10111CE701-DESIGN OF RC AND BRICK MASONRY STRUCTURES

E-LEARNING MATERIAL

BY,

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10111CE701

DESIGN OF REINFORCED CONCRETE & BRICK MASONRY STRUCTURES L T P C 3 1 0 4

OBJECTIVE

This course covers the design of Reinforced Concrete Structures such as Retaining Wall, Water Tanks, Staircases, Flat slabs and Principles of design pertaining to Box culverts, Mat foundation and Bridges. At the end of the course student has a comprehensive design knowledge related to structures, systems that are likely to be encountered in professional practice.

UNIT I RETAINING WALLS 12

Design of cantilever and counter fort retaining walls

UNIT II WATER TANKS 12

Underground rectangular tanks – Domes – Overhead circular and rectangular tanks – Design of staging and foundations

UNIT III SELECTED TOPICS 12

Design of staircases (ordinary and doglegged) – Design of flat slabs – Design of Reinforced concrete walls – Principles of design of mat foundation, box culvert and road bridges

UNIT IV YIELD LINE THEORY 12

Application of virtual work method to square, rectangular, circular and triangular slabs

UNIT V BRICK MASONRY 12

Introduction, Classification of walls, Lateral supports and stability, effective height of wall and columns, effective length of walls, design loads, load dispersion, permissible stresses, design of axially and eccentrically loaded brick walls

L : 45 , T : 15 TOTAL: 60 PERIODS

TEXT BOOKS

1. Krishna Raju, N., "Design of RC Structures", CBS Publishers and Distributors, Delhi, 2006

2. Dayaratnam, P., "Brick and Reinforced Brick Structures", Oxford & IBH Publishing House, 1997

3. Varghese, P.C., "Limit State Design of Reinforced Concrete Structures "Prentice hall of India Pvt Ltd New Delhi, 2007.

REFERENCES

1. Mallick, D.K. and Gupta A.P., "Reinforced Concrete", Oxford and IBH Publishing Company

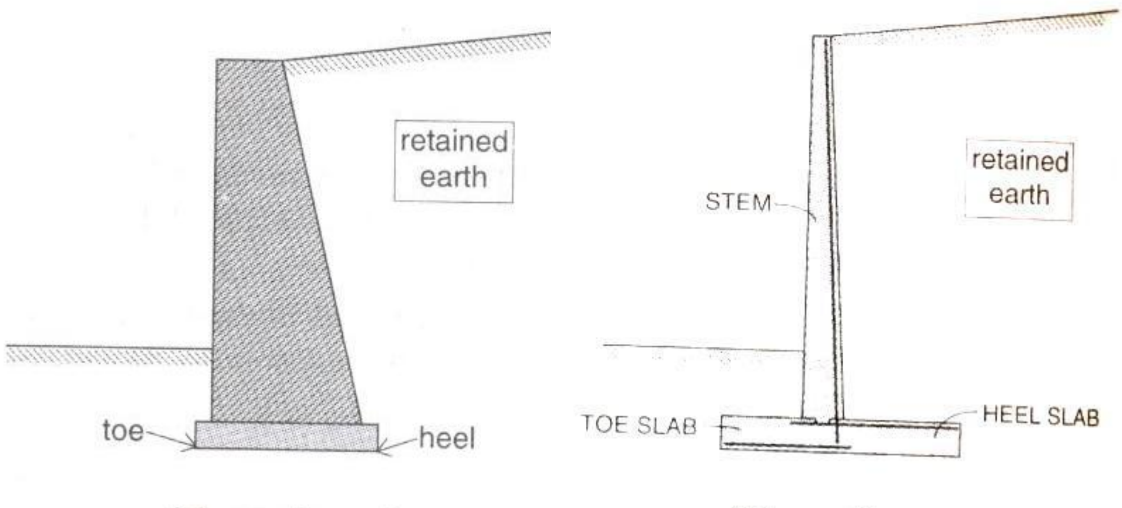
2. Syal, I.C. and Goel, A.K., "Reinforced Concrete Structures", A.H. Wheelers & Co. Pvt. Ltd., 1994

3. Ram Chandra.N. and Virendra Gehlot, "Limit State Design", Standard Book House.2004.

UNIT I
RETAINING WALL

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

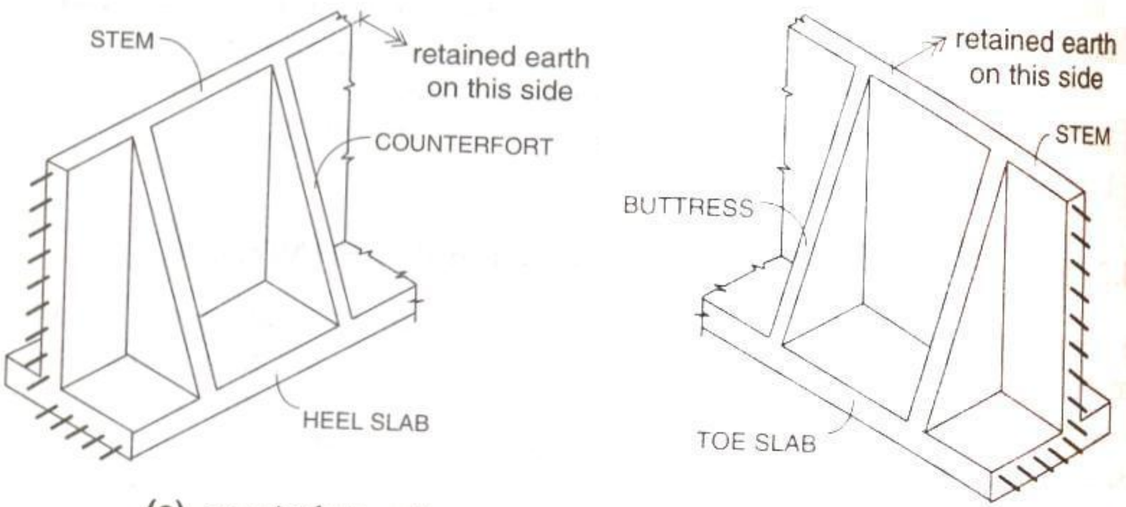
A) Gravity wall ($h<3m$) – Masonry or Stone



(a) gravity wall (b) cantilever wall

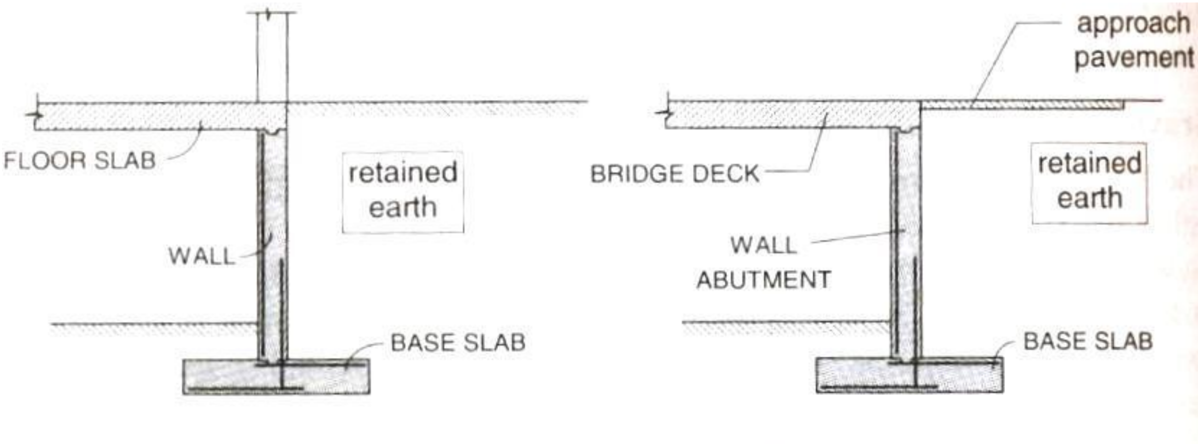
B) Cantilever wall (h>3m and h<6m)

C) Counterfort wall (h>8m)



(c) counterfort wall (d) buttress wall

D) Buttress wall [Transverse stem support provided on front side]

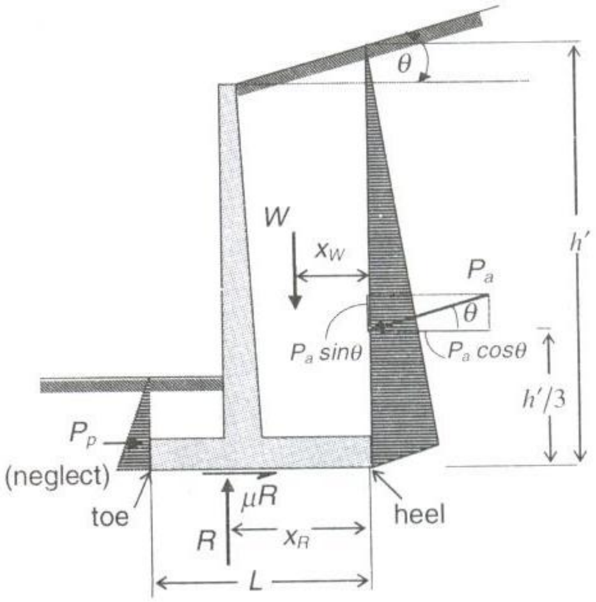


(e) basement wall **(f)** bridge abutment

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

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

<u>Earth pressure and stability requirements</u>:



Forces acting on a cantilever retaining wall

Pressure, $P = C\gamma_e Z$

Where, Z = depth, γ_e = Unit weight

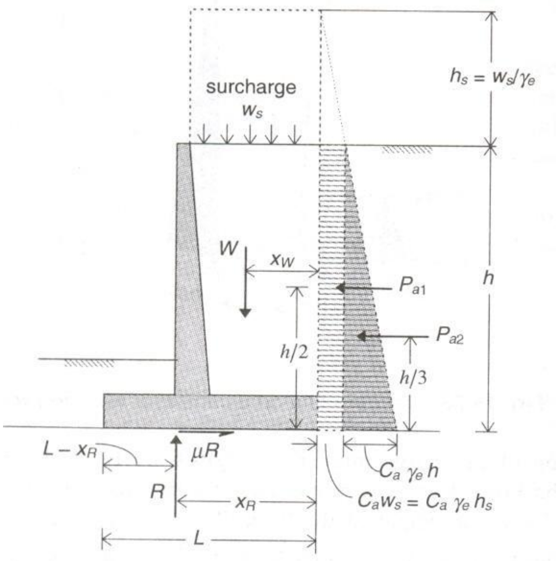
$C_a = \frac{1-Sin\phi}{1+Sin\phi}$; $Cp = \frac{1+Sin\phi}{1-Sin\phi}$

Always, Cp > Ca. Eg: If ɸ = 30^0, Ca = 1/3 and Cp = 3.

In sloped backfill,

$$C_a = \left[\frac{Cos\theta - \sqrt{Cos^2\theta - Cos^2\phi}}{Cos\theta + \sqrt{Cos^2\theta - Cos^2\phi}}\right]Cos\theta; \qquad Cp = \frac{1+Sin\phi}{1-Sin\phi}$$

1. <u>Effect of surcharge on level backfill</u>:



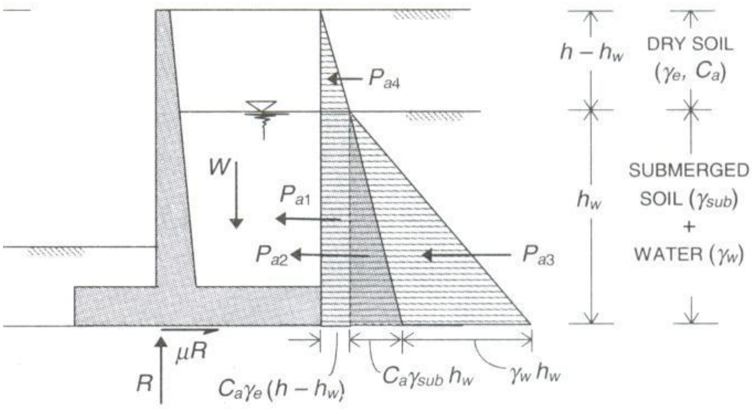
$P_a = P_{a1} + P_{a2}$, where,

$P_{a1} = C_a.W_s.h = C_a.\gamma_e.h_s.h$ & [h/2 above heel]

$P_{a2} = Ca.\ \gamma_e.h^2/2$ [h/3 above heel]

Note : 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. Effect of water in the backfill:



Effect of water in the backfill

Stability requirements FOS against $\dfrac{Overtuning}{Sliding} \geq 1.4$

i) Overturning:

$FOS_{overturning} = \dfrac{0.9Mr}{Mo} \geq 1.4$

a) For sloping backfill, $M_o = (P_a.Cos\,\theta).\dfrac{h'}{3} = C_a.\gamma_e.\dfrac{h'^3}{6}.Cos\,\theta$

$Mr = W(L - X_w) + (P_a.Sin\,\theta).L$

b) For level backfill [with surcharge, $M_o = (P_{a1}(h/2) + (P_{a2}(h/3)$.

$Mr = W(L - X_w)$ [as $\theta = 0^o$]

ii) Sliding: [Friction between base slab and supporting soil]

$F = \mu.R$ [where, R = W]

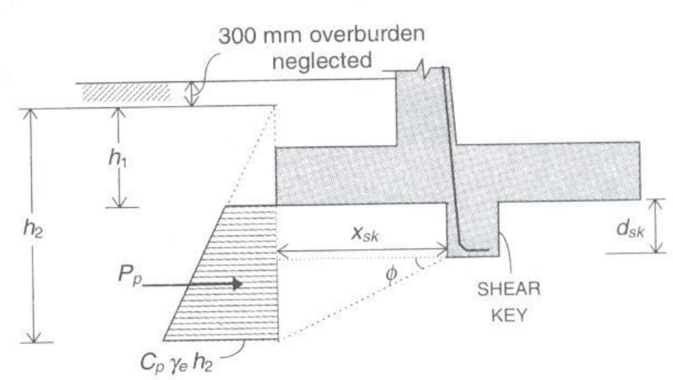
R -> Resultant soil pressure at footing base

μ -> Coefficient of static friction [0.35 – Silt & 0.60 – Rough rock]

$FOS_{sliding} = \frac{0.9F}{P_a Cos\theta} \geq 1.4$

When P_a is very high, shear key projection can be provided below footing base [Produces passive resistance P_{ps}, which is generally neglected, otherwise].

Sliding is reduced by providing shear key [like a plug, anchors inside]



Passive resistance due to shear key

$$P_{ps} = C\gamma_e(h_2^2 - h_1^2)/2$$

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.

Preliminary proportioning of cantilever retaining wall:

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

$$L_{min} = \left(\sqrt{\frac{C_a}{3}}\right)\frac{h'}{\alpha_R}$$

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

$$X = \left(\sqrt{\frac{C_a}{3}}\right)h'$$

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^o with the horizontal. Assume good soil for foundation at a depth of 1.25m below ground level with SBC of 160kN/m^2. Further assume the backfill to comprise of granular soil with unit weight of 16kN/m^3 and an angle of shearing resistance of 30^o. Assume the coefficient of friction between soil and concrete to be 0.5.

Given: h = 4.0 + 1.25m

θ = 15^o

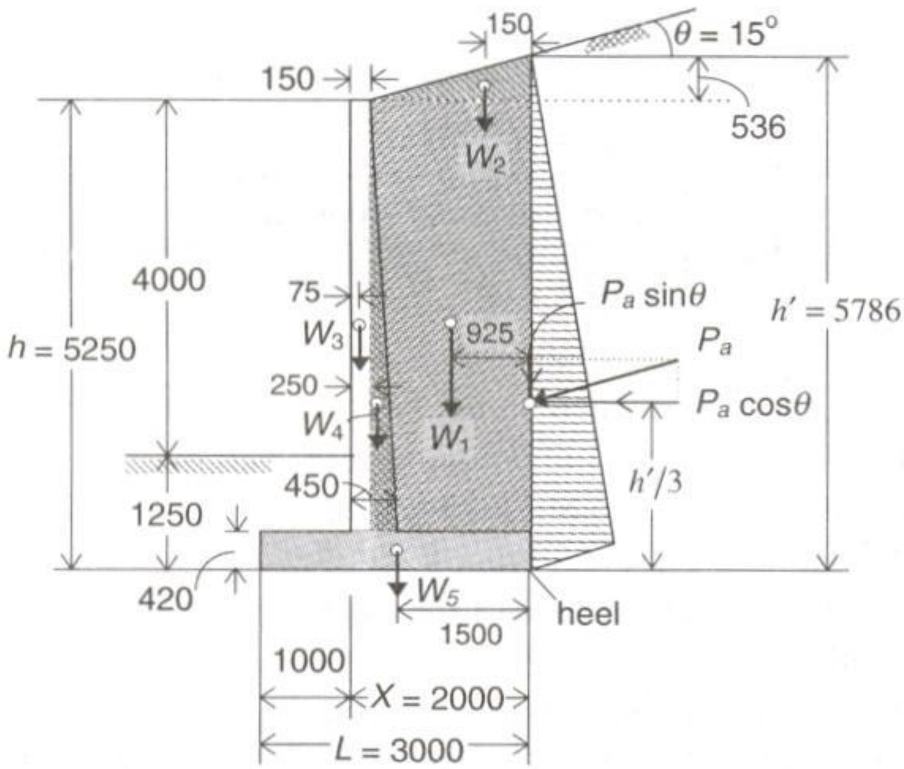
ф = 30^o

γ_e = 16kN/m^3

q_a = 160kN/m^2

μ = 0.5

Minimum depth of foundation (Rankine's),



$d_m = \frac{q_a}{\gamma_e}\left(\frac{1-\sin\phi}{1+\sin\phi}\right)^2 = \frac{160}{16}\left(\frac{1}{3}\right)^2 = 1.11m$

Earth pressure coefficient,

$$C_a = \left[\frac{Cos\theta - \sqrt{Cos^2\theta - Cos^2\phi}}{Cos\theta + \sqrt{Cos^2\theta - Cos^2\phi}}\right]Cos\theta = 0.373$$

$$Cp = \frac{1+Sin\phi}{1-Sin\phi} = 3.0$$

<u>Preliminary proportioning</u>:

Thickness of footing base slab = 0.08h = 0.08 x 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.

$X = \left(\sqrt{\frac{C_a}{3}}\right)h' = \sqrt{\frac{0.373}{3}}(5.25+0.4)$ = 2.0m [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.

<u>Stability against overturning</u>:

Force ID	Force (kN)	Distance from heel (m)	Moment (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^o=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**

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 ф for granular soil does not change significantly.

Force due to active pressure,

$P_a = Ca.\ \gamma_e.h'^2/2$

Where, $h' = h + X\tan\theta$

$= 5250 + 2000\tan15^o = 5786mm$

$P_a = 0.373(16)(5.786)^2/2 = 99.9kN$ [per m length of wall]

$$FOS = \frac{0.9 \text{ x Stabilising force or moment}}{\text{Destabilising force or moment}} \geq 1.4$$

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

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,

$X_w = M_w/W = 230.6/232.9 = 0.99m$

Stabilising moment (about toe),

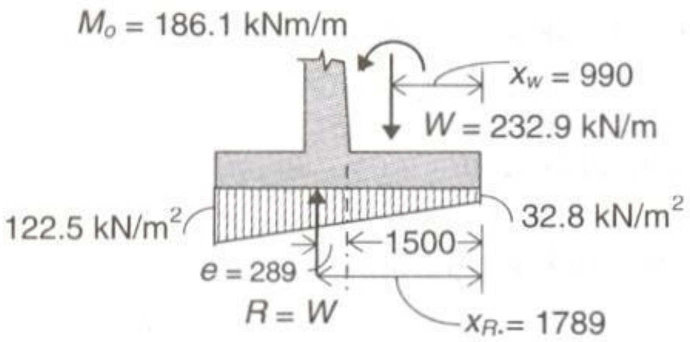
$M_r = W(L – X_w) + P_a\sin\theta(L) = 232.9(3 – 0.99) + 77.6$

$= 468.1kNm$ [per m length of wall]

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

<u>Soil pressure at footing base</u>:

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



calculation of soil pressures

Distance of R from heel, $L_R = (M_w + M_o) / R = (230.6 + 186.1)/232.9 = 1.789m$

Eccentricity, $e = L_R – L/2 = 1.789 – 3/2 = 0.289m < L/6$ ->[0.5m]

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

Maximum pressure as base,

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

$q_{min} = \frac{R}{L}\left(1-\frac{6e}{L}\right) = \frac{232.9}{3}(1-0.578) = 32.8kN/m^2 > 0$ [No tension develops] [Safe].

Stability against sliding:

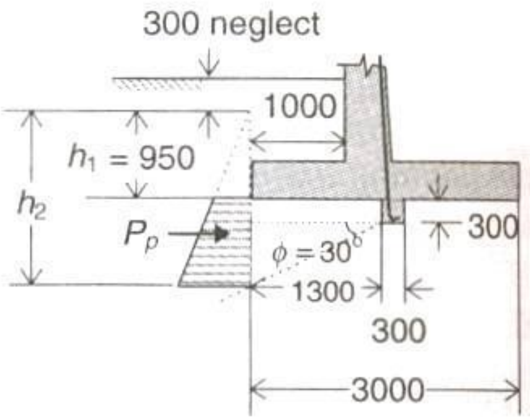
Sliding force, $P_a Cos\theta$ = 96.5kN

Resisting force, F = μR = 0.5 x 232.9 = 116.4kN [Ignoring passive pressure on toe side]

FOS(Sliding) = $\frac{0.9F}{P_a Cos\theta} = \frac{0.9x116.4}{96.5} = 1.085 < 1.40$ [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 + 1300tan30^o = 2001mm

Pp = $C_p.\gamma_e.(h_2^2 - h_1^2)/2$ = 3 x 16 x (2.001^2 – 0.950^2)/2 = 74.11kN.

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

Design of toe slab:

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

d = 420 – 75 – 8 = 337mm

V_u = $1.5\left[\frac{112+81.9}{2}\right]x(1-0.337) = 96.42kN$ [V_u is design shear at ‘d’ from face of stem]

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

= 76.48 kNm/m length

Nominal shear stress, $\tau_v = \frac{V_u}{bd} = \frac{96.42x10^3}{1000x337} = 0.286N/mm^2$

Using M20 concrete,

For a $\tau_c = 0.29$N/mm2, p_t (required) = 0.2% [Page 178, SP-16]

K = $\frac{M_u}{bd^2} = \frac{76.48x10^6}{1000x337^2} = 0.673N/mm^2$ [Page 48, SP-16]

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

Spacing = $\frac{1000x\pi x16^2/4}{674}$ = 298mm

Provide 16mmϕ @ 290mm c/c at bottom of toe slab

Ld = $\frac{\phi\sigma_s}{4\tau_{bd}} = \frac{(16).0.87f_y}{4\tau_{bd}}$ =16 x 47 = 752mm, beyond face of stem.

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

Design of heel slab:

V_u = $1.5\left[\frac{82.54+128.6}{2}\right]x(1.55-0.337) = 128.06kN$

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

= 203.96 kNm/m length

Nominal shear stress, $\tau_v = \frac{V_u}{bd} = \frac{128.06x10^3}{1000x337} = 0.38N/mm^2$

Using M20 concrete,

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

K = $\frac{M_u}{bd^2} = \frac{203.96x10^6}{1000x337^2} = 1.8N/mm^2$ [Page 48, SP-16]

For p_t = 0.565%, A_{st} = 0.565/100 x 1000 x 337 = 1904.05 mm^2 / m

Spacing = $\frac{1000x\pi x16^2/4}{1904.05}$ = 105.61mm

Provide 16mmϕ @ 100mm c/c at bottom of toe slab

L_d = $\frac{\phi\sigma_s}{4\tau_{bd}} = \frac{(16).0.87f_y}{4\tau_{bd}}$ =16 x 47 = 752mm, beyond face of stem.

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

Design of vertical stem:

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

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

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

M_u = 1.5(C_a.γ_e.h^3/6) = 1.5(1/3)(16 x 4.92^3/6) = 150.24kNm.

K = $\frac{Mu}{bd^2}$ = $\frac{150.24x10^6}{1000x4.92^2}$ = 1N/mm^2.

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

Spacing = 1000 x 201/1200 = 160mm

Provide 16mmϕ @ 160mm c/c in the stem, extending into the shear key upto 47Φ =

752mm.

Check for shear: [at 'd' from base]

V_u (stem) = 1.5(C_a.γ_e.Z^2/2) = 1.5(1/3 x 16 x (4.83 – 0.392)^2/2) = 53.83kN

$\tau_v = \frac{V_u}{bd} = \frac{53.83x10^3}{1000x392} = 0.135N/mm^2 < \tau_c$ [where, ζ_c = 0.39N/mm^2 for p_t = 0.3%))

Hence, SAFE.

Curtailment of bars:

Curtailments of bars in stem are done in two stages:

At 1/3rd and 2/3rd height of the stem above base.

Temperature and shrinkage reinforcement:

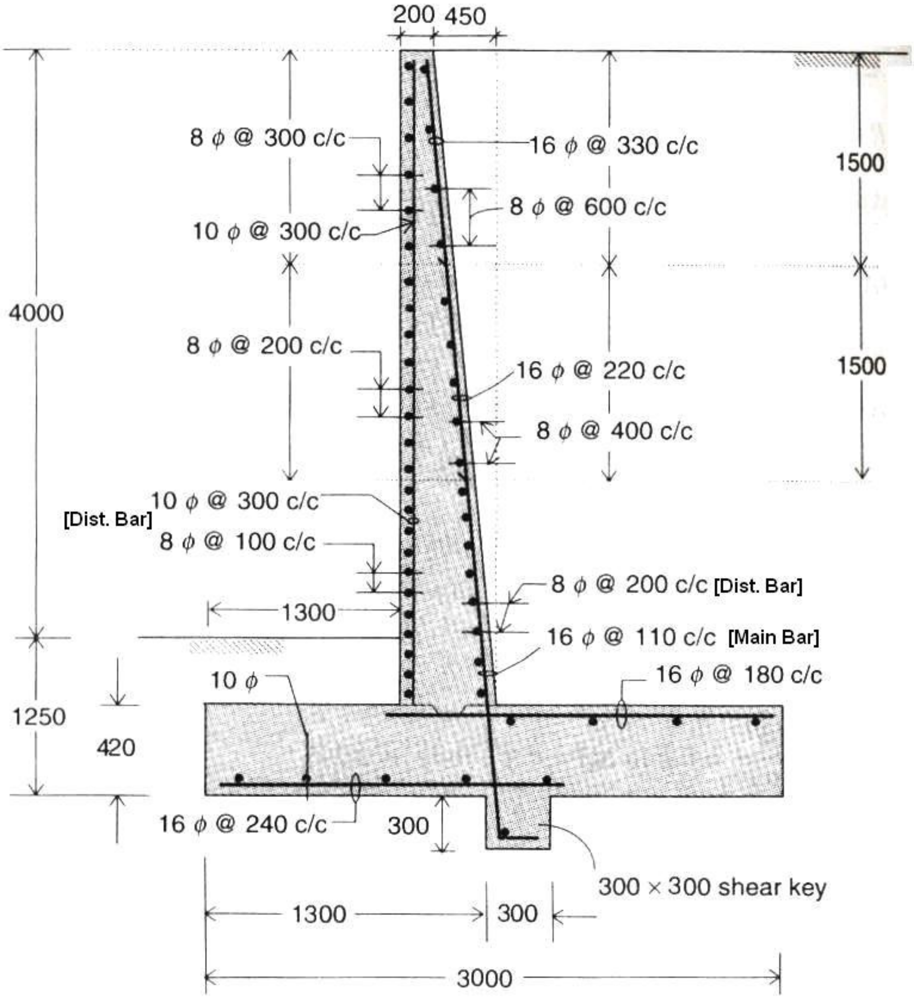
A_{st} = 0.12/100 x 10 x 450 = 540mm^2

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

For II 1/3rd height, provide 1/2 the above.

