2019

RANK improvement WORKBOOK

Answer key and Hint of Objective & Conventional *Questions*

Civil Engineering

Design of Concrete and Masonry Structures

MADE EASY Publications

1 WSM & LSM

LEVEL 1 Objective Questions

1. (c)
2. (d)
3. (a)
4. (b)

LEVEL 2 Objective Questions

5. (503)
6. (a)
7. (c)
8. (c)

LEVEL 3 Conventional Questions

Solution : 1

The assumptions made in the design for the limit state of collapse in flexure are:

(i) Plane sections normal to the longitudinal axis remain plane after bending.

(ii) The maximum strain in concrete at the outermost compression fibre is taken as 0.0035 in bending.

(iii) The relationship between the compressive stress distribution in concrete and the strain in concrete may be assumed to be a rectangle, trapezoid, parabola or any other shape which results in predictions of strength in substantial agreement with the results of the test. For design purposes, the compressive strength of concrete in the structure shall be assumed to be 0.67 times the characteristic strength. The partial safety factor γ_m = 1.5 shall be applied in addition to this.

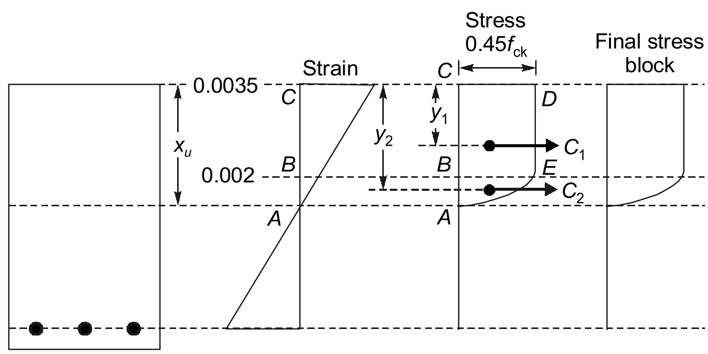
(iv) The tensile strength of the concrete is ignored.

(v) The stresses in reinforcement are derived from representative stress strain curve for the type of steel used. For design purposes, the partial safety factor γ_m = 1.15 shall be applied.

(vi) The maximum strain in the tension reinforcement in the section at failure shall not be less than

$$\frac{f_y}{1.15E_s} + 0.002$$

Derivation of stress block parameters:



Ratio of AB to AC i.e. $\frac{AB}{AC} = \frac{0.002}{0.0035}$ (from strain diagram)

$$\Rightarrow \quad \frac{AB}{AC} = \frac{4}{7}$$

$$\Rightarrow \quad AB = \frac{4}{7}AC$$

$$\Rightarrow \quad AB = \frac{4}{7}x_u$$

$$BC = AC - AB = x_u - \frac{4}{7}x_u = \frac{3}{7}x_u$$

Now, compressive force = Width of section × Area of stress diagram

Taking rectangle $BCDE$

Compressive force $\quad C_1 = B \times 0.45\, f_{ck} \times \frac{3}{7}x_u = 0.1929\, f_{ck}\, Bx_u$

Distance of line of action of compressive force C_1 from top

$$\Rightarrow \quad y_1 = \frac{1}{2}\times\frac{3}{7}x_u = \frac{3}{14}x_u$$

Taking parabola *ABEA*.

Compressive force, $$C_2 = B\times\frac{2}{3}\times 0.45\, f_{ck}\times\frac{4}{7}x_u = 0.1714\, f_{ck}\, Bx_u$$

Distance of line of action of compressive force C_2 from top

$$\Rightarrow \quad y_2 = \frac{3}{7}x_u + \frac{3}{8}\times\frac{4}{7}x_u = 0.64286x_u$$

$$\text{Total compressive force} = C_1 + C_2 = 0.1929\, f_{ck}\, B\, x_u + 0.1714\, f_{ck}\, Bx_u$$

$$C = 0.36\, f_{ck}\, B\, x_u$$

$$\bar{y} = \frac{C_1y_1 + C_2y_2}{C_1 + C_2} = \frac{0.1929\, f_{ck}\, B\, x_u\times\frac{3}{4}x_u + 0.1714\,B\,x_u\times 0.64286\,x_u}{0.36\, f_{ck}\,B\,x_u}$$

$$\Rightarrow \quad \bar{y} = 0.416\,x_u$$

Now total tensile force, $$T = 0.87\, f_y\times A_{st}$$

Lever arm, $$Z = d - 0.416\,x_u$$

$$MR\,(\text{wrt concrete}) = 0.36\, f_{ck}\, Bx_u\,(d - 0.416\,x_u)$$

$$MR\,(\text{wrt steel}) = 0.87\, f_y\, A_{st}\,(d - 0.416\,x_u)$$

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2 Shear, Torsion, Bond, Anchorage and Development Length

LEVEL 1 Objective Questions

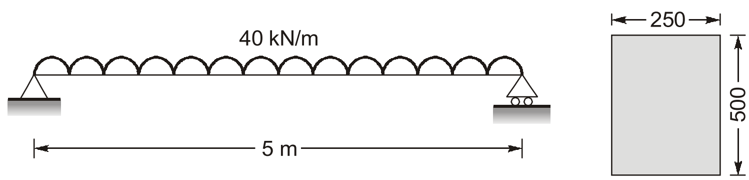
1.	(c)
2.	(6.4)
3.	(a)
4.	(a)
5.	(b)

LEVEL 2 Objective Questions

6.	(d)
7.	(d)
8.	(a)
9.	(b)
10.	(19)
11.	(c)
12.	(a)
13.	(a)
14.	(c)
15.	(b)
16.	(1309.73)
17.	(a)

LEVEL 3 Conventional Questions

Solution : 1



$$w = 40 \text{ kN/m}$$

Factored load $w_u = 1.5 \times 40 = 60$ kN/m

Max. SF, $V_u = \dfrac{w_u l}{2} = \dfrac{60 \times 5}{2} = 150$ kN

Effective depth, $d = 500 - 25 - \dfrac{20}{2} = 465$ mm

Let clear cover = 25 mm

Nominal shear stress, $\tau_v = \dfrac{V_u}{bd} = \dfrac{150 \times 10^3}{250 \times 465}$

$= 1.29 \text{ N/mm}^2 < \tau_{c\,max} = 2.8 \text{ N/mm}^2$ (OK)

$(\tau_{max} = 0.62\sqrt{f_{ck}} = 0.62\sqrt{20} = 2.8 \text{ MPa})$

Design shear strength of concrete:

$$P_t = \frac{A_{st} \text{ at critical section}}{bd} \times 100$$

At supports only 2-20ϕ bars are present on tension side.

$$\therefore \quad P_t = \frac{2 \times \frac{\pi}{4} \times (20)^2}{250 \times 465} \times 100 = 0.54\%$$

$$\therefore \quad \tau_c = 0.48 + \frac{0.56 - 0.48}{0.75 - 0.5} \times (0.54 - 0.5) = 0.4928 \text{ N/mm}^2$$

SF to be resisted by shear stirrups,

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

$$= 150 - 0.4928 \times 250 \times 465 \times 10^{-3} = 92.712 \text{ kN}$$

Using 2-legged 8 mm bars as shear stirrups

$$A_{Sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100.53 \text{ mm}^2$$

Spacing of 2-legged 8 mm stirrups

$$S_v = \frac{A_{Sv} \times (0.87 f_y) d}{V_{us}} = \frac{100.53 \times 0.87 \times 415 \times 465}{92.712 \times 10^3}$$

$$= 182.05 \text{ mm} \simeq 180 \text{ mm (say)}$$

Max. spacing corresponding to min. shear reinforcement

$$\frac{A_{Sv}}{b.S_v} = \frac{0.4}{0.87 f_y}$$

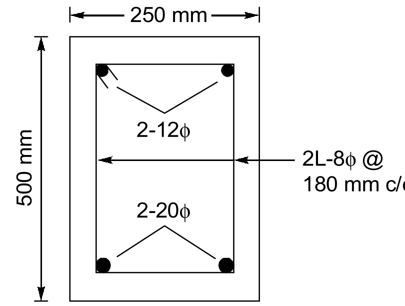
$$S_v = \frac{0.87 \times 415 \times 100.53}{0.4 \times 250} = 362.96 \approx 360 \text{ mm}$$

Max. spacing should not exceed:

(i) $0.75\, d = 0.75 \times 465 = 348.75$ mm (for vertical stirrups)

(ii) 300 mm

Hence max. spacing = 300 mm



Shear reinforcement details at support section

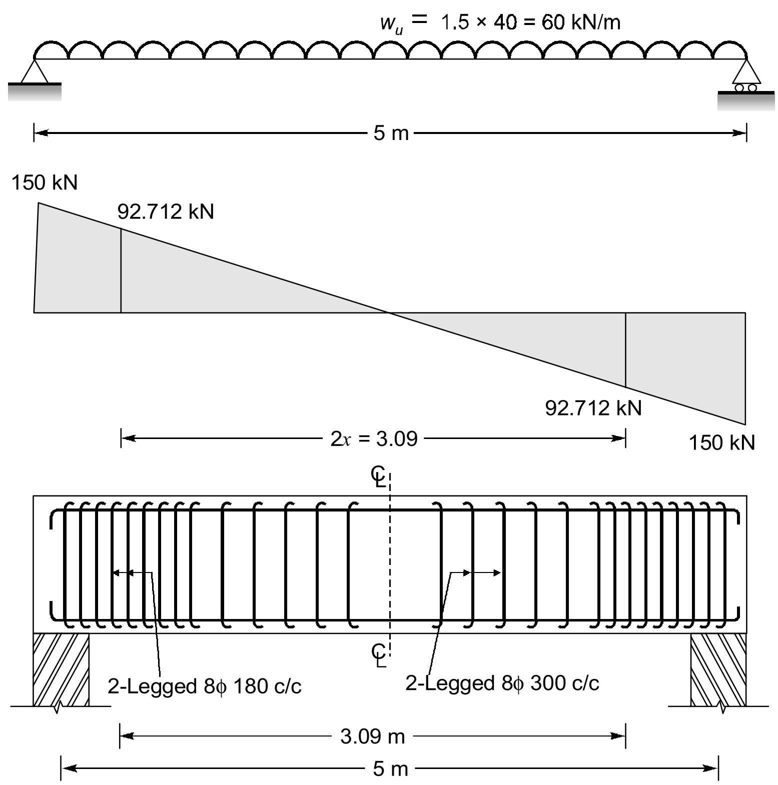
Location of minimum shear reinforcement (Max. spacing)

$$\frac{x}{92.712} = \frac{2.5}{150}$$

$$x = \frac{2.5 \times 92.712}{150} = 1.545 \text{ m from mid}$$

$\because$ Location of max. spacing, $2x = 3.09$ m

Provide 2L-8 mm ϕ 180 mm near support and @ 300 mm c/c at mid span. as shown in figure.



Shear reinforcement detailing

Solution : 2

Given: Beam cross-section = 230 × 450 mm; clear cover = 30 mm; Reinforcement = 3 – 20 mm ϕ Fe 415; Factored shear force, V_u = 110 kN; Grade of concrete = M20

$$\text{Effective cover} = 30 + \frac{20}{2} = 40 \text{ mm}$$

$$\therefore \quad \text{Effective depth} = 450 - 40 = 410 \text{ mm}$$

Nominal shear stress, $\tau_{v_u} = \frac{V_u}{bd} = \frac{110\times10^3}{230\times410}$

$$\tau_{v_u} = 1.166\ \text{N/mm}^2$$

For M20, maximum permissible shear stress, $\tau_{c,\,max} = 2.8\ \text{N/mm}^2\ \left(=0.62\sqrt{f_{ck}}\right)$

$$\tau_{v_u} < \tau_{c,\,max} \qquad \text{(OK)}$$

$$\text{Percentage of reinforcement, } p_t = \frac{3\times\frac{\pi}{4}\times20^2}{230\times410}\times100 = 1\%$$

$\therefore$ $\tau_c = 0.62\ \text{N/mm}^2$

Net shear force to be resisted by shear reinforcement, V_{us} is given by,

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

$\Rightarrow$ $V_{us} = 110 - \left(\frac{0.62\times230\times410}{1000}\right) = 51.534\ \text{kN}$

Using 2 legged – 8 mm ϕ stirrups,

$$S_v = \frac{A_{sv}\times0.87f_y\times d}{V_{us}} = \frac{\left(2\times\frac{\pi}{4}\times8^2\right)\times0.87\times415\times410}{51.534\times10^3}$$

$$= 288.77\ \text{mm, say 280 mm}$$

Maximum permissible spacing = Minimum of {$0.75d$, 300 mm}

= 300 mm

Check for minimum Reinforcement :

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

$\Rightarrow$ $\frac{\left(2\times\frac{\pi}{4}\times8^2\right)}{230\times S_v} = \frac{0.4}{0.87\times415} \Rightarrow S_v = 394.53\ \text{mm}$

$\therefore$ Provide 2L – 8 mm ϕ stirrups @ 280 mm c/c distance.

Solution : 3

Percentage of tensile reinforcement at support (p_t) $= \frac{4\times\frac{\pi}{4}\times25^2}{300\times550}\times100 = 1.19\%$

$\therefore$ For M20 concrete and 1.19% tensile reinforcement

$$\tau_c = 0.62 + \frac{(0.67-0.62)}{(1.25-1.0)}\times(1.19-1.0)$$

$$= 0.658\ \text{N/mm}^2$$

For M20 concrete, $\tau_{c,max}$ = 2.8 MPa

$$\tau_v = \frac{V_u}{bd} = \frac{250\times10^3}{300\times550}$$

$\Rightarrow \qquad \tau_v = 1.52$ MPa

$\therefore\ \tau_{c,max} > \tau_v > \tau_c;$

$\therefore$ Shear reinforcement is to be provided for

$$V_{us} = V_u - \tau_c bd$$
$$= 250 \times 1000 - 0.658 \times 300 \times 550$$
$$= 141430 \text{ N}$$

Shear resistance for a series of bent-up bars at different cross-section

$$V_{us_1} = \frac{0.87 f_y A_{sv_1} d}{s_v}(\sin\alpha + \cos\alpha)$$

$$= \frac{0.87 \times 250 \times 2 \times \frac{\pi}{4} \times 25^2 \times 550}{700}(\sin 45° + \cos 45°)$$

$$= 237267.79 \text{ N} > V_{us} (= 141430 \text{ N})$$

As the shear resistance of bent-up bars cannot exceed 0.5 × 141430 = 70715 N, vertical stirrups are to be provided for $V_{us_2} = (V_s/2)$

$$V_{us_2} = V_{us} - V_{us_1}$$
$$= 141430 - 70715 = 70715 \text{ N}$$

Providing 6 mm dia 2 legged vertical stirrups

$$A_{sv_2} = 2 \times \frac{\pi}{4} \times 6^2 = 56.55 \text{ mm}^2$$

$$\therefore \qquad s_{v_2} = \frac{0.87 f_y A_{sv_2} d}{V_{us_2}}$$

$$= \frac{0.87 \times 250 \times 56.55 \times 550}{70715} = 95.66 \text{ mm}$$

$$\therefore \qquad S_{v2} = 95.66 \text{ mm} < 300 \text{ mm} < 0.75d\ (= 412.5 \text{ mm})$$

From minimum reinforcement consideration the maximum spacing of vertical reinforcement

$$s_{v_{2,max}} = \frac{0.87 \times 250 \times 56.55}{0.4 \times 300}$$

$$= 102.49 \text{ mm} > 95.66 \text{ mm} \qquad \text{(OK)}$$

Hence, provide 6 mm dia. 2 legged vertical stirrups @ 95 mm c/c in addition to bent-up bars at different cross-sections to resist total shear force.

Solution : 4

Safe in bond.

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3 Beams, Slabs, Columns and Footings

LEVEL 1 Objective Questions

1.	(a)
2.	(a)
3.	(a)
4.	(b)
5.	(b)
6.	(b)
7.	(a)
8.	(a)
9.	(a)
10.	(a)
11.	(c)
12.	(28)

LEVEL 2 Objective Questions

13.	(d)
14.	(b)
15.	(c)
16.	(d)
17.	(d)
18.	(d)
19.	(a)
20.	(1309.73)
21.	(5)
22.	(0.20)
23.	(3111)
24.	(202.54)
25.	(1.06)
26.	(0.82)
27.	(1.84)

LEVEL 3 Conventional Questions

Solution : 1

(a) M_u = 4354.3 kN-m.

(b) 52.92 kN/m

(c) $\because$ w_u = 107.47 kN/m

$$V_u = \frac{w_u l}{2} = \frac{107.51 \times 18}{2} = 967.59 \text{ kN}$$

$\therefore$ Nominal shear stress
$$\tau_v = \frac{V_u}{Bd} = \frac{967.59 \times 10^3}{500 \times 1400} = 1.38 \text{ N/mm}^2$$

Now
$$p = 100 \times \frac{A_{st}}{Bd} = \frac{100 \times 10308.35}{500 \times 1400} = 1.47\%$$

For p = 1.25%, we have τ_c = 0.70 N/mm²

For p = 1.50%, we have τ_c = 0.74 N/mm²

$\therefore$
$$\tau_c \text{ for } 1.47\% = 0.70 + \frac{(0.74 - 0.70)}{(1.50 - 1.25)}(1.47 - 1.25) = 0.70 + 0.0352$$

$$\tau_c = 0.7352 \text{ N/mm}^2$$

$\therefore$ $V_c = \tau_c Bd = 0.7352 \times 500 \times 1400 = 514.64$ kN

$\because$ $\tau_v > \tau_c$, hence shear reinforcement is required.

$$V_{us} = V_u - V_c = 967.59 - 514.64 = 452.95 \text{ kN}$$

Now shear reinforcement at supports should be provided according to **IS:456 – 2000** i.e.

(d) Spacing,
$$S_v = \frac{0.87 f_y A_{sv} d}{V_{us}}$$

Adopting 2-legged 10 mm ϕ bars for vertical stirrups.

$$A_{sv} = 2 \times \frac{\pi}{4} \times 10^2 = 157.08 \text{ mm}^2$$

$\therefore$
$$S_v = \frac{0.87 \times 415 \times 157.08 \times 1400}{452.95 \times 10^3} = 175.3 \text{ mm c/c}$$

Hence, provide 170 mm spacing at supports.

