top and bottom. The load from slab is 100kN, including self weight of the brick pillar.

= 100kN

Load from slab

Self weight of brick pillar = $0.4 \times 0.4 \times 20 \times 3 = 9.6$ kN

Assume a column size of 400mm x 400mm.

Actual stress = $\frac{109.6 \times 10^3}{400 \times 400}$ = 0.685 N/mm²

Assume grade of mortar as M1 and compressive strength of 0.96N/mm²

 $h_{eff} = 0.75.H = 2.25m$

$$\lambda = \frac{2.25}{0.4} = 5.625 < 6$$

There is no need of stress reduction factor.

$$\begin{split} &\mathsf{K}_{st} = 1 \\ &\mathsf{A} = 0.4 \ x \ 0.4 = 0.16 \ m^2 < 0.2m^2 \\ &\mathsf{K}_{\mathsf{A}} = 0.7 + (1.5 \ x \ 0.4 \ x \ 0.4) = 0.94 \\ &\mathsf{K}_{\mathsf{sh}} = 1.06 \qquad [\text{Brick unit } 200 \ x \ 100 \ x \ 90] \\ &\sigma_{\mathsf{per}\ (\mathsf{modified})} = 0.9565 \ \mathsf{N/mm}^2 > \sigma_{\mathsf{act}} \end{split}$$

Hence the brick wall is safe with M1 mortar and compressive strength of 10N/mm².

6. Design an interior wall of a 3 storeyed building with ceiling height of each storey as 3m. The wall is unstiffened and 3.6m in length. Load from roof is 12kN/m and from each floor is 10kN/m. Select a cavity wall with overall thickness 250mm and length between each leaf as 50mm.



Load from roof

Load from each floor

= 10 + 10 kN/m

Self weight of wall = $3 \times 2 \times 0.1 \times 3 \times 20 = 36 \text{ kN/m}$

Total = 68 kN/m

= 12 kN/m

Actual stress = $\frac{68 \times 10^{-3}}{2 \times 100 \times 1000}$ = 0.34 N/mm² Use M1 mortar and brick of compressive strength 10N/mm², $\sigma_{per} = 0.96 \text{ N/mm}^2$ $h_{eff} = 0.75 \times 3 = 2.25 \text{m}$ $t_e = 2/3(t_w + t_w) = 0.133 \text{m}$ $\lambda = \frac{2.25}{t_e} = 16.875$ From Table 8, IS1905 – 1987, $K_{st} = 0.704$ Area of wall = 0.1 x 1 = 0.1 m² < 0.2 m² $K_A = 0.85$ $K_{sh} = 1.06$ σ_{per} (modified) = 0.704 x 0.85 x 1.06 x 0.96 = 0.61 N/mm² > σ_{act}

Hence the brickwork is safe with M1 mortar and brick of compressive strength 10N/mm².

MASONRY WALL SUBJECTED TO CONCENTRATED LOAD



The angle of dispersion below the concentrated load is 30° on each side.

Therefore, the actual stress is,

 $\sigma_{act} = P / A$, where, $A \rightarrow$ Area for 1m run = L x t, $L \rightarrow$ Length of load dispersion = $\frac{2H}{\tan 60^{\circ}}$, $H \rightarrow$ Height of wall

For the wall to be safe in carrying the load, $\sigma_{act} < \sigma_{per}$

1) Design a solid wall of a mill building 3m height securely tied with roof and floor units. The wall supports two beams on either side exerting reactions of 30kN and 20kN. Thickness of wall is 230mm and the beam bears on the wall for 115mm (width of beam). Neglect load due to self weight.

 $A_{br} = 230 \text{ x} 115 = 26450 \text{ mm}^2$

$$\sigma_{\rm br} = \frac{P}{A_{\rm br}} - \frac{(30 - 20)x10^3}{26450} = 1.89 \text{ N/mm}^2 < \sigma_{\rm per} \text{ in bearing}$$

The values given in Table 8 are increased by 50% for σ_{per} in bearing.