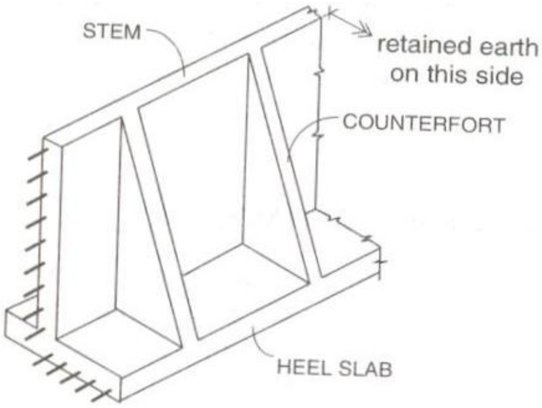
For III 1/3rd height, provide 1/3rd of I case.

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



Detailing of cantilever wall

DESIGN OF COUNTERFORT RETAINING WALL

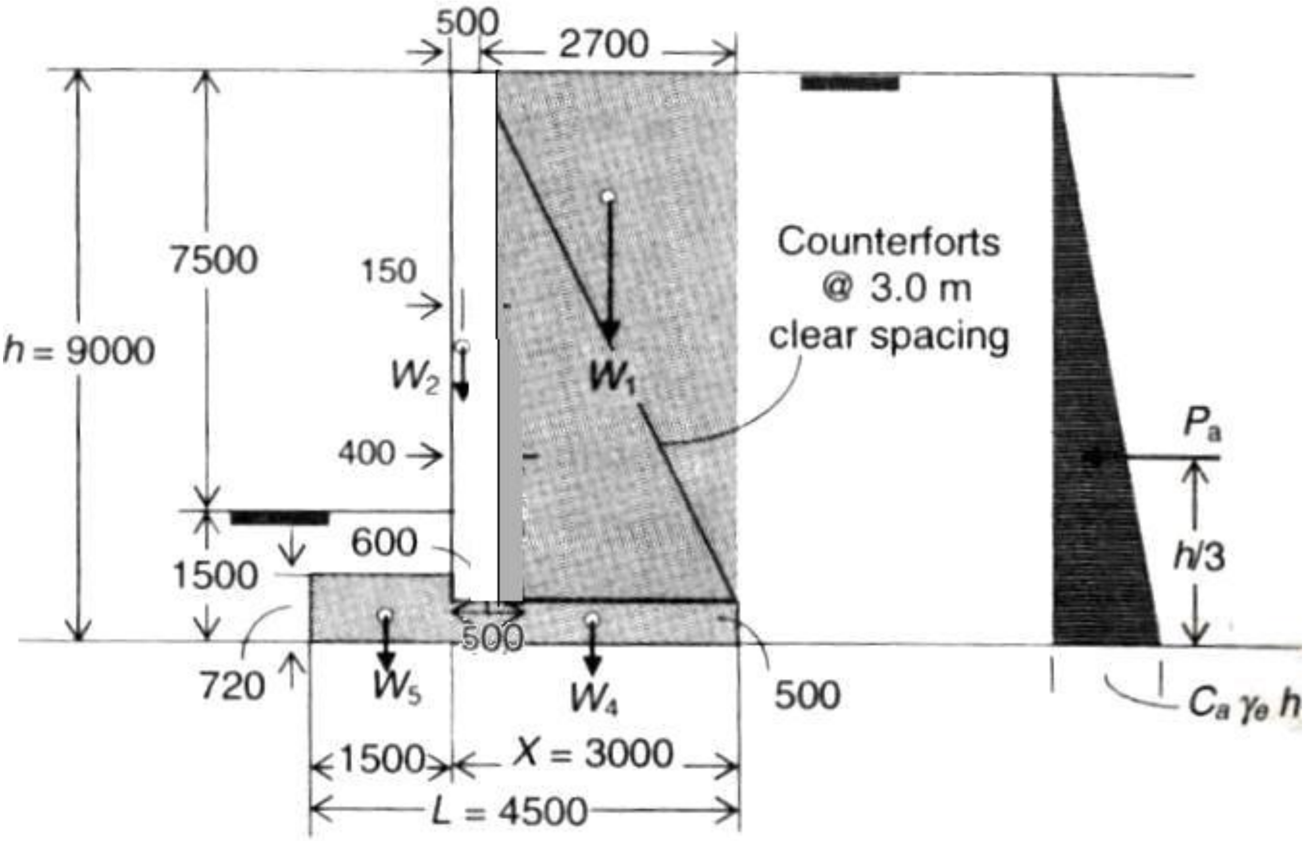


(c) counterfort wall

Preliminary proportioning of counterfort retaining wall:

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 = 1/3rd 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 $\varphi = 30^o$. The coefficient of friction between the soil and concrete is 0.5. Use M25 concrete and Fe415 steel.



Minimum depth of foundation = $\frac{P}{\gamma}\left(\frac{1-\sin\varphi}{1+\sin\varphi}\right)^2 = \frac{170}{16}\left(\frac{1-\sin 30}{1+\sin 30}\right)^2 = 1.181m < 1.5m$

Depth of foundation = 1.5m

Height of wall = 7.5 + 1.5 = 9m

Thickness of heel and stem = 5% of 9m = 0.45m ≈ 0.5m

Thickness of toe slab = 8% of 9m = 0.72m

$X_{\min} = \left(\sqrt{\frac{C_a}{3}}\right)h' = \sqrt{\frac{0.333}{3}}(9)$ = 3.0m [As C_a = 1/3 & C_p = 3]

L_{min} = 1.5 x 3 = 4.5m

Thickness of counterfort = 6% of 9 = 0.54m

Stability conditions:

Earth pressure calculations:

Force ID	Force (kN)	Distance from heel (m)	Moment (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 = $P_a.h/3 = C_a.\gamma_e.h^3/6$ = 0.33x16x93/6 = 647.35kNm.

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

FOS $_{(overturning)}$ = 1.98 > 1.4

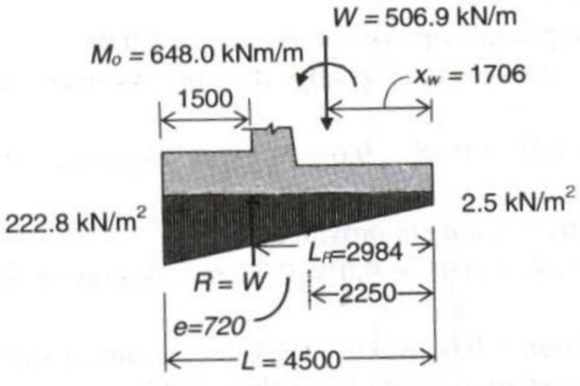
Hence, section is safe against overturning.

Sliding:

FOS $_{(sliding)}$ = $0.9(\mu R)/P_a\cos\theta$

F = μR = 0.5 x 510.75 = 255.375kN

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



Base pressure calculation:

$$q_{\max} = \frac{R}{L}\left(1+\frac{6e}{L}\right) = \frac{510.75}{4.5}(1+\frac{6x0.73}{4.5}) = 223.97kN/m^2 > q_q$$ [where q_a = 170kN/m²] [UnSafe].

$$q_{\min} = \frac{R}{L}\left(1-\frac{6e}{L}\right) = \frac{510.75}{4.5}(1-\frac{6x0.73}{4.5}) = 3.027kN/m^2 > 0$$ [No tension develops] [Safe].

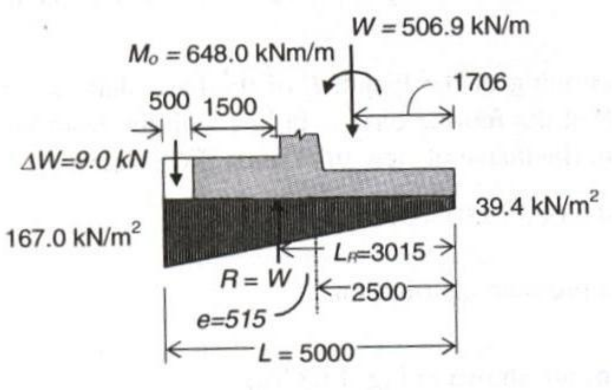
Where, L_R = (M + M_o)/R, e = L_R – L/2, where,

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

∑M = 874.69 + 42.75 = 917.438kNm.

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

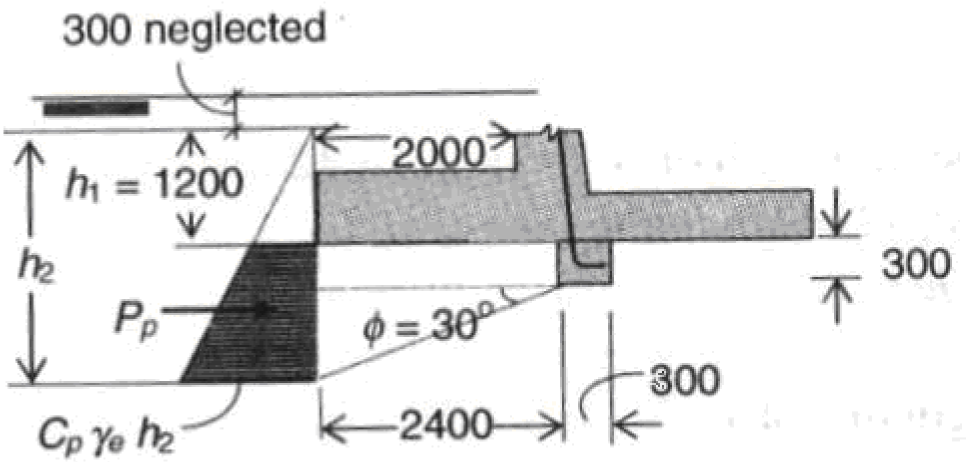
e = L_R – L/2 = 3.011 – (5/2) = 0.511m < L/6 → (0.833m)

$$q_{max} = \frac{R}{L}\left(1+\frac{6e}{L}\right) = \frac{519.75}{5}(1+\frac{6x0.511}{5}) = 166.32kN/m^2 < q_q$$ [where q_a = 170kN/m²] [Safe].

$$q_{min} = \frac{R}{L}\left(1-\frac{6e}{L}\right) = \frac{519.75}{5}(1-\frac{6x0.511}{5}) = 41.58kN/m^2 > 0$$ [No tension develops] [Safe].

FOS $_{(Sliding)}$ = 0.9(μR)/P_a = 0.9(0.5 x 519.75)/215.784 = 1.08 < 1.4.

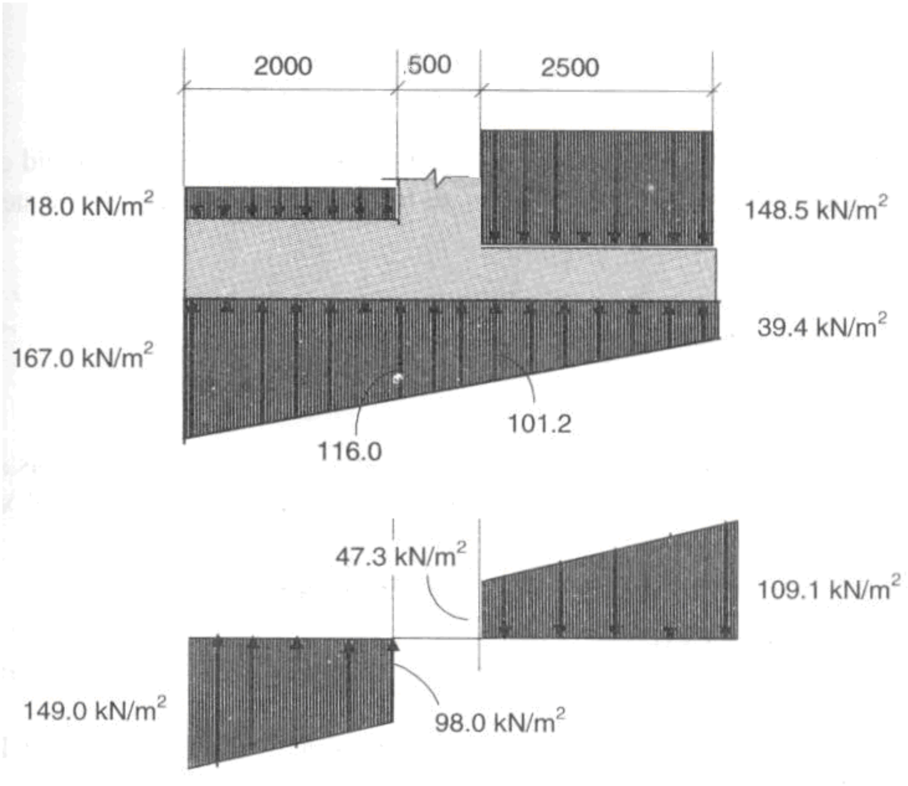
Hence the section is not safe against sliding. Shear key is provided to resist sliding. Assume shear key of size 300 x 300mm.



$P_{ps} = C_p.\gamma_e.(h2^2 – h1^2)/2 = 164.5kN/m$

FOS $_{(sliding)}$ = $0.9(\mu R + P_{ps})/P_a = 1.77 > 1.4$ [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

Design of Toe Slab:

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 = 80.38x2^2/2 + 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/bd^2 → A_{st} = 1551.25mm^2 → Spacing = 1000a_{st}/A_{st} → 16mm @125mmc/c.

Transverse reinforcement: = 0.12% of c/s
= 0.12/100 x 1000 x 720 = 864mm^2

Provide 10mm @100mm c/c.

<u>Design of heel slab</u>:

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 x 2.5^2/12 = 55.688kNm.

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 A_{st} required at midspan and support are found.

Factored M_{sup} = 83.53kNm → Ast = 570.7mm^2

Factored M_{mid} = 62.64kNm → Ast = 425.4mm^2

Using 16mm φ bar, Spacing = 1000ast / Ast = 110.02mm → 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 = 600mm^2

For 8mm bar, Spacing = 83.775mm → Provide 8mm @80mm c/c.

<u>Check for shear</u>:

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

Factored shear force = 134.0175kN

ζv = 0.33N/mm^2, ζc = 0.29N/mm^2, ζcmax = 3.1N/mm^2

Depth has to be increased.

Design of stem:

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

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

$M_{mid} = wl^2/16 = 1.5 \times 45.33 \times 3.54^2/16 = 53.26$kNm

Effective depth = 500 – (50 + 20/2) = 440mm

A_{st} at support = 1058mm^2, For 16mm φ, Spacing = 190mm. Provide 16mm @ 190mm c/c

A_{st} at midspan =718mm^2, For 16mm φ, Spacing = 280mm. Provide 16mm@280mm c/c

Max. SF = w (l/2 – d) = 60.29kN, Factored SF = 90.44kN

Transverse reinforcement = 0.12% of c/s → 8mm @ 80mmc/c

ζ_v = 0.188N/mm^2, ζ_c = 0.65N/mm^2, ζ_{cmax} = 3.1N/mm^2 Safe in Shear.

Design of Counterfort:

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, $M_{max} = K_a.\gamma_e.h^3/6 \times L_e$

Where, L_e → c/c distance from counterfort

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

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

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

Curtailment of reinforcement:

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.

Design of horizontal ties:

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.

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

T = 1/3 x 16 x (9 – 0.5) x 3.54 = 160.48kN

Factored force, T = 1.5 x 160.48kN

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

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

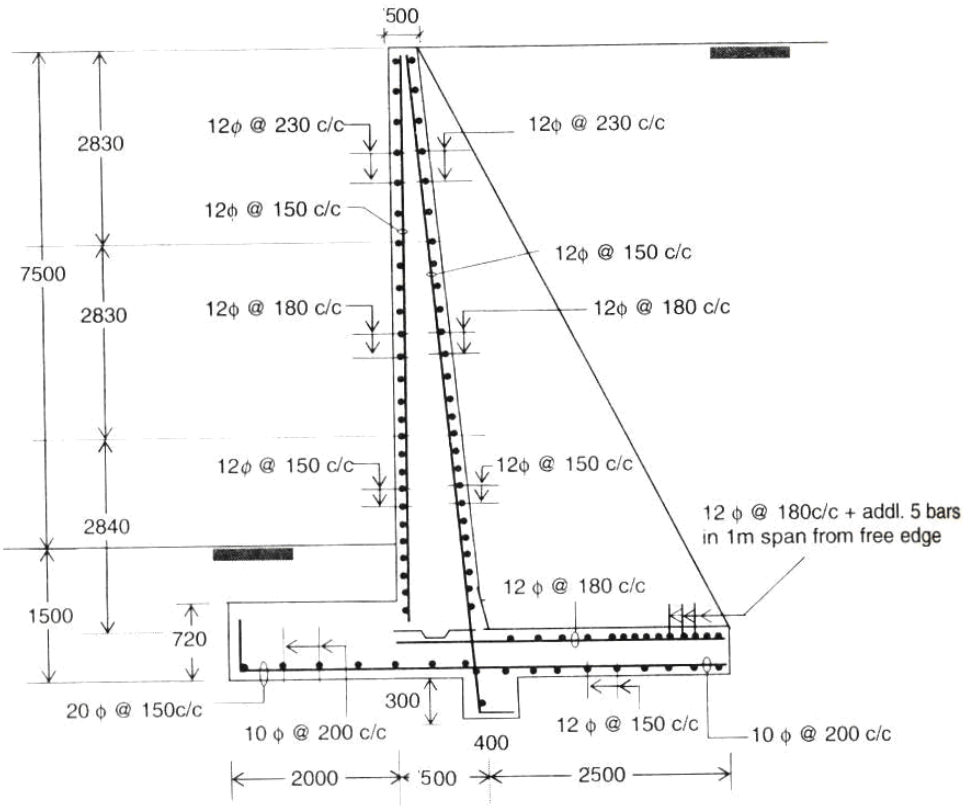
Design of vertical ties:

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

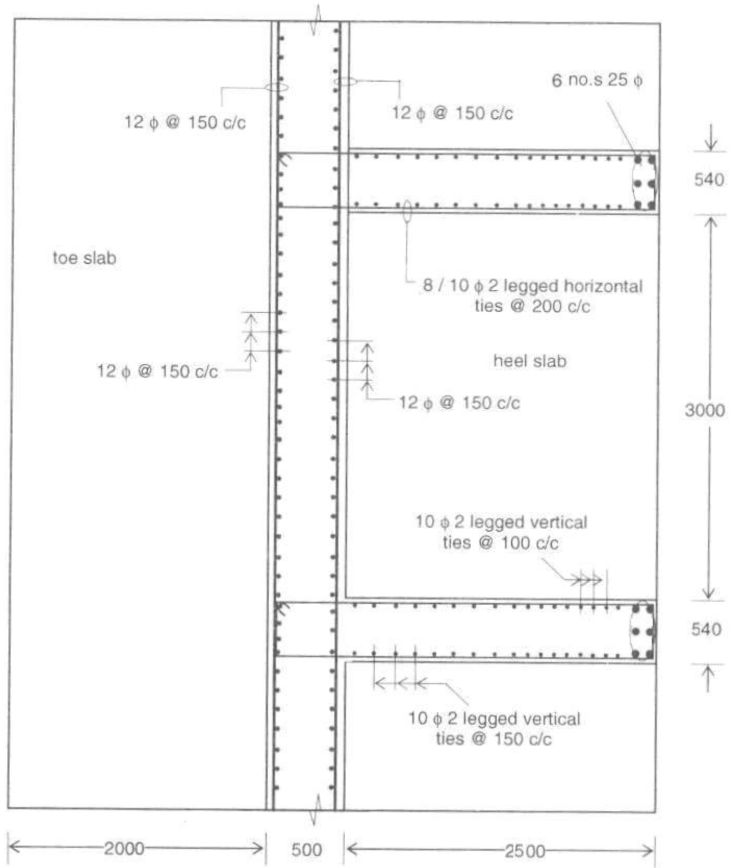
Factored T = 399.74 kN

Ast = 1107.15mm^2. For 10mm φ, Spacing = 70.93mm.

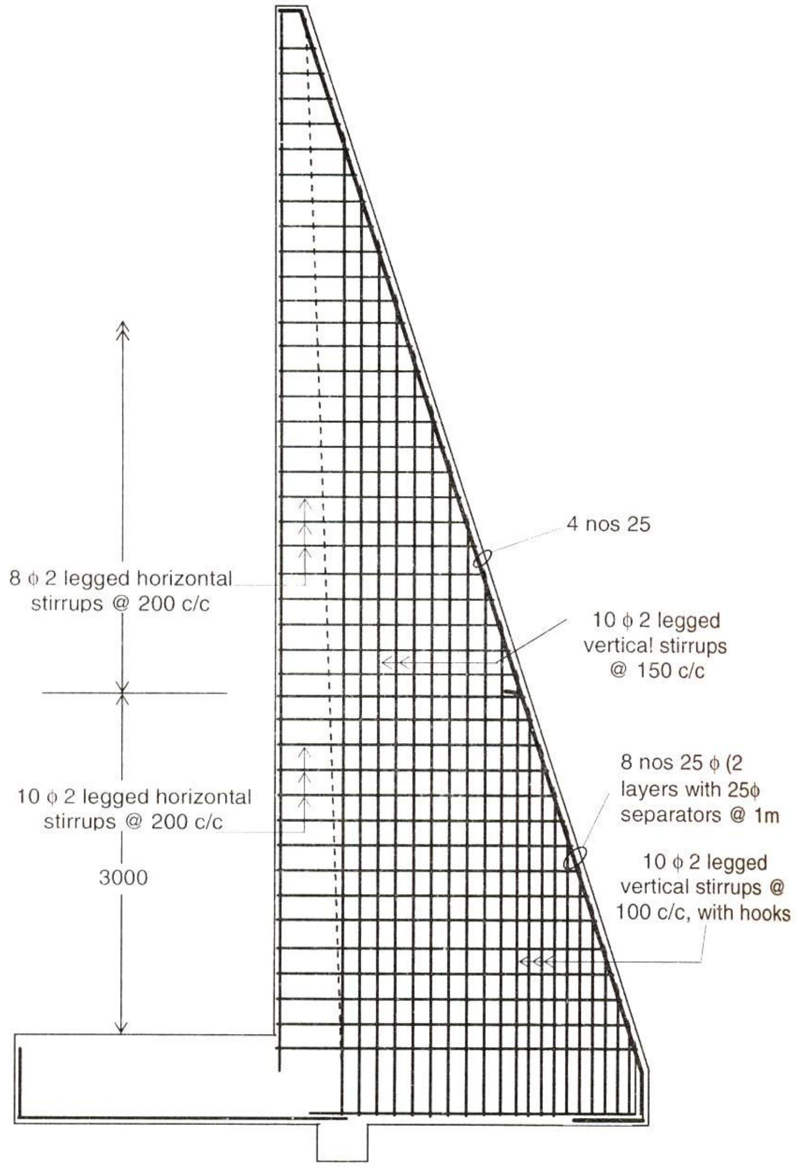
Provide 10mm @ 70mm c/c.



Reinforcement details of stem, toe slab and heel slab



Reinforcement details of stem and counterfort



Section through counterfort showing counterfort reinforcement

UNIT II

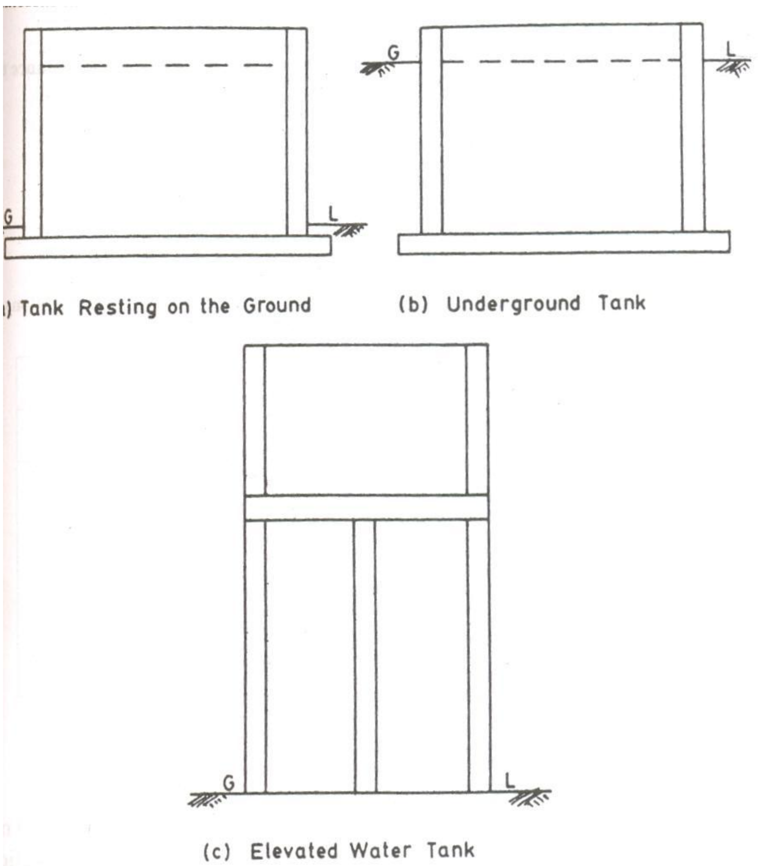
DESIGN OF WATER TANK

IS3370 [Part I – IV]

Cylindrical   Rectangular

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



Types of Water Tanks

Components of water tank:

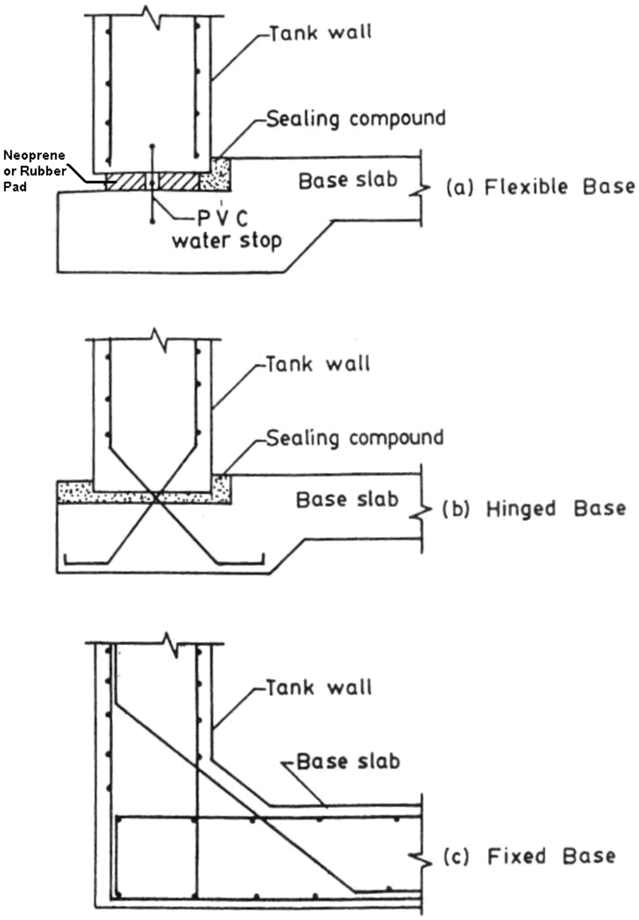
1. Side Walls [Rectangular or cylindrical]
2. Base slab
3. Cover slab or dome
4. Staging [Overhead] → Columns, Beams, Bracings

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 → σ_{cbc} = 7 N/mm^2, σ_t = 5 N/mm^2

For Fe250 steel, σst = 130 – 140 N/mm^2

For Fe415 steel, σst = 230 – 240 N/mm^2

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

Stress (N/mm^2)	Grade of Concrete					
	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 Stresses in Steel reinforcement for Strength Calculations in Water retaining Structures (IS: 3370-Part-II-1965)

Stress (N/mm^2)	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,

$$\sigma = \frac{Ft}{A_c + mA_{st}}$$ where, m → modular ratio = 280 / 3 σ_{cbc} = E_s/E_c

σcbc is the permissible compressive stress.