Minimum shear reinforcement,

$$S_v \le \frac{2.5 A_{sv} f_y}{b} \le \frac{2.5 \times 157.08 \times 415}{500} \le 325.94 \text{ mm} \quad [\because b = B]$$

But $S_v \le 300$ mm

Hence, at mid span, spacing between stirrups can be 300 mm c/c.

Maximum spacing = $0.75d$ = 0.75 × 1400 = 1050 mm

Hence provided reinforcement is OK.

Solution : 2

324.14 mm^2

Solution : 3

Given: l = 3.0 m; L.L. = 10.6 kN/m; M15 and Fe415 used

As the beam is a cantilever, approximate depth based on deflection, $d = \frac{l}{7} = \frac{3000}{7} = 428.57$ mm

Let us assume effective depth d = 450 mm and effective cover of 50 mm.

∴ Total depth D = 500 m

Assuming width of section, $B = \frac{D}{2} = 250$ mm

$$\text{D.L.} = 0.500 \times 0.250 \times 25 = 3.125 \text{ kN/m}$$

$$\text{L.L.} = 10.6 \text{ kN/m}$$

Total factored load $w_u = 1.5\,(3.125 + 10.6)$

$$= 20.5875 \text{ kN/m}$$

Maximum bending moment $M_u = \frac{w_u l^2}{2}$

$$M_u = \frac{20.5875 \times 3^2}{2} = 92.64 \text{ kN-m}$$

Check for depth: $d = \sqrt{\frac{M_u}{QB}} = \sqrt{\frac{92.64 \times 10^6}{0.138 \times 15 \times 250}} = 423.101$ mm

d = 423.101 mm < 450 mm ⇒ Hence safe.

With d = 450 mm, $M_{u,\text{lim}} = 0.138\, f_{ck}\, bd^2$

$$M_{u,\,\text{lim}} = 0.138 \times 15 \times 250 \times \frac{450^2}{10^6} = 104.79 \text{ kN-m} > 92.64 \text{ kNm}$$

∴ Section is under-reinforced.

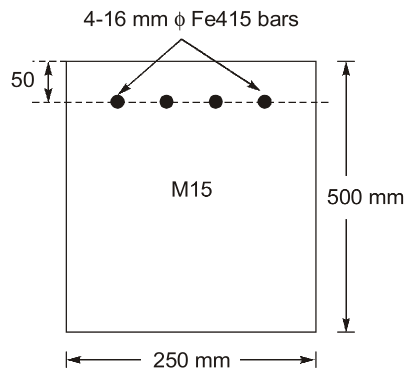
Area of steel, $A_{st} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_u}{f_{ck}.b.d^2}}\right] bd$

$$A_{st} = \frac{0.5 \times 15}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 92.64 \times 10^6}{15 \times 250 \times 450^2}}\right] 250 \times 450$$

$$= 686.31 \text{ mm}^2$$

Number of 16 mm ϕ bars $= \frac{686.31}{\frac{\pi}{4} \times 12^2} = 3.4$ say 4

Provide 416 mm ϕ bars at top of the section as it is a cantilever with effective cover of 50 mm.



Solution : 4

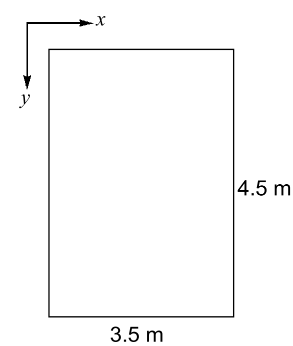
$M_{u\,lim}$ = 691.73 kNm, P_t% = 1.2916%

Solution : 5

Safe concenrtated load = 27.41 kN

and Safe UDL = 13.4 kN/m

Solution : 6



$$l_{x0} = 3.5 \text{ m and } l_{y0} = 4.5 \text{ m}$$

$$\text{Span ratio} = \frac{l_{y0}}{l_{x0}} = \frac{4.5}{3.5} = 1.29$$

Approximate depth based on deflection criteria.

$$D = \frac{l_{x0}}{32} = \frac{3500}{32} = 109.375 \text{ mm}$$

$$\text{Provide } D = 120 \text{ mm}$$

$$\text{Effective cover} = 20 \text{ mm}$$

$$\therefore \quad \text{Effective depth, } d = 120 - 20 = 100 \text{ mm}$$

$$\text{Assuming width of support, } w = 400 \text{ mm}$$

Effective length:

$$l_x = \left.\begin{matrix} l_{x0} + d \\ l_{x0} + w \end{matrix}\right\} \text{minimum of two}$$

$$l_x = \left.\begin{matrix} 3.5 + 0.1 \\ 3.5 + 0.4 \end{matrix}\right\} \text{minimum}$$

$$\therefore \quad l_x = 3.6 \text{ m}$$

Similarly,

$$l_y = 4.6 \text{ m}$$

$$\therefore \quad r = \frac{l_y}{l_x} = \frac{4.6}{3.6} = 1.28$$

Coefficients for $\frac{l_y}{l_x} = 1.28$ by interpolation:

$$\alpha_x^{(+)} = 0.045 + \frac{0.004}{0.1} \times 0.08 = 0.048$$

$$\alpha_x^{(-)} = 0.064 \quad \text{and} \quad \alpha_y = 0.047$$

D.L. $w_d = 25 \times 1 \times 0.12 = 3$ kN/m

Total factored load, $w_u = 1.5\,(3 + 2 + 1) = 9$ kN/m

Moment calculation: $M_x^{(+)} = \alpha_x^{(+)} \cdot w_u \cdot l_x^2 = 0.048 \times 9 \times 3.6^2 = 5.60$ kNm

$$M_x^{(-)} = \alpha_x^{(-)} \cdot w_u \cdot l_x^2 = 0.064 \times 9 \times 3.6^2 = 7.46 \text{ kN-m}$$

$$M_y = \alpha_y \cdot w_u \cdot l_x^2 = 0.047 \times 9 \times 3.6^2 = 5.48 \text{ kN-m}$$

Depth required: $d = \sqrt{\dfrac{M_u}{0.138\, f_{ck}.b}} = \sqrt{\dfrac{7.46 \times 10^6}{0.138 \times 20 \times 1000}} = 51.98 \text{ mm} < 100 \text{ mm}$

Hence OK

Reinforcement calculation: $A_{st} = \dfrac{0.5\, f_{ck}}{f_y}\left[1 - \sqrt{1 - \dfrac{4.6\, M_u}{f_{ck}.b.d^2}}\right] bd$

$$A_{st_x}^{(+)} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 5.6 \times 10^6}{20 \times 10^3 \times 100^2}}\right] 1000 \times 100$$

$$= 160.53 \text{ mm}^2$$

$$A_{st_x}^{(-)} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 7.46 \times 10^6}{20 \times 10^3 \times 100^2}}\right] 1000 \times 100$$

$$= 216.44 \text{ mm}^2$$

$$A_{st_y} = \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 5.48 \times 10^6}{20 \times 10^3 \times 100^2}}\right] 1000 \times 100$$

$$= 156.97 \text{ mm}^2$$

Spacing for $A_{st_x}^{(+)}$: $S_v = \dfrac{1000}{A_{st}} \times \dfrac{\pi}{4} \times \phi^2 = \dfrac{1000}{160.53} \times \dfrac{\pi}{4} \times 10^2$

$$= 489.25 \text{ mm}$$

Maximum permissible spacing $= 3d$, but < 300 mm

∴ Provide 10 mm – ϕ bars at 300 mm c/c distance

Spacing for $A_{st_x}^{(-)}$: $S_v = \dfrac{1000}{216.44} \times \dfrac{\pi}{4} \times 10^2 = 362.87 \text{ mm} > 300 \text{ mm}$

∴ Provide 10 mm – ϕ bars at 300 mm c/c distance

Spacing for A_{st_y}: $S_v = \dfrac{1000}{156.97} \times \dfrac{\pi}{4} \times 10^2 = 500.35 > 300 \text{ mm}$

∴ Provide 10 mm – ϕ bars at 300 mm c/c distance

Check for shear: $V_{u_x} = \dfrac{w_u l_x}{3} = \dfrac{9 \times 3.6}{3} = 10.8$ kN

$$V_{u_y} = w_u l_x\left(\frac{r}{2+r}\right) = 9\times 3.6\times\left(\frac{1.28}{2+1.28}\right) = 12.64\text{ kN}$$

$$\tau_{Vu} = \frac{V_{uy}}{bd} = \frac{12.64\times 10^3}{1000\times 100} = 0.1264\text{ N/mm}^2 < 0.40\text{ N/mm}^2 \qquad \text{(OK)}$$

Check for development length:

For A_{st_y} with 50% curtailment at support:

$$M_{u_1} = 0.87 f_y\cdot\frac{A_{st_y}}{2}\cdot j\cdot d$$

$$= 0.87\times 415\times\frac{156.97}{2}\times 0.8\times 100/10^6$$

$$= 2.27\text{ kN-m}$$

$$V_{u_x} = 10.8\text{ kN}$$

$$L_d = \frac{\phi\times 0.87 f_y}{4\tau_{bd}} = \frac{10\times 0.87\times 415}{4\times 1.2\times 1.6} = 470.12\text{ mm}$$

$$1.3\frac{M_{u_1}}{V_{u_x}} + L_0 = 1.3\times\frac{2.27\times 10^3}{10.8} + \frac{400}{2} - 30$$

$$= 273.24 + 170 = 443.24\text{ mm} < L_d$$

Hence it is not safe

Let us increase depth, $d = 120$ mm

$$1.3\frac{M_{u_1}}{V_{u_x}} + L_0 = 327.89 + 170 = 497.89\text{ mm} > L_d$$

Hence it is safe.

For $A_{st_x}^{(+)}$ with 50% curtailment at support

$$M_{u_1} = 0.87 f_y\cdot\frac{A_{st}^{(+)}}{2}x\cdot j\cdot d = 0.87\times 415\times\frac{160.53}{2}\times 0.8\times\frac{120}{10^6}$$

$$= 2.78\text{ kN-m}$$

$$V_{u_y} = 12.64\text{ kN}$$

$$1.3\frac{M_{u_1}}{V_{u_y}} + L_0 = \frac{1.3\times 2.78\times 10^3}{12.64} + \frac{400}{2} - 30 = 456.13\text{ mm} < L_d$$

Hence it is not safe

Let us increase depth, $d = 130$ mm

$$1.3\frac{M_{u_1}}{V_{u_y}} + L_0 = 479.74 > L_d$$

Hence it is safe.

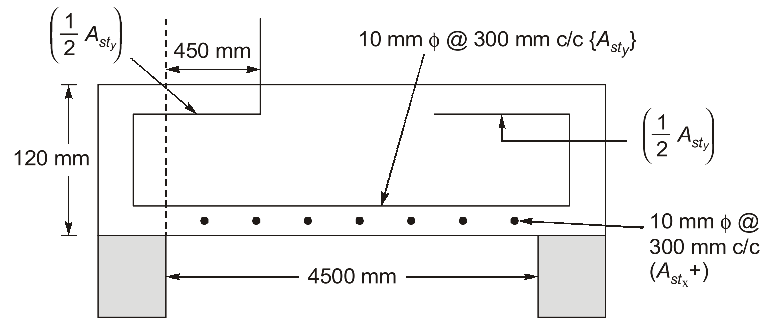
Torsion Reinforcement:

$$\text{Size of mesh} = \frac{l_x}{5} = \frac{3600}{5} = 720\text{ mm}$$

$$\text{Area of steel} = \frac{3}{4}A_{st_x}^{(+)} = \frac{3}{4} \times 160.53 = 120.40 \text{ mm}^2$$

$$\text{Spacing of bars} = \frac{720}{120.40} \times \frac{\pi}{4} \times 10^2 = 469.67 \text{ mm}$$

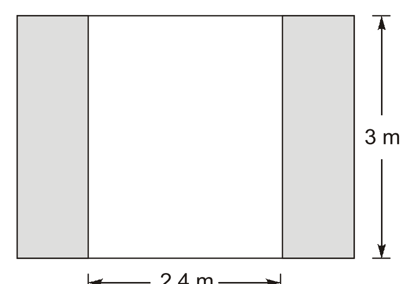
∴ Provide 10 ϕ bars at a spacing of 300 mm.



Solution : 7

The give slab is a one-way simply supported slab.

Approximate depth based on deflection criteria



$$d = \frac{3000}{20} = 150 \text{ mm}$$

Let us assume $d = 160$ mm

$$D = 160 + 30 = 190 \text{ mm}$$

$$L_{\text{eff.}} = \left.\begin{array}{r}\text{c/c distance between supports} = 2.4 + 0.3 \\ \text{clear span + d} = 2.4 + 0.16\end{array}\right\} \text{minimum}$$

∴ $L_{\text{eff.}} = 2.56$ m

$$\text{Dead load} = 25 \times 0.19 \times 1 = 4.75 \text{ kN/m}$$

$$\text{Total load} = 30 + 4.75 = 34.75 \text{ kN/m}$$

$$\text{Maximum bending moment} = \frac{wl^2}{8} = \frac{34.75 \times 2.56^2}{8}$$

$$\text{B.M} = 28.47 \text{ kN-m}$$

For M20 : $m = 13.33$, $\sigma_{cbc} = 7 \text{ N/mm}^2 = c$

For Fe415 : $\sigma_{st} = 230 \text{ N/mm}^2$

$$k = \frac{mc}{t + mc} = \frac{13.33 \times 7}{230 + 13.33 \times 7} = 0.29$$

$$j = 1 - \frac{k}{3} = 0.903$$

$$Q = \frac{1}{2}c \cdot j \cdot k = \frac{1}{2} \times 7 \times 0.903 \times 0.29 = 0.9165$$

Depth of slab required, $d = \sqrt{\frac{\text{B.M.}}{QB}}$ where B.M. = Bending moment

$$d = \sqrt{\frac{28.47 \times 10^6}{0.9165 \times 1000}} = 176.25 \text{ mm}$$

∴ Let us provide $d = 180$ mm

$D = 180 + 30 = 210$ mm (Assume effective cover of 30 mm)

$\text{D.L.} = 0.21 \times 1 \times 25 = 5.25$ kN/m

l_{eff} = clear span + d = 2.4 + 0.18 = 2.58 m

$$\text{B.M.} = \frac{35.25 \times 2.58^2}{8} = 29.33 \text{ kN-m}$$

Depth check: $d = \sqrt{\frac{BM}{QB}} = \sqrt{\frac{29.33 \times 10^6}{0.9165 \times 1000}} = 178.89 \text{ mm} < 180$

Hence OK

Main Reinforcement along l_x: $A_{st} = \frac{BM}{\sigma_{st} \cdot j \cdot d} = \frac{29.33 \times 10^6}{230 \times 0.903 \times 180} = 784.56 \text{ mm}^2$

Using 16 mm ϕ bars Spacing $= \frac{1000}{A_{st}} \times \frac{\pi}{4} \times \phi^2 = \frac{1000}{784.56} \times \frac{\pi}{4} \times 16^2$

$= 256.27$ mm

Let us provide 16 mm ϕ bars at 250 mm c/c

Distribution reinforcement along l_y: $A_{st} = \frac{0.12}{100} \times BD = \frac{0.12}{100} \times 1000 \times 210 = 252 \text{ mm}^2$

$$\text{Spacing} = \frac{1000}{252} \times \frac{\pi}{4} \times 16^2 = 797.86 \text{ mm}$$

Maximum permissible spacing = minimum {5d, 450 mm}

∴ Provide 16 mm-ϕ bars at 450 mm c/c

Shear check: $V = \frac{wl_x}{2} = 45.4725$ kN

$$\tau_v = \frac{V}{Bd} = \frac{45.4725}{1000 \times 180} \times 10^3 = 0.2526 \text{ N/mm}^2$$

$$p_t(\%) = \frac{A_{st}}{Bd} \times 100 = \frac{784.56}{1000 \times 180} \times 100 = 0.44\%$$

Corresponding $\tau_c = 0.2872 \text{ N/mm}^2$

Also $\tau_{c_{max}} = 1.8 \text{ N/mm}^2$

∴ $\tau_v < \tau_c < \tau_{c_{max}}$

Hence safe in shear

Check for development length:

$$L_d = \frac{\phi \times \sigma_{st}}{4\tau_{bd(per)}} = \frac{16 \times 230}{4 \times 0.8 \times 1.6} = 718.75 \text{ mm}$$

$$M_1 = \sigma_{st} \times \frac{A_{st}}{2} \times j \times d = 230 \times \frac{784.56}{2} \times 0.903 \times \frac{180}{10^6}$$

$$M_1 = 14.66 \text{ kN-m}$$

$$V = 45.4725 \text{ kN}$$

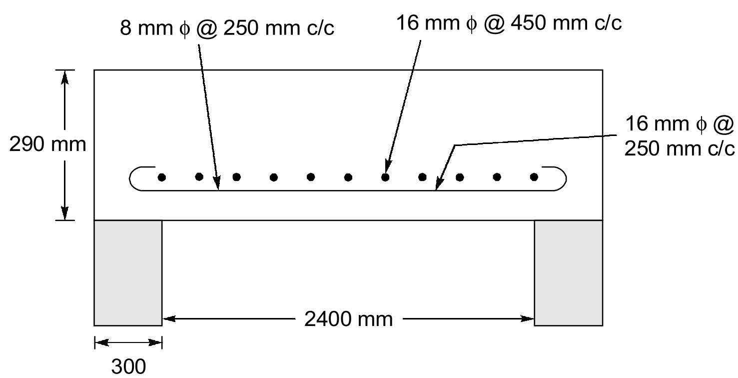
$$1.3\frac{M_1}{V} + L_0 = 1.3 \times \frac{14.66 \times 10^3}{45.4725} + \frac{300}{2} - 30 = 539.26 \text{ mm} < L_d$$

Hence it is not safe.