Therefore, assume H1 grade of mortar and brick of compressive strength 15 N/mm².

 $\sigma_{per} = 1.31 \text{ N/mm}^2$

 $\sigma_{per br} = 1.5 \text{ x } 1.31 = 1.965 \text{ N/mm}^2$

Check for compressive stress:

$$σ_{act} = P / A, A = L x t, where, L → Length of load dispersion$$

$$L = \frac{2H}{tan 60^{0}} = \frac{2x3}{tan 60^{0}} = 3.464 m$$
A = 3.464 x 1000 x 230 = 796720 mm²

 $σ_{act} = \frac{50x10^{3}}{796720} = 0.063 N/mm^{2}$

 $σ_{per} = 1.31 N/mm^{2}$

 $K_{st} = 0.89$

 $K_{A} = 1$

 $K_{sh} = 1.06$

 $σ_{per modified} = K_{st} x K_{A} x K_{sh} x σ_{per}$

 $= 0.89 x 1 x 1 x 1.31 = 1.1659 N/mm^{2} > σ_{act} [0.063 N/mm^{2}]$

The wall is safe in carrying a concentrated load with H1 mortar and brick of compressive strength 15N/mm².

Design the exterior wall of a workshop building 3.6m height carrying steel trusses at
 5m spacing. The wall is securely tied at roof and floor levels. The wall is of thickness
 200mm and the truss bears on the wall for 200mm and load from the truss is 30kN.

[Length is considered only for piers and cross walls]

 $A_{br} = 200 \text{ x } 200 = 40000 \text{ mm}^2$

$$\sigma_{\rm br} = \frac{30 \times 10^3}{40000} = 0.75 \text{ N/mm}^2 < \sigma_{\rm br \, per} \rightarrow [1.5(\sigma_{\rm per}) = 1.5 \text{ x } 0.96 = 1.44 \text{ N/mm}^2]$$

Assume M1 mortar and brick of compressive strength 10 $\ensuremath{\text{N/mm}^2}\xspace$.

Check for compressive stress:

$$\sigma_{ac} = P/A$$

$$L = \frac{2H}{\tan 60^{0}} = 4.157 \text{ m}$$

$$A = 4.157 \text{ x } 10^{3} \text{ x } 200 = 956110 \text{ mm}^{2}$$

$$\sigma_{act} = \frac{30 \times 10^{3}}{956110} = 0.036 \text{ N/mm}^{2}$$

$$H_{eff} = 0.75 \text{ x } 3.6 = 2.7 \text{ m}$$

$$L_{eff} = 0.8 \text{ x } 4.157 = 3.3256 \text{ m}$$

$$\lambda = \frac{2.7 \times 10^{3}}{2300} = 13.5$$

$$A = 4.157 \text{ x } 0.2 = 0.8314 \text{ m}^{2} > 0.2 \text{ m}^{2}$$

$$K_{A} = 1$$

$$K_{st} = 0.795$$

$$K_{sh} = 1.06$$

$$\sigma_{per modified} = 0.81 \text{ N/mm}^{2}$$

3) In the above problem, design the wall if piers are available below the truss and size of pier is 200 x 400mm.

Here, we need to take the length.

If the truss fully rests on
pier, bearing area,

$$A_{br} = 80000 \text{ mm}^2$$

 $P = 30 \times 10^3 \text{ N}$
 $\sigma_{br} = \frac{30x10^3}{200x400}$
 $\sigma_{br} = 0.375 \text{ N/mm}^2 < \sigma_{br \, per} \rightarrow [1.5(0.96) = 1.44 \text{ N/mm}^2]$
Using M1 mortar and brick of compressive strength 10N/mm²,
Check for compressive stress:
 $\sigma_{ac} = P/A$
 $L = \frac{2H}{\tan 60^0} = 4.16 \text{m}$
 $H_{eff} = 0.75 \times 3.6 = 2.7 \text{m}$