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

Tensile stress: Fe250 → 115 N/mm^2, Fe415 → 150 N/mm^2

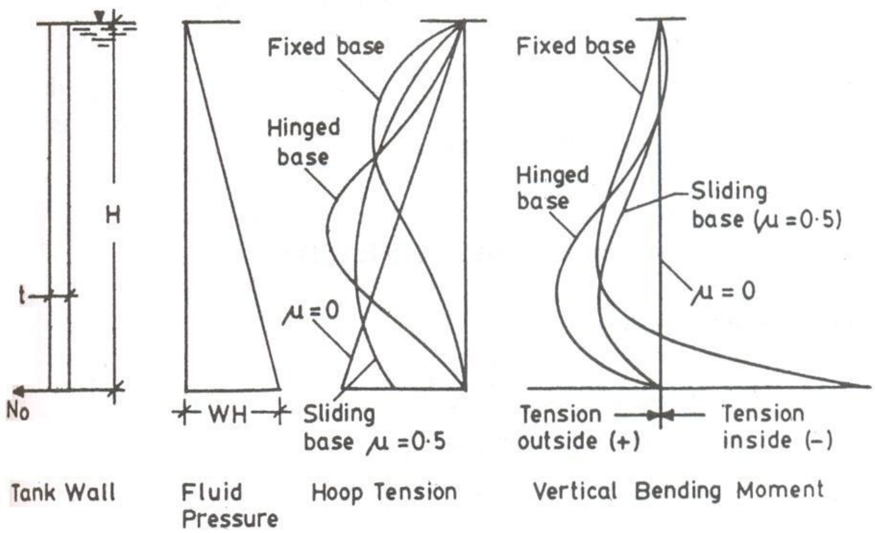
Bending stress:

	< 225mm		> 225mm	
	On Face	On face away	On Face	On face away
Fe250	115	115	115	125
Fe415	150	150	190	175

Reinforcement requirements: [As per IS3370]

1. Minimum A_{st} is 0.3% for 100mm section and 0.2% for 450mm section.
2. 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 m^3]

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

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 = $\frac{\gamma.h.D}{2}$ = (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/mm^2.

A_{st} required = 214.34 x 10^3 / 150 = 1428.9 mm^2

Spacing = 1000.ast / Ast, → 16mm @ 140mm c/c

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

$$\sigma_{ct} = \frac{Ft}{A_c + mA_{st}}$$

Ac = 1000t, F = 214.34kN, m = 280 / 3 σ_{cbc} = 280 / (3 x 7) = 13.33 ≈ 13

$$\sigma_{ct} = 1.2 = \frac{214.34}{1000t + (13.33x1428.9)}$$

➔ t = 160mm

Minimum thickness as per empirical formula is,

t_{min} = (30h + 50)mm where h → 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) x 1000 x 170 = 476mm^2

Provide 8mm @ 100mm c/c

Curtailment of reinforcement:

At 2m height,

Hoop tension = $\frac{\gamma.h.D}{2}$ = (9.81 x 1.8 x 11.5) / 2 = 101.5335 kN

Ast = 101.535 x 103 / 150 = 676.89 mm^2

Provide 10mm @ 110mm c/c (or) 16mm @ 290mm c/c

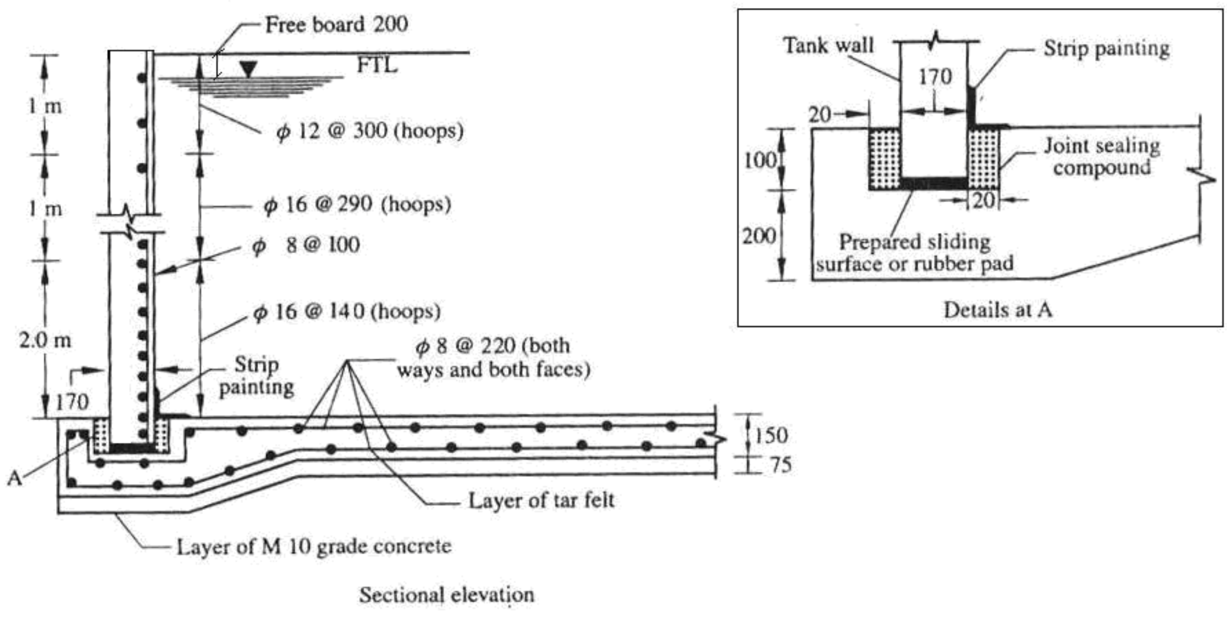
Design of base slab:

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 → 0.3/100 x 150 x 1000 = 450 mm^2, 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.

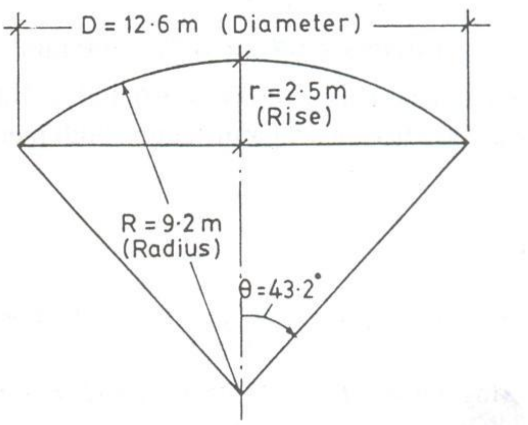


Design of dome: - Roof covering for cylindrical water tanks

[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 x 11.5 = 2.3m



Details of Dome of Circular Water Tank

Radius of curvature of the dome [R]

$R^2 = (R – 2.3)^2 + 5.75^2$

→ R = 8.33m

Cosθ = 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.

σ_c = 5 N/mm^2

Assume thickness of dome as 100mm

Meridonial thrust, T = $\frac{WR}{1+Cos\theta}$, 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 x 25	= 2.5 kN/m^2
Live load		= 2kN/m^2
	W	= 4.5 kN/m^2

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

Meridonial thrust = T / (c/s area) = 21.46 x 10^3 / (100 x 1000) = 0.21 N/mm^2 < 5 N/mm^2

The hoop stress developed in dome slab is given by,

Hoop stress = $\frac{WR}{t}\left(Cos\theta - \frac{1}{1+Cos\theta}\right)$ = $\frac{4.5x8.3}{100}\left(0.74 - \frac{1}{1+0.74}\right)$ = 0.06 N/mm^2 < 5 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 = 300mm^2 → 8mm @ 160mm 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, $H_{L\ Comp} = T.Cos\theta \times D/2 = 21.46 \times 10^3 \times 0.74 \times 5.75 = 91.3kN$

$A_{st} = 91.3 \times 103 / 150 = 608\ mm^2$

Provide (2# 16mmφ + 2# 12mmφ) → (626mm²)

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

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

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.

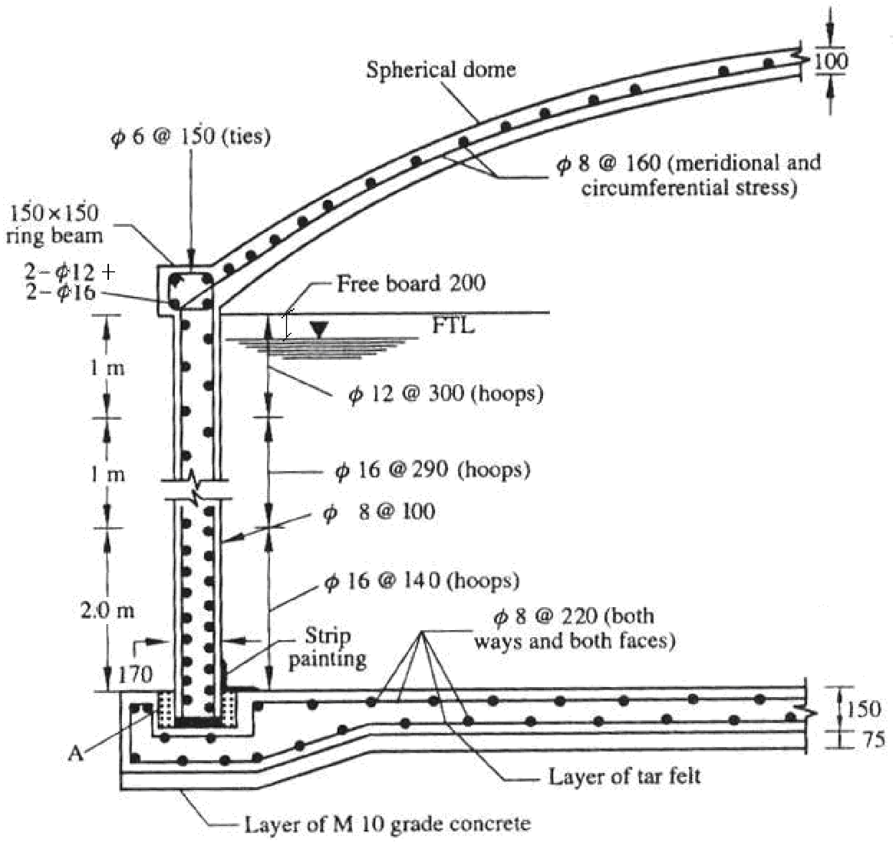
$\sigma_{ct} = 2.8\ N/mm^2$

→ $2.8 = \frac{F_t}{A_c + 13.3x608}$ → $A_{st} = 608\ mm^2$

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

→ $A_c = 25314 mm^2$.

Provide 150 x 150 mm size of ring beam with (2# 16mmφ + 2# 12mmφ) 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, $\frac{H^2}{Dt}$. 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 $\frac{H^2}{Dt}$ 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 → Radius of tank, D → Diameter of tank

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

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

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

➔ $\frac{H^2}{Dt}$ → 8.18, Hoop Tension Coeff. → 0.578 @ 0.6H

Therefore, T = Coeff. X wHR = 0.578 x 9.81 x 4 x 11.5/2 = 130.41 kN

$\frac{H^2}{Dt}$ → 8, BM Coeff. → -0.0146

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

➔ $\frac{H^2}{Dt}$ → 8.18, BM Coeff. → 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 x 10^3 / 150 = 869.33 mm^3.

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

For bending moment,

$A_{st} = \frac{M}{\sigma_{st}.j.d}$

Where, for M20, j = 0.84, R = 1.16, σ_{cbc} = 7 N/mm^2

$A_{st} = \frac{9.03}{150x0.84x129}$ = 555 mm^2

d = D – φ_{HT} – φ_{BM}/2 - C_C
= 170 – 12 – 8/2 – 25 = 129mm

$d = \sqrt{\frac{M}{R.b}} = \sqrt{\frac{9.2x10^6}{1.16x1000}}$ = 88.23mm < d [129mm]

Min. depth required = 88.23mm

0.3% of c/s (vertical steel) = 0.28/100 x 1000 x 170 = 476 mm^2

Provide 8mm @ 100mm c/c

The maximum of Ast for BM at A_{st} {maximum} is provided as vertical reinforcement [555mm^2].

Provide 8mm @ 90mm c/c

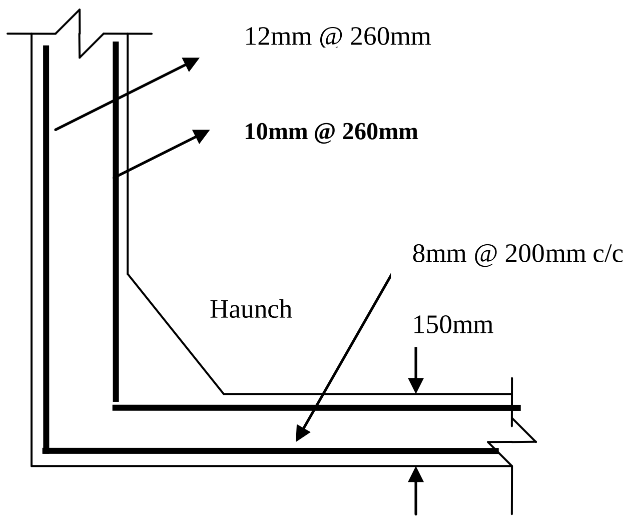
Provide 10mm @ 130mm c/c as vertical reinforcement. If the reinforcement is provided as double layers on both the faces, the spacing is doubled.

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.

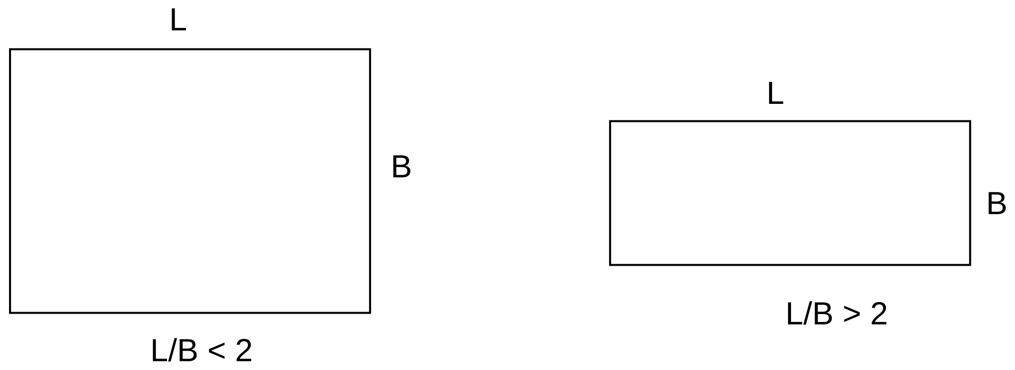
Design of base slab:

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.



DESIGN OF RECTANGULAR WATER TANK



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 → Hoop Tension → Transverse reinforcement]

When L/B < 2,

i) Cantilever moment at base = $w.H.h^2 / 6$

where, h = 1m or H/4, whichever greater

ii) For horizontal bending of the walls, the maximum moment is found from,

a) Fixed end moments,

$PL^2/12$ & $PB^2/12$

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 → 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,

$A_{stL} = \frac{M - P_L x}{\sigma_{st}.j.d} + \frac{P_L}{\sigma_{st}}$; $A_{stB} = \frac{M - P_B x}{\sigma_{st}.j.d} + \frac{P_B}{\sigma_{st}}$

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 x B/2 & P_B = p

1. Design a rectangular tank of size 4m x 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 or H/4 ➔ 1m or 0.75m = 1m [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	AD
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 x 6^2 / 12 = 58.86 kNm (3p)

M_{FAD} = pB^2 / 12 = 19.62 x 4^2 / 12 = 26.16 kNm (1.33p)

Joint A,

	AB	AD
FEM	3p	-1.33p
DF	0.4	0.6
BM	(-1.67 x 0.4)	(-1.67 x 0.6)
	= -0.67p	= -1.002p
CO	-	-
M_f	+ 2.33p	- 2.33p

Mf = 2.33p = 2.33 x 19.62 = 45.72kNm

Moment at midspan,

Long wall:

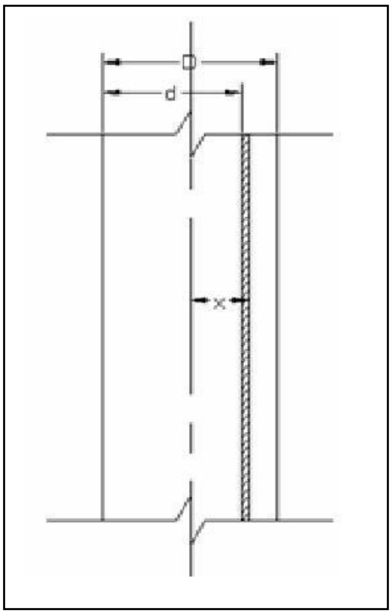
$\frac{pL^2}{8} - M_f$ = 42.57 kNm,

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.



$A_{stL} = \frac{M - P_L x}{\sigma_{st}.j.d} + \frac{P_L}{\sigma_{st}}$; $A_{stB} = \frac{M - P_B x}{\sigma_{st}.j.d} + \frac{P_B}{\sigma_{st}}$

P_L = p x B / 2 = 19.62 x 4/2 = 39.24 kN

P_B = p x L / 2 = 19.62 x 6/2 = 58.86 kN

$d_{req} = \sqrt{\frac{M}{R.b}} = \sqrt{\frac{45.72x10^6}{1.32x1000}}$ = 186mm

Provide d = 190mm, D = 220mm

Provide effective depth of 190mm and effective cover of 30mm

x = D /2 – Eff. Cover

= 220 / 2 – 30

= 80mm

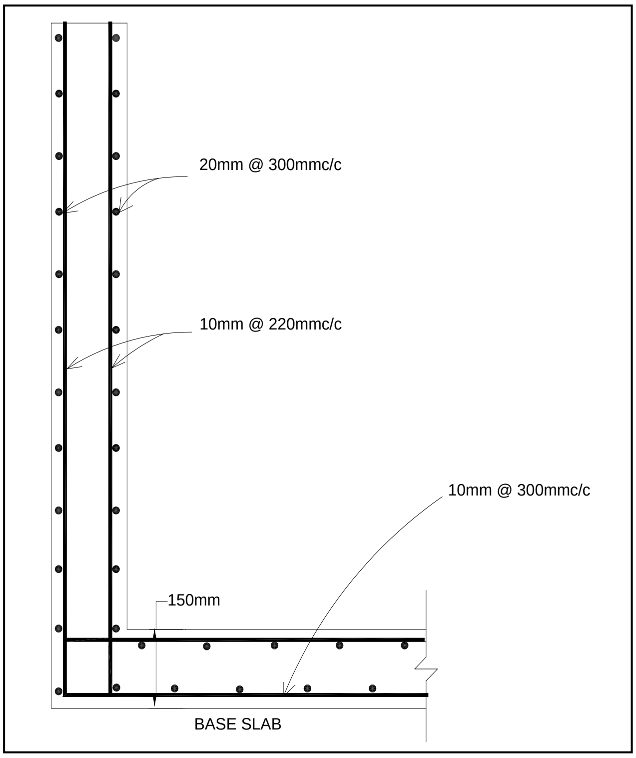
Area of steel required for Tension in the wall along the longer side and shorter side,

$A_{stL} = \frac{45.72x10^6 - (39.24x80x10^3)}{150x0.853x190} + \frac{39.24x10^3}{150}$ = 2013mm^2

$A_{stB} = \frac{45.72x10^6 - (58.86x80x10^3)}{150x0.853x180} + \frac{58.86x10^3}{150}$ = 2079mm^2

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 $= w.H.h^2 / 6$ [L / B < 2]

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

$A_{st} = \dfrac{M}{\sigma_{st}.j.d}$ = 201.76mm^2

Min. Ast = 0.3% of c/s

= 0.3/100 x 1000 x 220

= 660 mm^2

Spacing is provided for the maximum of the above two [66mm^2]

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 x 1000 x 150 = 450 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 x 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.

Long wall:

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

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

Short wall:

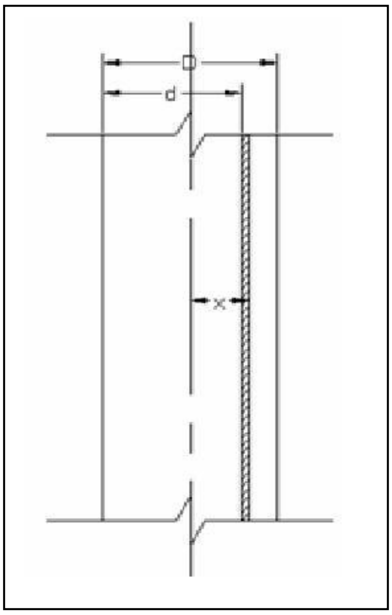
Cantilever moment = $w.H.h^2/12 = 9.81 \times 3 \times 1^2 /2 = 14.715$ kNm

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

$A_{stL} = \dfrac{M - P_L x}{\sigma_{st}.j.d} + \dfrac{P_L}{\sigma_{st}}$, where, $P_L = p \times B/2 = 19.62 \times 4/2 = 39.24$ kN and $P_B = p = 19.62$ kN



$d_{req} = \sqrt{\frac{M}{R.b}} = \sqrt{\frac{44.145x10^6}{1.32x1000}}$ = 182.87mm

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,

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

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

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

Along horizontal direction,

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

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

Distribution steel:

100mm → 0.3%

450mm → 0.2%

220mm → 0.26%

Min A_{st} = 0.26% of c/s = 0.26 / 100 x 1000 x 220 = 572mm^2

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

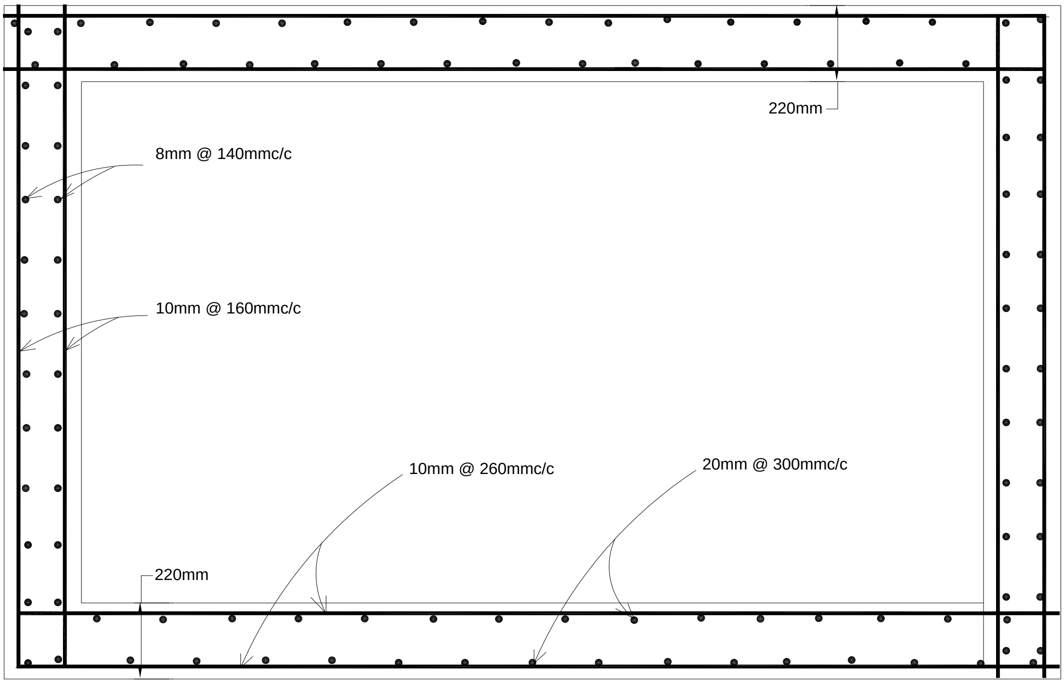
Base slab:

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 = 465mm^2.

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.