Let us increase depth, $d = 260$ mm

$$1.3\frac{M_1}{V} + L_0 = 605.59 + 120 = 725.59 \text{ mm} > L_d$$

Hence OK



Solution : 8

Effective length, $$L_{eff} = L_0 + \frac{W}{2} = 1500 + \frac{400}{2} = 1700 \text{ mm}$$

Factored bending moment at support,

$$M_u = 1.5\,(10 + 12) = 33 \text{ kN-m}$$

Factored shear force, $$V_u = 1.5 \times 15 = 22.5 \text{ kN}$$

Depth required, $$d = \sqrt{\frac{M_u}{QB}} = \sqrt{\frac{33 \times 10^6}{0.138 f_{ck} \times 1000}}$$

$$= \sqrt{\frac{33 \times 10^6}{0.138 \times 30 \times 1000}} = 89.28 \text{ mm}$$

Provide $d = 90$ mm

Using effective cover $= 30$ mm

∴ $D = 90 + 30 = 120$ mm

Area of steel requirement:

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

$$= \frac{0.5\times 30}{415}\left[1-\sqrt{1-\frac{4.6\times 33\times 10^6}{30\times 1000\times 90^2}}\right]1000\times 90$$

$$= 1260.14 \text{ mm}^2$$

$$\text{Use 12 mm} - \phi \text{ bars, spacing} = \frac{1000}{1260.14}\times\frac{\pi}{4}\times 12^2$$

$\Rightarrow$ $$S = 89.75 \text{ mm}$$

$\therefore$ Provide 12 mm – ϕ bars with 80 mm c/c distance along x-direction.

Distribution bars:

$$A_{st_y} = \frac{0.12}{100}BD = \frac{0.12}{100}\times 1000\times 120 = 144 \text{ mm}^2$$

$$\text{Use 12 mm-}\phi \text{ bars at spacing} = \frac{1000}{144}\times\frac{\pi}{4}\times 12^2$$

$$S = 785.39 \text{ mm}$$

$$\text{Maximum permissible spacing} = 5d, \text{ but} < 450 \text{ mm}$$

$$= 450 \text{ mm}$$

$\therefore$ Provide 12 mm – ϕ bars at 450 mm c/c distance along y-direction.

Check for shear:

$$\tau_{vu} = \frac{V_u}{Bd} = \frac{22.5\times 10^3}{1000\times 90} = 0.25 \text{ N/mm}^2$$

$$\tau_{c,\,max} = 3.5 \text{ N/mm}^2$$

$$p_t(\%) = \frac{1260.14}{1000\times 90}\times 100 = 1.4\ \%$$

$$\tau_c = 0.75 \text{ N/mm}^2$$

$\therefore$ $$\tau_{vu} < \tau_c < \tau_{c_{max}}$$

Hence safe in share

Check for development length:

$$L_0 = \frac{400}{2} - 30 = 170 \text{ mm}$$

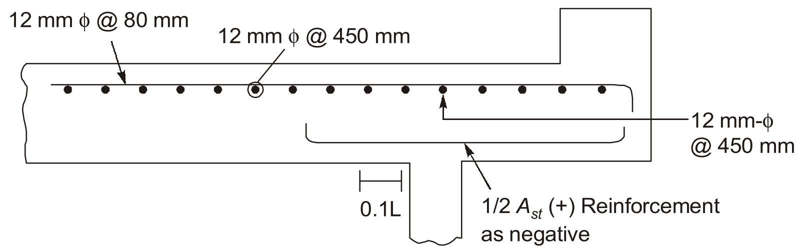
$$M_1 = 0.87f_y\times\frac{A_{st_x}}{2}\times jd = 0.87\times 415\times\frac{1260.14}{2}\times\frac{0.8\times 90}{10^6}$$

$$= 16.38 \text{ kN-m}$$

$$V_u = 22.5 \text{ kN}$$

$$L_d = \frac{\phi\times 0.87f_y}{4\tau_{bd}} = \frac{12\times 0.87\times 415}{4\times 1.6\times 1.6} = 423.105 \text{ mm}$$

$$1.3\frac{M_1}{V_u} + L_0 = 1.3\times\frac{16.38\times 10^6}{22.5\times 10^3} + 170 = 1116.4 \text{ mm} > L_d \quad \text{(OK)}$$



Solution : 9

Given: $P = 1500$ kN, $P_u = 1.5 \times 1500 = 2250$ kN, Circular dia. = 450 mm, $L_{eff} = 3.5$ m

Check for slenderness ratio $\quad \dfrac{L_{eff}}{D} = \dfrac{3500}{450} = 7.78 < 12$

Therefore, column can be designed as short column

Check for minimum eccentricity $\quad e_{min} = \dfrac{L_{eff}}{500} + \dfrac{D}{30}$ or 20 mm whichever is greater

$$e_{min} = \frac{3500}{450} + \frac{450}{30} = 22.8 \text{ mm}$$

As, $\quad 0.05\,D = 22.5$ mm

$$\approx e_{min}$$

∴ Codal formula for axially compressed short column may be used.

$$P = 1.05\,(0.4\,f_{ck}\,A_c + 0.67\,f_y A_{sc})$$

Area of concrete $\quad A_c = \dfrac{\pi}{4} \times 450^2 - A_{sc}$

A_{sc} = Area of steel

$$\therefore \quad 2250 \times 10^3 = 1.05\left(0.4 \times 25 \times \left(\frac{\pi}{4} \times 450^2 - A_{sc}\right) + 0.67 \times 415 A_{sc}\right)$$

$$\Rightarrow \quad A_{sc} = 2060.9 \text{ mm}^2$$

$$A_{sc,min} \text{ at } 0.8\% \text{ of } A_g = \frac{0.8}{100} \times \frac{\pi}{4} \times 450^2 = 1272.35 \text{ mm}^2$$

Provide 8 bars of 20 mm φ

$$\therefore \quad A_{sc} \text{ provided} = 8 \times \frac{\pi}{4} \times 20^2 = 2513.27 \text{ mm}^2 > 2060.9 \text{ mm}^2 \text{ (OK)}$$

Design of spiral reinforcement

Assuming a clear cover of 40 mm over spirals,

Core diameter = 450 – (40 × 2) = 370 mm

Assuming a spiral bar diameter of 8 mm and pitch S_t

$$p_s = \frac{\text{Volume of spiral reinforcement}}{\text{Volume of core}} \text{ per unit length of column}$$

$$= \frac{\left(\frac{\pi}{4} \times 8^2\right) \times \pi \times \frac{(370-6)}{S_t}}{\left(\frac{\pi}{4} \times 370^2\right)} = \frac{0.5317}{S_t}$$

As per code: $$p_s \geq 0.36\left[\frac{A_g}{A_{core}} - 1\right]\left[\frac{f_{ck}}{f_y}\right]$$

$$\Rightarrow \quad \frac{0.5317}{S_t} \geq 0.36\left[\frac{\frac{\pi}{4} \times 450^2}{\frac{\pi}{4} \times 370^2} - 1\right]\left[\frac{25}{415}\right]$$

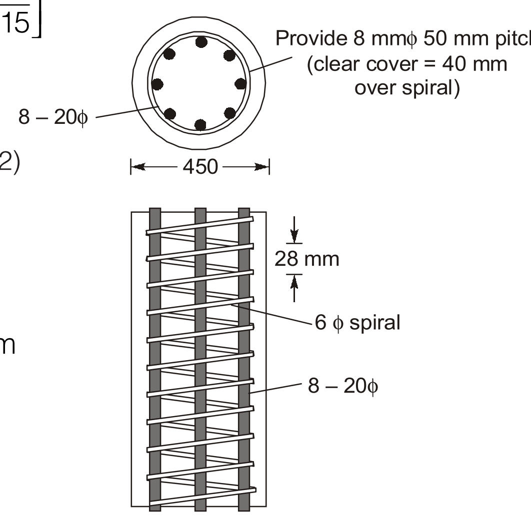
$$\Rightarrow \quad \frac{0.5317}{S_t} \geq 0.36 \times (0.479) \times (0.0602)$$

$$\Rightarrow \quad S_t \leq 51 \text{ mm}$$

As per code $$S_t < \begin{cases} 75 \text{ mm} \\ \frac{\text{Core dia.}}{6} = 61.67 \text{ mm} \end{cases}$$

and $$S_t > \begin{cases} 25 \text{ mm} \\ 3\phi_h = 24 \text{ mm} \end{cases}$$

Provide 8ϕ spiral at 50 mm c/c pitch.



Solution : 10

Given: M20 | Fe415; Unsupported length l = 3 m; P_u = 1600 kN

Let us assume effective length = 3 m

Let us design a square short column of '$B \times B$' mm

For short column, $$\frac{l_{eff}}{B} \leq 12$$

$$\frac{3000}{B} \leq 12$$

$$\Rightarrow \quad B \geq 250 \text{ mm}$$

Let us assume B = 420 mm

(i) $$\frac{l_{eff}}{B} = \frac{3000}{420} = 7.14 < 12$$

(ii) Minimum eccentricity, $$e_{min} = \left.\begin{matrix} \frac{l}{500} + \frac{B}{30} \\ \text{or } 20 \text{ mm} \end{matrix}\right\} \text{maximum}$$

$$\therefore \quad e_{min} = \left.\begin{matrix} \frac{3000}{500} + \frac{420}{30} \\ \text{or } 20 \text{ mm} \end{matrix}\right\} \text{maximum} \Rightarrow e_{min} = 20 \text{ mm}$$

(iii) $$0.05B = 0.05 \times 420 = 21 \text{ mm} > e_{min}$$

$$\therefore \quad e_{min} < 0.05\,B$$

Hence,
$$P_u = 0.4\,f_{ck}\,A_c + 0.67\,f_y\,A_{sc}$$
$$1600 \times 10^3 = 0.4 \times 20 \times (420^2 - A_{sc}) + 0.67 \times 415 \times A_{sc}$$
$$\Rightarrow \quad A_{sc} = 699.13\ \text{mm}^2$$

Minimum percentage of reinforcement = 0.8%

$$A_{sc,\,min} = 0.8 \times \frac{420^2}{100} = 1411.2\ \text{mm}^2 > A_{sc}$$

$$\therefore \quad \text{Provide } A_{sc,\,min} = 1411.2\ \text{mm}^2$$

Let us provide 4 – 20 mm ϕ and 2 – 12 mm ϕ bars

$$4 \times \frac{\pi}{4} \times 20^2 + 2 \times \frac{\pi}{4} \times 12^2 = 1482.83\ \text{mm}^2 > A_{sc,\,min}$$

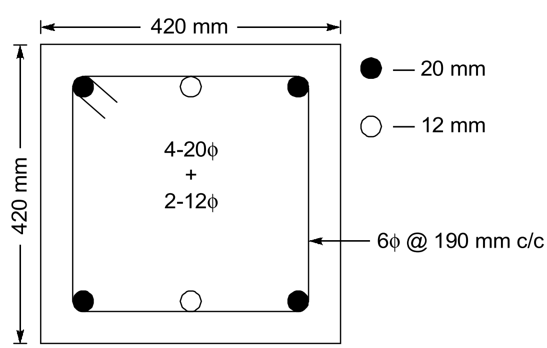
Laterial ties :

Tie diameter,
$$\phi_t = \left.\begin{matrix} \dfrac{\phi_{d,\,max}}{4} \\ 6\ \text{mm} \end{matrix}\right\}\ \text{maximum} = 6\ \text{mm}$$

$$\Rightarrow \quad \phi_t = \left.\begin{matrix} \dfrac{20}{4}\text{mm} \\ 6\ \text{mm} \end{matrix}\right\}\ \text{maximum} = 6\ \text{mm}$$

$$\text{Spacing} = \text{minimum}\ \{420,\ 16\,\phi_{d,\,min},\ 300\ \text{mm}\}$$
$$\Rightarrow \quad \text{Spacing} = \{420,\ 16 \times 12,\ 300\ \text{mm}\}\text{minimum} = 192\ \text{mm}$$

$\therefore$ Use 6 mm ϕ_t @ spacing 190 mm c/c



Solution : 11

Size of footing:

Given, P = 1400 kN, q_0 = 100 kN/m^2 at 1 m depth

Assuming the weight of footing + backfill to be 10% of load P.

$$\therefore \quad \text{Footing area required} = \frac{1400 \times 1.1}{100} = 15.40\ \text{m}^2$$

$$\text{Minimum size of footing} = \sqrt{15.40} = 3.92\ \text{m}$$

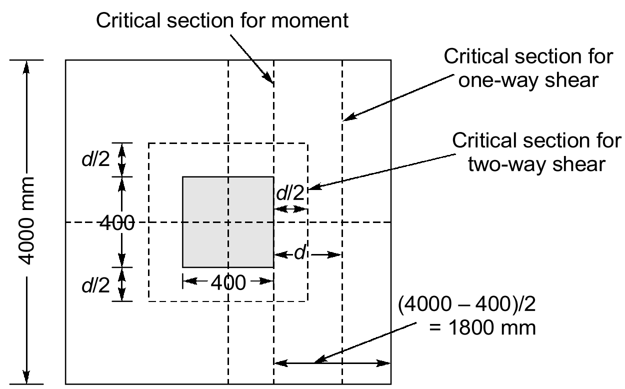
Assume a 4 m × 4 m footing size.

Thickness of footing slab based on shear:

Net soil pressure at ultimate loads (Load factor = 1.5)

$$\therefore \qquad q_u = \frac{1400 \times 1.50}{4.0 \times 4.0} = 131.25 \text{ kN/m}^2 = 0.131 \text{N/mm}^2$$

(a) One-way shear:



Plan

The critical section for one way shear is at distance d from column face.

$\therefore$ Factored shear force, $V_{u1} = 0.131 \times 4000\,(1800 - d)$

$= 943200 - 524d$

Given, $\tau_c = 0.35$ MPa

$\therefore$ One way shear resistance, $V_{c1} = 0.35 \times 4000 \times d = 1400\,d$

$\therefore \quad V_{u1} \le V_{c1}$

$\Rightarrow \quad 943200 - 524d \le 1400\,d$

$\Rightarrow \quad d \ge 490.23 \text{ mm} \simeq 491 \text{ mm}$

(b) Two way shear:

The critical section for two way shear is act $d/2$ from column periphery

$\therefore$ Factored shear force, $V_{u2} = 0.131[(4000)^2 - (400 + d)^2]$

$\Rightarrow \quad V_{u2} = 0.131\,[(4000)^2 - (400 + 491)^2]$

$\Rightarrow \quad = 1992 \times 10^3$ N

Two way shear resistance, $V_{c2} = k\tau_c\,[4 \times (400 + d)]d \quad k = 1$ (for square column)

$\tau_c = 0.25\sqrt{f_{ck}} = 0.25\sqrt{25} = 1.25 \text{ N/mm}^2$

$\therefore \quad V_{c2} = 1.25 \times 1 \times 4d(400 + d)$

$\Rightarrow \quad V_{c2} = (2000d + 5d^2)$N

$\therefore \quad V_{u2} \le V_{c2}$

$\Rightarrow \quad 1992 \times 10^3 \le 2000d + 5d^2$

$\Rightarrow \quad d \ge 462.12$ mm

Evidently, one way shear governs the thickness. Assuming clear cover of 75 mm and 16 ϕ bars in both directions with an average $d = 491$ mm.

$\therefore$ Total thickness, $D = (491 + 75 + 16/2) = 574 \text{ mm} \simeq 590$ mm

Now, $d_{eff} = 590 - 75 - 16/2 = 507$ mm

Assuming unit weight of concrete = 24 kN/m^3

Given: Unit weight of soil = 20 kN/m^3

$$\therefore \quad q = \frac{1400}{4 \times 4} + 24 \times 0.59 + 20 \times (1 - 0.59) = 109.86 > 100 \text{ kN/m}^2 \text{ (unsafe)}$$

∴ Increase footing size to say 4.25 m × 4.25 m

$$\therefore \quad q = \frac{1400}{4.25 \times 4.25} + 24 \times 0.59 + 20(1 - 0.59)$$

$$= 99.87 \text{ kN/m}^2 < 100 \text{ kN/m}^2 \quad \text{(OK)}$$

Design for flexural reinforcement:

$$M_u = \frac{0.131 \times 4250 \times (1925)^2}{2} = 1031.55 \times 10^6 \text{Nmm}$$

$$\therefore \quad R_u = \frac{M_u}{Bd^2} = \frac{1031.55 \times 10^6}{4250 \times (507)^2} = 0.944 \text{ MPa}$$

$$\therefore \quad \frac{A_{st}}{Bd} = \frac{f_{ck}}{2f_y}\left\{1 - \sqrt{1 - \frac{4.598 R_u}{f_{ck}}}\right\} Bd$$

$$\Rightarrow \quad (A_{st})_{\text{required}} = \frac{25}{2 \times 415}\left[\left\{1 - \sqrt{1 - \frac{4.589 \times 0.944}{25}}\right\}\right] \times 4250 \times 507$$

$$\Rightarrow \quad (A_{st})_{req} = 5902.57 \text{ mm}^2$$

$$(A_{st})_{min} = 0.0012\, BD = 0.0012 \times 4250 \times 590 = 3009 \text{ mm}^2$$

∴ Provide, $A_{st} = 5902.57 \text{ mm}^2$

Using 16 ϕ bars, number of bars required $= \dfrac{5902.57}{\frac{\pi}{4} \times 16^2} = 29.4 \simeq 30$

Corresponding spacing, $s = \dfrac{\{4250 - 75 \times 2 - 30 \times 16\}}{(30 - 1)} = 124.83 \text{ mm} \simeq 120 \text{ mm}$

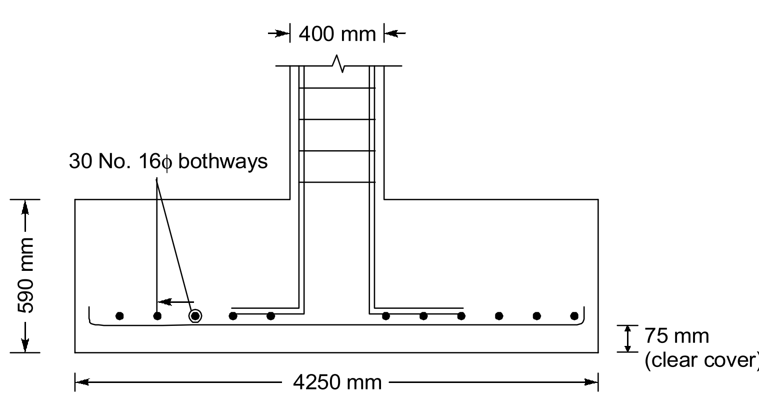
Provide 16 mm 30 bars both ways.