 $L_{eff} = 0.8 \times L = 0.8 \times 4.5 = 3.6m$ $t_{e} = S_{c} \times t$ $\frac{S_{p}}{W_{p}} = \frac{4.5}{0.2} = 22.5, \qquad \frac{t_{p}}{t_{w}} = \frac{0.4}{0.2} = 2$ $S_{c} = 1$ $t_{e} = 1 \times 0.2 = 0.2m$ $\lambda = \frac{2.7}{0.2} = 13.5$ $A = L \times t = 4.5 \times 0.2 = 0.9 \text{ m}^{2} > 0.2 \text{ m}^{2}$ $K_{A} = 1$ $K_{st} = 0.795$ $\sigma_{act} = \frac{30x10^{3}}{4.5x0.2} = 0.03N/mm^{2}$

 $\sigma_{\text{per modified}} = 1 \times 1 \times 0.795 \times 1.06 \times 0.96 = 0.8089 \text{ N/mm}^2$

Hence the wall is safe with M1 mortar and brick of compressive strength 10N/mm².

ECCENTRICALLY LOADED BRICK MASONRY

Eccentricity - Offset distance from CG of member to CG of load

Occurs in,

- i) Exterior wall –Bearing not sufficient
- ii) Flexible slab Excessive deflection (timber)
- iii) When span is very large, code recommends to take some amount of eccentricity

$$\sigma_{act} = \frac{P}{A} \quad \frac{M}{Z} < 1.25(\sigma_{per modified}),$$

where, $M \rightarrow P x e$, $Z \rightarrow b.t^2/6$, b = 1m (wall)

GuidelinesgiveninAppendixA

Eccentricity in loading occurs in loading on walls due to various reasons causing reduction in stress. The cases where eccentricity needed to be assumed are given in Appendix A of IS1905-1987.

- 1. For an exterior wall, when span of roof is more than 30 times the thickness of wall, the eccentricity assumed is one sixth of the bearing width.
 - $L > 30 t_w$

$$\mathsf{E} = 1/6(\mathsf{t}_\mathsf{w})$$

2. When bearing is not sufficient, eccentricity assumed is $t_{\ensuremath{w}}/12$

- 3. When flexible floors are adopted, for full width bearing, eccentricity assumed is $t_w/6$.
- For interior walls, when there is unequal length of slabs on both sides and then the difference between the loading is greater than 15%, moment is generated for which, e=M/P.



Actual stress (σ_{act}) is the sum of direct compressive stress P/A and bending stress M/Z. The permissible stress given in Table 8 can be increased by 25% and modification factors applied on it.

1) Design an exterior wall of height 3.5m, which is unstiffened. The slab is light weight flexible slab of length 3.5m. Assume the unit weight of slab as 15kN/m³ with thickness 0.2m.



Half of the load from the slab comes to the wall and since the slab is flexible, eccentricity considered as per Appendix A of IS1905 – 1987.

Assume 200mm thick wall with M1 mortar and brick of compressive strength of 10N/mm².

Loading on wall:

Load from slab = $15 \times 0.2 \times 3.5/2 = 5.25 \text{ kN/m}$ Self weight of wall = $0.2 \times 3.5 \times 20 = 14 \text{ kN/m}$

Total = $\underline{19.25kN/m}$

 $e = t_w/6 = 33.33mm$

M = P x e

Moment due to eccentricity = $19.25 \times 10^3 \times 33.33 = 641.67 \times 10^3$ Nmm Z = $bt^2/6 = 1000 \times 200^2 / 6 = 6.67 \times 10^6$ mm³ $M/Z = 0.096 \text{ N/mm}^2$

Total stress = $\frac{P}{A} = \frac{M}{Z} = \frac{19.25 \times 10^3}{1000 \times 200} + 0.096 = 0.192 \text{ N/mm}^2$