DESIGN OF UNDERGROUND WATER TANK

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 $16kN/m^3$ 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, $p = \gamma_w(H - h)$

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

ii) Earth pressure exerted on wall under tank empty condition, $p = H/3.(\gamma-\gamma_w)+\gamma_w H$

$= 48kN/m^2$

Where, h = 1m or (H/4 = 1m)

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

Long wall,

Cantilever moment = $\dfrac{\gamma_w H^3}{6}$ = $10 \times 4^3/6$ = 106.67kNm

Short wall,

Cantilever moment = $\dfrac{\gamma_w H h^2}{2}$ = 20kNm

Horizontal bending moment = $\dfrac{pB^2}{16}$ = $\dfrac{30x5^2}{16}$ = 46.875 kNm &

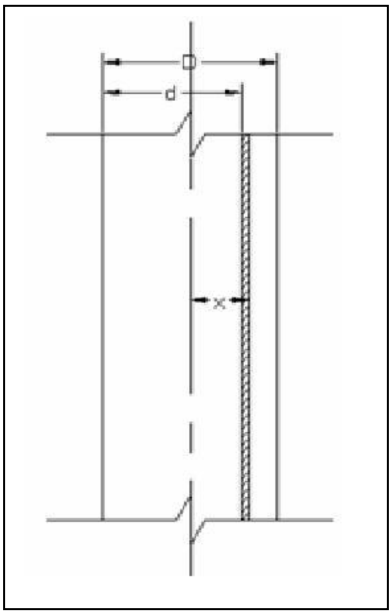
= $\dfrac{48x5^2}{16}$ = 75 kNm

<u>Long wall</u>:

$A_{stL} = \dfrac{M - P_L x}{\sigma_{st}.j.d} + \dfrac{P_L}{\sigma_{st}}$, where, P_L = p x B/2 = 48 x 2.5 = 120 kN

P_B = p = 48 kN

$d_{req} = \sqrt{\dfrac{M}{R.b}} = \sqrt{\dfrac{106.67x10^6}{1.32x1000}}$ = 284.27mm



Provide d = 290mm

Provide, D = 290 + 30 = 320mm

x = 320 / 2 – 30 = 130mm

$A_{st(VL)}$ = 2454.35 + 800 = 3254.35 mm^2

$A_{st(HL)}$ = 0.237% of c/s

(For 100mm 0.3%

For 450mm 0.2%

For 320mm 0.237%)

$A_{st(HL)}$ = 0.237 x 1000 x 320 = 758.4 mm^2

Provide 10mm @ 200mm c/c

Short wall:

$A_{st(VL)}$ = 370.8 + 320 = 690.8 mm^2 < A_{stmin}

Provide Astmin, 10mm @ 200mm c/c

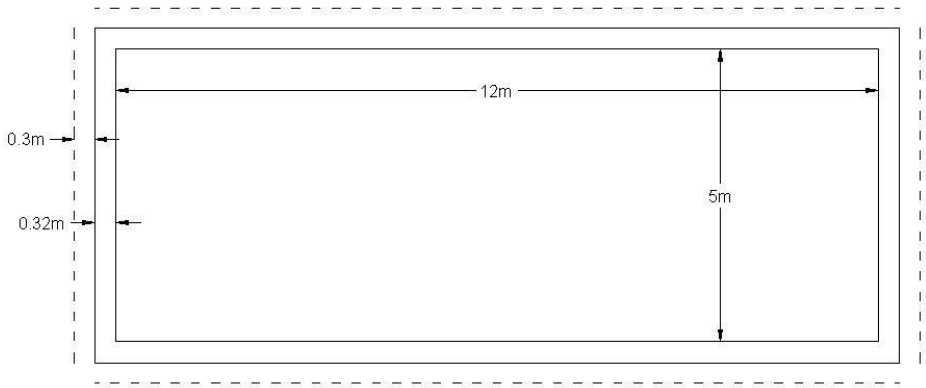
$A_{st(HL)}$ = 2173.09 mm^2

Provide 16mm @ 180mm c/c

Check for uplift:

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 x 12.64 x 0.32 x 4 x 25 = 808.36 kN

Short wall = 2 x 5 x 0.32 x 4 x 25 = 320 kN

Base slab = (12.64 + 0.6).(5.64 + 0.6) x 0.4 x 25 = 826.176kN

Total = 1955.136 kN

Weight of earth retained over projection,

Long wall = 2 x (12.54 + 0.6) x 4 x 0.3 x 16 = 508.42 kN

Short wall = 2 x 5.64 x 4 x 0.3 x 16 = 216.576 kN

= 724.996 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,

$p = H / 3.(\gamma - \gamma_w) + \gamma_w.H$

$= \frac{4.4}{3}(16 - 10) + 10 \times 4.4 = 52.8$ kN/m^2

Considering 1m length of wall, the force exerted is = ½ x H x 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 x 12.64 x 0.32 x 4 x 25 = 808.36 kN

Short wall = 2 x 5 x 0.32 x 4 x 25 = 320 kN

Base slab = (12.64 + 1.4).(5.64 + 1.4) x 0.4 x 25 = 988.416kN

Total = 2116.78 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

= 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 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.

Design of base slab:

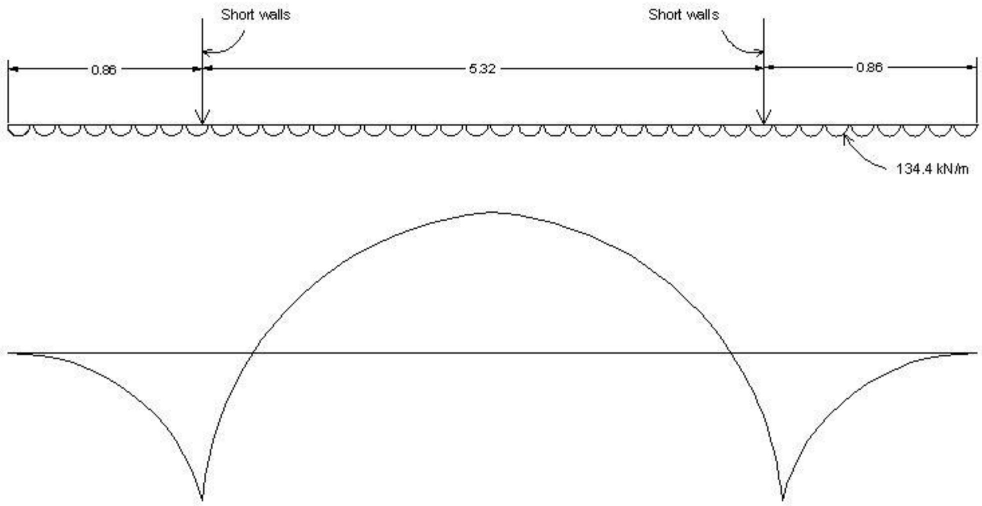
Loading on the base slab includes the self weight of base slab and weight of side walls.

Considering 1m strip,

Weight of base slab = 7.04 x 0.4 x 25 = 70.4 kN/m

Weight of walls = 2 x 4 x 0.32 x 25 = 64 kN/m

Total = 134.4 kN/m



Reaction at the support = 134.4 x 7.04 / 2 = 473.09 kN

B.M. at the support = $w.l^2/2$ = 134.4 x 0.86^2 / 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 = $\frac{M}{\sigma_{st}.j.d} = \frac{425.8x10^6}{150x0.853x365} = 9118.2mm^2$

[D = 400mm, d = 400 – 35 = 365mm]

Provide 25mmø in two layers [both faces].

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

UNIT III

YIELD LINE THEORY

Yield lines – Typical crack patterns – generated when ultimate moment is reached

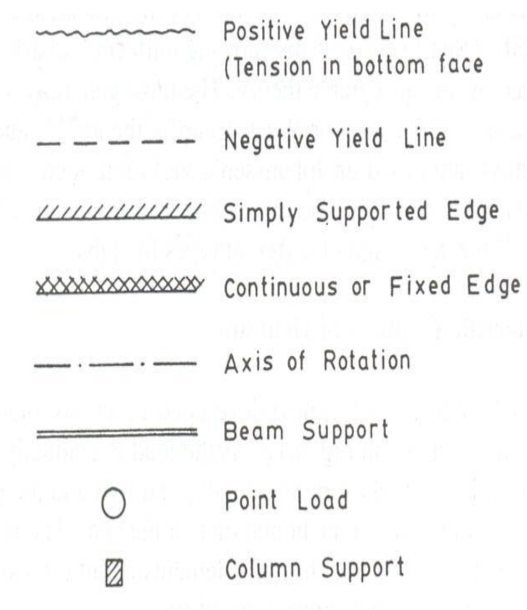
Characteristics of yield lines are,

i) Yield lines are straight

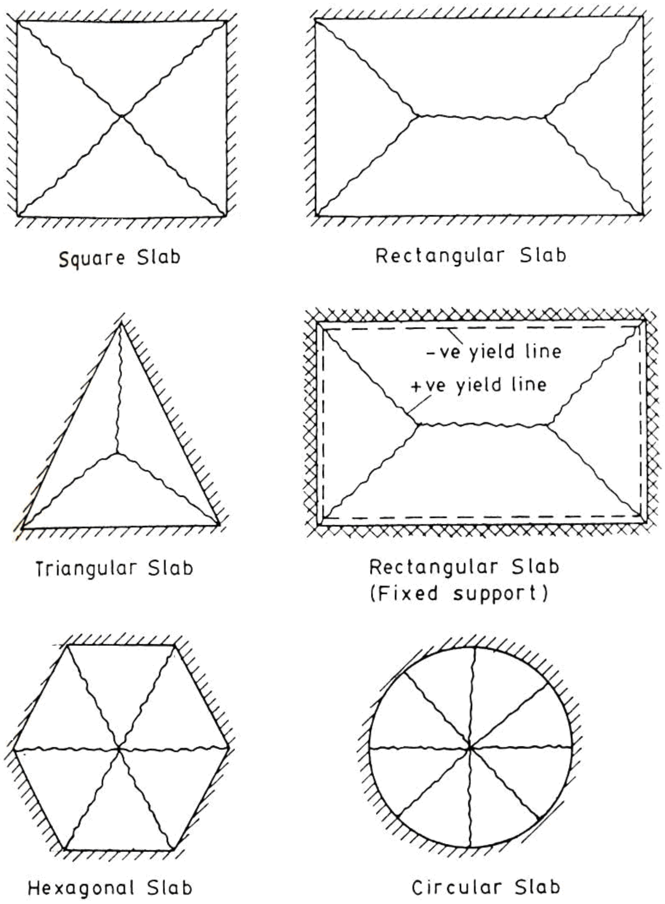
ii) Yield lines end at supporting edges of slab

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



Notations For Yield Lines and Supports



Typical Yield Line Patterns in Reinforced Conc.ete Slabs

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.

Rules of yield lines:

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.

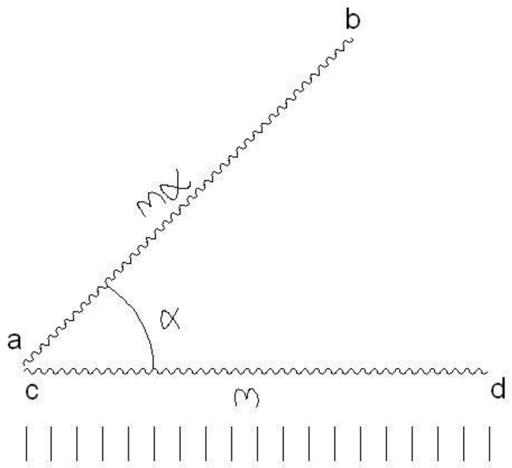
(B) Upper bound theorem: The upper bound of the true collapse load is that external load for which the internal work done by the slab for a small increment of displacement, assuming that moment at every plastic hinge is equal to the yield moment and satisfying the boundary conditions, is equal to the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

Yield moments:

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

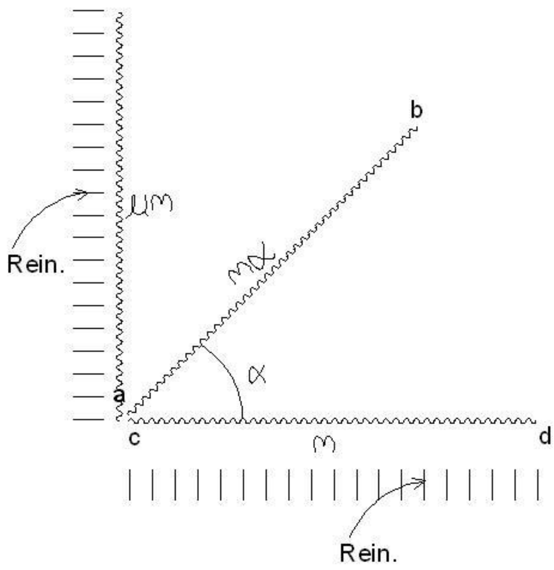


$m_\alpha.a.b = m.\cos\alpha.c.d.$ → $m_\alpha = \left(\frac{c.d}{a.b}\right) m.\cos\alpha$ → $m_\alpha = m.\cos^2\alpha$

where, m → yield moment per m length along the yield line

→ One mesh reinforcement

ii)



$m_\alpha = m.\cos^2\alpha + \mu m.\cos^2.(90 - \alpha)$

$= m.\cos^2\alpha + \mu m.\cos^2 (90 - \alpha)$

$= m.\cos^2\alpha + \mu m.\sin^2\alpha$

For isotropically reinforced slab,

($\mu = 1$) & $m_\alpha = m$

When reinforcement is same horizontally and vertically, $\mu = 1$ & $m_\alpha = m$

Methods of yield line analysis:

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

Virtual work method – Applied load causing virtual displacement is equal to internal work done or energy dissipated in rotation along the yield lines.

I) Isotropically reinforced – square slab – simply supported – udl

External work done = w.δ

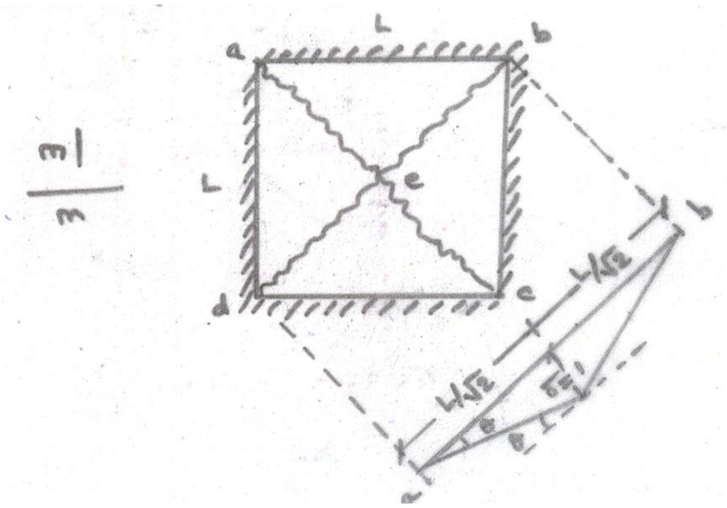
Where, w → load, δ → virtual displacement

Internal work done = M.θ = Σm.L.θ

m → Ultimate moment / unit length

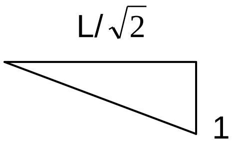
L → Length of yield line

M → 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,



Tan θ = 1 / (L/$\sqrt{2}$) = $\sqrt{2}$/L

$\Sigma(M.\theta)_{ac} = \Sigma(m.L.\theta)_{ac} = m.\sqrt{2}\,L.\frac{2\sqrt{2}}{L} = 4m$ [$\frac{2\sqrt{2}}{L}$ → since defl. Is on two sides]

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

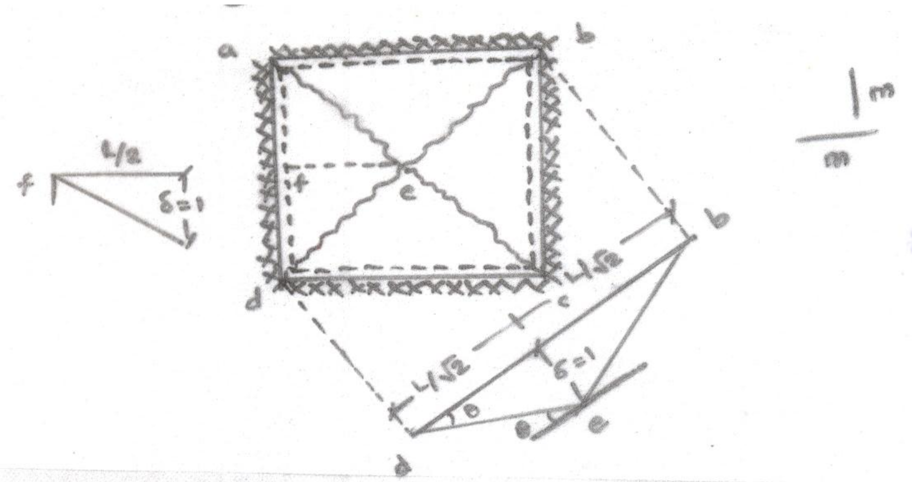
Total internal work done = ΣM.θ = 8m

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

ΣW.δ = [(1/2 x L x L/2 x w) x 1/3] x 4 = $\frac{wL^2}{3}$, where, w → udl on slab

ΣM.θ = ΣW.δ → 8m = $\frac{wl^2}{3}$ → m = $\frac{wl^2}{24}$

II) Isotropically reinforced – square slab – fixed on all edges – udl



External work done = w.δ

Where, w → load, δ → virtual displacement

Internal work done = ΣM.θ = Σm.L.θ = m.$\sqrt{2}$ L.$\frac{2\sqrt{2}}{L}$ = 4m

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

Total internal work done = ΣM.θ = 8m

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

Θ = 1 / L/2 = 2/L, ΣM.θ = 4 [w x L x 2/L] = 8m

Total internal work done = 16m

External work done,

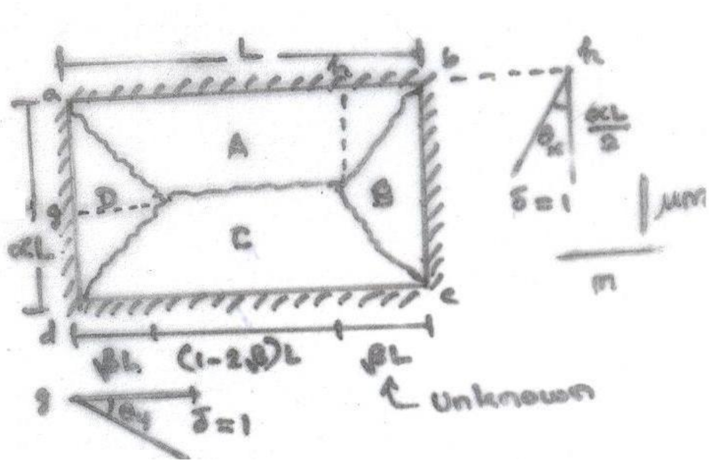
ΣW.δ = [(1/2 x L x L/2 x w) x 1/3] x 4 = wL^2 / 3

ΣM.θ = ΣW.δ

16m = wL^2 / 3 → m = wL^2 / 48

M → moment per metre length along the yield line

III) Orthotropically reinforced (Diff. rein. bothways) – rectangular slab – simply supported – udl



For element A,

$\Theta_x = \dfrac{2}{\alpha L}$ & $\Theta_y = 0$

$M_x = mL$

$(M_x.\ \Theta_x + M_y.\ \Theta_y]_A = \dfrac{2m}{\alpha}$

For element D,

Internal work done,

$\Theta_x = 0,\ \Theta_y = 1/(\beta L),\ M_y = \alpha.L.\mu m$

$\Sigma M.\theta = 2 \times \left[\dfrac{2m}{\alpha} + \dfrac{\alpha\mu m}{\beta}\right]$

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

$m = \dfrac{w.\alpha^2 L^2}{24}[\sqrt{(3+\mu\alpha^2} - \alpha\sqrt{\mu}]^2$

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

$m = \dfrac{w_u {L_x}^2}{24}\left(\dfrac{\tan^2\phi}{\mu}\right)$, where, $\tan\phi = \left(1.5\mu + \dfrac{\mu^2}{4\alpha^2}\right) - \left(\dfrac{\mu}{2\alpha}\right)$

μ → Coefficient of orthotrophy [Ratio between reinforcement provided along shorter direction and longer direction]

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

$m = \dfrac{w_u {L_y}^2}{48}\left(\dfrac{\tan^2\phi}{\mu}\right)$

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 kN/m^2
Live load		= 4 kN/m^2
Floor finish		= 1 kN/m^2
	Total	= 10 kN/m^2

Factored load (w_u) = 1.5 x 10 = 15 kN/m^2

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

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

Mu < Mulim

K = $\frac{M_u}{bd^2} = 0.241$

$A_{st} = 122mm^2$

Provide 8mm @ 300mm c/c

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

$l_y / l_x = 1$

Four edges are discontinuous

$\alpha_x = \alpha_y = 0.056$

$M_x = \alpha_x.w.l_x^2 = 0.056 \times 15 \times 5^2 = 21$ kNm

$M_y = \alpha_y.w.l_x^2 = 0.056 \times 15 \times 5^2 = 21$ 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.

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

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 kN/m^2
Live load		= 4 kN/m^2
Floor finish		= 1 kN/m^2
	Total	= 10 kN/m^2

Factored load (w_u) = 1.5 x 10 = 15 kN/m^2

By yield line theory, $m = \frac{wL^2}{24} = 15.625 kNm$

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

Mu < Mulim

A_{st} = 247.59mm^2

Provide 8mm @ 200mm c/c

3. Design a rectangular slab of size 4m x 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,

m = $\frac{w.\alpha^2 L^2}{24}[\sqrt{(3+\mu\alpha^2} - \alpha\sqrt{\mu}]^2$

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/m^2
Live load		= 4 kN/m^2
Floor finish		= 0.75 kN/m^2
	Total	= 9 kN/m^2

Factored load (w_u) = 1.5 x 9 = 13.5 kN/m^2

m = $\frac{w.\alpha^2 L^2}{24}[\sqrt{(3+\mu\alpha^2} - \alpha\sqrt{\mu}]^2$ = 8.8209 x 1.602 = 14.13 kNm

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

Mu < Mulim. The section is under-reinforced.

K = Mu/b.d^2 ➔ A_{st} = 277mm^2

Minimum A_{st} = 0.12% of c/s = 204mm^2

A_{st} along longer span = μ x A_{st} = 0.7 x 237 = 194mm^2 < 204 mm^2

Provide 8mm @ 240mm c/c

<u>Check for shear</u> (factored):

$V_u = \frac{w_u . l_x}{2}$ = 13.5 x 4 / 2 = 27 kN

K.$\zeta_v = \frac{V_u}{bd} = \frac{27x10^3}{1000x150}$ = 0.18 N/mm^2

ζ_c for [100A_{st}/bd = (100 x 277)/(1000 x 150) = 0.185N/mm^2]

0.15 → 0.28

0.25 → 0.36

0.185 → 0.308

As per Cl.42.4 of IS456-2000,

K → 1.3 for d = 150mm

K → 1 for d = 300mm

K → ((1.126 + 0.133) = 1.26) for d = 170mm

K. ζ_c = 1.26 x 0.308 = 0.388 N/mm^2 > ζ_v (0.18 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,

$m = \frac{w.L_y{}^2}{48}[\frac{\tan^2 \phi}{\mu}]$,

where, $\tan \varphi = \left(1.5\mu + \frac{\mu^2}{4\alpha^2}\right) - \frac{\mu}{2\alpha}$

μ = 0.7, α = 0.66, tanφ = 0.8, φ = 38.65^o

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/m^2
Live load		= 4 kN/m^2
Floor finish		= 0.75 kN/m^2

Total = 9 kN/m^2

Factored load (w_u) = 1.5 x 9 = 13.5 kN/m^2

m = $\frac{w.L_y^{\ 2}}{48}[\frac{\tan^2\phi}{\mu}]$ = $\frac{13.5x6^2}{48}[\frac{0.801^2}{0.7}]$ = 9.28 kNm

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

$M_u < M_{ulim}$. The section is under-reinforced.

K = Mu/b.d^2 ➔ A_{st} = 175mm^2

Minimum A_{st} = 0.12% of c/s = 204mm^2

A_{st} along longer span = μ x A_{st} = 0.7 x 175 = 122.5mm^2 < 204 mm^2

Provide 8mm @ 240mm c/c bothways [Min. A_{st}]

Check for shear (factored): {No change}

V_u = $\frac{w_u.l_x}{2}$ = 13.5 x 4 / 2 = 27 kN

K.ζ_v = $\frac{V_u}{bd}$ = $\frac{27x10^3}{1000x150}$ = 0.18 N/mm^2

ζ_c for [100A_{st}/bd = (100 x 277)/(1000 x 150) = 0.185N/mm^2]

0.15 → 0.28

0.25 → 0.36

0.185 → 0.308

As per Cl.42.4 of IS456-2000,

K → 1.3 for d = 150mm

K → 1 for d = 300mm

K → ((1.126 + 0.133) = 1.26) for d = 170mm

K. ζ_c = 1.26 x 0.308 = 0.388 N/mm^2 > ζ_v (0.18 N/mm^2)

Section is safe in shear.

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

m = $\frac{w_u L_x^{\ 2}}{24}\left(\frac{\tan^2\phi}{\mu}\right)$

Provide 8mm @ 240mm bothways.

Triangular slab:

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

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

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

M = $\frac{w_u \alpha L^2}{6} Sin^2 \frac{\beta}{2}$

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

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

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.

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 x 25	= 5 kN/m^2
Live load		= 3 kN/m^2
Floor finish		= 1 kN/m^2
	Total	= 9 kN/m^2

Factored load (w_u) = 1.5 x 9 = 13.5 kN/m^2

m = $\frac{w_u L^2}{72}$ = 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.d^2 ➔ A_{st} = 72.754mm^2

Minimum A_{st} = 0.12% of c/s = 240mm^2

A_{st} < min Ast [240 mm^2]

Provide 8mm @ 200mm c/c

<u>Check for shear</u> (factored):

$V_u = \frac{w_u . l_x}{2}$ = 13.5 x 5 / 2 = 33.75 kN

K.$\zeta_v = \frac{V_u}{bd} = \frac{27x10^3}{1000x150}$ = 0.1875 N/mm^2

ζ_c for [100A_{st}/bd = (100 x 277)/(1000 x 150) = 0.185N/mm^2]

0.13 → 0.28

As per Cl.42.4 of IS456-2000,

K → 1.3 for d = 150mm

K → 1 for d = 300mm

K → ((1.04 + 0.2) = 1.24) for d = 180mm

K. ζ_c = 1.24 x 0.28 = 0.3472 N/mm^2 > ζ_v (0.18 N/mm^2)

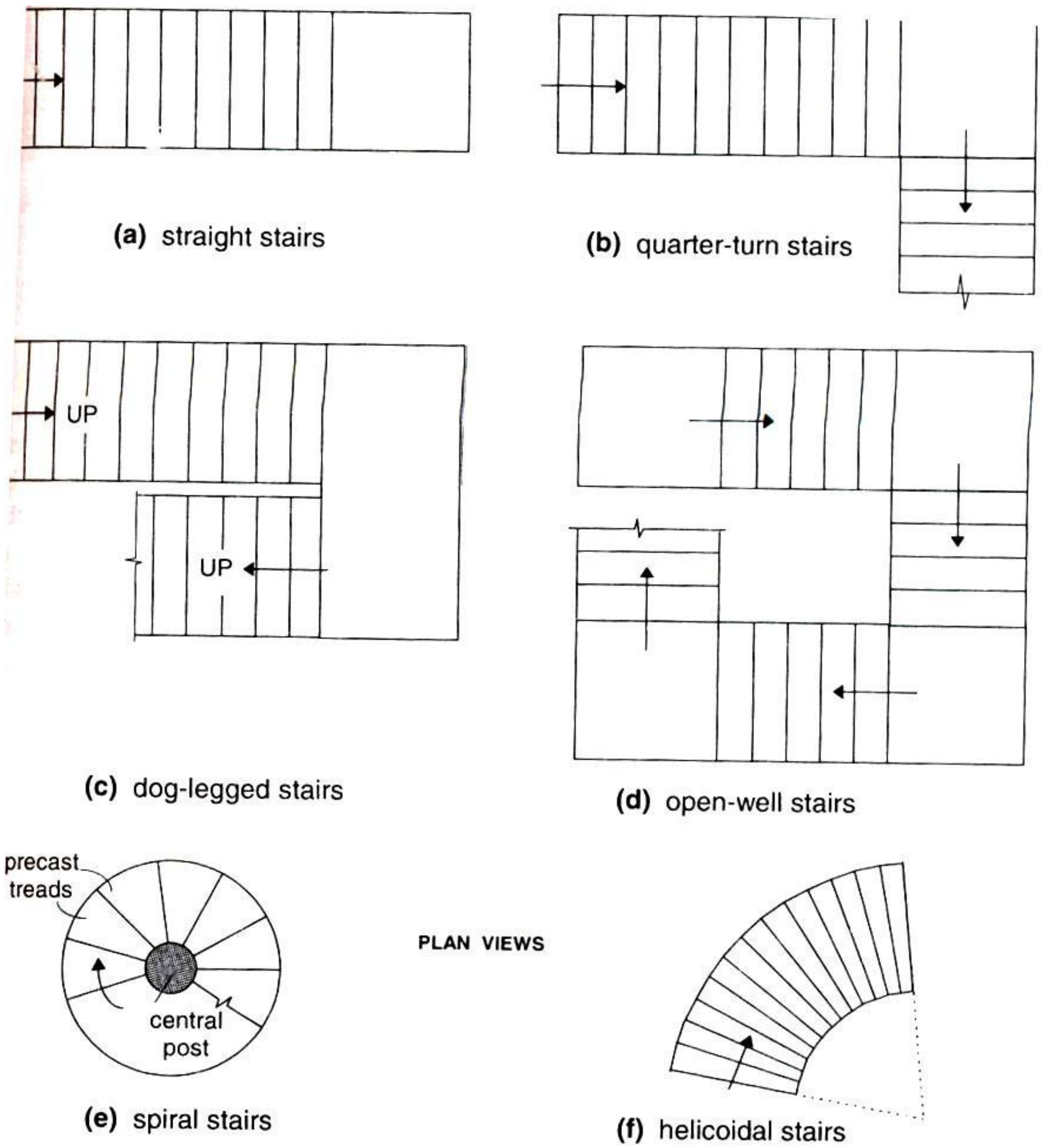
Section is safe in shear.