Check for development length.

$$L_d = \frac{\phi f_{st}}{4\tau_{bd}} = \frac{0.87 \times 415}{4 \times 1.2 \times 1.6} = 47\phi$$

For 16 ϕ bars in footing, $L_d = 47 \times 16 = 752$ mm

Length available = (1925 – 75) = 1850 mm > 752 mm. (OK)



Solution : 12

Given: Column : 400 mm × 400 mm; P = 200 kN {Inclusive of self weight}; M = 50 kN-m; q_0 = 100 kN/m^2

Let us design isolated unsymmetrical footing

$$\text{Area of footing required, } A = \frac{P}{q_0} = \frac{200}{100} = 2 \text{ m}^2$$

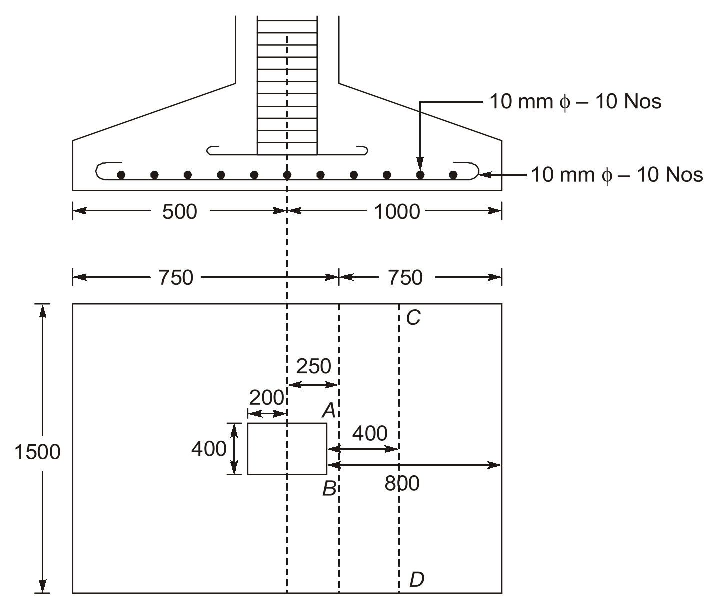
Let dimension be $B \times B$,

$$\therefore \quad B = \sqrt{2} = 1.44 \text{ m}$$

Let us take B = 1.5 m

$$\text{eccentricity} \quad e = \frac{M}{P} = \frac{50}{200} \times 10^3 = 250 \text{ mm}$$

Hence place C.G. of footing 250 mm away from the axis of column as shown in fig.



$$W_0 = \frac{P}{B^2} = \frac{200}{1.5^2} = 88.89 \text{ kN/m}^2$$

$$\text{Cantilever length to the right of } AB = \frac{1}{2}[1500 - 400] + 250 = 800 \text{ mm}$$

$$\text{Bending moment at the face } AB = W_0 \cdot B \cdot \frac{(0.8)^2}{2}$$

$$M = 88.89 \times 1.5 \times \frac{0.8^2}{2} = 42.67 \text{ kN-m}$$

The section of footing at AB will be trapezoidal. However its moment of resistance can be approximately determined by considering is to be a rectangle of width = $b + \frac{1}{8}(B - b)$

$$= 400 + \frac{1}{8}(1500 - 400) = 537.5 \text{ mm}$$

$$\therefore \quad d = \sqrt{\frac{M}{QB}} = \sqrt{\frac{42.67 \times 10^6}{0.914 \times 537.5}} = 294.7 \text{ mm}$$

Let d = 400 mm and effective cover = 75 mm

$$D = 475 \text{ mm}$$

Check for shear:

1. For two way shear : Section is situated at distance $d/2$ from column face all around.

$$b_0 = b + d = 400 + 400 = 800 \text{ mm}$$

$$\text{Punching shear force} = F = W_0\,[B^2 - b_0^2]$$

$$F = 88.89\,[1.5^2 - 0.8^2] = 143.11 \text{ kN}$$

$$\tau_v = \frac{F}{4b_0 d_0} \quad \Rightarrow \quad d_0 = \frac{F}{4b_0 \tau_v}$$

$$\tau_{per} = k_B \times 0.16 \times \sqrt{f_{ck}}$$

$$k_B = \left(0.5 + \frac{b}{a}\right) \not> 1 \quad \Rightarrow \quad k_B = 1$$

$$\tau_{per} = 0.16 \times \sqrt{20} = 0.715 \text{ N/mm}^2$$

$$d_0 = \frac{143.11 \times 10^3}{4 \times 800 \times 0.715} = 62.55 \text{ mm}$$

If an effective depth of 140 mm is provided at outer edge, and if effective depth d = 400 mm is kept at face AB, the available effective depth at $d/2$ from AB will be $= 140 + \frac{400 - 140}{800} \times (800 - 200) = 335$ mm

It is more than the required depth d_0 = 62.55 mm

2. For one way shear : Critical section CD will be at distance d = 400 from AB.

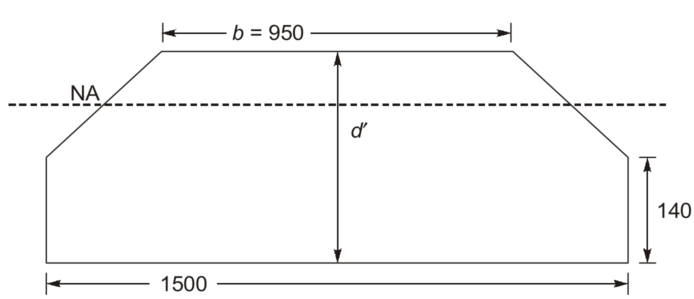
$$\text{Cantilever length to the right of } CD = 800 - 400 = 400 \text{ mm}$$

$$\text{The shear force } V = W_0 B \times 0.4$$

$$= 88.89 \times 1.5 \times 0.4 = 53.334 \text{ kN}$$

The section will be trapezoidal in shape as shown in fig.

$$\text{The width } b \text{ at top} = 400 + \frac{1500 - 400}{800} \times 400 = 950 \text{ mm}$$



$$\text{Effective depth } d' = 140 + \frac{400 - 140}{800} \times (800 - 400) = 270 \text{ mm}$$

$$\text{Depth of N.A.} = kd' = 0.289 \times 270 = 78.03 \text{ mm}$$

$$\text{Width of section at N.A. } b' = 950 + \frac{1500 - 950}{(270 - 140)} \times 78.03 = 1280.13 \text{ mm}$$

$$\tau_v = \frac{V}{b'd'} = \frac{53.334 \times 10^3}{1280.13 \times 270} = 0.15 \text{ N/mm}^2$$

$$\tau_{c,\,min} = 0.18 \text{ N/mm}^2 \text{ (WSM)} > \tau_V$$

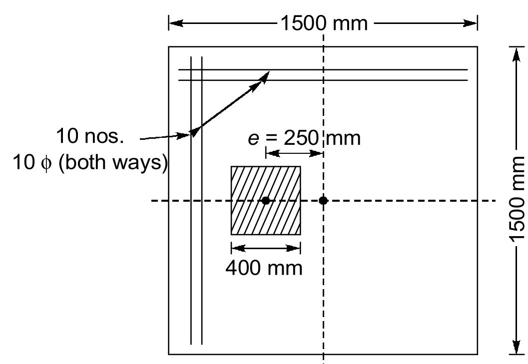
Hence safe.

Steel reinforcement:
$$A_{st} = \frac{M}{\sigma_{st} \cdot j.d} = \frac{42.67 \times 10^6}{230 \times 0.904 \times 400} = 513.06 \text{ mm}^2$$

$$\text{Minimum reinforcement} = \frac{0.12}{100} \times 1500 \times 400 = 720 \text{ mm}^2$$

$$\text{Use 10 mm} - \phi \text{ bars in number} = \frac{720}{\frac{\pi}{4} \times 10^2} = 9.16$$

∴ Provide 10 – 10 mm ϕ bars reinforcement in both directions.



Reinforcement details

Solution : 13

Given: Column : 300 × 400 mm; Axial load, $P = 1500$ kN

$$\gamma = 17 \text{ kN/m}^3, \ \phi = 30° \text{ and } q_0 = 125 \text{ kN/m}^2$$

$$\text{Weight of footing} = 10\% \text{ of } P = 0.1 \text{ P}$$

$$\text{Area of footing required} = \frac{1.1P}{q_0} = \frac{1.1 \times 1500}{125}$$

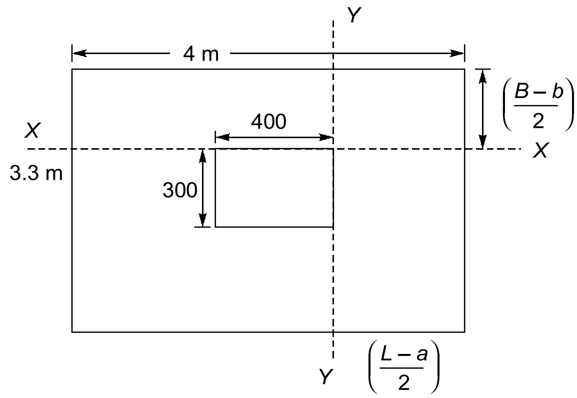
$$A = 13.2 \text{ m}^2$$

$$\text{Let width of footing } B = 3.3 \text{ m}$$

$$\therefore \quad \text{Length, } L = \frac{A}{B} = 4 \text{ m}$$

Net soil pressure for design using LSM:

$$W_{u0} = 1.5 \times \frac{P}{A} = 1.5 \times \frac{1500}{13.2} = 170.45 \text{ kN/m}^2$$



Check for Bending Moment:

Critical section is face of the column

$$M_{u_x} = W_{u_0} \times \left(\frac{B-b}{2}\right) \times \frac{\left(\frac{B-b}{2}\right)}{2} \times L$$

$$= 170.45 \times \frac{(3.3-0.3)^2}{8} \times 4$$

$$= 767.025 \text{ kN-m}$$

$$M_{u_y} = W_{u_0} \times B \times \frac{(L-a)^2}{8} = 170.45 \times 3.3 \times \frac{(4-0.4)^2}{8}$$

$$= 911.23 \text{ kN-m}$$

Depth required,

$$d_x = \sqrt{\frac{M_{u_x}}{QL}} = \sqrt{\frac{767.025 \times 10^6}{0.138 \times 20 \times 4000}} = 263.59 \text{ mm}$$

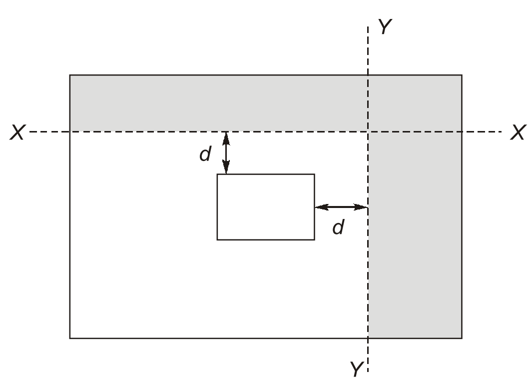
$$d_y = \sqrt{\frac{M_{u_y}}{QB}} = \sqrt{\frac{911.23 \times 10^6}{0.138 \times 20 \times 3300}} = 316.3 \text{ mm}$$

∴ Let us assume $d = 320$ mm with effective cover of 80 mm.

So, depth (D) = 400 mm

Check for one-way shear:

Critical section is at distance 'd' from face of the column.



$$\frac{L-a}{2} = \frac{4-0.4}{2} = 1.8 \text{ m}$$

$$\frac{B-b}{2} = \frac{3.3-0.3}{2} = 1.5 \text{ m}$$

As overhang is more for *y-y* face, therefore, it is the critical section.

$$V_u = W_{u_0} \times B \times \left(\frac{(L-a)}{2} - d\right) = 170.45 \times 3.3 \times (1.8 - 0.32)$$

$$= 832.48 \text{ kN}$$

$$\tau_v = \frac{V_u}{Bd} = \frac{832.48 \times 10^3}{3300 \times 320} = 0.78 \text{ N/mm}^2 > 0.28 \text{ N/mm}^2$$

∴ Change depth

Let us keep d = 620 mm

$$V_u = 170.45 \times 3.3 \times (1.8 - 0.62) = 663.73 \text{ kN}$$

$$\tau_v = \frac{663.73 \times 10^3}{3300 \times 620} = 0.32 \text{ N/mm}^2 > 0.28 \text{ N/mm}^2 (= \tau_{c,\min})$$

∴ Change depth

Let us keep d = 720 mm

$$V_u = 170.45 \times 3.3 \times (1.8 - 0.72) = 607.48 \text{ kN}$$

$$\tau_v = \frac{607.48 \times 10^3}{3300 \times 720} = 0.256 \text{ N/mm}^2 < \tau_c (= 0.28 \text{ N/mm}^2)$$

Hence safe.

So, d = 720 mm and D = 800 mm

Check for punching shear:

$$V_p = P_u - W_{u0}(a+d)(b+d)$$

$$\tau_{P_u} = \frac{P_u - W_{u_0}(a+d)(b+d)}{2\{(a+d)+(b+d)\}d}$$

$$\tau_{P_u} = \frac{(1.5 \times 1500 \times 10^3) - 170.45 \times 10^{-3} \times (1120) \times (1020)}{2 \times \{400 + 720 + 300 + 720\} \times 720} = 0.67 \text{ MPa}$$

$$\tau_{P_{per}} = k_B \times 0.25 \times \sqrt{f_{ck}}$$

$$k_B = \left(0.5 + \frac{b}{a}\right) \not> 1$$

$$k_B = 0.5 + \frac{300}{400} = 1.25 > 1$$

∴ $k_B = 1$

$$\tau_{P_{per}} = 1 \times 0.25 \times \sqrt{20} = 1.118 \text{ MPa} > \tau_{P_u} \quad \text{(OK)}$$

Area of steel required:

1. For moment M_{ux} :

$$A_{st_x} = \frac{0.5 f_{ck}}{f_y}\left[1 - \sqrt{1 - \frac{4.6 M_{u_x}}{f_{ck} \cdot L \cdot d^2}}\right] Ld$$

$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 767.025\times 10^6}{20\times 4000\times 720^2}}\right]4000\times 720$$

$$= 3469.88\text{ mm}^2$$

$$\text{Minimum reinforcement } (A_{st_x})\text{min} = 0.12\times\frac{L\times D}{100}$$

$$= 0.12\times\frac{4000\times 800}{100} = 3840\text{ mm}^2$$

$$\therefore \quad A_{st_x} = 3840\text{ mm}^2$$

Use 16 mm – ϕ bars

$$n_T = \frac{3840}{\frac{\pi}{4}\times 16^2} = 19.09,\ \text{say } 20$$

$$n_C = 20\times\frac{2}{1+\frac{4}{3.3}} = 18$$

∴ Provide 18 – 16 ϕ bar at centre and one-one bar at side.

2. For moment M_{uy} :

$$A_{st_y} = \frac{0.5\,f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6M_{u_y}}{f_{ck}\cdot B\cdot d^2}}\right]Bd$$

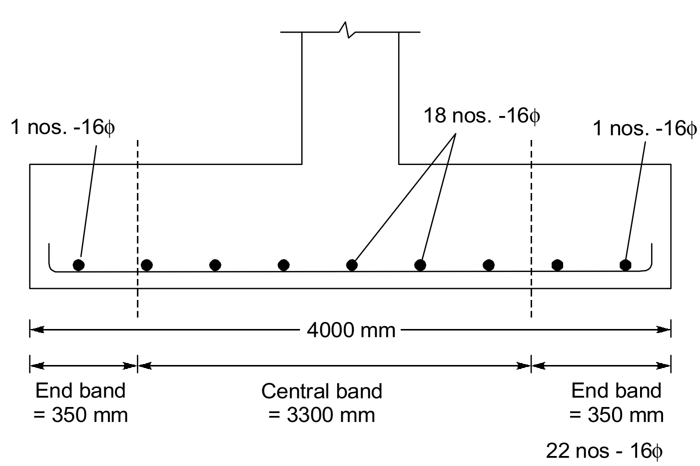
$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 911.23\times 10^6}{20\times 3300\times 720^2}}\right]3300\times 720$$

$$= 4327.56\text{ mm}^2$$

$$\text{Minimum reinforcement } (A_{sty})_{\text{min}} = \frac{0.12\times BD}{100} = \frac{0.12\times 3300\times 720}{100} = 3168\text{ mm}^2 < A_{st_y}$$

$$n = \frac{4327.56}{\frac{\pi}{4}\times 16^2} = 21.5\simeq 22\ (\text{say})$$

∴ Provide 22 – 16 ϕ bars equally spaced.



Solution : 14

Nominal shear stress = 0.28 MPa

Solution : 15

Taking effective cover = 40 mm, $d = 800 - 40 = 760$ mm

(i) Design of tensile reinforcement at support

$$\text{Factored B.M. at support, } M_u = -1.5\times\frac{Wl_{eff}^2}{2} = -1.5\times\frac{30\times3^2}{2} = -202.5 \text{ kNm}$$

Tensile reinforcement for M_u is given by the equation

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

$\because$ {Putting the value of x_u (from, $0.36\ f_{ck}\ B x_u = 0.87\ f_y\ A_{st}$) in above equation}

$$202.5\times10^6 = 0.87\times415\times A_{st}\times760\left(1-\frac{A_{st}\times415}{300\times760\times20}\right)$$

or $24.97\ A^2_{st} - 274398\ A_{st} + 202.5\times10^6 = 0$

or $A_{st} = 795.58 \text{ mm}^2$ ...(A)

$$\text{Min. tensile reinforcement} = \frac{0.85}{f_y}\times100 = \frac{0.85\times100}{415} = 0.2\%$$

i.e. $A_{st} = 0.002\times300\times760$

$= 456 \text{ mm}^2$ ...(B)

Considering, (A) and (B)

Provide 3 numbers 20 mm diameter having $A_{st} = 942.47 \text{ mm}^2$.