 σ_{per} :

Use M1 mortar and brick of compressive strength 10N/mm²,

 $\sigma_{per} = 0.96 \text{ N/mm}^2$

Stress reduction factor (K_{st})

Slenderness ratio (Least of l_e/t & h_e/t)

From Table 4,

 $h_e = 0.75 H = 0.75 x 3 = 2.25m$ [Both ends fixed]

 $h_{e}/t = 2.625 / 0.2 = 13.125 < 27$

Therefore, the stress reduction factor from Table 10 for λ = 13.125 and no eccentricity condition is,

For $12 \rightarrow 0.78$ (e = 0) For $14 \rightarrow 0..7$ For $13.125 \rightarrow 0.735$

 $K_{st} = 0.735$

<u>Area reduction factor (K_A)</u> [CI.5.4.1.2, IS1905-1987]

For thickness t = 0.2m, A = 0.2 m^2

<u>K_A</u>= 1

Shape modification factor (K_{sh}) [Cl. 5.4.1.3, IS1905-1987]

 K_{sh} for block of size 200 x 100 x 90 mm laid along 100mm side, from Table 10 for Height to Width ratio of 90 x 100,

```
\frac{Height}{Width} = \frac{90}{100} = 0.9
For 0.75x_0 \rightarrow 1
For 1x_0 \rightarrow 1.1
For 0.9x_0 \rightarrow 1.06
K_{sh} = 1.06
\sigma_{per modified} = K_{st}.K_A.K_{sh}. \sigma_{per}
= 0.735 \times 1 \times 1.06 \times 1.25 \times 0.96 = 0.935 \text{ N/mm}^2
\sigma_{act} [0.192\text{N/mm}^2] < 1.25 \sigma_{per} [1.17\text{N/mm}^2]
```

Note : For brick masonry columns laterally supported by beams,

 $H_e = H$

Only when the column is not laterally supported (laterally unsupported),

 $H_e = 2H$

2) Design a masonry column tied effectively at top and bottom. Load from slab is 100kN. Assume size of column as 400 x 400mm

$$\begin{split} \sigma_{act} &= \frac{100 \times 10^3}{400 \times 400} = 0.625 \text{ N/mm}^2 \\ \sigma_{per} &= 0.96 \text{ N/mm}^2 \\ \text{Heff} &= \text{H} = 2\text{m} \\ \lambda &= 5 < 6 \\ K_{st} &= 1 \\ K_{sh} &= 1.06 \\ K_A &= 0.94 \\ \sigma_{per \text{ modified}} &= 0.957 \text{ N/mm}^2 \\ \sigma_{act} &< \sigma_{per} \end{split}$$

3) Design an interior wall of a single storeyed building supporting unequal concrete roof slabs. The plan is as shown in figure. Assume triangular pressure distribution and unit weight of slab is 10kN/m³. The height of the wall is 3.8m and the wall is fixed to the foundation block below.

Height = 3.8m

$$e = \frac{t_w}{6} \quad \frac{t_w}{6} = \frac{2t_w}{6} =$$

 $\frac{t_w}{3}$

$$\sigma_{act} = \frac{P}{A} - \frac{M}{Z}$$

Loading from slab 1:



= Area of triangle x Load intensity on slab = $\frac{(1/2)x3.6x1.8x10}{3.6}$ = 9 kN/m Loading from slab 2: = Area of rectangle x Load intensity on slab = $\frac{(1/2)x(1.8/2)x3.6x10}{3.6}$ = 4.5 kN/m Self weight of brickwork = 0.2 x 3.8 x 20 = 15.2 kN/m = 28.7 kN/m

Since the difference in loading is 100% [> 15%], there is an eccentricity in the loading. The eccentricity in the loading as per Appendix A of IS1905-1987 is,