UNIT IV

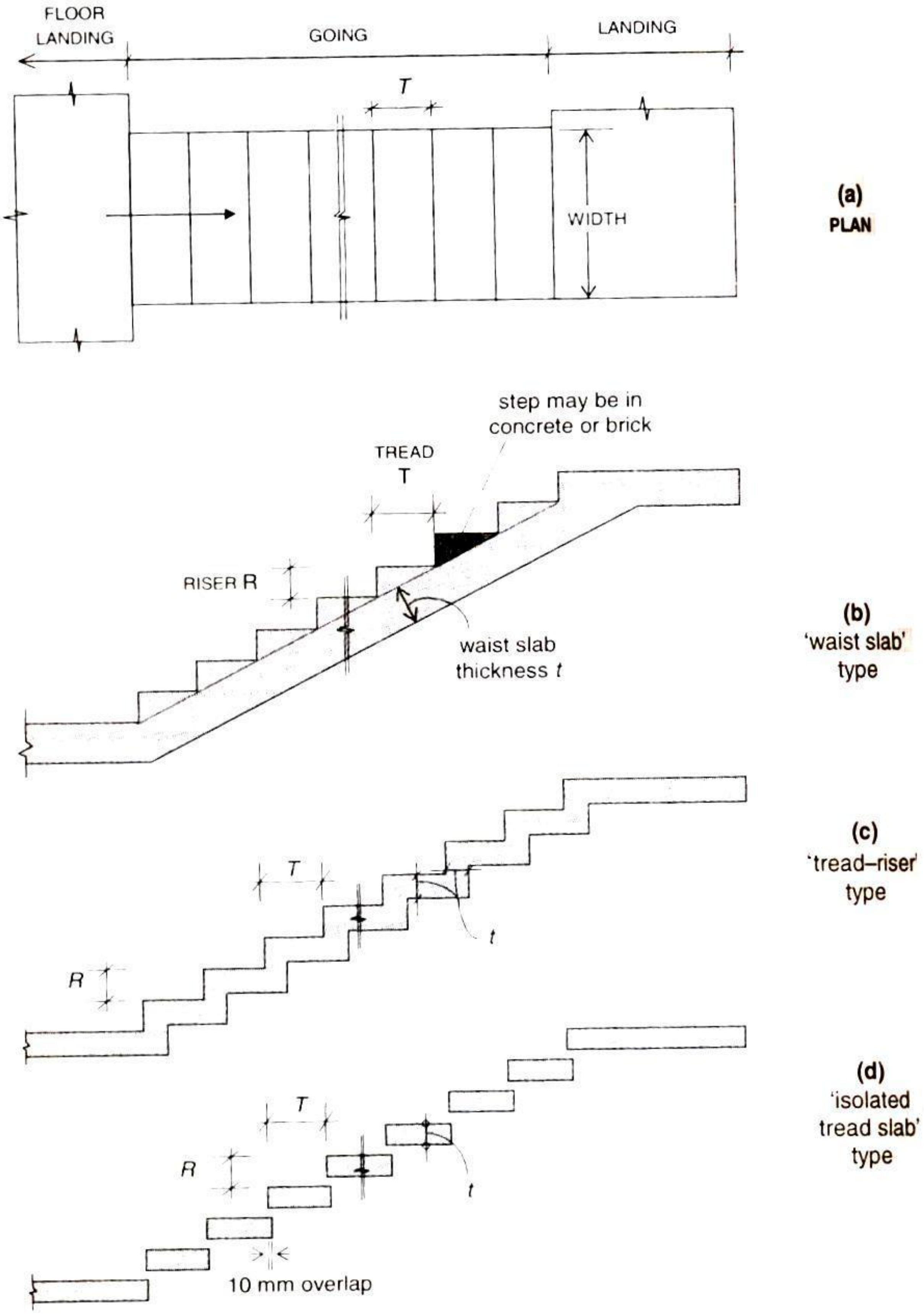
MISCELLANEOUS STRUCTURES [STAIRCASE, FLAT SLAB, RC WALL]

DESIGN OF STAIRCASE

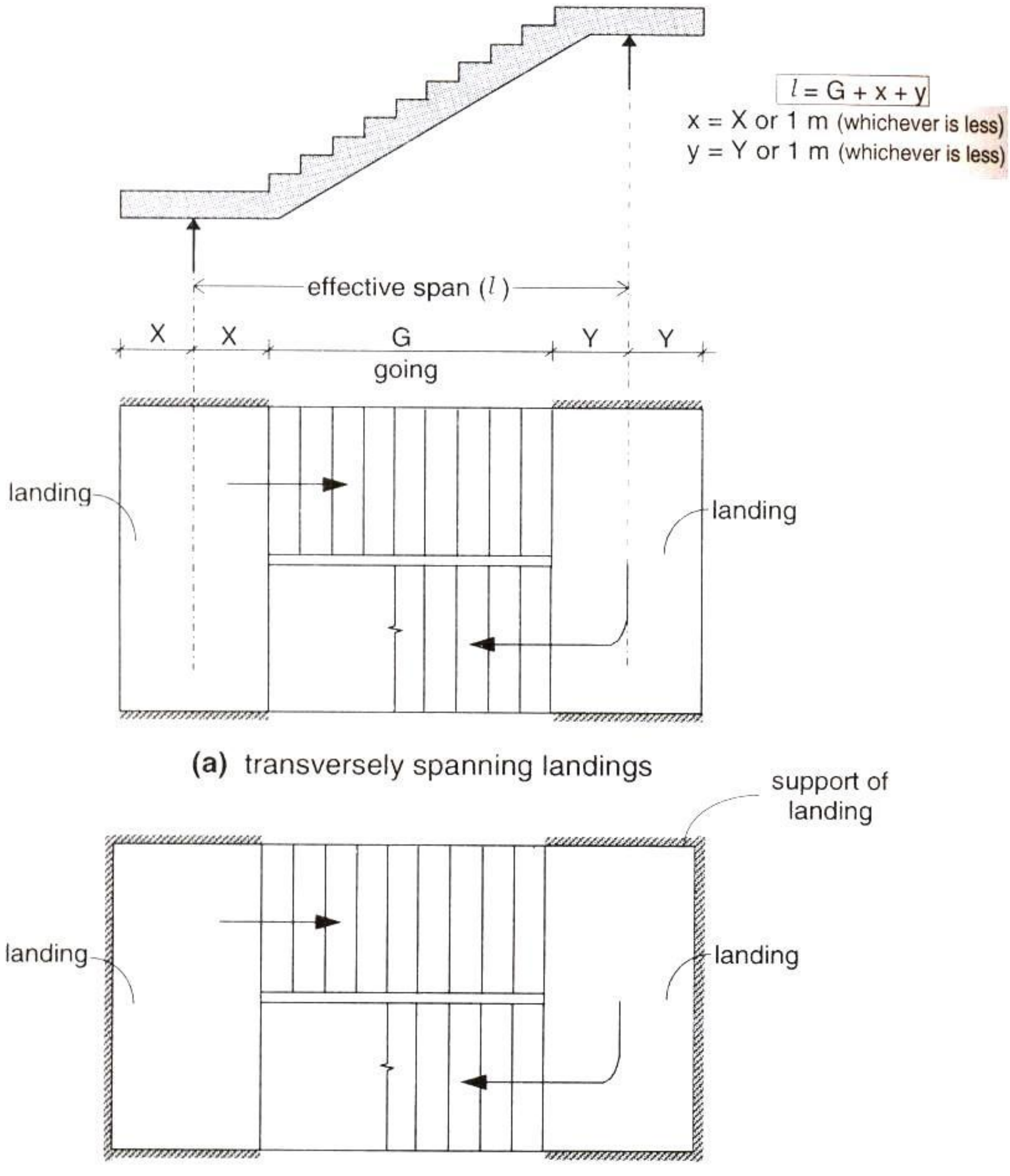
GEOMETRICAL TYPES OF STAIRCASE:



Common geometrical configurations of stairs

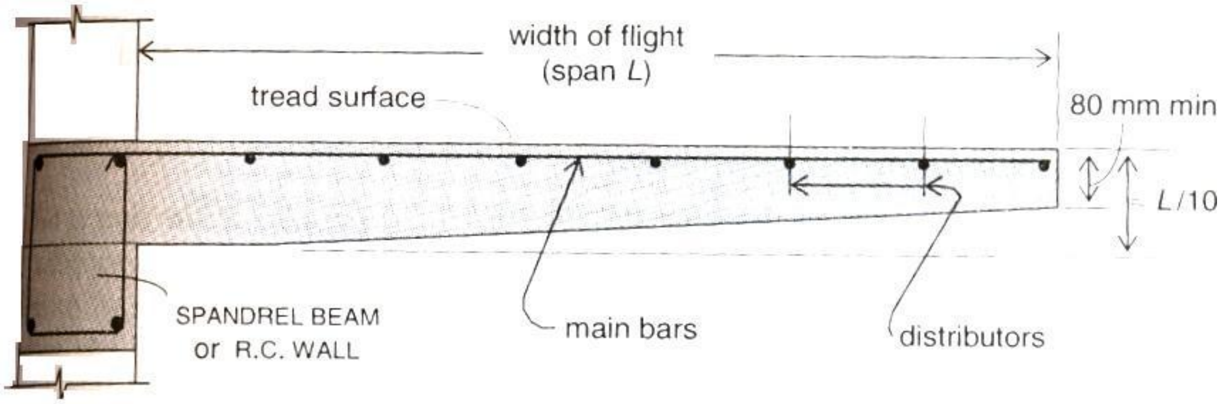


A typical flight in a staircase

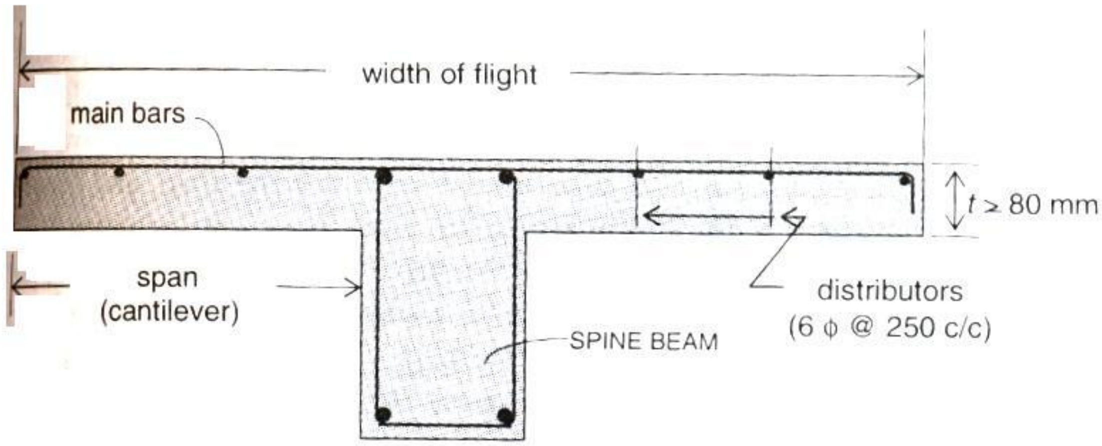


Special support conditions for longitudinally spanning stair slabs

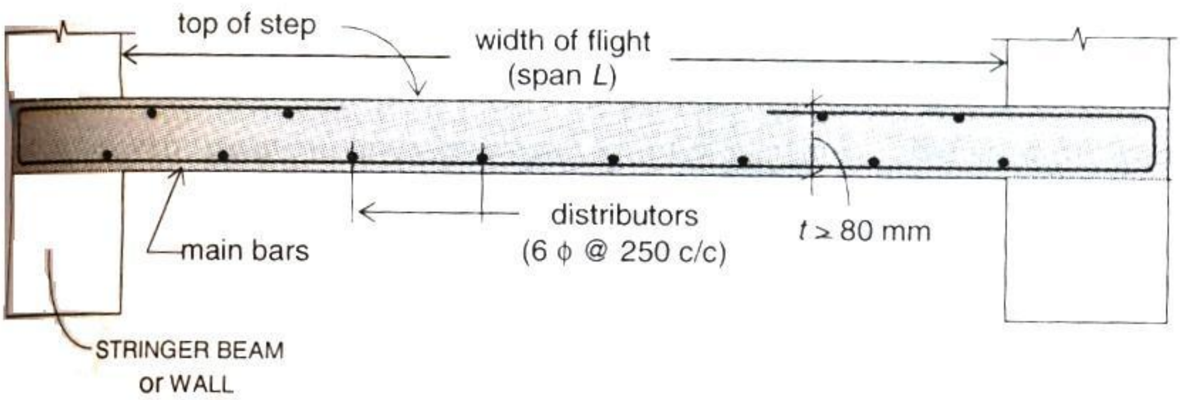
- Based on loading and support conditions:
  - Spanning along transverse direction
    - Cantilever staircase
    - Slab supported between stringer beams



(a) slab cantilevered from a spandrel beam or wall



(b) slab doubly cantilevered from a central spine beam

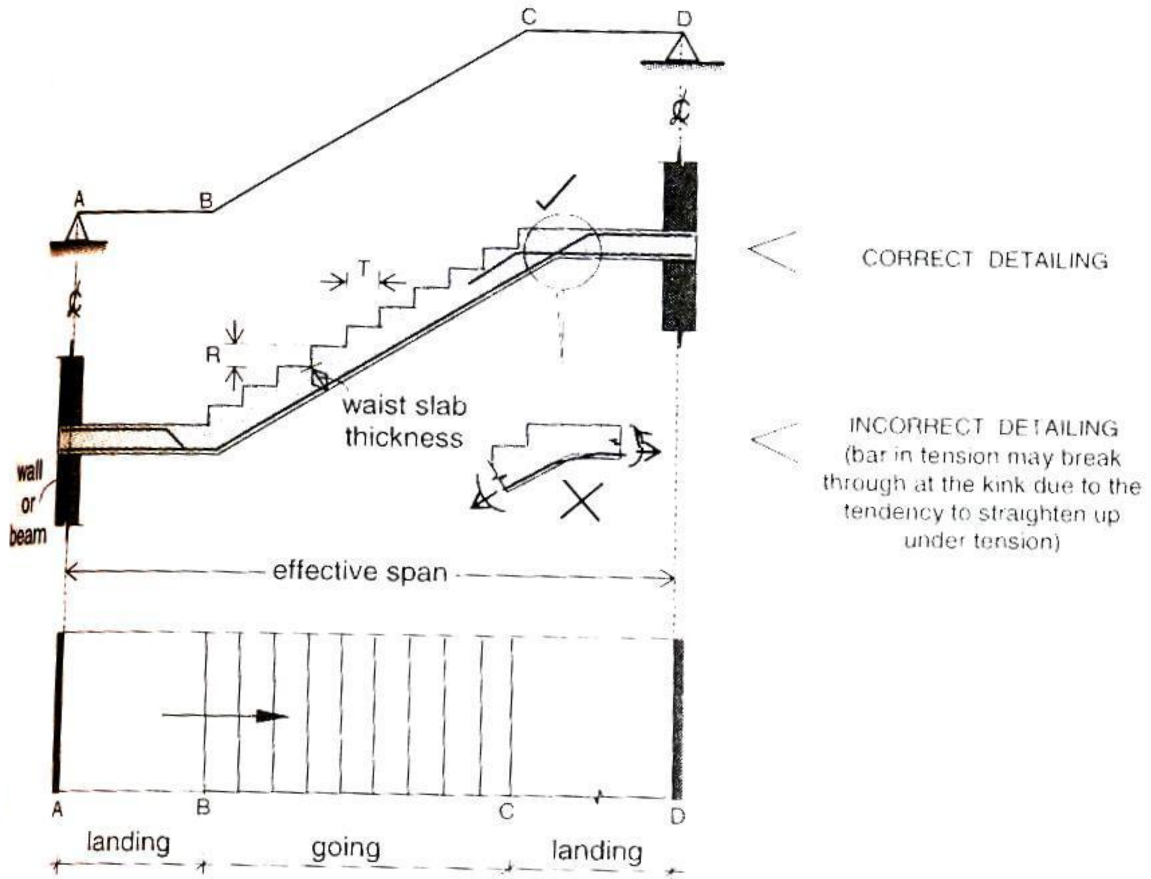


(c) slab supported between two stringer beam or walls

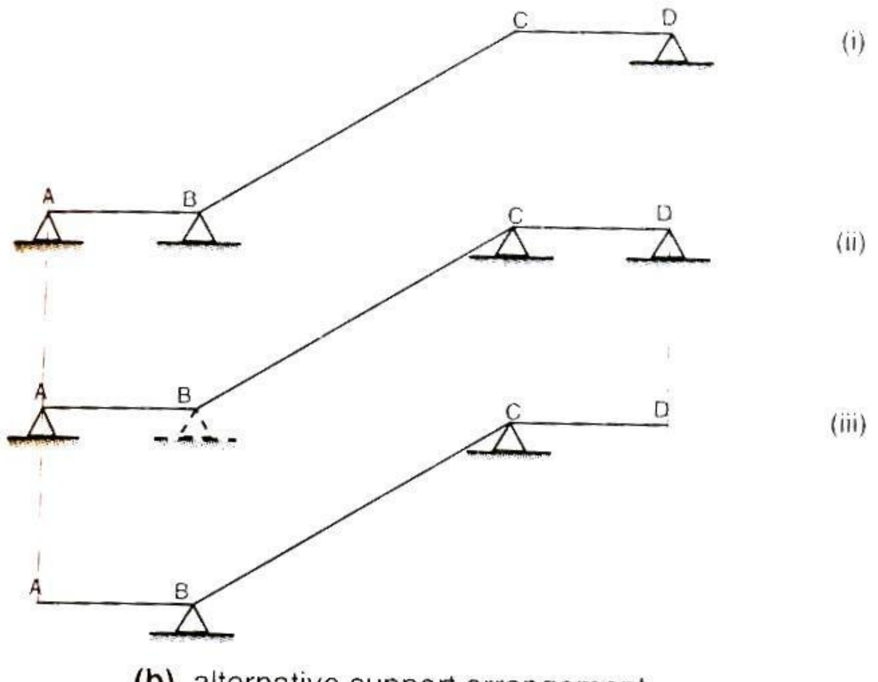
Typical examples of stair slabs spanning transversely

- Spanning along longitudinal direction

[Most commonly adopted]



(a) simply supported arrangement



(b) alternative support arrangement

Typical examples of stair slabs spanning longitudinally

Loading on staircase:

Dead load:

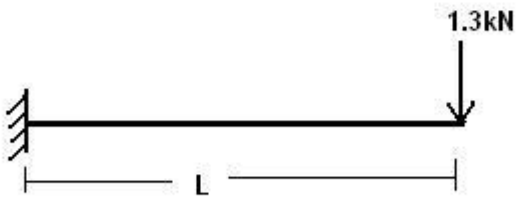
- Self weight of slab
- Self weight of step
- Tread finish [0.6 – 1 kN/m^2]

Live load:

For overcrowding ➔ 5 kN/m^2

No overcrowding ➔ 3 kN/m^2

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 x 0.15 x 0.3 = 1.125 kN/m

Floor finish (0.6 kN/m2) = 0.6 x 0.3 = 0.18 kN/m

Total = 1.305 kN/m

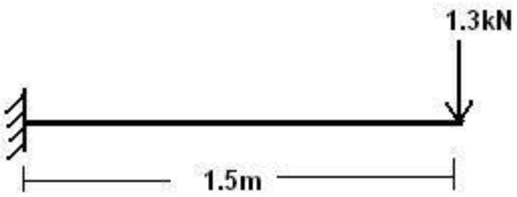
Dead load moment, $M_D = 1.305 \times 1.5^2 / 2 = 1.468$ kNm

Live load: Maximum of,

i) Overcrowding → 5 kN/m^2

L.L. = 5 x 0.3 = 1.5 kN/m

$M_L = wl^2/2 = 1.5 \times 1.5^2 / 2 = 1.69$ kNm

ii) 

$M_L = 1.3 \times 1.5 = 1.95$ kNm

The maximum of the above two values is M_L = 1.95 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 → 15mm to 20 mm]

$M_u = 0.87.f_y.A_{st}.\left(d - 0.42\left(\frac{0.87.f_y A_{st}}{0.36.f_{ck}.b}\right)\right)$ [OR] K = $\frac{M_u}{bd^2}$ → Take p_t from SP16

$5.13 \times 10^6 = 65.25 \times 10^3.A_{st} - 9.19.A_{st}^2$

→ $A_{st} = 202$ mm^2

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

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

Distribution steel:

MS → 0.15% of c/s → 0.15/100 x 300 x 150 = 67.5mm^2

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 x 1.5 = 1.958 kN

Shear force due to live load,

i) 1.5 x 1.5 = 2.25 kN

ii) 1.3 kN

$V_u = 2.25 + 1.958 = 4.208$ 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, → K = 1.3

When depth is > 300mm, → K = 1

Development length = $\frac{\phi.\sigma_s}{4\tau_{bd}}$ = 453.125 mm

Provide a development length of 450mm

Providing a 90^0 bent, length required = 450 – 80 = 370mm

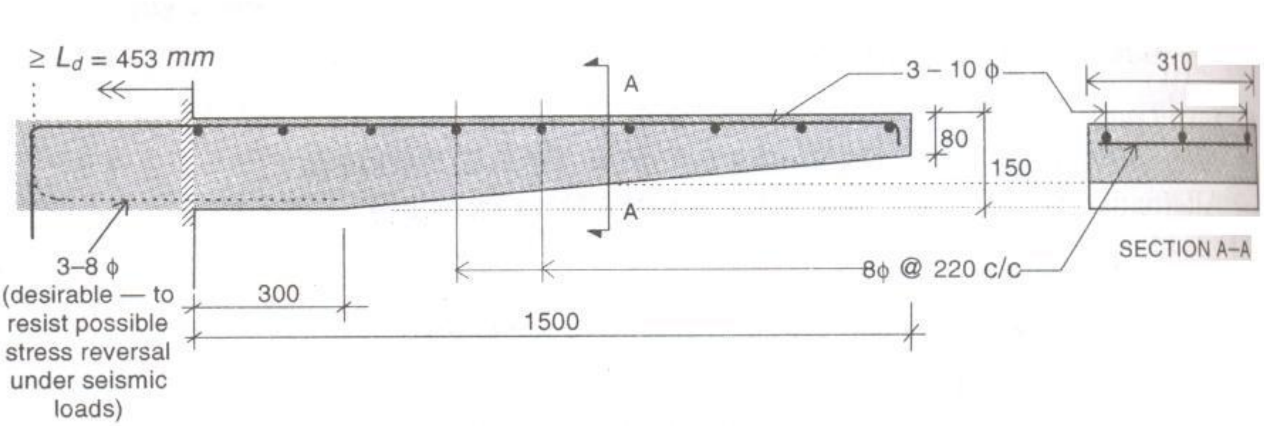
$\zeta_v = \frac{V_u}{b.d}$ = 0.168 N/mm^2

For $p_t = \frac{100A_s}{b.d}$ = 0.54%,

ζ_c = 1.3 x 0.4928 = 0.6406N/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/m^2. 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 ➔ 1500/d = 30 ➔ 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	= 2.135 kN/m

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 x 1/0.309 = 6.02 kN/m

Factored load = 9.03 kN/m

M = wl^2/8 = 9.03 x 1.5^2 / 8 = 2.54 kNm

$$2.54x10^6 = 0.87.250.A_{st}.\left(60 - 0.42\left(\frac{0.87x250xA_{st}}{0.36x20x1000}\right)\right)$$

➔ A_{st} = 205.72 mm^2/m

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

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

Provide 8mm @ 180mm c/c

Distribution steel:

MS → 0.15% of c/s

= 0.15/100 x 1000 x 60 = 90mm^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/m^2 and a live load of 5 kN/m^2. 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

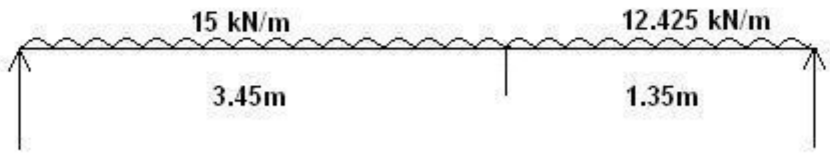
Self weight of waist slab	= 25 x (0.265 x (0.3354/0.3)	= 7.4 kN/m^2
Self weight of step	= 25 x ½ x 0.15	= 1.875 kN/m^2
Floor finish		= 0.8 kN/m^2
Live load		= 5 kN/m^2
	Total	= 15 kN/m^2

Loading on going slab:

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

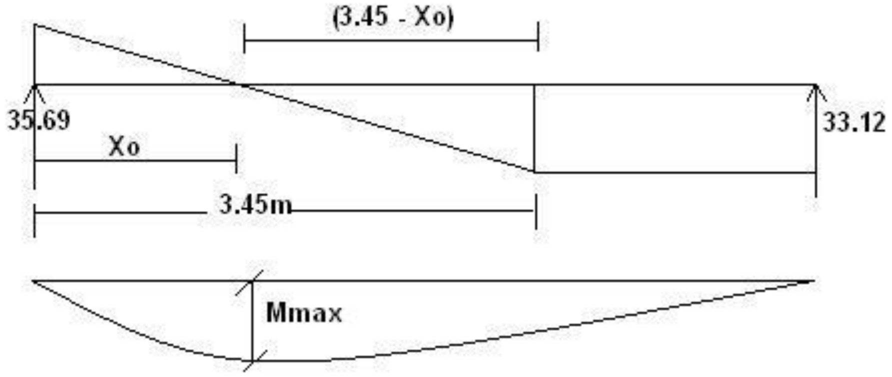
Considering 1m strip,

The staircase slab is idealized as given below:



R_A x 4.8 – (15 x 3.45 x 3.075) – (12.425 x 0.675 x 1.35) = 0

R_A = 35.69 kN, R_B = 33.11 kN



Xo = R_A / udl = 35.69/15 = 2.37 m

Moment at 2.37m is,

35.69 x 2.37 – (15 x 2.37 x 2.37/2) = 42.23 kNm

Factored moment = 63.35 kNm

$M_u = 0.87.f_y.A_{st}.\left(d - 0.42\left(\frac{0.87.f_y A_{st}}{0.36.f_{ck}.b}\right)\right)$ [OR] K = $\frac{M_u}{bd^2}$ ➔ Take p_t from SP16

63.35 x 10^6 = 86.625 x 10^3.A_{st} – 7.604.A_{st}^2

➔ A_{st} = 785.19 mm^2

Provide 10mm @ 100mm c/c

Distribution steel:

0.12% of c/s → 0.12/100 x 1000 x 265 = 318mm^2

Provide 8mm @ 150mm c/c

Check for shear:

ζv = V_u/b.d

Maximum shear force = [35.69 – (15.08 x 0.24)] x 1.5 = 48.1 kN

$\zeta_v = \frac{V_u}{b.d} = \frac{48.1x10^3}{1000x240}$ = 0.2 N/mm^2

p_t = 100.A_{st}/b.d = 0.327%

For 0.25% → 0.36

For 0.5% → 0.48 For 0.33% → 0.0.398

ζ_c = 0.398 N/mm^2

The ζ_c value is modified based on Cl.40.2.1.1 for D = 265mm

For D = 150mm, 1.3

For D = 300mm, 1 For D = 265mm, 1.07

$\zeta_{c\ modified}$ = 1.07 x 0.398 = 0.426 N/mm^2

$\zeta_v < \zeta_c$

Hence, safe.