(ii) Design of shear reinforcement at support

$V_u = 1.5\times30\times3$

$= 135$ kN

$$\tau_v = \frac{V_u - \frac{M_u}{d}\tan\beta}{bd} = \frac{135\times10^3 - \frac{202.5\times10^6}{760}\times\frac{450}{3000}}{300\times760}$$

$$\left\{\because \tan\beta = \left(\frac{800-350}{3000}\right)\right\}$$

$= 0.42 \text{ N/mm}^2$

$$\text{Percentage of tensile steel} = \frac{942\times100}{300\times760}\% = 0.413\%$$

For M20 concrete,
$$\tau_c = 0.36+\frac{0.48-0.36}{(0.5-0.25)}\times(0.413-0.25)$$

(From table 19 of IOS 456 : 2000)

$= 0.438 \text{ N/mm}^2 > 0.42\ (\tau_v)$

Hence only nominal shear reinforcement is to be provided.

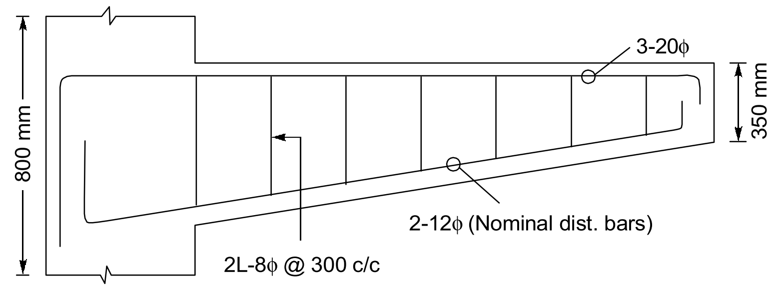
Providing 8 mm dia two legged stirrups

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2 = 100 \text{ mm}^2$$

$$s_{v\,max} = \frac{0.87 f_y A_{sv}}{0.4b} = \frac{0.87 \times 415 \times 100}{0.4 \times 300} = 300.87 \text{ mm}$$

∴ 300.8 mm < 450 mm < 570 mm (0.75 d)

Hence provided 8 mm dia two-legged stirrups @ 300 c/c.



Reinforcement details

Solution : 16

(1) Loading

Dead load

(i) 125 mm thick slab = (0.125 × 25) = 3.125 kN/m²

(ii) 6 mm thick ceiling plaster = (0.006 × 24) = 0.144 kN/m²

(iii) 30 mm thick floor finish = (0.03 × 24) = 0.72 kN/m²

Total Dead load = 3.989 kN/m²

Live load = 4.0 kN/m²

(2) Load on beam

Dead load from slab = (3.989 × 3) = 11.967 kN/m

Assuming 5% extra and rib depth = 250 mm, dead load = 1.05 (0.25 × 0.25 × 25) = 1.640625 kN/m

Total dead load = 13.607625 kN/m

Total live load = (4 × 3) = 12 kN/m

$$w_{DL} = 13.60 \text{ kN/m}$$

$$w_{LL} = 12 \text{ kN/m}$$

(3) Factored load, moment and shear force

$$w_f = 1.5\,(13.60 + 12) = 38.41 \text{ kN/m}$$

$$M_u = \frac{w_f . l_e^2}{8} = 145.23 \text{ kN-m}$$

$$V_u = \frac{w_f . l_e}{2}$$

$$= \frac{(38.41)(5.5)}{2} = 105.63 \text{ kN}$$

(4) Effective width and depth

(i) b_f = Lesser of **(a)** and **(b)** as computed below.

(a) $$\frac{L_e}{6}+b_w+6D_f = \frac{5.5}{6}+0.25+6(0.125)$$

$$= 1.9166 \text{ m} = 1.917 \text{ m}$$

(b) Centre to centre distance of beams = 3.0 m

∴ $$b_f = 1.917 \text{ m} = 1917 \text{ mm}$$

(ii) Overall depth = 250 + 125 = 375 mm

Effective depth = 375 – (Clear cover + ϕ/2) [Assume 20 mm dia bar]

$$= 375 - 25 - \frac{20}{2} = 340 \text{ mm}$$

(5) $$M_u = 0.36 f_{ck} \frac{x}{d}\left(1-0.416\frac{x}{d}\right) b_f d^2$$

$$\frac{x}{d} = 1.2 \pm \sqrt{1.2^2 - \frac{6.68 M_u}{f_{ck} b_f d^2}}$$

$$\frac{x}{d} = 1.2 - \sqrt{1.44 - \frac{6.68(145.23\times10^6)}{20(1917)(340)^2}} = 1.2 - 1.105$$

∴ $$\frac{x}{d} = 0.095$$

⇒ $$x = (0.095)\,340 = 32.3 \text{ mm} < D_f = 125 \text{ mm}$$

(6) Steel to be provided

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

$$= \frac{145.23\times10^6}{0.87(415)(340-0.42(32.3))}$$

$$= 1232 \text{ mm}^2$$

Provide 4 – 20 ϕ (A_{st} provided = 1256 mm²)

Check on steel

(7) Steel percentage

$$p_t = \frac{1256\times100}{250\times340} = 1.47\%$$

$$\text{Min steel} = \frac{0.85\times100}{f_y} = \frac{0.85\times100}{415} = 0.2\%$$

(8) Shear reinforcement

$$V_u = 105.63 \text{ kN}$$

Percentage of tensile reinforcement = 1.47%

Maximum shear stress = 2.8 N/mm²

$$\tau_v = \frac{V_u}{bd} = \frac{105.63\times10^3}{(250)(340)}$$

$$= 1.2427 \text{ N/mm}^2$$

From Table 19 of IS 456 : 2000

$$\tau_c = 0.67 + \frac{0.72 - 0.67}{0.25} \times (1.47 - 1.25)$$

$$= 0.714 \text{ N/mm}^2$$

$$\tau_v > \tau_c \text{ and } \tau_v < \tau_{c\,max}$$

(9) Shear force due to to concrete = V_c

$$V_c = (0.714)(250)(340) = 60690 \text{ N}$$

$$= 60.69 \text{ kN}$$

Shear to be taken by steel

$$V_{us} = V_u - V_c$$

$$= 105.63 - 60.69$$

$$= 44.94 \text{ kN}$$

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

$$\therefore \quad \frac{A_{sv}}{s_v} = \frac{V_{us}}{0.87 f_y d}$$

$$\frac{A_{sv}}{s_v} = \frac{44.94 \times 10^3}{0.87(415)(340)} = 0.366$$

Using 8 mm ϕ – 2 legged stirrups

$$A_{sv} = 2 \times \frac{\pi}{4} \times 8^2$$

$$= 2 \times 50 = 100 \text{ mm}^2$$

$$\frac{A_{sv}}{s_v} = 0.366 \text{ mm}$$

$$\Rightarrow \quad s_v = \frac{A_{sv}}{0.366} = \frac{100}{0.366} = 273 \text{ mm}$$

Use 8 mm ϕ – 2 legged stirrups @ 250 mm c/c

Maximum spacing

(a) $0.75d = 0.75\,(340) = 255$ mm

(b) ≯ 300 mm

Minimum or lesser of the above two

i.e. spacing = 255 mm

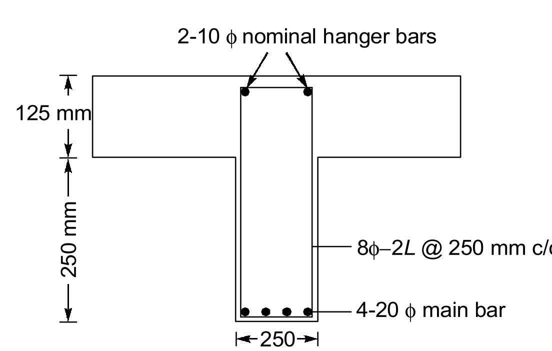
Use 8ϕ–2 Legged @ 250 mm c/c

Concrete *M* 20

Steel *Fe* 415

Clear cover = 25 mm

Main longitudinal tensile reinforcement = 4 – 20 ϕ



Solution : 17

Given that P_u = 600 kN, M_u = 100 kN-m, f_{ck} = 20 N/mm^2, l_{eff} = 4.5 m and column is short.

Now, for column to be short,

$$\frac{l_{eff}}{D} \leq 12$$

$$\Rightarrow \quad D \geq \frac{l_{eff}}{12} \geq \frac{4500}{12} \geq 375\,\text{mm}$$

Since the width of support of beam = 300 mm, therefore, provide width of column = 300 mm

Assume size of column = (300 × 400) mm and diameter of lateral ties = 6 mm

Using 20 mm diameter bars,

$$\text{Effective cover } (d') = 40 + 6 + \frac{\phi}{2} = 40 + 6 + \frac{20}{2} = 56 \text{ mm}$$

Now,
$$\frac{d'}{D} = \frac{56}{400} = 0.14$$

$$f_{ck}.b.D = 20 \times 300 \times 400 = 2400 \text{ kN}$$

$$\frac{P_u}{f_{ck}b.D} = \frac{600}{2400} = 0.25$$

$$\frac{M_u}{f_{ck}.b.D^2} = \frac{100 \times 10^6}{2400 \times 10^3 \times 400} = 0.104$$

Using chart 2 and chart 3 (Appendix) of SP 16

For $\frac{d'}{D}$ = 0.10 and 0.15, we get,

$\frac{d'}{D}$	$\frac{p}{f_{ck}}$
0.10	0.042
0.15	0.045

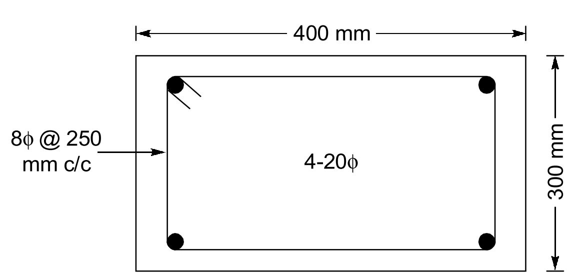
By interpolation, we get,

$$\frac{d'}{D} = 0.14, \frac{p}{f_{ck}} = 0.044$$

$$\therefore \quad p = 0.044 \times f_{ck} = 0.044 \times 20 = 0.89\% > A_{st,\,min} = 0.8\%$$

$$\therefore \quad A_{st} = p\frac{bD}{100} = 0.89 \times \frac{300 \times 400}{100} = 1066 \text{ mm}^2$$

Provide 4-20 mm ϕ bars = 4 × 314 = 1256 mm^2



Reinforcement details

Solution : 18

$$\frac{l_{eff}}{b} = \frac{3000}{450} = 6.67 < 12$$

So, column is short

$$e_{min} \text{ with respect to major axis } = \frac{l}{500} + \frac{D}{30}$$

e_{min} is greater of
$$\begin{cases} \frac{3000}{500} + \frac{600}{30} = 26 \text{ mm} \\ 20 \text{ mm} \end{cases}$$

$\Rightarrow$ $e_{min} = 26$ mm

Also $e_{min} \leq 0.05$ D
$= 0.05 \times 600 = 30$ mm

$\because$ e_{min} (= 26 mm) $<$ 30 mm   Hence OK

e_{min} with respect to minor axis

$$e_{min} = \frac{3000}{500} + \frac{450}{30} = 21 \ (> 20 \text{ mm})$$

$e_{min} = 21$ mm

Also $e_{min} \leq 0.05 \times 450 = 22.5$ mm

e_{min} (= 21 mm) $\leq$ 22.5 mm   Hence OK

Now we can use $P_u = 0.4 f_{ck} A_c + 0.67 f_y A_{sc}$

$\Rightarrow$ $1.5 \times 2000 \times 10^3 = 0.4 \times 20 \times (450 \times 600 - A_{sc}) + 0.67 \times 415 \times A_{sc}$

$\Rightarrow$ $A_{sc} = 3110.5 \text{ mm}^2$

Provide 4-25 mm dia bars at corner

$$\text{Area} = 4 \times \frac{\pi}{4} \times 25^2 = 1964 \text{ mm}^2$$

Provide 4-20 mm dia bars at centre

$$\text{Area} = 4 \times \frac{\pi}{4} \times 20^2 = 1256 \text{ mm}^2$$

Total steel provided = 1964 + 1256 = 3220 mm^2

$$\text{Percentage of steel} = \frac{\frac{\pi}{4} \times (25)^2 \times 4 + \frac{\pi}{4} \times 4 \times (20)^2}{450 \times 600} \times 100 = 1.192\%$$

$>$ 0.8% (minimum % of steel)

and $<$ 6% (maximum % of steel)

Lateral ties

Diameter of tie bars (ϕ_t) should be greater of

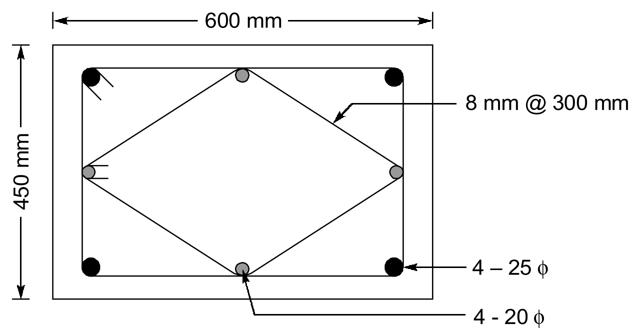
(i) $\frac{1}{4} \times 25 = 6.25$ mm

(ii) 6 mm

Hence, take $\phi_t = 8$ mm.

Spacing should be lesser of

(i) 450 mm (least lateral dimension)
(ii) $16 \times \phi_{long,\,min} = 16 \times 20 = 320$ mm
(iii) 300 mm
Provide 8 mm dia tie bars @ 300 mm c/c



Solution : 19

(1) Calculation of factored loads

Assume, $\dfrac{\text{Span}}{d} = 25$

$$d = \frac{\text{Span}}{25} = \frac{3500}{25} = 140 \text{ mm}$$

Toral depth $= d + \text{Clear cover} + \phi/2$

$= 140 + 15 + \dfrac{10}{2}$ (Assuming 10 mm dia.bar)

$= 160$ mm

Dead load

(i) Slab $0.16 \times 25 = 4.0 \text{ kN/m}^2$
(ii) Finish load $= 1.0 \text{ kN/m}^2$
∴ Total $DL = 5.0 \text{ kN/m}^2$
$LL = 4.0 \text{ kN/m}^2$

Factored Load (Design Load)
$= 1.5\,(DL + LL)$
$= 1.5\,(5 + 4)$
$= 13.5 \text{ kN/m}^2$

Effective span
(i) $3.5 + 0.25 = 3.75$ m (Width of wall = 250 mm)
(ii) $3.5 + 0.14 = 3.64$ m
∴ Effective span $= 3.64$ m

Total load per metre width
$= 13.5 \times 3.64$
$= 49.14$ kN

(2) Ultimate moment and shear force

$$M_u = \frac{WL}{8} = \frac{49.14 \times 3.64}{8} = 22.3587 \text{ kNm}$$

$$V_u = \frac{W}{2} = \frac{49.14}{2} = 24.57 \text{ kN}$$

(3) Depth from bending moment consideration

$$M_u = 0.138 f_{ck} bd^2$$

$$d = \sqrt{\frac{M_u}{0.138 f_{ck} b}} = \sqrt{\frac{22.3587 \times 10^6}{0.138(20)(1000)}} = 90.00 \text{ mm}$$

$$d_{computed} = 90 \text{ mm} < d_{provided} = 140 \text{ mm}$$

$$\left(d_{provided} = 160 - 15 - \frac{10}{2} = 140 \text{ mm}\right)$$

(4) Check for Shear

$$\tau_v = \frac{V_u}{bd} = \frac{24.57 \times 10^3}{(1000)(140)} = 0.1755 \text{ N/mm}^2$$

From Table 19 of IS 456 : 2000

For $p_t = \dfrac{100 A_{st}}{bd} \le 0.15\%$

$$\tau_c = 0.28 \text{ N/mm}^2$$

$$\tau_v < \tau_c$$

(5) Calculation of steel area required

$$\frac{x}{d} = 1.2 - \sqrt{(1.2)^2 - \frac{6.6 M_u}{f_{ck} bd^2}}$$

$$\frac{6.6 M_u}{f_{ck} bd^2} = \frac{6.6\left(22.3587 \times 10^6\right)}{20(1000)(140)^2} = 0.376$$

$$\therefore \quad \frac{x}{d} = 1.2 - \sqrt{1.44 - 0.376} = 1.2 - 1.0315 = 0.1685 < 0.48$$

Lever Area, $Z = d\left(1 - 0.416\dfrac{x}{d}\right) = 140\,(1 - 0.416 \times 0.1685) = 130.186 \text{ mm}$

$$A_{st} = \frac{M_u}{0.87 f_y Z} = \frac{22.3587 \times 10^6}{0.87(415)(130.186)} = 475.6 \text{ mm}^2$$

$$\text{Spacing required} = \frac{1000 \times 78.5}{475.6} = 165.05 \text{ mm}$$

(For 10 mm dia bar, A_s bar $= \dfrac{\pi}{4} \times 10^2 = 78.5 \text{ mm}^2$)

Provide 10 ϕ @ 150 mm c/c ($A_{st} = 523.6 \text{ mm}^2$)