 $e = \frac{\underline{t}_{\underline{w}}}{6} \quad \frac{\underline{t}_{\underline{w}}}{6} = \frac{2\underline{t}_{\underline{w}}}{6} = \frac{\underline{t}_{\underline{w}}}{3}$ $M \rightarrow$ (Difference in load from slab x Eccentricity) $= (9 - 4.5) \times 10^3 \times 200/3 = 300 \times 10^3 \text{ Nmm}$ $\sigma_{act} = \frac{P}{A} - \frac{M}{Z}$, where $Z = \frac{b.t^2}{6} = \frac{1000x200^2}{6} = 6.67 \times 10^6 \text{ mm}^3$ $\sigma_{act} = \frac{28.7 \times 10^3}{1000 \times 200} \quad \frac{300 \times 10^3}{6.67 \times 10^3} = 0.1885 \text{ N/mm}^2$ $e = t_w/3$ → $e/t_w = 1/3$ λ is least of (H_e/t_e and L_e/t_e) Assume width of the cross wall as 200mm $t_e = t \times S_c$ $\frac{S_p}{W_p} = \frac{3.8}{0.2} = 19, \qquad \frac{t_p}{t_w} = \frac{0.6}{0.2} = 3$ $S_c = 1.04$ $t_e = t \times Sc = 1.04 \times 0.2 = 0.208m$ $H_{eff} = 0.75 \text{ x } 3.8 = 2.85 \text{ m}$ $\lambda = \frac{2850}{208} = 13.7$ $L_{eff} = 0.8 L = 0.8 \times 3.60 = 2.88 m$ $\lambda = 13.8$ $\lambda \rightarrow 12$ 0.72 $\lambda \rightarrow 14$ 0.66 0..67 $\lambda \rightarrow 13.7$ $K_{st} = 0.67$ $A = 0.1 \times 1 = 0.1 m^2 < 0.2 m^2$

$$K_{A} = 1$$

$$K_{sh} = 1.06$$

$$\sigma_{per modified} = 0.96 \times 0.67 \times 1 \times 1.06 = 0.68 \text{ N/mm}^{2}$$

$$\sigma_{act} [0.1885 \text{ N/mm}^{2}] < 1.25(\sigma_{per}) \rightarrow [0.851 \text{ N/mm}^{2}]$$
4) Design an exterior wall of height 4m, unstiffened and supports a flexible slab 150mm thick with unit weight 12 kN/m^{3}. The length of the slab is 4m.
Load from slab = 2 \times 0.15 \times 12 = 3.6 kN/m
Self weight of wall = 0.2 \times 4 \times 20 = 16 kN/m
Total = 19.6 kN/m
e = tw/6 = 200/6 = 25mm
M = P x e = 19.6 \times 10^{3} x 25 = 6.53 \times 10^{5} \text{ Nmm}
 $Z = \frac{bt^{2}}{6} = \frac{1000 x 200^{2}}{6} = 6.67 \times 10^{6} \text{ mm}^{3}$
 $\sigma_{act} = \frac{P}{A} \quad \frac{M}{Z} = 0.098 + 0.097 = 0.195 \text{ N/mm}^{2}$
H_{eff} = 0.75H = 3m
Assuming M1 mortar and brick of compressive strength 10 N/mm²,
 $\lambda = \frac{3000}{200} = 15$
 $\lambda \Rightarrow 14 \qquad 0.7$
 $\lambda \Rightarrow 16 \qquad 0.63$
 $\lambda \Rightarrow 15 \qquad 0.35 + 0.315 = 0..665$
 $K_{st} = 0.665$
 $A = 0.1 \times 1 = 0.1m^{2} < 0.2m^{2}$
 $K_{A} = 1$
 $K_{sh} = 1.06$
 $\sigma_{per modified} = 0.96 \times 0.665 \times 1 \times 1.06 = 0.677 \text{ N/mm}^{2}$

 $\sigma_{act} [0.195 \text{ N/mm}^2] < 1.25(\sigma_{per}) \rightarrow [0.846 \text{ N/mm}^2]$