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 kN/m^2, FF = 0.6 N/mm^2

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 kN/m^2
Tread finish		= 0.6 kN/m^2
Live load		= 5 kN/m^2
	Total	= 15.82 kN/m^2

Loading on landing slab:

Self weight of slab = 25 x 0.283 = 7.075 kN/m^2

l/d = 20 ➔ 5.16/d = 20

➔ d = 258mm

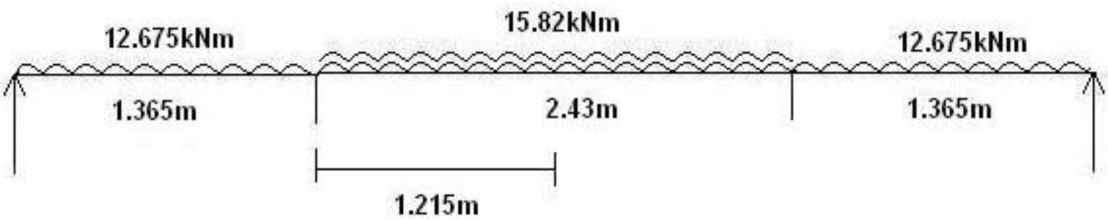
D = 258 + 20 + 10/2 = 283mm

Length of inclination of one step,

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

Self weight of slab = 25 x 0.283 = 7.075 kN/m^2

FF		= 0.6 kN/m^2
LL		= 5 kN/m^2
	Total	= 12.675 kN/m^2



R_A x 5.16 – (12.675 x 1.365 x 4.4775) – (15.82 x 2.43 x 2.58) – (12.675 x 1.365 x 0.6875) = 0

R_A = 36.54 kN

R_B = 36.51 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

$$M_u = 0.87.f_y.A_{st}.\left(d - 0.42\left(\frac{0.87.f_y A_{st}}{0.36.f_{ck}.b}\right)\right)$$

b = 1000mm, d = 258mm

$74.49 \times 10^6 = 93.15 \times 10^3 A_{st} - 7.4 A_{st}^2$

A_{st} = 868.99 mm^2

Provide 12mm φ @ 130mm c/c

Distribution steel:

0.15% of c/s = 0.15/100 x 1000 x 283 = 424.5 mm^2

8mm @ 110mm c/c

Check for shear:

$ζ_v = V_u/b.d$

Maximum shear force = [36.5 – (12.675 x 0.258)] x 1.5 = 49.84 kN

$ζ_v$ = 0.193 N/mm^2

$ζ_c$:

Pt = 100.Ast/b.d = 0.336%

For pt = 0.25% → 0.36

For pt = 0.5% → 0.48

For pt = 0.336% → 0.40

$ζ_c$ = 0.40 N/mm^2

Modification factor,

For D = 150mm, → 1.3

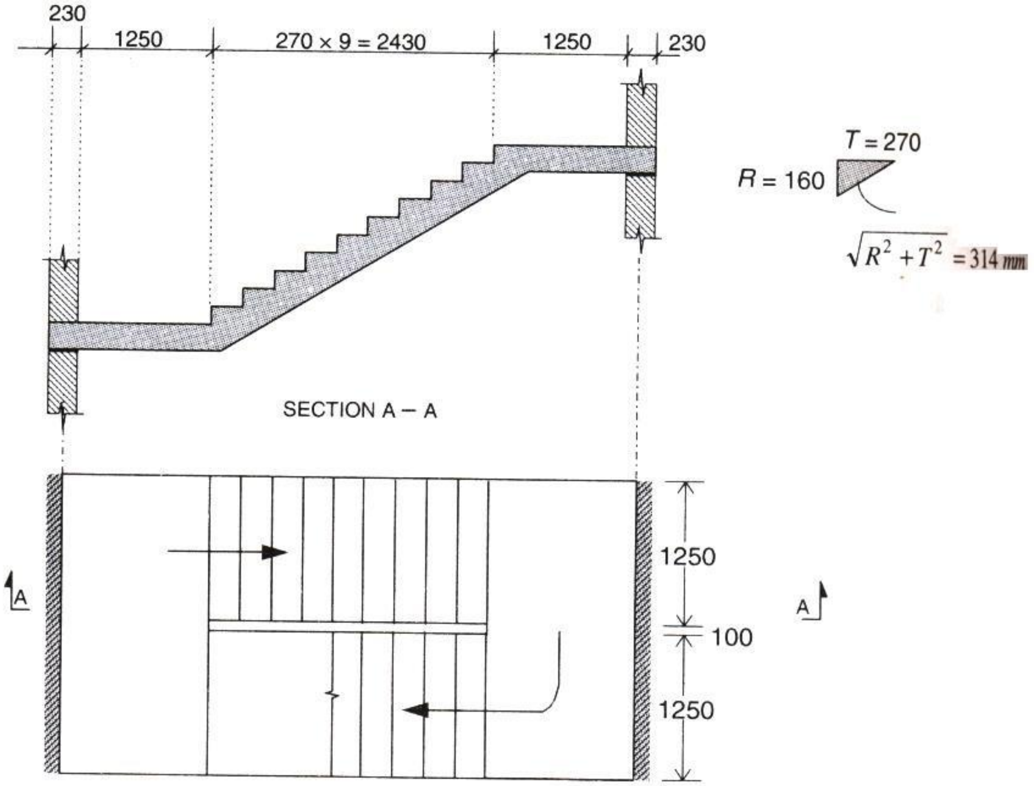
For D = 300mm, → 1

For D = 283mm, → 1.03

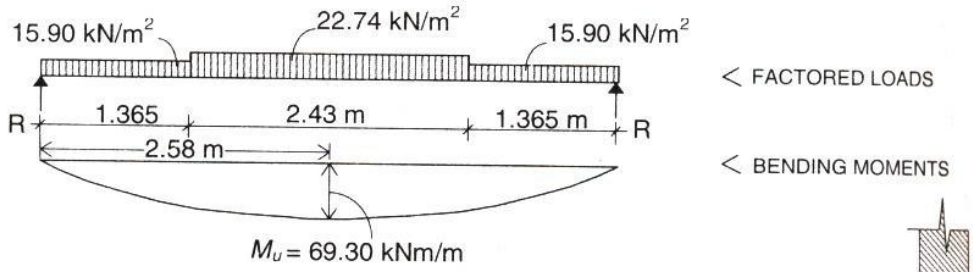
$ζ_{c\ modified}$ = 1.03 x 0.40 = 0.412 N/mm^2

$ζ_v < ζ_c$

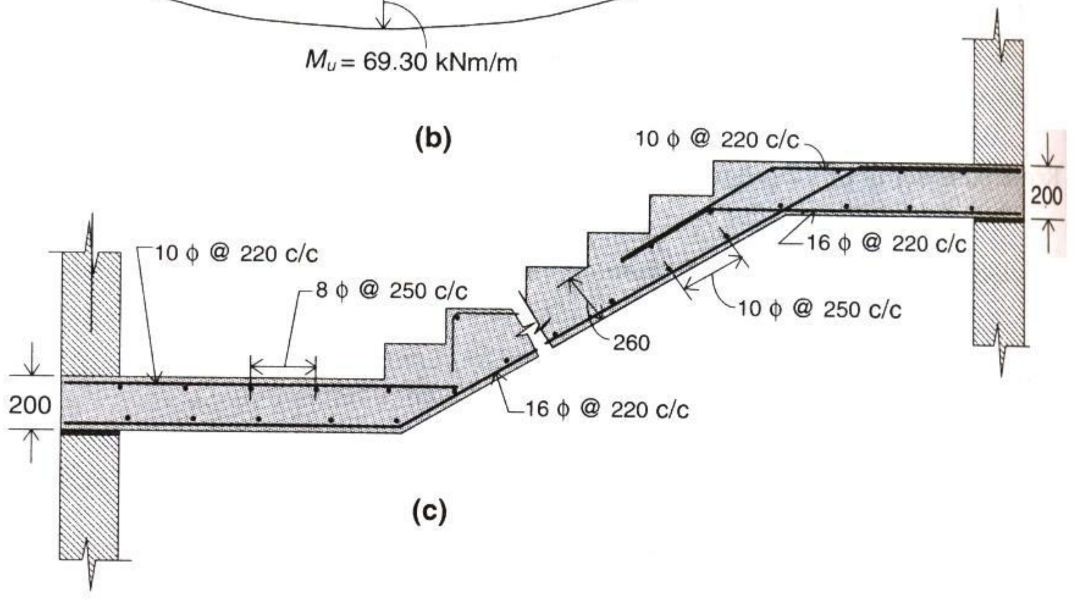
Safe in shear.


230
1250
270 × 9 = 2430
1250
230
T = 270
R = 160
√(R² + T²) = 314 mm
SECTION A – A
A
A
1250
100
1250

(a)


15.90 kN/m²
22.74 kN/m²
15.90 kN/m²
< FACTORED LOADS
R
1.365
2.43 m
1.365 m
R
2.58 m
< BENDING MOMENTS
Mu = 69.30 kNm/m

(b)


10 φ @ 220 c/c
200
16 φ @ 220 c/c
10 φ @ 220 c/c
8 φ @ 250 c/c
10 φ @ 250 c/c
260
200
16 φ @ 220 c/c

(c)

TREAD RISER STAIRCASE

Actual analysis → Theory of plates

l/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 → 5 kN/m^2 (overcrowded), 3 kN/m^2 (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 kN/m^2, FF = 0.6 N/mm^2

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 = 3.68m

Assume 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 x (0.27 + 0.16) x 0.175/0.27 x 1 = 6.97 kN/m2

Floor finish		= 0.6 kN/m2
Live load		= 5 kN/m2
	Total	= 12.57 kN/m2

Considering 1m strip, w = 12.57 kN/m

Loading on landing slab:

Self weight of slab = 25 x 0.175		= 4.375 kN/m^2
Floor finish		= 0.6 kN/m^2
Live load		= 5 kN/m^2
	Total	= 9.975 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 x 3.68 – (4.99 x 3.3675) – (12.57 x 1.84 x 2.43) – (4.99 x 0.3125 x 0.625) = 0

➔ R_A = 18.39 kN

➔ R_B = 18.39 kN

Moment at centre, (i.e. 1.84m),

M_{max} = 18.39 x 1.84 – (4.99 x 0.625 x 1.5275) – (12.57 x 1.215 x 0.675)

= 19.79 kNm

Factored moment = 29.69 kNm

For b = 1000mm, d = 150mm,

K = M_u/b.d^2

➔ A_{st} = 598.36 mm^2 / 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 mm^2

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

$\zeta_v = 0.1764$ N/mm^2

ζ_c :

$100A_{st}/b.d. = 0.3989\%$

For pt = 0.25 → 0.36

For pt = 0.5 → 0.48

For pt = 0.39 → (0.1584 + 0.2688) = 0.427

Modification factor (K):

For D = 150mm → 1.3

For D = 300mm → 1

For D = 175mm → 1.08 + 0.17 = 1.25

$\zeta_{c\ modified} = 0.534$ 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,

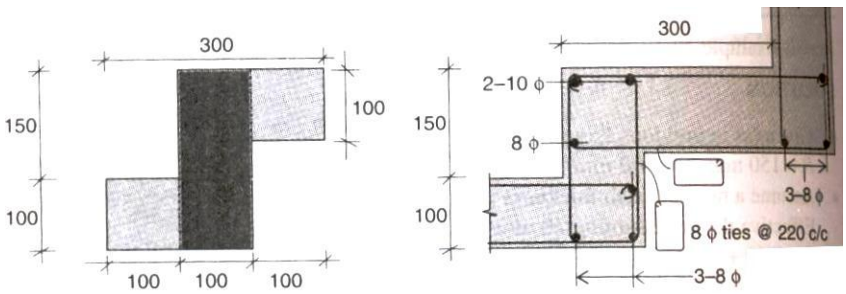
i) Directly on landing = 9.98 kN/m

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

w = 25.25 kN/m

l = 2.6m

$M_u = wl^2/8 \times 1.5 = 32$ kNm



(a) (b)

b = 1000mm, d = 150mm

$K = M_u/b.d^2$

$A_{st} = 650.19$ mm^2

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%
    - 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,

l → effective length of wall, h → effective height of wall, t → 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 (λ < 12), $P_u = 0.4.f_{ck}.A_c + 0.67.f_y.A_{st}$
3. 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 (λ < 45):

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

   $$e_a = \frac{L_e^{\,2}}{2000.t}, \quad M_a = P \times e_a$$

   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 $\varphi_L/4$.

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) → $l_{eff} = 0.65l_o – 0.75l_o$
2. Both ends hinged → $l_{eff} = l_o$
3. One end fixed and other end free → $l_{eff} = 2l_o$
4. One end fixed and other end hinged → $l_{eff} = l_o/\sqrt{2}$

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.

λ = h/t or l/t

= 3000/120 or 4000/120

= 30 or 40

Assume both ends fixed (restrained against rotation and displacement)

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

= 0.75 x 3 = 2.25m

λ = 2250/100 = 22.5 > 12

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

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,

$$e_a = \frac{L_e^2}{2000t} = \frac{2.25^2 x 1000^2}{2000 x 100} = 25.3125\text{mm}$$

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

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.

$\frac{P_u}{f_{ck}bt}$ = $\frac{600x10^3}{20x100x1000}$ = 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,

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

Area of steel = 1.4/100 x 100 x 1000 = 1400 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 x 1000 x 100 = 400 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 = $\frac{M_u}{P_u}$ = $\frac{25x10^6}{600x10^3}$ = 41.67mm

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

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

d = 5000/22 = 227.27

Assume a thickness of 225mm

L_{eff} = 0.75 x 5 = 3.75m

λ_{act} = 3750/225 = 16.67 > 12

The given wall is slender.

e_{min} = t/20 or 20mm

= 225/20 or 20mm

= 11.25mm or 20mm < 41.67mm

Additional eccentricity due to slenderness,

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

Total eccentricity = $e_{min} + e_a$ = 41.67 + 31.25 = 72.92mm

Moment $M_u = P_u$ x 72.92 = 600 x 1000 x 72.92 = 43.75 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.

$\frac{P_u}{f_{ck}bt} = \frac{600x10^3}{20x100x225}$ = 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 x 225 x 1000 = 900 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 x 1000 x 100 = 400 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.

P_u = 1000kN, M_u = 50kNm

e = 50 x 10^6/1000 x 10^3 = 50mm

l/d = 22 [Generally l/d taken from 20 – 25]

d = 5000/22 = 227.27mm

Adopt a thickness of 225mm.

l_{eff} = 0.75 x 5000 = 3750mm

λ_{act} = 3750/225 = 16.67 > 12

λ_{min} = 45 > 16.67

Wall is slender.

e_{min} = t/20 or 20mm

= 225/20 or 20mm

= 11.25mm or 20mm < 41.67mm

Additional eccentricity due to slenderness,

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

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.

$\frac{P_u}{f_{ck}bt} = \frac{1000x10^3}{20x100x225}$ = 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,

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

Area of steel = 0.6/100 x 225 x 1000 = 1350 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 x 225 x 1000 = 900 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 $l_2/4$, where l_2 is the span transverse to l_1. l_2 – longer span, moment is considered along the span l1

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 (l/d)modified by 0.9 for flat slabs with drops.

Take → l/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: (Cl.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 $M_o = Wl_o/2$, 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%

$$M_o = \frac{WL_o}{8}$$

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

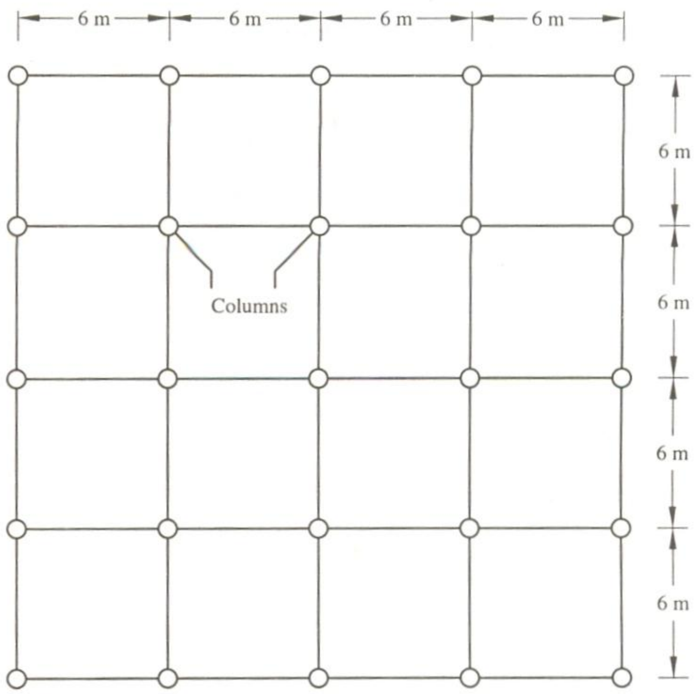
1) Design a flat slab system (interior panel) to suit the following data:

Size of the floor = 20 x 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 l/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

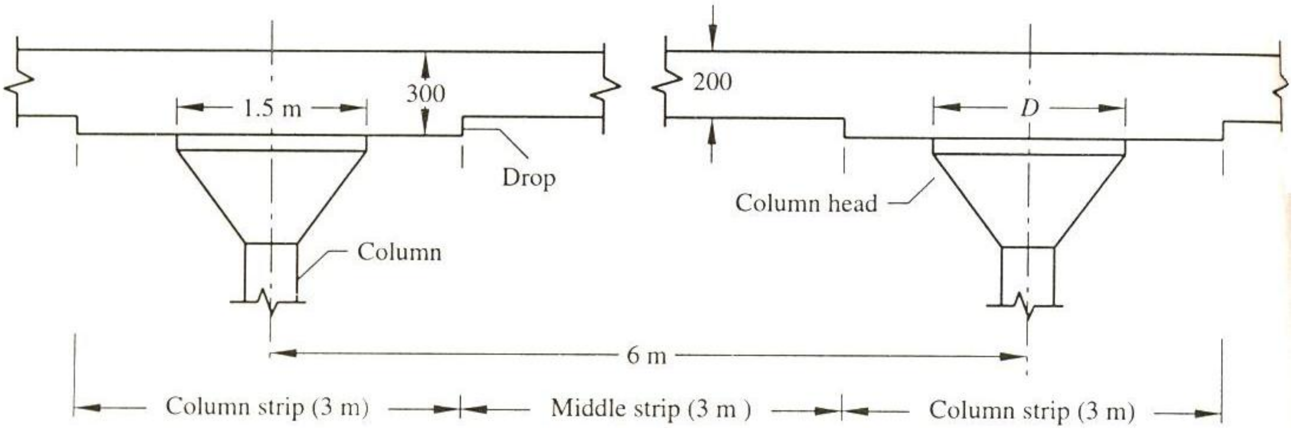
Provide length of drop as 2.5m. For the panel, 1.25m is the contribution of drop.

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 x 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 x 12 = 18 kN/m^2

$W = w_u \times L_2 \times L_n$ = 18 x 5 x 3.75 = 337.5 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% 0.49 x 158.2 = 77.52kNm	65 x 0.2 = 15% 0.15 x 158.2 = 23.73kNm
Positive moment (35%)	35 x 0.6 = 21% 0.21 x 158.2 = 33.22 kNm	35 x 0.4= 15% 0.15 x 158.2 = 23.73kNm

Check for depth adopted:

Column strip:

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

$77.5 \times 10^6 = 0.138 \times 20 \times 2.5 \times 1000 \times d^2$

➔ d = 105.98mm ~ 106mm < 275mm

Middle strip:

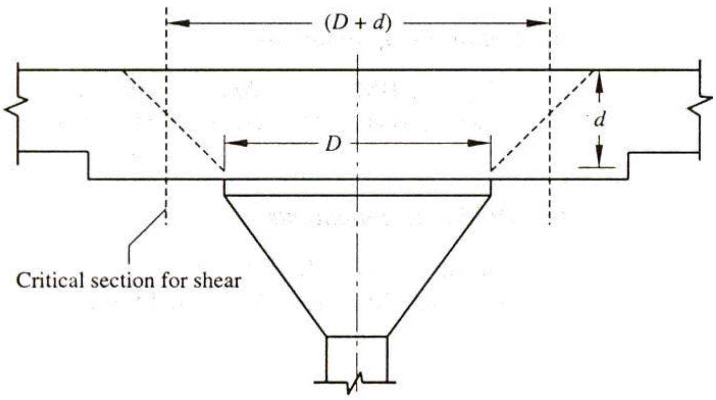
$M_u = 0.138.f_{ck}.b.d^2$ b = 2.5m

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

➔ 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

Shear force = Total Load – (Load on circular area)

= 18 x 5 x 5 – (π(D + d)^2/4) x w_n

= 417.12 kN

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

Nominal shear stress: (b = 1m)

$\varsigma_v = \frac{V_u}{b.D} = \frac{87.06x10^3}{1000x275}$ = 0.317 N/mm^2

Design shear stress: $\varsigma_c = K.\varsigma_c'$

Where, K = (0.5 + β) ≤ 1

= 1.5 ≤ 1

$\varsigma_c' = 0.25.\sqrt{f_{ck}}$ = 1.118 N/mm^2

ς_c = 1 x 1.118 = 1.118 N/mm^2

$\varsigma_v < \varsigma_c$

Safe in shear.

Reinforcement:

Column strip: (b=2.5m), (d = 275mm)

Negative moment = 77.5 x 10^6 Nmm

$M_u = 0.87.f_y.A_{st}.\left(d - 0.42\left(\frac{0.87.f_y A_{st}}{0.36.f_{ck}.b}\right)\right)$ [OR] K = $\frac{M_u}{bd^2}$ ➔ Take p_t from SP16

77.6 x 10^6 = 99.29 x 10^3.A_{st} – 3.04.A_{st}^2

➔ A_{st} = 800.16 mm^2

Required 10mm @ 240mm c/c

Min A_{st}: 0.12% of c/s = 0.12/100 x 1000 x 275 = 825 mm^2

Provide 10mm @ 230mm c/c

Positive moment = 33.2 kNm

A_{st} = 337.876 mm^2

Provide 8mm @ 370mm c/c

Min. steel: Provide 10mm @ 230mm c/c

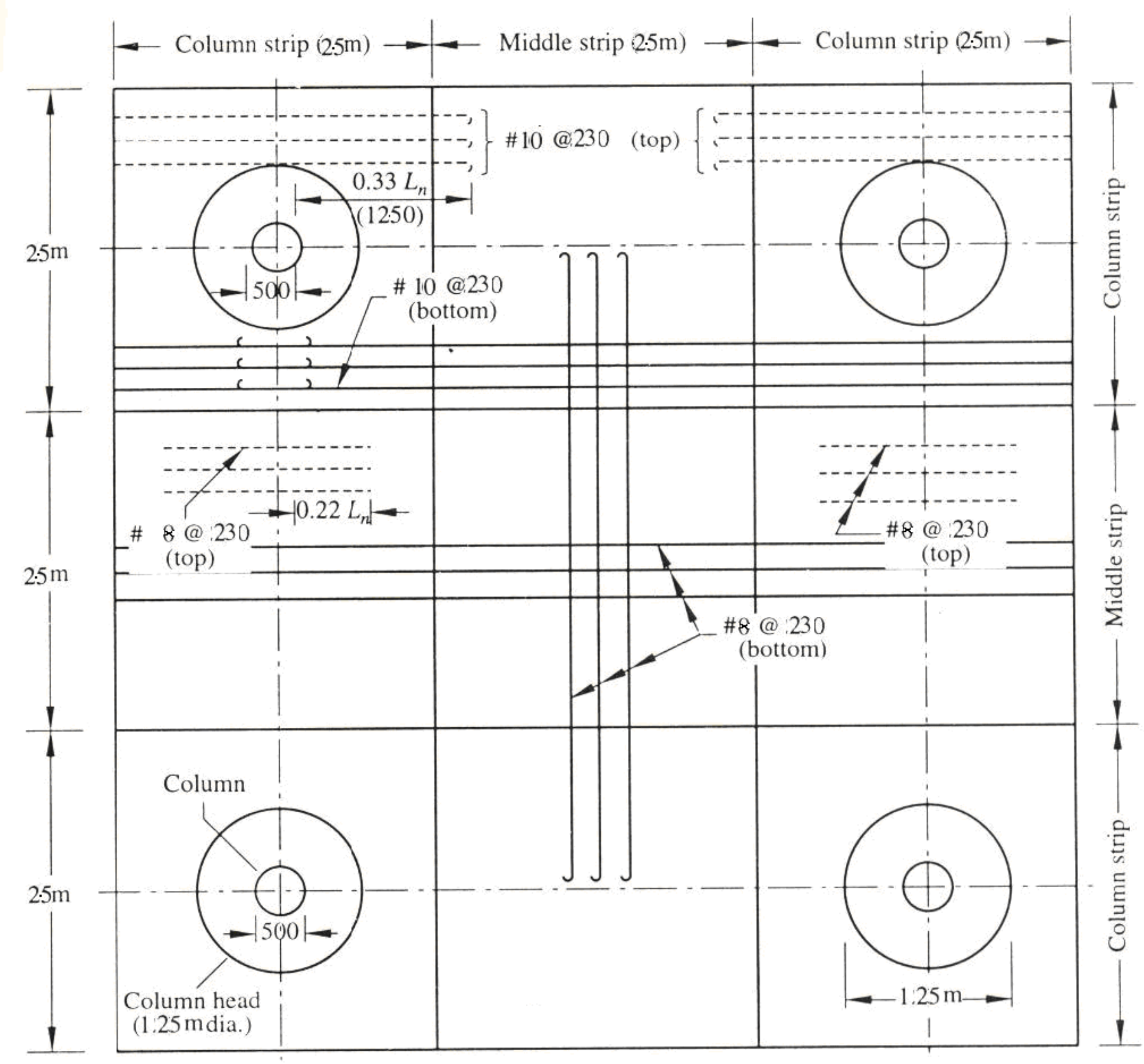
Middle strip: (b = 2.5m), (d = 175mm)

Negative and positive moment: 23.7 kNm

A_{st} = 382.6 mm^2

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

Provide 8mm @ 230mm c/c.