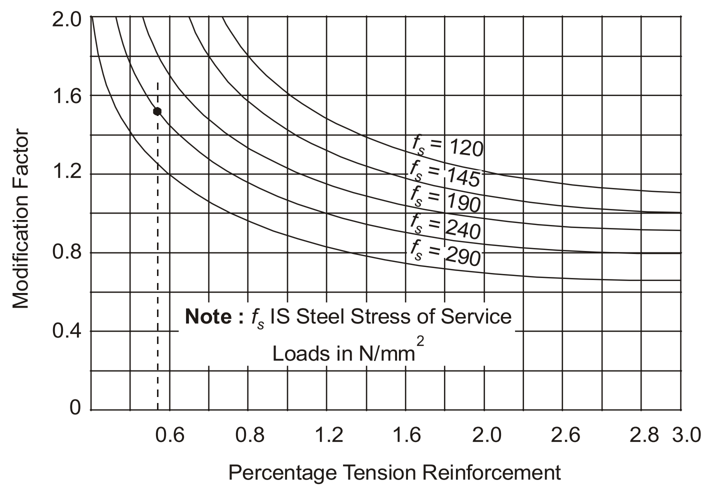
$$A_{st} \text{ provided} = 523 \text{ mm}^2$$

$$\text{Percentage of tension steel} = \frac{523 \times 100}{1000 \times 140} = 0.373\%$$

$$f_s = 0.58 f_y \frac{\text{Area of c/s of steel required}}{\text{Area of c/s of steel provided}}$$

$$A_{st} \text{ required} = 475.6 \text{ mm}^2 \text{ and } A_{st} \text{ provided} = 523 \text{ mm}^2$$

$$\therefore \quad f_s = 0.58\,(415)\frac{475.6}{523} = 218 \text{ N/m}^2$$



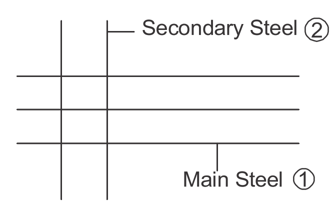
$$f_s = 0.58\, f_s \frac{\text{Area of cross-section of steel required}}{\text{Area of cross-section of steel provided}}$$

$$\text{Modification factor} = 1.55$$

$$\frac{L}{d} = 1.55 \times 20 = 31 \qquad \left(\textit{Assumed } \frac{L}{d} = 25\right)$$

$$d = \frac{L}{31} = \frac{3640}{31} = 117.42 \text{ mm}$$

Secondary steel @ 0.12% of gross area



$$A_{st} = \frac{0.12bD}{100}$$

$$= \frac{0.12(1000)(160)}{100}$$

$$= 192 \text{ mm}^2$$

Provide 8 mm dia bar

$$A_s = \frac{\pi}{4}8^2 = 50 \text{ mm}^2$$

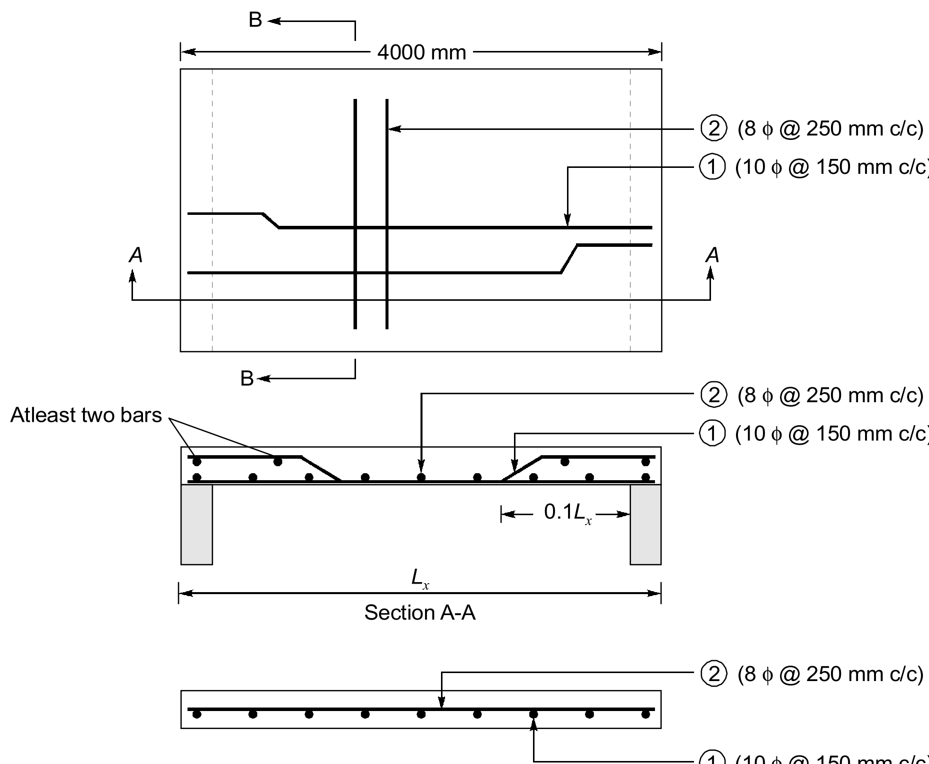
$$\text{Spacing required} = \frac{1000 \times 50}{192} = 260 \text{ mm c/c}$$

Spacing of reinforcement

(i) 3d or 300 mm whichever is less against shrinkage and temperature

(ii) $\ngtr$ 5d or 450 mm, whichever is less

Distribution steel: 8 ϕ @ 250 mm c/c



Section B-B

$$\text{Thickness of slab} = 160 \text{ mm}$$

$$L_x = 3500 + 250 = 3750 \text{ mm}$$

$$0.1\, L_x = 375 \text{ m}$$

Solution : 20

(1) Thickness of Slab

$$L_x = 4.0 \text{ m}$$

$$\text{Cover to mild exposure} = 15 \text{ mm}$$

Clause 24.1. IS 456 : 2000

Note: Span/overall depth

Simply supported	35
Continuous	40

For high strength deformed bars of grade Fe 415, the above values should be multiplied by 0.8

$$\frac{\text{Span}}{\text{Overall depth}} = 35$$

$$\frac{\text{Span}}{\text{Overall depth}} \text{ for this problem} = 35 \times 0.8 = 28$$

$$\frac{L}{D} = 28$$

$$D = \frac{L}{28}$$

$$= \frac{4000}{28} = 142.85$$

Let us provide, $D = 175$ mm

Effective depth, $d = 175 - 15 - \frac{10}{2} = 155$ mm

(2) Design load

(A) *Dead load*

(i) Slab 0.175 × 25 × 1 = 4.375 kN/m

(ii) Floor finish (25 mm thick) 0.025 × 24 × 1 = 0.6 kN/m

(iii) Plastering (6 mm thick) 0.006 × 24 × 1 = 0.144 kN/m

Total DL = 5.119 kN/m.

(B) *Live Load* = 8 kN/m.

Total load = 8 + 5.119 = 13.119 kN/m.

w = factored load = 13.119 × 1.5 = 19.6785 kN/m.

(3) Maximum Factored Moment

$$\frac{L_y}{L_x} = \frac{5.5}{4.0} = 1.375$$

From Table 27, IS 456 : 2000

$\frac{L_y}{L_x}$	1.3	1.4
α_x	0.093	0.099
α_y	0.055	0.051

For $\frac{L_y}{L_x} = 1.375$

$$\alpha_x = 0.093 + \frac{(0.099 - 0.093)}{0.1} \times (1.375 - 1.3)$$

$$= 0.0975$$

$$\alpha_y = 0.055 - \frac{(0.055 - 0.051)}{0.1} \times (1.375 - 1.3)$$

$$= 0.052$$

$$M_x = \alpha_x w l_x^2$$

$$= 0.0975\,(19.6785)\,(4.0)^2$$

$$= 30.69846 \text{ kNm}$$

$$M_y = \alpha_y w l_x^2$$

$$= 0.052\,(19.6785)\,(4.0)^2$$

$$= 16.372512 \text{ kNm}$$

(4) Area of steel

$$\frac{x}{d} = 1.2 - \sqrt{(1.2)^2 - \frac{6.6M_u}{f_{ck}bd^2}}$$

$$\frac{6.6M_u}{f_{ck}bd^2} = \frac{6.6(30.69846 \times 10^6)}{25(1000)(155)^2} = 0.3373316$$

$$\frac{x}{d} = 1.2 - \sqrt{1.44 - 0.3373316} = 1.2 - 1.05$$

$$\therefore \quad \frac{x}{d} = 0.15 < 0.48$$

Lever Area,
$$Z = d\left(1 - 0.416\frac{x}{d}\right)$$
$$= 155\,(1 - 0.416 \times 0.15) = 145.328 \text{ mm}$$

Area of steel, in the shorter direction

$$A_{st} = \frac{M_u}{0.87 f_y Z}$$

$$= \frac{30.69846 \times 10^6}{0.87(415)145.328} = 585 \text{ mm}^2$$

Providing 10 mm dia bar

$$\text{Area of steel bar} = \frac{\pi}{4}(10)^2 = 78.5 \text{ mm}^2$$

$$\text{Spacing required} = \frac{1000 \times 78.5}{585} = 134 \text{ mm}$$

Provide 10 ϕ @ 125 mm c/c.

Area of steel in the longer direction

$$\text{Effective depth in the longer side} = 175 - 15 - 10 - \frac{10}{2} = 145 \text{ mm}$$

$$\frac{x}{d} = 1.2 - \sqrt{1.44 - \frac{6.6M_u}{f_{ck}bd^2}}$$

$$\frac{6.6M_u}{f_{ck}bd^2} = \frac{6.6 \times 16.37252 \times 10^6}{25(1000)(145)^2} = 0.20558$$

$$\frac{x}{d} = 1.2 - \sqrt{1.44 - 0.20558}$$
$$= 1.2 - 1.11 = 0.089 < 0.48$$

Lever Area,
$$Z = d\left(1 - 0.416\frac{x}{d}\right) = 145\,(1 - 0.416 \times 0.089)$$
$$= 139.6 \text{ mm}$$

$$j = 1 - 0.416\frac{x}{d}$$

$$A_{st} = \frac{M_u}{0.87f_y jd} = \frac{M_u}{0.87f_y Z}$$

$$A_{st} = \frac{M_u}{0.87f_y Z}$$

$$A_{st} = \frac{16.372512\times10^6}{(0.87)(415)(139.6)} = 324.83 \text{ mm}^2$$

$$\text{Spacing required} = \frac{1000\times78.5}{324.83} = 241.66 \text{ mm}$$

Provide 10 mm dia bar @ 240 mm c/c (A_{st} = 327 m^2)

(5) Check of Deflection

Percentage of steel along l_x

$$p_t = \frac{585\times100}{1000(155)} = 0.3774 \text{ (needed)}$$

$$f_s = 0.58f_y\frac{A_{st}\text{ required}}{A_{st}\text{ provided}}$$

$$= 0.58(415)\frac{585}{628}$$

$$f_s = 224 \text{ N/mm}^2$$

from p_t = 0.3774% and f_s = 224 N/mm^2

Modification factor, F = 1.5 **(Fig. 4 page 38 of IS 456 : 2000)**

From Clause 23.2.1 IS 456 : 2000

Allowable $\frac{L}{D} = 20 \times 1.5 = 30$

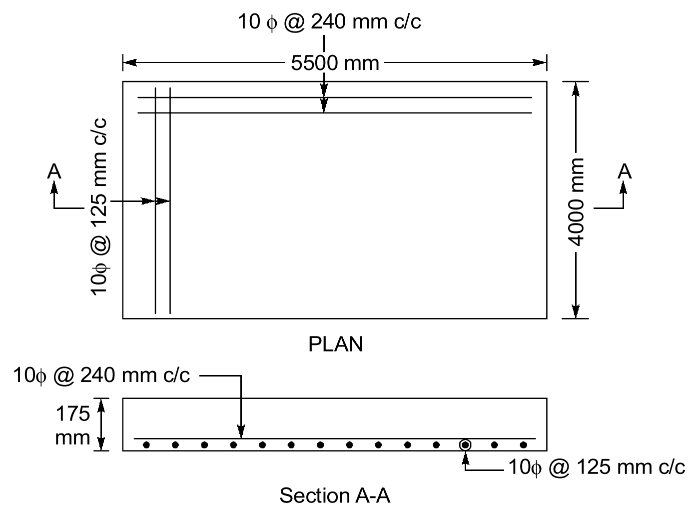
$$\frac{L}{D}\text{ provided} = \frac{4000}{175} = 22.85$$

$$\frac{L}{d}\text{ provided} = \frac{4000}{155} = 25.80$$

Hence Safe.

Summary of Design

- Overall Depth = 175 mm
- Clear Cover = 15 mm
- Reinforcement in the shorter direction; 10 ϕ @ 125 mm c/c
- Reinforcement in the longer direction; 10 ϕ @ 240 mm c/c
- Concrete = M25
- Steel = Fe 415



Solution : 21

(i) Soil pressure

$$\text{Axial load} = 1400 \text{ kN}$$

$$\text{Approximate area of footing required} = \frac{1400}{100} = 14 \text{ m}^2$$

$$\text{Weight of footing including earth} = 20 \times 1 \times 10 = 200 \text{ kN}$$

$$\text{Total weight on soil} = 1600 \text{ kN}$$

$$\text{Actual area of footing required} = \frac{1600}{100} = 16 \text{ m}^2$$

Provide 4.0 m × 4.0 m square footing giving total area = 16 m^2

(ii) Bending moment

The net earth pressure acting upward due to factored load is

$$p = \frac{1400 \times 1.5}{16} = 131.25 \text{ kN/m}^2$$

where 1.5 is the partial safety factor.

Bending moment about axis X-X passing through the face of the column as shown in figure.

$$M_u = \left[131.25 \times 4 \times \left(\frac{4 - 0.04}{2}\right)^2 \times \frac{1}{2}\right] = 851 \text{ kN-m}$$

The effective depth required is given by

$$\text{BM} = 0.138\,\sigma_{ck}\,bd^2$$

$$\Rightarrow \qquad d = \sqrt{\frac{851 \times 10^6}{0.138 \times 25 \times 4000}} = 248 \text{ mm}$$

Adopt 550 mm effective depth and 600 mm overall depth. Increased depth is taken due to shear consideration.

$$\text{Area of tension steel is given by BM} = 0.87\sigma_y A_t \left(d - \frac{\sigma_y A_t}{\sigma_{ck} b}\right)$$

$$\Rightarrow \quad 851 \times 10^6 = 0.87 \times 415 A_t \left(550 - \frac{415 A_t}{25 \times 4000}\right)$$

$$\Rightarrow \quad A_t = 4434 \text{ mm}^2$$

$$\therefore \quad \text{No. of 12 } \phi \text{ bars required} = \frac{4434}{\frac{\pi}{4}(12)^2} = 39.2 \simeq 40 \text{ (say)}$$

$$p = \frac{40 \times \frac{\pi}{4} \times 12^2 \times 100}{4000 \times 550} = 0.20\%$$

(iii) One way shear action

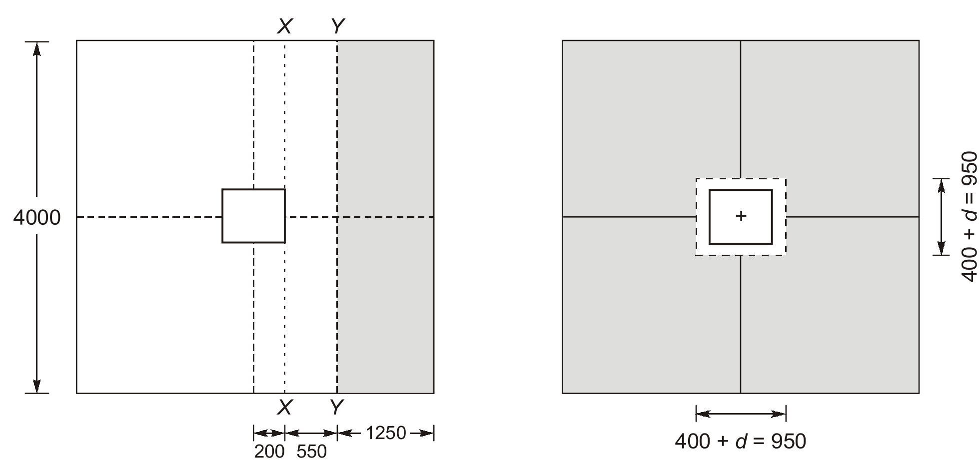
The critical section is taken it distance '*d*' away from the face of column as shown in Figure.

$$\text{Shear force } V_u = 131.25 \times 4\left[\left(\frac{4-0.40}{2}\right) - 0.55\right] = 6567 \text{ kN}$$

$$\text{Nominal shear stress } \tau_v = \frac{V_u}{bd} = \frac{656000}{4000 \times 550} = 0.298 \text{ N/mm}^2$$

Shear strength of M25 concrete with 0.20% steel

$$\tau_c = 0.327 \text{ N/mm}^2 > \tau_v$$



Critical Section for Moment and One-way Shear **Critical Section for Two-way Shear**

(iv) Two-way shear action

The critical section taken at a distance '0.5*d*' away from the face of column as shown in Figure.

$$\text{Shear force } V_u = 131.25\left[16 - (0.40 + 0.55)^2\right] = 1982 \text{ kN}$$

$$\text{Nominal shear stress } \tau_v = \frac{V_u}{b_0 d} = \frac{1982 \times 1000}{4(400+550)550} = 0.948 \text{ N/mm}^2 = 0.95 \text{ N/mm}^2$$

Shear strength of M 25 concrete

$$\tau_c' = k_s \tau_c, \; k_s = (0.5 + \beta_c)$$

$$\beta_c = \frac{\text{Length of shorter side of column}}{\text{Length of longer side of column}}$$

$$k_s = 0.5 + 1 \not> 1.0 \therefore k_s = 1.0$$

$$\tau_c' = \tau_c = 0.25\sqrt{\sigma_{ck}}$$

$$= 1.25 \text{ N/mm}^2 > 0.95 \text{ N/mm}^2 \quad \text{...OK}$$

This shows that a footing having an effective depth of 248 mm would be safe in shear.

(v) Development length: Development length of 10 mm bars

$$L_d = \frac{\sigma_s\phi}{4\pi_{bd}} = \frac{0.87\times 415\phi}{4\times(1.6\times 1.4)} = 40\ \phi$$

$\Rightarrow$ $L_d = 640$ mm

Actual embedment length provided from face of the column is

$$= \left(\frac{4000-400}{2}\right) - 50 \text{ mm (cover)}$$

$$= 1750 \text{ mm} > L_d \quad \text{...OK}$$

(vi) Load transfer from column to footing: Nominal bearing stress in the column concrete

$$\sigma_{br} = \frac{P_u}{A_c} = \frac{1.5\times 1400\times 1000}{400\times 400} = 13.125 \text{ N/mm}^2$$

$$\text{Allowable bearing stress} = 0.45\ \sigma_{ck} = 11.25 \text{ N/mm}^2$$

$$< 13.125 \text{ N/mm}^2 \quad \text{...OK}$$

Thus, the column load cannot be transferred by bearing alone

$$\text{To carry the excess load } P = (13.125 - 11.25)\ 400 \times 400 = 3000000 \text{ N}$$

$$\text{Required area of steel } A_s = \frac{300000}{0.67\times 415} = 1079 \text{ mm}^2$$

$$\text{Minimum } A_s = 0.5\% \text{ of column area}$$

$$= \frac{0.5}{100}\times 400^2 = 800 \text{ mm}^2$$

$$< 1079 \text{ mm}^2 \quad \text{...OK}$$

Provide 6-16 mm bars as dowels, $A_s = 1206$ mm^2. The stress in 16 mm dowels must be developed above and below junction of column and footing.

$$L_d \text{ for compression} = \frac{\sigma_s\phi}{4\pi_{bd}} = \frac{0.87\times 415\phi}{4\times(1.6\times 1.25\times 1.4)}$$

$$= 33\ \phi = 33 \times 16 = 528 \text{ mm}$$

The available vertical length L_1 foranchorage is

$$L_1 = 600 - 50 \text{ (clear cover)} - 2 \times 12 \text{ (footing bars)} - 16 \text{ (dowel)}$$

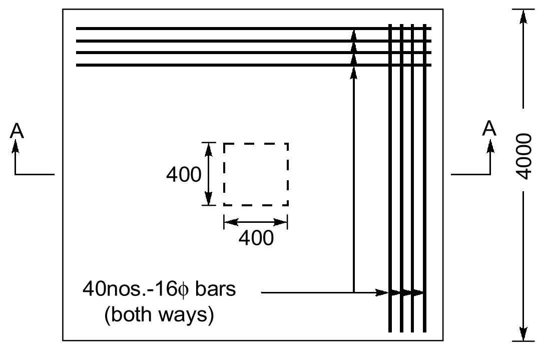
$$= 510 \text{ mm} < 528 \text{ mm} \quad \text{...OK}$$

Let us provide smaller diameter bars as dowels so that the available vertical length in the footing is sufficient for anchorage. Use 12 mm bars as dowels.

$$\text{Development length} = 33 \times 12 = 396 \text{ mm} < L_1 \quad \text{...OK}$$

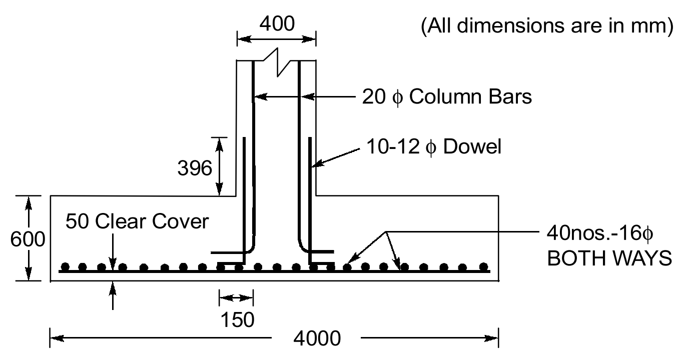
$$\text{Provide 10 – 12 mm bars, } A_s = 1130 \text{ mm}^2 > 1079 \text{ mm}^2 \quad \text{...OK}$$

In this example no need to check the bearing stress on the footing since both the column and footing contain the same grade of concrete. The details of reinforcement are shown in figure.



Plan of Reinforcement

Reinforcement in Footing Base



Section A-A

Sectional Elevation Showing Reinforcement

■■■■

4 Prestressed Concrete

LEVEL 1 Objective Questions

1. (a)
2. (0.15)
3. (60.75)
4. (b)
5. (d)

LEVEL 2 Objective Questions

6. (b)
7. (a)
8. (b)
9. (b)
10. (86.205)
11. (c)
12. (d)
13. (0.47)
14. (1.55)
15. (10)
16. (a)
17. (d)
18. (8.4)

LEVEL 3 Conventional Questions

Solution : 1

(i) f_{top} = 2.172 MPa, f_{bottom} = 7.728 MPa

(ii) Max comp. stress = 11.172 MPa

(iii) Load factor = 1.47

Solution : 2

(i) Min. possible depth = 855.35 mm say 860 mm

(ii) Min prestressing force = 1586.2 kN, eccentrricity = 228.46 mm

Solution : 3

At transfer

f_{top} = 1.96 MPa, f_{bottom} = 8.94 MPa

At service load condition

f_{top} = 13.03 MPa

f_{bottom} = –3.75 MPa

Solution : 4

At transfer

f_{top} = 0.784 MPa, f_{bottom} = 11.716 MPa

At final stage

f_{top} = 9.228 MPa

f_{bottom} = 0.972 MPa

Solution : 5

Due to elastic deformation = 58.68 MPa

Due to creep of concrete = 93.89 MPa

Due to creep of steel = 90 MPa

Due to shrinkage of concrete = 63 MPa

Solution : 6

(i) 15 mm deflection

(ii) Final deflection = 8.54 mm

Solution : 7

Min. prestressing force = 6151.58 kN, UDL = 28 kN/m

Eccentricity = 1283.35 mm

Solution : 8

Loss due to

(i) Elastic shortening = 56.2 MPa

(ii) Creep of concrete = 79.3 MPa

(iii) Shrinkage of concrete = 60 MPa

(iv) Relaxation of steel = 40 MPa

Solution : 9

Stress at transfer stage

$f_t = -0.090\ \text{N/mm}^2$

$f_b = 14.483\ \text{N/mm}^2$

At final stage

$f_t = 10.97\ \text{N/mm}^2$

$f_b = -0.6241\ \text{N/mm}^2$

■■■■

5 Stair Cases, Earthquake Engg., Water Tanks and Retaining Wall

LEVEL 1 Objective Questions

1.	(a)
2.	(a)
3.	(c)
4.	(d)
5.	(a)

LEVEL 2 Objective Questions

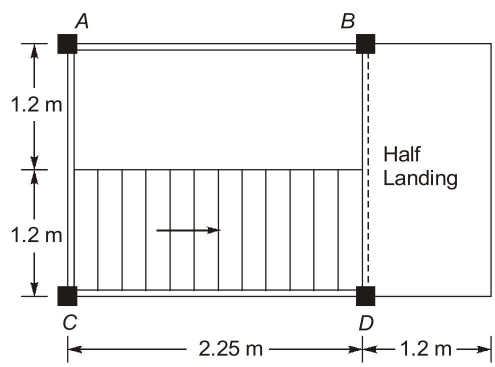
6.	(b)
7.	(c)
8.	(b)
9.	(d)
10.	(c)

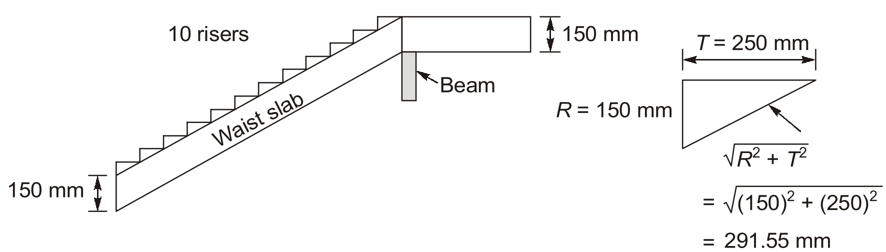
LEVEL 3 Conventional Questions

Solution : 1

σ_{cbt} = Stress in concrete in bending tension = 0.177 MPa
f_{ct} = Direct tensile stress in concrete = 0.38 MPa
f_{st} = Stress in steel = 37.23 MPa

Solution : 2





Given R = 150 mm and T = 250 mm
Slab thickness in landing region = 150 mm
Waist slab thickness = 150 mm

Loads on going flight on projected plan area

1. Self weight of waise slab $= 25 \times 0.150 \times \dfrac{291.55}{250} = 4.37$ kN/m^2
2. Self weight of steps $= 25 \times \dfrac{1}{2} \times 0.15 \times 4 \times 0.25 = 1.875$ kN/m^2
3. Live loads = 3.0 kN/m^2

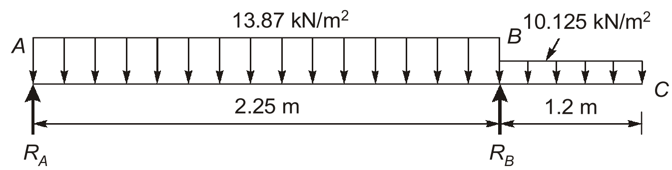
Total load = 9.245 kN/m^2
Factored load = 1.5 × 9.245 kN/m^2 = 13.87 kN/m^2

Loads on landing

1. Self weight of slab @ 25 × 0.150 = 3.75 kN/m^2
2. Live loads = 3 kN/m^2

Factored load = 6.75 × 1.5 = 10.125 kN/m^2

Design moment:



For portion *BC*;
x measured from *C* ($0 < x < 1.2$)

$$M_x = \frac{-10.125 \times x^2}{2} = -5.0625x^2$$

At x = 1.2 m $\qquad M_B$ = 7.29 kNm/m

$\Sigma M_A = 0 \qquad R_B \times 2.25 = 13.87 \times \frac{(2.25)^2}{2} + 10.125 \times 1.2 \times (2.25 + 0.6)$

$\Rightarrow \qquad R_B$ = 31 kN/m

$\therefore \qquad R_A$ = 13.87 × 2.25 + 10.125 × 1.2 – 31 = 12.36 kN/m

Design bending moment between A and B

$$M_x = 12.36x - \frac{13.87x^2}{2}$$

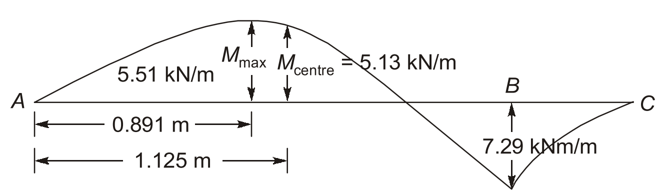
For M_x to be maximum

$$\therefore \qquad \frac{dMx}{dx} = 0$$

$\Rightarrow \qquad 12.36 - 13.87x = 0 \quad \Rightarrow \quad x = 0.891$ m

$$M_{max} = 12.36 \times 0.891 - \frac{13.87 \times (0.891)^2}{2} = 5.507 \text{ kNm/m}$$

At x = 1.125 m $\qquad M_{centre}$ = 5.13 kN/m



Bending moment diagram

Solution : 3

Given : F_t = 60 kN/m and M = 7.5 kNm/m; σ_{td} = 1.5 MPa, σ_{tb} = 2 MPa and m = 9

Since both F_t and M are acting on the wall of a liquid retaining structure, the design must satisfy.

$$\left(\frac{f_{td}}{\sigma_{td}} + \frac{f_{tb}}{\sigma_{tb}}\right) \leq 1.0 \qquad \text{...(i)}$$

Design constants : σ_{st} = 230 N/mm^2, σ_{cbc} = 10 MPa

$$k_b = \frac{m.\sigma_{cbc}}{(\sigma_{st} + m.\sigma_{cbc})} = \frac{9 \times 10}{230 + 9 \times 10} = 0.28$$

$$j = 1 - \frac{k_b}{3} = 0.91$$

Assuming percentage of steel $p_t = 0.4\%$

$$M = A_{st} \times \sigma_{st} \cdot j \cdot d$$

$$\Rightarrow \quad M = \left(\frac{p_t}{100}\right) b \cdot d \cdot \sigma_{st} \cdot j \cdot d$$

$$\Rightarrow \quad d = \left(\frac{M \times 100}{0.4b \times \sigma_{st} \cdot j}\right)^{1/2} = \left(\frac{7.5 \times 10^6 \times 100}{0.4 \times 1000 \times 230 \times 0.91}\right)^{1/2}$$

$$\Rightarrow \quad d = 94.65 \text{ mm}$$

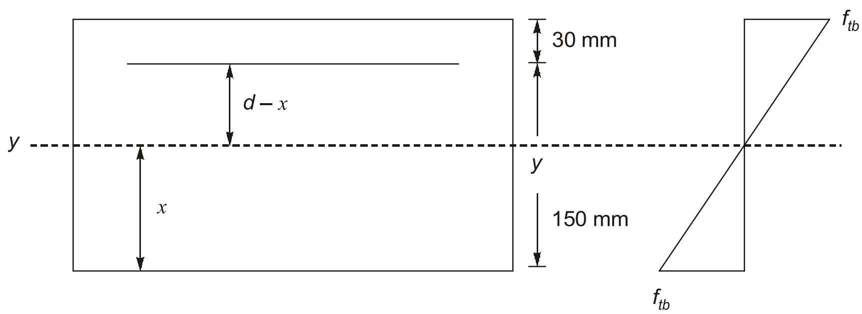
Provide effective depth of 150 mm with 30 mm effective cover, therefore, total depth = 180 mm

$$A_{st_1} = \frac{0.4}{100} \times 180 \times 1000 = 720 \text{ mm}^2$$

$$A_{st_2} \text{ for } F_t = \frac{60 \times 10^3}{230} = 260.87 \text{ mm}^2$$

$$A_{st} = A_{st_1} + A_{st_2} = 980.87 \text{ mm}^2$$

Check:



$$x = \frac{A_1x_1 + A_2x_2}{A_1 + A_2} = \frac{(1000 \times 180 \times 90) + (9-1) \times 980.87 \times 150}{(1000 \times 180) + (9-1) \times 980.87}$$

$$= 92.51 \text{ mm}$$

$$\therefore \quad D - x = 87.49 \text{ mm}, \; d - x = 57.49 \text{ mm}$$

$$I_{yy} = \left[\frac{1000 \times 180^3}{12} + (9-1) \times 980.87 \times (57.49)^2 + (1000 \times 180 \times 2.51^2)\right]$$

$$\Rightarrow \quad I_{yy} = 513069006.3 \text{ mm}^4$$

$$A_{eq} = (1000 \times 180) + (9-1) \times 980.87 = 187846.96 \text{ mm}^2$$

$$\sigma_{tb} = \frac{M}{I_{yy}} \times (D - x) = \frac{7.5 \times 10^6}{513069006.3} \times (87.49) = 1.28 \text{ Mpa}$$

$$\sigma_{td} = \frac{60 \times 10^3}{187846.96} = 0.32 \text{ Mpa}$$

Applying equation (i) :

$$\frac{1.28}{2} + \frac{0.32}{1.5} = 0.85 < 1.0$$

Hence design is O.K.

Reinforcement details:

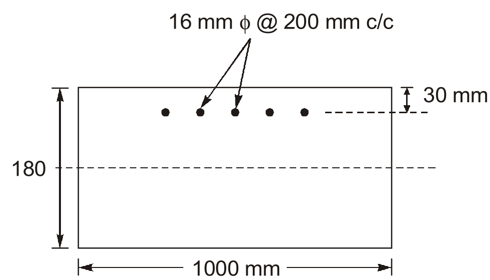
$$A_{st} = 980.87 \text{ mm}^2$$

Using 16 mm ϕ bars,

$$\text{spacing} = \frac{1000}{980.87} \times \frac{\pi}{4} \times 16^2$$

$$= 204.98 \text{ mm}$$

$\therefore$ Provide 16 mm ϕ bars at 200 mm c/c.



Solution : 4

$$\text{Effective length} = 1.2 + (8 \times 0.25) + 1 - 0.23 = 3.97 \text{ m}$$

$$\text{Assuming waist slab} = 200 \text{ mm}$$

Dead weight of horizontal area,

$$w_1 = w' \times \frac{\sqrt{R^2 + T^2}}{T}$$

$$w_1 = (0.2 \times 25) \times \frac{\sqrt{250^2 + 150^2}}{250} = 5.83 \text{ kN/m}^2$$

Dead weight of steps,

$$w_2 = \frac{R \text{ (in mm)}}{2 \times 1000} \times 25 \times 4 \times \left(\frac{250}{1000}\right)$$

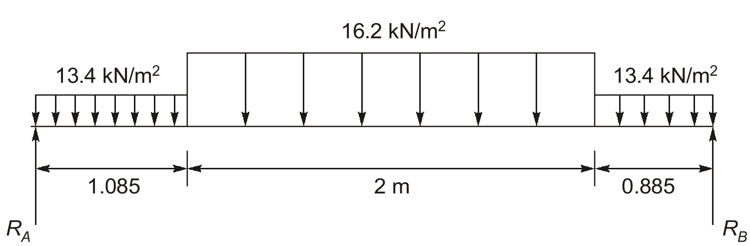
$$w_2 = 1.875 \simeq 1.87 \text{ kN/m}^2$$

Assuming live load and finishing load = 3.1 kN/m^2

$$\text{Total factored load} = 1.5\,(5.83 + 1.87 + 3.1) = 16.2 \text{ kN/m}^2$$

$$\text{Factored load on landings} = 16.2 - (1.5 \times 1.87) = 13.395 \simeq 13.4 \text{ kN/m}^2$$

Loading diagram :



$$R_A + R_B = 13.4 \times (1.085 + 0.885) + 16.2 \times 2$$

$$= 58.8 \text{ kN}$$

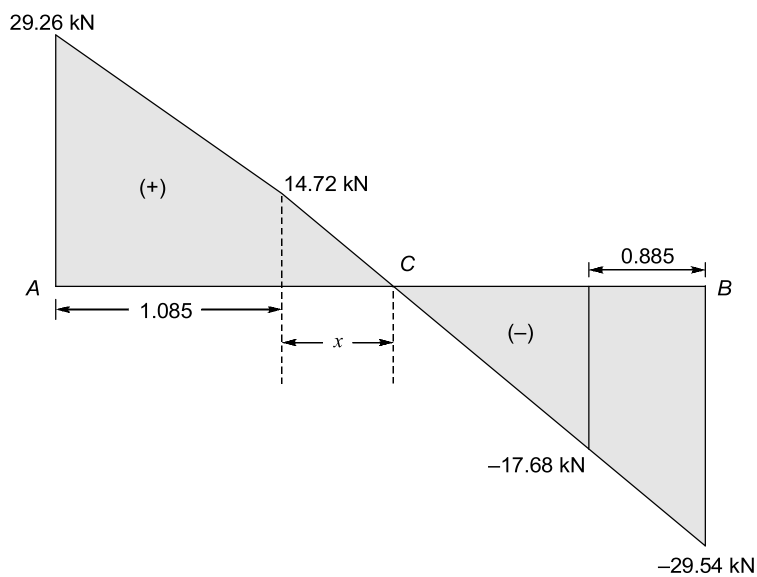
$$\Sigma M_B = 0$$

$\Rightarrow$

$$R_A \times 3.97 = [\{(13.4 \times 1.085) \times 3.4275\} + \{(16.2 \times 2) \times 1.885\} + \left\{13.4 \times \frac{0.885^2}{2}\right\}]$$

$$\therefore \qquad R_A = 29.26 \text{ kN}$$
$$R_B = 29.54 \text{ kN}$$

Shear force diagram:



$$\therefore \quad \text{Critical shear force } V_u = 29.54 \text{ kN}$$

Point of maximum bending moment is point of zero shear force.

$$\therefore \qquad 14.72 - 16.2\,x = 0$$

$$\Rightarrow \qquad x = \frac{14.72}{16.2} = 0.91 \text{ m}$$

$$\therefore \quad \text{Location of point of maximum B.M.} = 1.085 + 0.91$$
$$= 1.995 \text{ m from } A$$

$$\text{Max. B.M.} = \left[R_A \times 1.995 - \left\{(13.4 \times 1.085) \times \left(\frac{1.085}{2} + 0.91\right)\right\} - \left\{16.2 \times \frac{0.91^2}{2}\right\}\right]$$
$$= 30.55 \text{ kN-m}$$

Depth required : $d = \sqrt{\dfrac{M_u}{0.138 f_{ck} B}}$ {Using M20 and Fe415}

$$\Rightarrow \qquad d = \sqrt{\frac{30.55 \times 10^6}{0.138 \times 20 \times 10^3}} = 105.21 \text{ mm}$$

Adopt d = 110 mm and effective cover of 30 mm.