Plan (interior panel)

FLAT SLAB [EXTERIOR PANEL] (Cl31.4.3.3, IS456-2000)

Stiffness of slab and column = $\frac{4EI}{L}$, where, I = bd^3/12 (or) πd^4/64, E = 5000$\sqrt{f_{ck}}$

α_c is checked with $\alpha_{c\ min}$ given in Table 17 of IS456. From Cl.31.4.3.3, the interior and exterior negative moments and the positive moments are found.

Interior negative design moment is,

$$0.75 - \frac{0.10}{1+\frac{1}{\alpha_c}}$$ where, $\alpha_c = \frac{\sum K_c}{K_s}$

Interior positive design moment is,

$$0.63 - \frac{0.28}{1 + \frac{1}{\alpha_c}}$$

Exterior negative design moment is,

$$\frac{0.65}{1 + \frac{1}{\alpha_c}}$$

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.

l/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 = l/4 = 6/4 = 1.5m

Loading on slab:

Self weight of slab = (0.2 + 0.3)/2 x 25 = 0.25 x 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 x 12 = 18 kN/m^2

To find the value of $\alpha_c = \frac{\sum K_c}{K_s}$

as per Cl.3.4.6.,

α_c = flexural stiffness of column and slab

ΣK_c = summation of flexural stiffness of columns above and below

ΣK_s = summation of flexural stiffness of slab

$$\Sigma K_c = 2\left(\frac{4EI}{L}\right) = 2\left(\frac{4xExI_c}{L_c}\right) = \frac{2x4x22.3606x10^3x1.25x10^9}{3000}$$

Where, I = $\pi d^4/64 = \pi \times 400^4/64$, E = $5000\sqrt{f_{ck}}$ = 22.3606×10^3

$$\Sigma K_s = \frac{4xEx6000x(250)^3}{12x6000} = 5.208 \times 10^6 E$$

α_c = 0.644

From Table 17 of IS456-2000

$\alpha_{c\ min} = L_2/L_1$ = 6/6 = 1

$$L_L / D_L = \frac{5}{(6.25+0.75)} = 0.71 \sim 1$$

$\alpha_{c\ min}$ = 0.7

$\alpha_{c\ min}$ should be < $\alpha_{c\ min}$

α_c = 0.7

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

W = $w_u \times L_2 \times L_n$ = 18 x 6 x 4.5 = 486 kN

L_n = 6 – 1.5 = 4.5m

As per Cl.31.4.3.3 of IS456-2000,

Exterior negative design moment is,

$$\frac{0.65}{1+\frac{1}{\alpha_c}} \times M_o = \underline{73.168} \text{ kNm} \qquad \text{where, } \alpha_c = 0.7$$

Interior negative design moment is,

$$0.75 - \frac{0.10}{1+\frac{1}{\alpha_c}} \times M_o \qquad \text{where, } \alpha_c = \frac{\sum K_c}{K_s}$$

= 193.775 kNm

For column strip (75%),

= 0.75 x 193.775 = <u>145.3309</u> kNm

For middle strip (25%),

= 0.25 x 193.775 = <u>48.44</u> kNm

Interior positive design moment is,

$$0.63 - \frac{0.28}{1+\frac{1}{\alpha_c}} \text{ x } M_o$$

= 140.708 kNm

For column strip (60%),

= 0.6 x 140.708 = 84.43 kNm

For middle strip (40%),

= 0.4 x 140.708 = 56.28 kNm

Check for depth:

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

➔ 145.331 x 106 = 0.138 x 20 x 3 x d^2

➔ d_{cs} = 132.484 mm < 275 mm

M_{ms} = 82.4 kNm

➔ d_{ms} = 82.4 mm < 175 mm

Check for punching shear:

SF = TL – (Load on circular area)

= 18 x 6 x 6 – [π(1.775)^2/4] x 18 [w_n = 18]

= 648 – 44.54 = 603.45 kN [D + d = 1.5 + 0.275 = 1.775m]

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

Nominal shear stress = ς_v = $\frac{V_u}{b.d}$ = $\frac{108.216x10^3}{1000x275}$ = 0.394 N/mm2

Design shear stress: ς_c = K.ς_c’

where, K = (0.5 + β) ≤ 1

= (0.5 + 6/6) ≤ 1

= 1.5 ≤ 1 ➔ K = 1

ς_c’ = 0.25 $\sqrt{f_{ck}}$ = 1.118 N/mm^2

$\varsigma_v < \varsigma_c$

Section is safe in shear.

A_{st} for exterior negative moment (73.168 kNm), b = 3000mm, d = 275mm,

$M_u = 0.87.f_y.A_{st}.\left(d - 0.42\left(\frac{0.87.f_y A_{st}}{0.36.f_{ck}.b}\right)\right)$ [OR] K = $\frac{M_u}{bd^2}$ ➔ Take p_t from SP16

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

➔ A_{st} = 751.373 mm^2

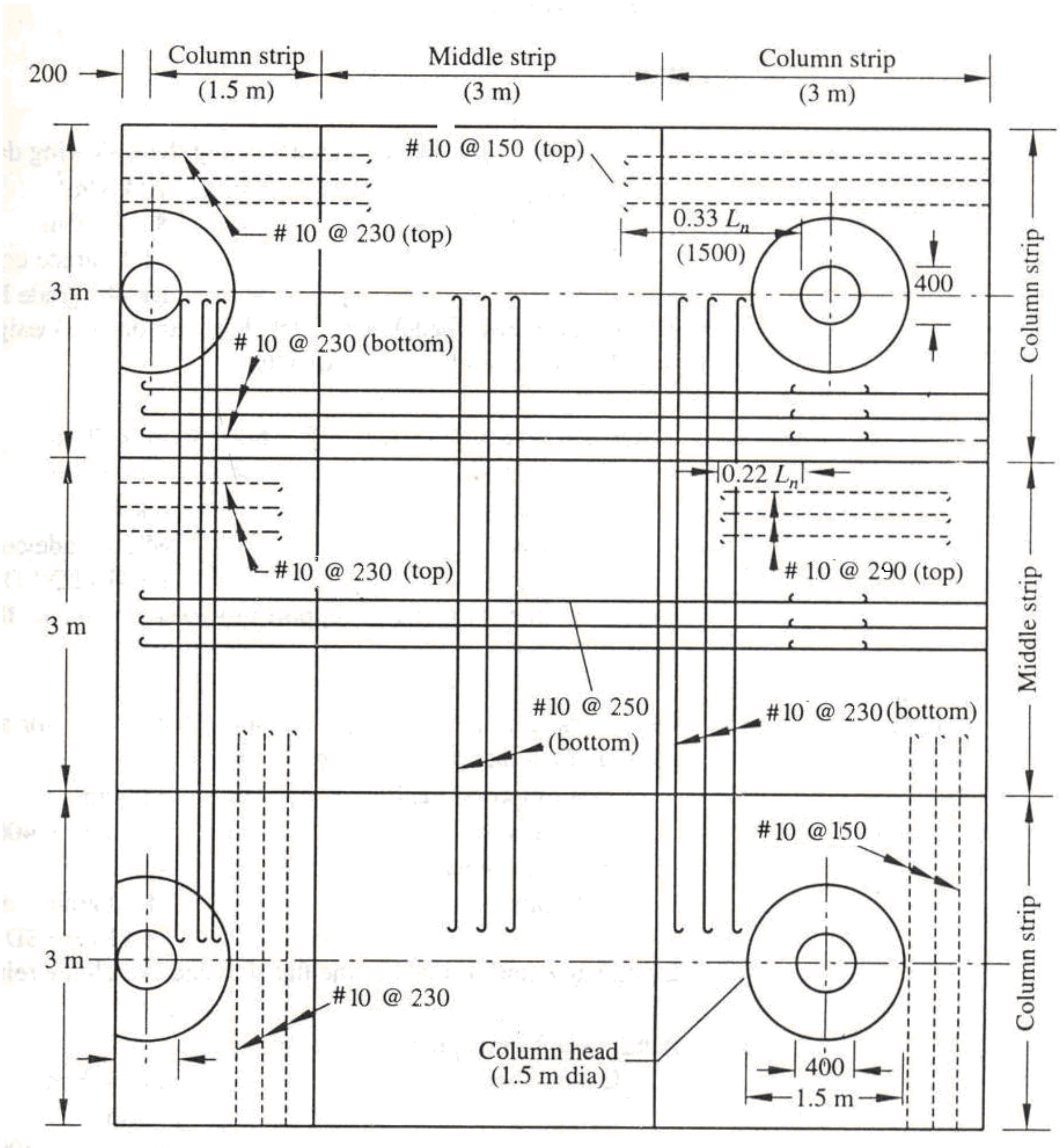
Required 10mm @ 310mm c/c

Min A_{st}: 0.12% of c/s = 0.12/100 x 3000 x 275 = 990 mm^2

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
Ext. –ve 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



Plan (exterior panel)

UNIT V

DESIGN OF BRICK MASONRY

[IS1905-1987] – Reaffirmed 1998

Brick

(Comp. strength → 3.5 to 40 N/mm^2)

Clay | Calcium Silicate (Cement, FAL-G)

Used for,

1. External and internal bearing walls
2. Load bearing piers and columns
3. Parititionwalls
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.

<u>Classification of bricks: [Based on shapes]</u>

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/mm^2.
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/mm^2.

<u>Tests on bricks:</u>

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

    i) upto 12.5 N/mm^2 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/mm^2

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) : ($\lambda_{max} = 27$)

($\lambda = 60$ for RC columns, $\lambda = 45$ & 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 $\lambda \leq 6$ 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 Table 4, IS1905-1987.

Table 4 – Effective height:

Support condition	Effective height
Fixed – Fixed	0.75H
Fixed – Hinged	0.85H
Hinged – Hinged	H
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

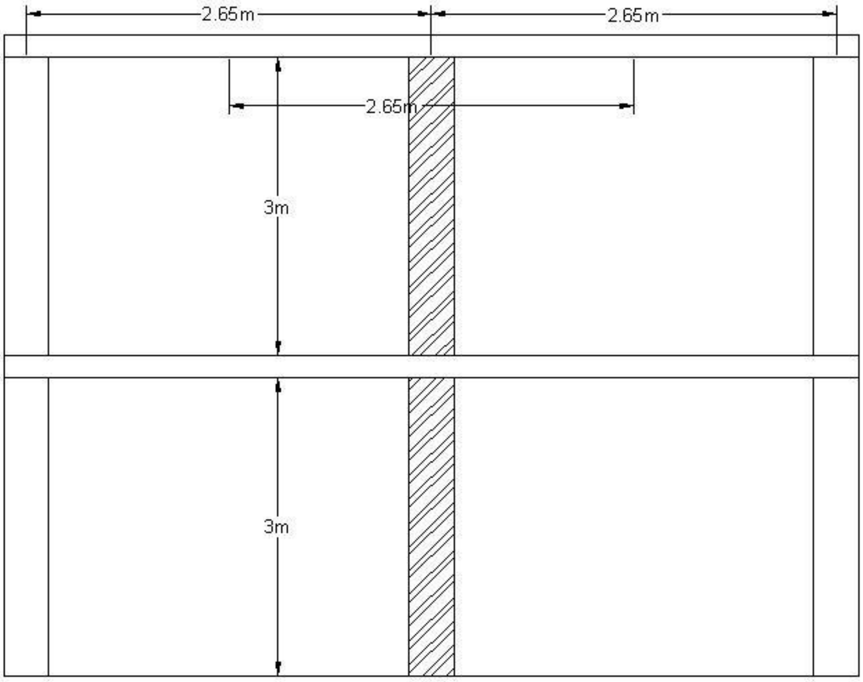
1. Design an interior cross wall for a two storeyed building to carry 100mm thick RC slab with 3m storey height. The wall is unstiffened and supports 2.65m wide slab. Loading on the slab is given as below:

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/mm^2 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 x 0.1 x 25 = 5 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 x 0.1 x 20 x 3 = 12 kN/m

Total = 40.36 kN/m

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength 10N/mm^2 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 Cl.5.4.

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 = 2.25 / 0.1 = 22.5 < 27

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

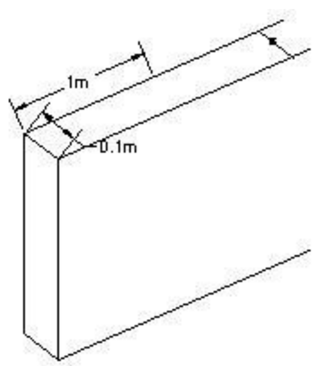
For 22 → 0.56 (e = 0)

For 24 → 0.51

For 22.5 → 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) = 0.85

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

Ksh 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 $0.75x_o$ → 1

For $1x_o$ → 1.1

For $0.9x_o$ → 1.06

$\underline{K_{sh} = 1.06}$

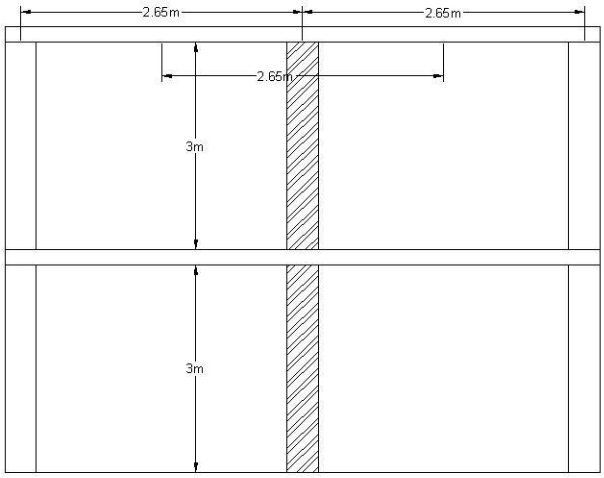
$\sigma_{per\ modified} = K_{st}.K_A.K_{sh}.\ \sigma_{per}$

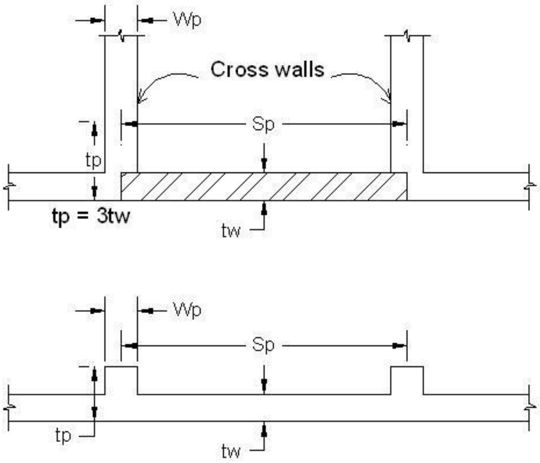
$= 0.55 \times 0.85 \times 1.06 \times 0.96 = 0.48\ N/mm^2$

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

Hence the adopted thickness of 100mm with M1 mortar and masonry unit with compressive strength $10N/mm^2$ 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/mm² [From Table 8, IS1905 – 1987]

Slenderness ratio, λ → Least of H_e/t & L_e/t

$H_e = 0.75\ H = 0.75 \times 3 = 2.25m$

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

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

t_p → Thickness of pier

t_p = 3tw [for cross walls] cl.4.6.3, IS1905-1987

S_p = 3.7m, [S_p → c/c spacing of pier]

t_w = 0.1m [t_p → thickness of pier]

t_p = 3tw = 0.3m [t_w → thickness of wall]

w_p = 0.1m [w_p → width of pier] 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 $\frac{S_p}{w_p} = 37$, $\frac{t_p}{t_w} = 3$

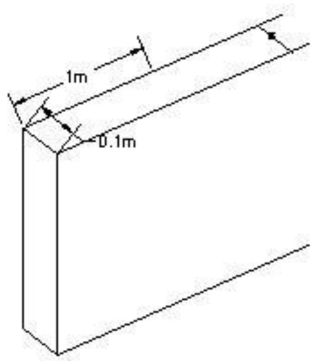
$S_e = 1$

Thickness of wall = 1 x 0.1 = 0.1m

[Considering stiffness]

λ = 2.25 / 0.1 = 22.5 < 27

The stress reduction factor (K_{st}) for Table no.10 for λ = 22.5 with no eccentricity (e=0) condition,



For λ = 22 0.56

For λ = 24 0.51

For λ = 22.5 0.55

K_{st} = 0.55

Area reduction factor for area = 0.1 x 1 = 0.1m2 < 0.2m2,

KA = 0.7 + (1.5 x 0.1) = 0.85

Shape modification factor: [Cl.5.4.1.3]

Ksh for block size of 200 x 100 x 90mm laid along 100mm side from Table 10 for height to width ratio of 90 x 100mm,

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

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

$\sigma_{per\ modified} = K_{st}.K_A.K_{sh}.\ \sigma_{per}$

$= 0.55 \times 0.85 \times 1.06 \times 0.96 = 0.48\ N/mm^2 > \sigma_{act}$

Provided masonry wall of thickness 100mm with M1 mortar and compressive strength of each unit 10 N/mm^2 is safe.

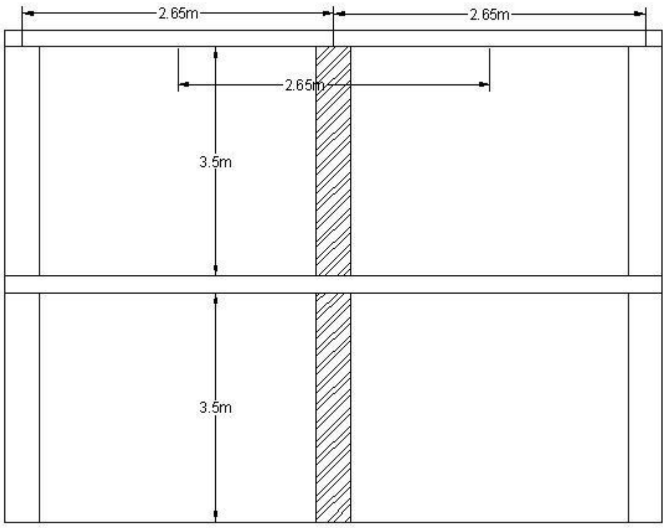
3. Design an interior cross wall for a two storeyed building to carry 100mm thick RC slab. Check the safety of the wall if the wall is continuous and cross wall is available on only one side and the storey height is 3.5m. The wall supports 2.65m wide slabs on both sides. Loading on the slab is given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

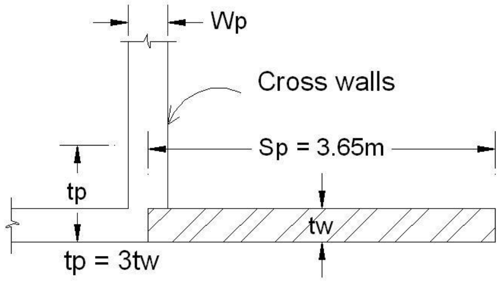
i) Live load on floor slab = 2 kN/m^2

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/mm^2 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 x 0.1 x 25 = 5 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 x 0.1 x 20 x 3.5 = 14 kN/m
Total	= 42.36 kN/m

$\sigma_{act\ /m} = \dfrac{42.36x10^3}{100x1000}$ = 0.4236 N/mm^2

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength $10N/mm^2$ 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 Cl.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 H = 0.75 x 3.5 = 2.625m [Both ends fixed]

Effective length, le = 1.5 L = 1.5 x 3.65 = 5.475m [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,

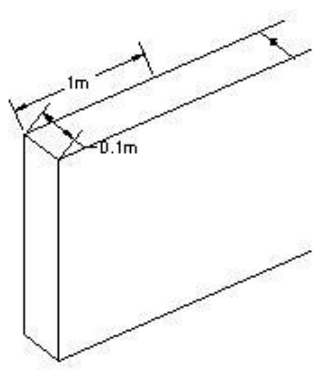
For 26 → 0.45 (e = 0)

For 27 → 0.43

For 26.25 → 0.3375 + 0.1075 = 0.445

K_{st} = 0.445

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) = 0.85

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 $0.75x_o$ → 1

For $1x_o$ → 1.1

For $0.9x_o$ → 1.06

<u>K_{sh} = 1.06</u>

$\sigma_{per\ modified}$ = $K_{st}.K_A.K_{sh}.\ \sigma_{per}$

= 0.445 x 0.85 x 1.06 x 0.96 = 0.385 N/mm^2

< $\sigma_{act\ /m}$ [0.4236 N/mm^2]

Hence the adopted thickness of 100mm with M1 mortar and masonry unit with compressive strength 10N/mm^2 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 x 0.2 x 20 x 3.5 = 28 kN/m

Total = <u>56.36 kN/m</u>

Loading on masonry wall = 56.36 kN/m

Actual stress $\sigma_{act} = \frac{56.36x1000}{200x1000}$ = 0.2818 N/mm^2

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength 10N/mm^2 is taken from Table 8, IS 1905 – 1987.

Permissible stress σ_{per} = 0.96 N/mm^2

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

<u>Stress reduction factor (K_{st})</u>

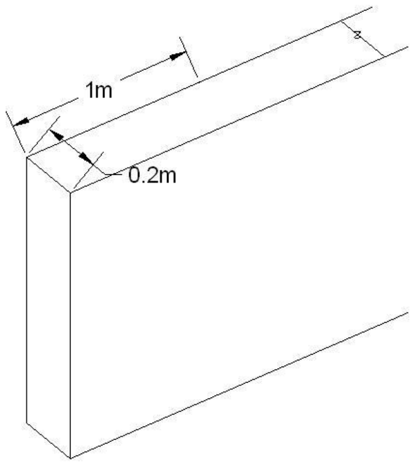
Slenderness ratio (Least of l_e/t & h_e/t)

From Table 4,

Effective height, h_e = 0.75 H = 0.75 x 3.5 = 2.625m [Both ends fixed]

Effective length, le = 1.5 L = 1.5 x 3.65 = 5.475m [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 λ = 22.5 and no eccentricity condition is,

For 12 → 0.84 (e = 0)

For 14 → 0.78

For 13.125 → 0.3675 + 0.439 = 0.806

<u>K_{st} = 0.806</u>

<u>Area reduction factor (K_A)</u> [Cl.5.4.1.2, IS1905-1987]

A = 0.2 x 1 = 0.2m^2

<u>K_A = 1</u>

<u>Shape modification factor (K_{sh})</u> [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 $0.75x_o$ → 1

For $1x_o$ → 1.1

For $0.9x_o$ → 1.06

<u>K_{sh} = 1.06</u>

$\sigma_{per\ modified}$ = $K_{st}.K_A.K_{sh}.\ \sigma_{per}$

= 0.806 x 1 x 1.06 x 0.96 = 0.82 N/mm^2

> $\sigma_{act\ /m}$ [0.2818 N/mm^2]

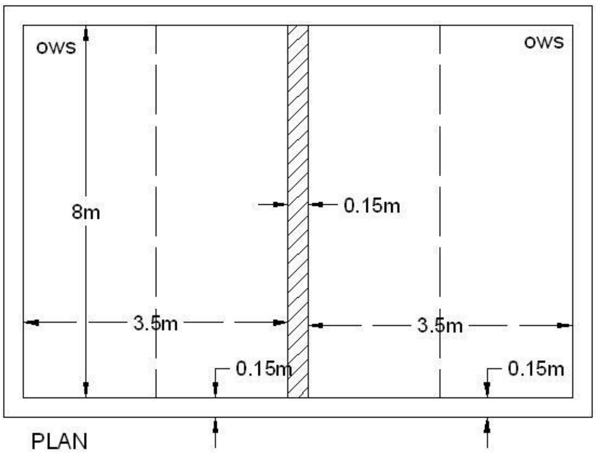
Hence the adopted thickness of 200mm with M1 mortar and masonry unit with compressive strength 10N/mm^2 is safe in carrying the load from slab.

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/m^2.

Load from slab = 30 x 3.65 = 109.5 kN/m

Self weight of wall = 0.15 x 3.5 x 1 x 20 = 10.5 kN/m

Total = 120 kN/m



PLAN

Actual stress = $\frac{120x1000}{150x1000}$ = 0.8 N/mm^2

Permissible stress of masonry for M1 mortar and masonry unit of compressive strength 10N/mm^2 is taken from Table 8, IS 1905 – 1987.

Permissible stress σ_{per} = 0.96 N/mm^2

Stress reduction factor, Area reduction factor, Shape modification factor are applied as per Cl.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 H = 0.75 x 3.5 = 2.625m [Both ends fixed]

Effective length, le = 1 L = 1.0 x 8.15 = 8.15m

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

S_p = 8.15m, [S_p → c/c spacing of pier/wall]

t_w = 0.15m [t_p → thickness of pier/wall]

t_p = 3tw = 0.45m [t_w → thickness of wall]

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

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

From Table 6, for $\frac{S_p}{w_p}$ = 54.3, $\frac{t_p}{t_w}$ = 3

S_e = 1

Thickness of wall = 1 x 0.2 = 0.2m

[Considering stiffness]

λ = 2.625 / 0.2 = 17.5 < 27

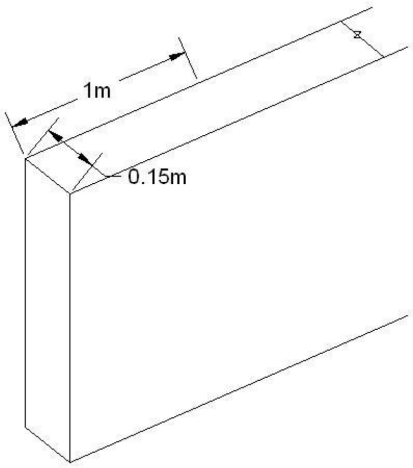
Therefore, the stress reduction factor from Table 10 for λ = 17.5 and no eccentricity condition is,

For 16 → 0.73 (e = 0)

For 18 → 0.67

For 13.125 → 0.1825 + 0.5025 = 0.685

K_{st} = 0.685



Area reduction factor (K_A) [Cl.5.4.1.2, IS1905-1987]

A = 0.15 x 1 = 0.15m^2

K_A = 0.7 + (1.5 x 0.15) = 0.925

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 → 1

For 1x_o → 1.1

For 0.9x_o → 1.06

K_{sh} = 1.06

$\sigma_{per\ modified}$ = $K_{st}.K_A.K_{sh}.\ \sigma_{per}$

= 0.685 x 0.925 x 1.06 x 0.96 = 0.647 N/mm^2

< $\sigma_{act\ /m}$ [0.8 N/mm^2]

Hence the adopted M1 mortar and masonry unit with compressive strength 10N/mm^2 is **not** sufficient in carrying the load.

Increase the strength of brick unit and mortar as,

H1 mortar and masonry unit compressive strength 15N/mm^2

σ_{per} = 1.31 N/mm^2

$\sigma_{per\ modified}$ = 0.88 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.

5. Design a masonry wall of height 4m subjected to a load of 20kN/m. Use M1 mortar. The 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/mm^2

σ_{per} = 0.96 N/mm^2

H_e = 0.75 H = 0.75 x 4 = 3m

λ = $\frac{3}{0.3}$ = 10

K_{st} = 0.89

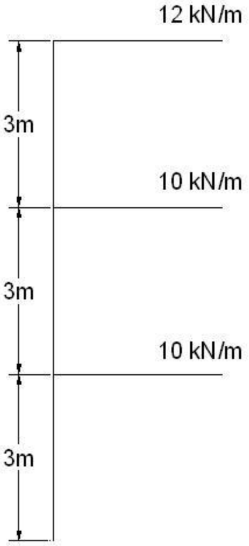
For A = 0.3 x 1 = 0.3 m2, KA = 1

K_{sh} = 1.06

$\sigma_{per\ modified}$ = 0.27 N/mm^2 > σ_{act} [0.066 N/mm^2]

Hence, safe.

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



Loading on brick wall:

Load from slab	= 12 + 10 + 10	= 32 kN/m
Weight of wall (self wt.)	= 3 x 2 x 0.1 x 3 x 20	= 36 kN/m
Total		= 68 kN/m

Actual stress = $\frac{68x1000}{2x100x1000}$ = 0.34 N/mm^2

Use M1 mortar and brick of compressive strength 10 N/mm^2.

σ_{per} = 0.96 N/mm^2 [From Table 8, IS1905 – 1987]

(λ ≤ 6)

h_{eff} = 0.75 x h = 0.75 x 3 = 2.25m

t_e = 2/3(t_w + t_w) = 2/3(0.1 + 0.1) = 0.133m

λ = $\frac{2.25}{t_e}$ = $\frac{2.25}{0.133}$ = 16.875

From Table 8, IS1905 – 1987,

λ → 16, 0.75

λ → 18, 0.67

λ → 16.875, (0.421875 + 0.293125 = 0.715)

Area of wall (each leaf) = 0.1 x 1 = 0.1m^2 < 0.2 m^2

K_A = 0.85

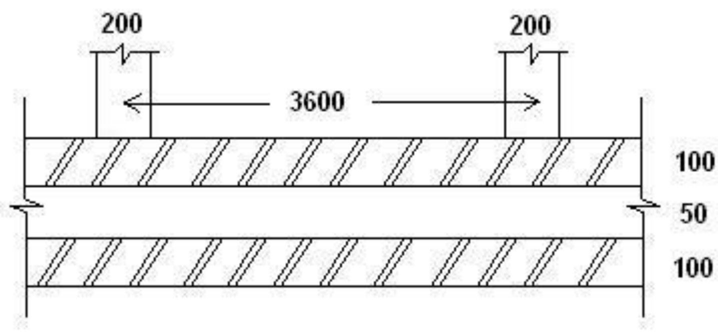
K_{sh} = 1.06

$\sigma_{per\ (modified)}$ $= K_{st} \times K_A \times K_{sh} \times \sigma_{per}$

$= 0.704 \times 0.85 \times 1.06 \times 1.96 = 0.61\ N/mm^2 > \sigma_{act}$

Therefore, the cavity wall is safe with M1 mortar and masonry unit of compressive strength $10N/mm^2$.

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/m

Load from floor = 12.5 + 12.5 kN/m

Wall load [3x0.2x20] = 36 kN/m

Total = <u>77 kN/m</u>

Actual stress = $\sigma_{ac} = \frac{77x10^3}{200x1000}$ = 0.39 N/mm^2

σ_{per} :

Assume M1 mortar and brick of compressive strength 10 N/mm^2.

From Table 7, IS1905 – 1987,

σ_{per} = 0.96 N/mm^2 ($\lambda \le 6$)

Effective height = h_{eff} = 0.75 x 3 = 2.25m

Effective length = l_{eff} = 0.8 l = 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, $\frac{t_p}{t_w} = \frac{3t_w}{t_w} = \frac{0.3}{0.1} = 3$

From Table 8, IS1905 – 1987,

→ 15, 1.2

→ 20, 1

→ 18, 0.48 + 0.6 = 1.08

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

$\lambda = \frac{2.25}{0.139} = 16.18$

λ → 16, 0.73

$\lambda \rightarrow 20$, 0.67

$\lambda \rightarrow 16.18$, $0.664 + 0.0603 = 0.7243$

$K_{st} = 0.72$

Area = $0.1 \times 1 = 0.1 m^2 < 0.2 m^2$

$K_A = 0.85$

$K_{sh} = 1.06$

$\sigma_{per\ (modified)} = 0.62\ N/mm^2 > \sigma_{act}\ [0.39\ N/mm^2]$

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/mm^2.

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

σ_{per}:

h_{eff} = h[Table 4, IS1905-1987]

$\lambda = 2000/400 = 5 < 6$

There is no need for Stress reduction factor ($K_{st} = 1$)

From Table 7, $\sigma_{per} = 0.96\ N/mm^2$

$A_{st} = 0.4 \times 0.4 = 0.16 m^2 < 0.2 m^2$

$K_A = 0.7 + (1.5 \times 0.4 \times 0.4) = 0.94$

$K_{sh} = 1.06$

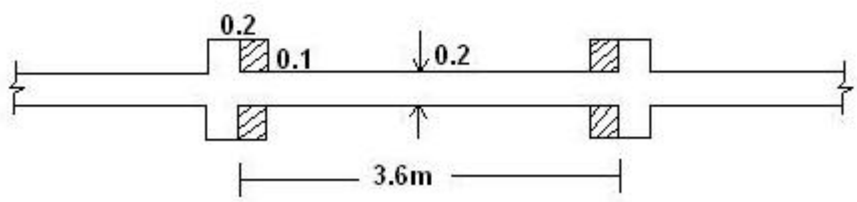
$\sigma_{per\ (modified)} = 0.94 \times 0.96 \times 1 \times 1.06 = 0.98\ N/mm^2$.

$\sigma_{act} < \sigma_{per}$

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.

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 RC roof. Assume roof load as 45kN/m. The wall is stiffened by piers at equal intervals shown in figure.



Height = 5.4m, w = 45 kN/m

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 x 0.2) + 4(0.1 x 0.1) = 0.76 m^2

Loading per bay (for 3.6m length) = 45 x 3.6 = 162 kN

Load from brick wall = 0.76 x 5.4 x 20 = 82.08 kN

Total = 244.08 kN

Actual stress = $\sigma_{act} = \frac{244.08x10^3}{0.76x10^6}$ = 0.321 N/mm^2

$\sigma_{per:}$

h_{eff} = 0.75h = 4.05m [Table 4, IS1905-1987] demise

leff = 0.8l = 0.8 x 3.6 = 2.88m

λ = 2.88 < 6

There is no need for Stress reduction factor (K_{st} = 1)

From Table 7, σ_{per} = 0.96 N/mm^2

A_{st} = 0.4 x 0.4 = 0.16m^2 < 0.2m^2

K_A = 0.7 + (1.5 x 0.4 x 0.4) = 0.94

K_{sh} = 1.06

$\sigma_{per\ (modified)}$ = 0.94 x 0.96 x 1 x 1.06 = 0.98 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,

$\frac{S_p}{w_p} = \frac{3.6}{0.2}$ = 18, $\frac{t_p}{t_w} = \frac{0.4}{0.2} = 2$

(tp → depth of pier (0.4))

Sc = 1.04

t_{eff} = 1.04 x 0.2 = 0.208m

λ is the least of H_{eff} and L_{eff}

λ = $\frac{2.88}{0.208}$ = 13.85

Kst = 0.785

Area reduction coefficient = $\frac{0.76}{3.6}$ = 0.211 > 0.2

K_A = 1 [Cl.5.4.1.2, IS1905 – 1987]

Ksh = 1.06

$\sigma_{per\ (modified)}$ = 0.785 x 0.96 x 1 x 1.06 = 0.79 N/mm^2 > σ_{act}

Inference : Hence the brick wall is safe with M1 mortar and brick of compressive strength 10N/mm^2.

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.

Load from slab = 100kN

Self weight of brick pillar = 0.4 x 0.4 x 20 x 3 = 9.6kN

Total = 109.6 kN

Assume a column size of 400mm x 400mm.

Actual stress = $\frac{109.6x10^3}{400x400}$ = 0.685 N/mm^2

Assume grade of mortar as M1 and compressive strength of 0.96N/mm^2

h_{eff} = 0.75.H = 2.25m

$\lambda = \frac{2.25}{0.4}$ = 5.625 < 6

There is no need of stress reduction factor.

K_{st} = 1

A = 0.4 x 0.4 = 0.16 m^2 < 0.2m^2

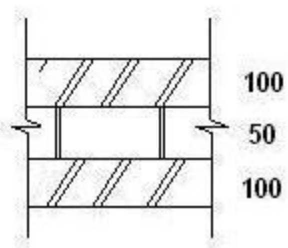
K_A = 0.7 + (1.5 x 0.4 x 0.4) = 0.94

K_{sh} = 1.06 [Brick unit 200 x 100 x 90]

$\sigma_{per\ (modified)}$ = 0.9565 N/mm^2 > σ_{act}

Hence the brick wall is safe with M1 mortar and compressive strength of 10N/mm^2.

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.



h = 3m, l = 3.6m

Load from roof = 12 kN/m

Load from each floor = 10 + 10 kN/m

Self weight of wall = 3 x 2 x 0.1 x 3 x 20 = 36 kN/m

Total = 68 kN/m

Actual stress = $\frac{68x10^3}{2x100x1000}$ = 0.34 N/mm^2

Use M1 mortar and brick of compressive strength 10N/mm^2,

σ_{per} = 0.96 N/mm^2

h_{eff} = 0.75 x 3 = 2.25m

t_e = 2/3(t_w + t_w) = 0.133m

$\lambda = \frac{2.25}{t_e}$ = 16.875

From Table 8, IS1905 – 1987,

K_{st} = 0.704

Area of wall = 0.1 x 1 = 0.1 m^2 < 0.2 m^2

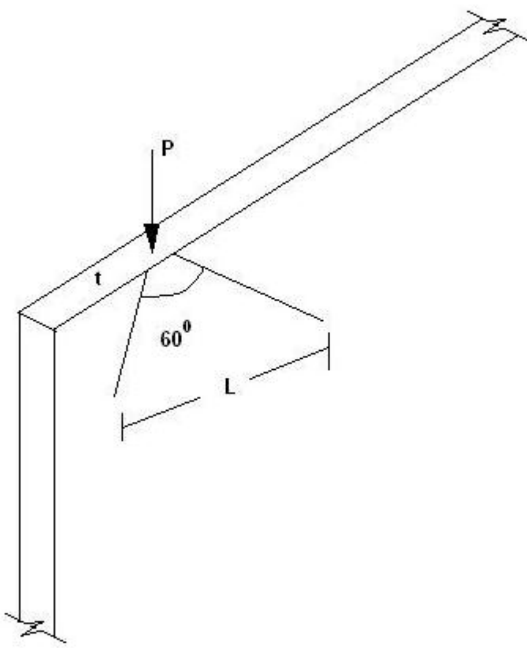
K_A = 0.85

K_{sh} = 1.06

$\sigma_{per\ (modified)}$ = 0.704 x 0.85 x 1.06 x 0.96 = 0.61 N/mm^2 > σ_{act}

Hence the brickwork is safe with M1 mortar and brick of compressive strength 10N/mm^2.

MASONRY WALL SUBJECTED TO CONCENTRATED LOAD



$\sigma_{br} < \sigma_{br\ per}$

where, $\sigma_{br} = \frac{Load\_from\_beam}{Bearing\_area} = \frac{P}{A_{br}}$,

where, P → reaction from conc. load [Eg: From Beam]

A_{br} → Bearing area

$\sigma_{br\ per}$ = 1.5 (σ_{per}) [σ_{per} given in Table 8, IS1905-1987]

When concentrated load is applied on a masonry wall, the wall is checked for load bearing stress. Permissible stress in bearing is taken as 50% more than the value given in Table 7 of IS1905-1987.

The angle of dispersion below the concentrated load is 30^o on each side.

Therefore, the actual stress is,

σ_{act} = P / A,

where, A → Area for 1m run = L x t,

L → Length of load dispersion = $\frac{2H}{\tan 60^0}$, H → 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 x 115 = 26450 mm^2

$\sigma_{br} = \frac{P}{A_{br}} = \frac{(30+20)x10^3}{26450}$ = 1.89 N/mm^2 < σ_{per} 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/mm^2.

σ_{per} = 1.31 N/mm^2

$\sigma_{per\ br}$ = 1.5 x 1.31 = 1.965 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 mm^2

$\sigma_{act} = \frac{50x10^3}{796720}$ = 0.063 N/mm^2

σ_{per} = 1.31 N/mm^2

K_{st} = 0.89

K_A = 1

K_{sh} = 1.06

$\sigma_{per\ modified} = K_{st} \times K_A \times K_{sh} \times \sigma_{per}$

= 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/mm^2.

2) Design the exterior wall of a workshop building 3.6m height carrying steel trusses at 4.5m 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 x 200 = 40000 mm^2

$\sigma_{br} = \frac{30x10^3}{40000}$ = 0.75 N/mm^2 < $\sigma_{br\ per}$ → [1.5(σ_{per}) = 1.5 x 0.96 = 1.44 N/mm^2]

Assume M1 mortar and brick of compressive strength 10 N/mm^2.

Check for compressive stress:

σ_{ac} = P/A

L = $\frac{2H}{\tan 60^0}$ = 4.157m

A = 4.157 x 10^3 x 200 = 956110mm^2

$\sigma_{act} = \frac{30x10^3}{956110}$ = 0.036 N/mm^2

H_{eff} = 0.75 x 3.6 = 2.7m

L_{eff} = 0.8 x 4.157 = 3.3256m

$\lambda = \frac{2.7x10^3}{2300}$ = 13.5

A = 4.157 x 0.2 = 0.8314 m^2 > 0.2 m^2

K_A = 1

K_{st} = 0.795

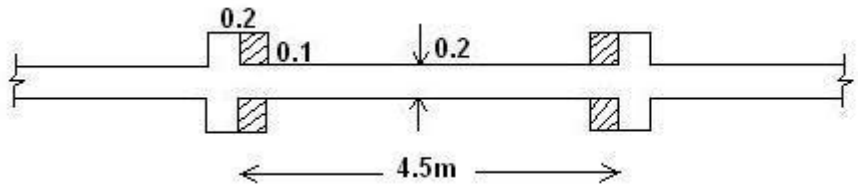
K_{sh} = 1.06

$\sigma_{per\ modified}$ = 0.81 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 mm^2

P = 30 x 10^3 N

$\sigma_{br} = \frac{30x10^3}{200x400}$

σ_{br} = 0.375N/mm^2 < $\sigma_{br\ per}$ → [1.5(0.96) = 1.44 N/mm^2]

Using M1 mortar and brick of compressive strength 10N/mm^2,

Check for compressive stress:

σ_{ac} = P/A

L = $\frac{2H}{\tan 60^0}$ = 4.16m

H_{eff} = 0.75 x 3.6 = 2.7m

L_{eff} = 0.8 x L = 0.8 x 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 x 0.2 = 0.2m

$$\lambda = \frac{2.7}{0.2} = 13.5$$

A = L x t = 4.5 x 0.2 = 0.9 m^2 > 0.2 m^2

$K_A = 1$

$K_{st} = 0.795$

$$\sigma_{act} = \frac{30x10^3}{4.5x0.2} = 0.03\text{N/mm}^2$$

$\sigma_{per\ modified}$ = 1 x 1 x 0.795 x 1.06 x 0.96 = 0.8089 N/mm^2

Hence the wall is safe with M1 mortar and brick of compressive strength 10N/mm^2.

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

$$\sigma_{act} = \frac{P}{A} + \frac{M}{Z} < 1.25(\sigma_{per\ modified}),$$

where, M → P x e,   Z → b.t^2/6,   b = 1m (wall)

Guidelines given in Appendix A

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.

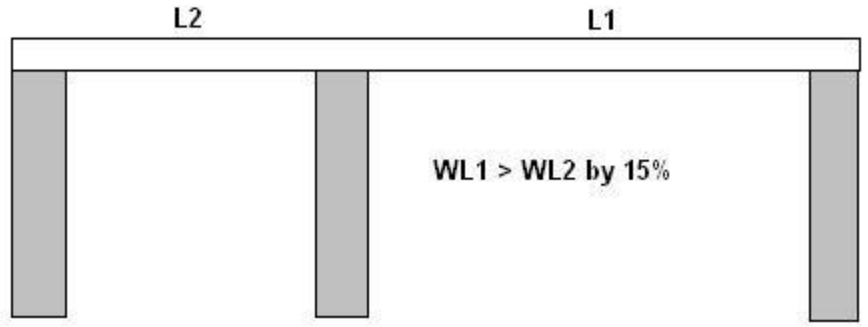
1. For an exterior wall, when span of roof is more than 30 times the thickness of wall, the eccentricity assumed is one sixth of the bearing width.

   L > 30 t_w

   E = 1/6(t_w)

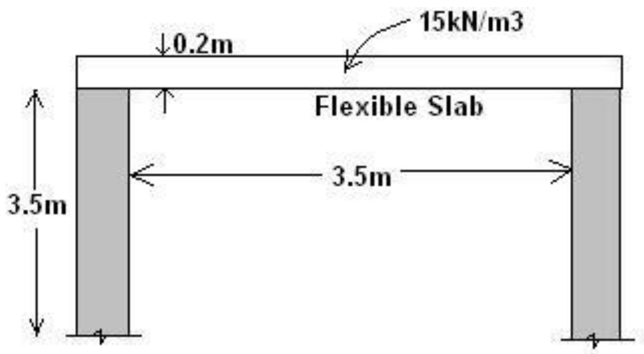
2. When bearing is not sufficient, eccentricity assumed is t_w/12

3. When flexible floors are adopted, for full width bearing, eccentricity assumed is $t_w/6$.
4. 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 $15 kN/m^3$ 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 $10 N/mm^2$.

Loading on wall:

Load from slab = 15 x 0.2 x 3.5/2 = 5.25 kN/m

Self weight of wall = 0.2 x 3.5 x 20 = 14 kN/m

Total = <u>19.25 kN/m</u>

$e = t_w/6 = 33.33mm$

M = P x e

Moment due to eccentricity = $19.25 \times 10^3 \times 33.33 = 641.67 \times 10^3$ Nmm

$Z = bt^2/6 = 1000 \times 200^2 / 6 = 6.67 \times 10^6\ mm^3$

M/Z = 0.096 N/mm^2

Total stress = $\frac{P}{A}+\frac{M}{Z}$ = $\frac{19.25x10^3}{1000x200}$ + 0.096 = 0.192 N/mm^2

σ_{per} :

Use M1 mortar and brick of compressive strength 10N/mm^2,

σ_{per} = 0.96 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 → 0.78 (e = 0)

For 14 → 0..7

For 13.125 → 0.735

K_{st} = 0.735

Area reduction factor (K_A) [Cl.5.4.1.2, IS1905-1987]

For thickness t = 0.2m, A = 0.2 m^2

K_A = 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_o → 1

For 1x_o → 1.1

For 0.9x_o → 1.06

K_{sh} = 1.06

$\sigma_{per\ modified}$ = $K_{st}.K_A.K_{sh}.\ \sigma_{per}$

= 0.735 x 1 x 1.06 x 1.25 x 0.96 = 0.935 N/mm^2

σ_{act} [0.192N/mm^2] < 1.25 σ_{per} [1.17N/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

$\sigma_{act} = \frac{100x10^3}{400x400} = 0.625$ N/mm^2

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

Heff = H = 2m

$\lambda = 5 < 6$

$K_{st} = 1$

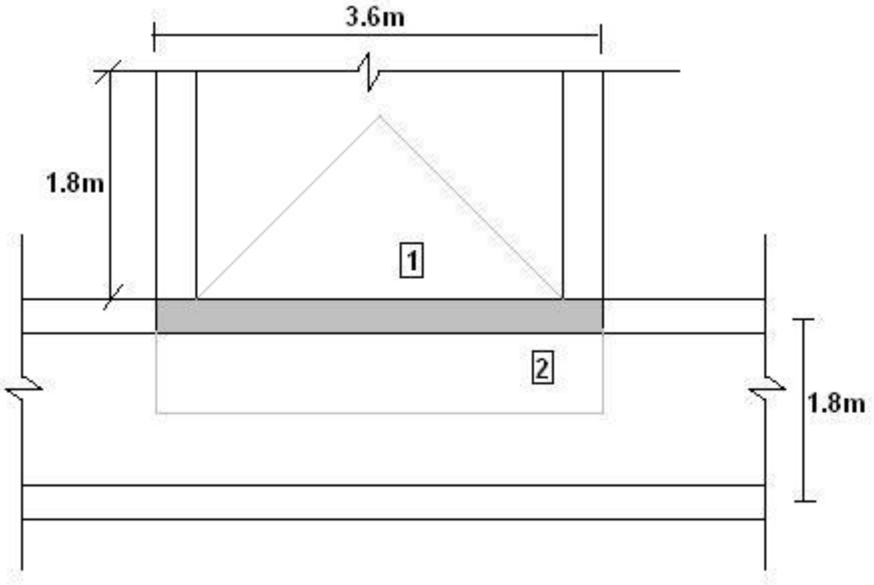
$K_{sh} = 1.06$

$K_A = 0.94$

$\sigma_{per\ modified} = 0.957$ N/mm^2

$\sigma_{act} < \sigma_{per}$

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/m^3. The height of the wall is 3.8m and the wall is fixed to the foundation block below.



Height = 3.8m

$e = \frac{t_w}{6} + \frac{t_w}{6} = \frac{2t_w}{6} = \frac{t_w}{3}$

$\sigma_{act} = \frac{P}{A} + \frac{M}{Z}$

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 = $\frac{t_w}{6}+\frac{t_w}{6}$ = $\frac{2t_w}{6}$ = $\frac{t_w}{3}$

M → (Difference in load from slab x Eccentricity)

= (9 – 4.5) x 10^3 x 200/3 = 300 x 10^3 Nmm

σ_{act} = $\frac{P}{A}+\frac{M}{Z}$, where Z = $\frac{b.t^2}{6}$ = $\frac{1000x200^2}{6}$ = 6.67 x 10^6 mm^3

σ_{act} = $\frac{28.7x10^3}{1000x200}+\frac{300x10^3}{6.67x10^3}$ = 0.1885 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 x S_c

$\frac{S_p}{W_p}=\frac{3.8}{0.2}$ = 19, $\frac{t_p}{t_w}=\frac{0.6}{0.2}=3$

S_c = 1.04

t_e = t x Sc = 1.04 x 0.2 = 0.208m

H_{eff} = 0.75 x 3.8 = 2.85m

λ = $\frac{2850}{208}$ = 13.7

L_{eff} = 0.8 L = 0.8 x 3.60 = 2.88m

λ = 13.8

λ → 12 0.72

λ → 14 0.66

λ → 13.7 0..67

K_{st} = 0.67

A = 0.1 x 1 = 0.1m^2 < 0.2m^2

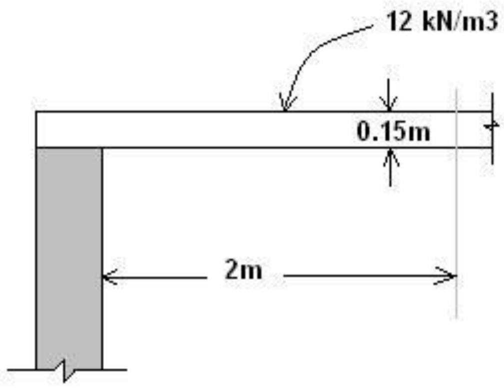
K_A = 1

K_{sh} = 1.06

$\sigma_{per\ modified}$ = 0.96 x 0.67 x 1 x 1.06 = 0.68 N/mm^2

σ_{act} [0.1885 N/mm^2] < 1.25(σ_{per}) → [0.851 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 x 0.15 x 12 = 3.6 kN/m

Self weight of wall = 0.2 x 4 x 20 = 16 kN/m

Total = 19.6 kN/m

e = t_w/6 = 200/6 = 25mm

M = P x e = 19.6 x 10^3 x 25 = 6.53 x 10^5 Nmm

$Z = \frac{b.t^2}{6} = \frac{1000x200^2}{6}$ = 6.67 x 10^6 mm^3

$\sigma_{act} = \frac{P}{A} + \frac{M}{Z}$ = 0.098 + 0.097 = 0.195 N/mm^2

H_{eff} = 0.75H = 3m

Assuming M1 mortar and brick of compressive strength 10 N/mm^2,

$\lambda = \frac{3000}{200}$ = 15

λ → 14 0.7

λ → 16 0.63

λ → 15 0.35 + 0.315 = 0..665

K_{st} = 0.665

A = 0.1 x 1 = 0.1m^2 < 0.2m^2

K_A = 1

K_{sh} = 1.06

$\sigma_{per\ modified}$ = 0.96 x 0.665 x 1 x 1.06 = 0.677 N/mm^2

σ_{act} [0.195 N/mm^2] < 1.25(σ_{per}) → [0.846 N/mm^2]