$\therefore$ Total depth = 140 mm

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

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 30.55 \times 10^6}{20 \times 1000 \times 110^2}}\right] 1000 \times 110$$

$$= 934.25 \text{ mm}^2$$

Using 12 mm ϕ bars, number of bars in 1.5 m width

$$= 1.5 \times \frac{934.25}{\frac{\pi}{4} \times 12^2} = 12.39, \text{ say } \mathbf{13}$$

$$\text{Spacing} = \frac{1500}{13} = 115.38 \text{ mm, say } 110 \text{ mm}$$

∴ Provide 12 mm ϕ at 110 mm c/c (13 nos)

Distribution reinforcement, $A_{sd} = \frac{0.12}{100} \times 1000 \times 140$

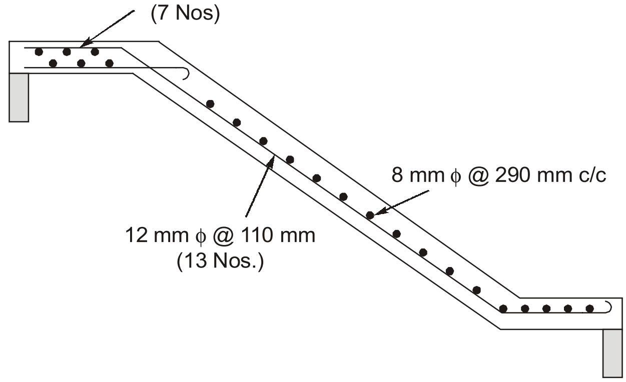
$$= 168 \text{ mm}^2$$

$$\text{Using 8 mm ϕ at spacing} = \frac{1000}{168} \times \frac{\pi}{4} \times 8^2 = 299.2 \text{ mm}$$

∴ Provide 8 mm ϕ at 290 mm c/c.

$$\text{Nominal shear stress} = \frac{V_u}{Bd} = \frac{29.54 \times 10^3}{1000 \times 110}$$

$$\tau_v = 0.27 \text{ N/mm}^2$$



Solution : 5

Given : 14 steps; T = 300 mm; R = 180 mm; 2 Landings = 1.25 m

Design constants: M20lFe415 used; $Q = 0.138 f_{ck}$

Loading on flight:

Let us assume bearing of landing slab in wall be 160 mm

$$\text{Effective span} = 1.25 + \frac{13 \times 300}{1000} + 0.16 + 1.25 = 6.56 \text{ m}$$

Let us assume waist slab = 280 mm {Assuming @ rate of 40 mm to 50 mm per m span}

Weight of slab on slope, $w' = 0.28 \times 1 \times 1 \times 25 = 7 \text{ kN/m}^2$

$$\text{Dead weight of horizontal area, } w_1 = \frac{w'\sqrt{R^2+T^2}}{T}$$

$$w_1 = \frac{7\sqrt{180^2+300^2}}{300} = 8.16 \text{ kN/m}^2$$

$$\text{Dead weight of steps} = \frac{180}{2\times 1000}\times 25 = 2.25 \text{ kN/m}^2$$

$$= \left\{\frac{1}{2}\times 0.18\times\left(\frac{1000}{250}\right)\times 0.25\times 25\right\}$$

$$\text{Live load} = 5 \text{ kN/m}^2$$

$$\text{Assuming load due to finishing, etc} = 0.1 \text{ kN/m}^2$$

$$\text{Total load } w = 8.16 + 2.25 + 5 + 0.1$$

$$= 15.51 \text{ kN/m}^2$$

$$\text{Load } w \text{ on landing} = 15.51 - 2.25$$

$$= 13.26 \text{ kN/m}^2$$

But assuming uniform weight for design.

Design of Waist slab:

$$\text{Factored Bending moment} = 1.5\frac{wl^2}{8} = \frac{1.5\times 15.51\times 6.56^2}{8} = 125.15 \text{ kN-m}$$

$$\text{Depth required, } d = \sqrt{\frac{M_u}{Q.B}} = \sqrt{\frac{125.15\times 10^6}{0.138\times 20\times 1000}} = 212.94$$

Adopt effective depth, $d = 230$ mm and effective cover of 30 mm to get total depth = 260 mm

Reinforcement:

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

$$= \frac{0.5\times 20}{415}\left[1-\sqrt{1-\frac{4.6\times 125.15\times 10^6}{20\times 1000\times 230^2}}\right]1000\times 230$$

$$= 1800.2 \text{ mm}^2$$

Using 16 mm ϕ bars, no. of bars needed in 1.4 m

$$\text{Width} = 1.4\times\frac{1800.2}{\frac{\pi}{4}\times 16^2} = 12.53, \text{ say } 13$$

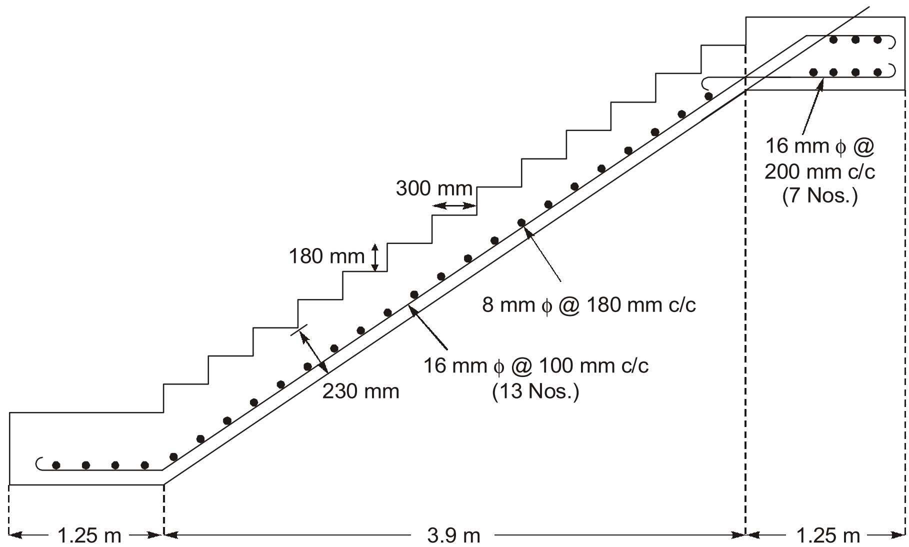
$$\text{Spacing of bars} = \frac{1400}{13} = 107.7 \text{ mm, say } 100 \text{ mm}$$

Distribution reinforcement :

$$A_{sd} = \frac{0.12}{100}\times 1000\times 230 = 276 \text{ mm}^2$$

$$\text{Using 8 mm } \phi \text{ at spacing} = \frac{1000}{276}\times\frac{\pi}{4}\times 8^2 = 182.12 \text{ mm}$$

∴ Provide 8 mm ϕ at 180 mm c/c spacing.



Solution : 6

$(FOS)_{over\text{-}turning} = 2.21$

$(FOS)_{sliding} = 0.971$

Solution : 7

Base pressures = 108.96 kN/m² and 3.37 kN/m²

Solution : 8

$H = 4.35$ m, $k_a = \frac{1}{3}$, $\gamma = 18$ kN/m³

Consider 1 m length of retaining wall.

Maximum B.M. at bottom of stem $= \frac{1}{6} \cdot k_a \cdot \gamma \cdot H^3$

$$M = \frac{1}{6} \times \frac{1}{3} \times 18 \times 4.35^3 = 82.31 \text{ kNm}$$

Factored B.M., $M_u = 1.5\text{ M} = 123.48$ kNm

Effective depth required, $d = \sqrt{\frac{M_u}{0.138 f_{ck} \cdot B}}$

$$d = \sqrt{\frac{123.48 \times 10^6}{0.138 \times 20 \times 1000}} = 211.52 \text{ mm}$$

Let d = 240 mm with effective cover of 60 mm, therefore, total depth = 300 mm

Let us keep the same thickness throughout.

Reinforcement in stem:

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

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 123.48 \times 10^6}{20 \times 1000 \times 240^2}}\right] 1000 \times 240$$

$$= 1665.57 \text{ mm}^2$$

$$\text{Use 16 mm } \phi \text{ bar at spacing} = \frac{1000}{1665.57} \times \frac{\pi}{4} \times 16^2 = 120.71 \text{ mm, say 120 mm}$$

∴ Provide 16 mm ϕ bars @ 120 mm c/c distance. Continue alternate bars to toe slab for distance 45 ϕ = 720 mm.

Curtailment of 50% A_{st} can be provided for h = 0.79 H.

However bars should be extended for 12ϕ or d whichever is more.

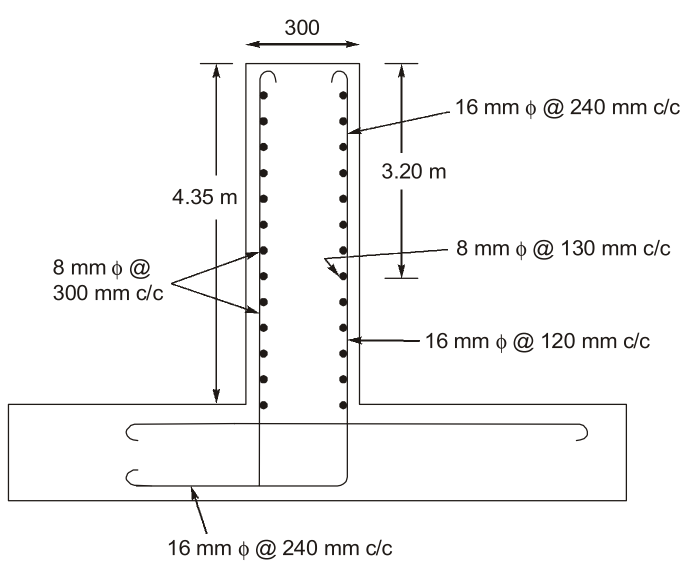
∴ Depth of curtailment from top = 0.79 × 4.35 – 0.240 ≃ 3.20 m

$$\text{Distribution reinforcement} = \frac{0.12}{100} \times 1000 \times 300 = 360 \text{ mm}^2$$

$$\text{Using 8 mm } \phi \text{ at spacing} = \frac{1000}{36} \times \frac{\pi}{4} \times 8^2 = 139.6 \text{ mm}$$

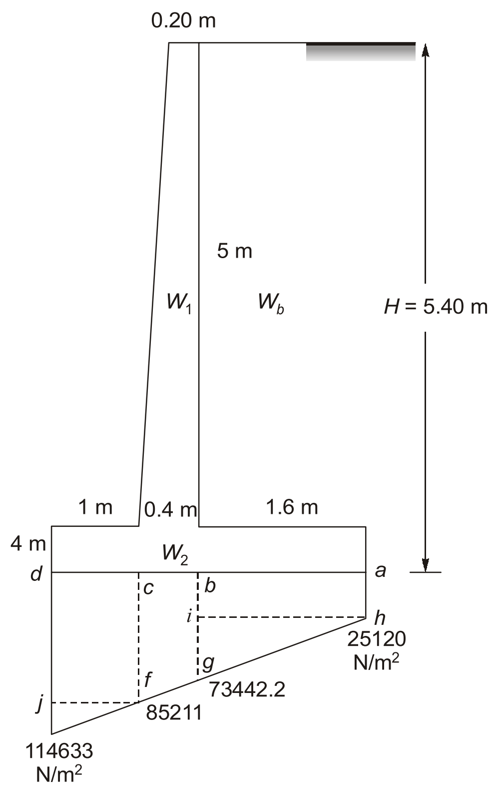
∴ Provide 8 mm ϕ @ 130 mm c/c at inner face of wall along length.

Temperature reinforcement : Provide 8 mm ϕ @ 300 mm c/c both ways on outer face.



Solution : 9

Wall proportions.



Thickness of the stem of the top = 200 mm

Thickness of the stem at the bottom

Consider one metre run of the wall.

Maximum bending moment per metre run of the wall

$$= M = C_p \frac{wh^3}{6} = \frac{1}{3} \times 18000 \times \frac{5^3}{6} = 125000 \text{ Nm}$$

Ultimate moment, $M_u = 1.5 \times 125000 = 187500$ Nm

$0.138 f_{ck} bd^2 = 0.138 \times 20 \times 1000\, d^2 = 187500 \times 10^3$

$d = 261$ mm

Effective cover to stem reinforcement = 40 mm

Total thickness of stem = 261 + 40 = 301 mm

The thickness may be increased by 30% to 35% for an economical design.

Provide a thickness of 400 mm at the bottom of the stem.

The base slab also will be made 400 mm thick.

Total height of the wall = $H = 5 + 0.40 = 5.40$ m

Width of the base slab, $b = 0.5$ H to 0.6 H

$0.5\text{ H} = 0.5 \times 5.4 = 2.70$ m

$$0.6\,H = 0.6 \times 5.4 = 3.24 \text{ m}$$

Provide a base width of 3 m

Toe projection This may be made about one-third the base width.

Provide a toe projection of 1 m Stability calculations

See table below for stability calculations

Load due to	Magnitude of the load (N)	Distance from a (m)	Moment about a (Nm)
W_1 0.2 × 5 × 25000	25000	1.7	42500
$\frac{0.2 \times 5}{2} \times 25000$	12500	$\frac{28}{15}$	23333.33
W_2 3 × 5 × 25000	30000	1.5	45000
W_b 1.6 × 5 × 18000	144000	0.8	115200
Moment of lateral pressure			
$C_p w \frac{H^3}{6} = \frac{1}{3} \times 18000 \times \frac{5.4^3}{6}$			157464
Total	211500		383497.33

Distance of the point of application of the resultant force from the heel end *a*,

$$= Z = \frac{383497.33}{211500} = 1.813 \text{ m}$$

$\therefore$ Eccentricity $e = Z - \frac{b}{2} = 1.813 - 1.50 = 0.313$ m

$$\frac{b}{6} = \frac{3}{6} = 0.5 \quad \therefore \quad e < \frac{b}{6}$$

Extreme pressure intensity at the base

$$= \frac{W}{b}\left[1 \pm \frac{6e}{b}\right] = \frac{211500}{3}\left[1 \pm \frac{6 \times 0.313}{3}\right] \text{N/m}^2$$

$\therefore$ $p_{max} = 114633 \text{ N/m}^2$ and $p_{min} = 26367 \text{N/m}^2$

Safe bearing capacity of the soil = 200 kN/m^2 = 200000 N/m^2

Design of the stem

$$\text{Maximum B.M} = M = 125000 \text{ Nm}$$

$$\text{Ultimate moment} = M_u = 1.5 \times 125000 = 187500 \text{ Nm}$$

$$\text{Effective depth} = d = 400 - 40 = 360 \text{ mm}$$

$$\frac{M_u}{bd^2} = \frac{18750 \times 10^3}{1000 \times 360^2} = 1.447$$

Percentage of steel $p_t = 50\left[\dfrac{1 - \sqrt{1 - \dfrac{4.6 \times 1.447}{20}}}{\dfrac{415}{20}}\right] = 0.441\%$

$$A_{st} = \frac{0.441}{100}(1000 \times 360) = 1558 \text{ mm}^2$$

$$\text{Spacing of 16 mm diameter bars} = \frac{201 \times 1000}{1588} = 126 \text{ mm}$$

Provide 16 mm ϕ bars @ 120 mm c/c

$$\text{Distribution steel} = \frac{0.12}{100}(1000 \times 400) = 480 \text{ mm}^2$$

$$\text{Spacing of 8 mm diameter bars} = \frac{50 \times 1000}{480} = 104 \text{ mm} \;\; \text{say 100 mm c/c}$$

If the distribution steel is provided near both the faces, the spacing of 8 mm diameter bars will be 200 mm near each face.

Design of the toe slab

The bending moment calculations for a 1 metre wide strip of the toe slab are shown in the table below.

B.M. Calculations for a 1 metre wide strip of the toe slab

Load due to	Magnitude of the load (N)	Distance from *c* (m)	Moment about *c* (Nm)
Upward pressure			
c d j f 85211 × 1	85211	0.5	42605.50
j f e $\frac{1}{2}$ × 1 × 29422	14711	$\frac{2}{3}$	9807.33
			52412.83
Deduct for self weight of toe slab			
1 × 0.40 × 25000	10000	0.5	5000
B.M. for toe slab			47412.83

$$\text{B.M. for toe slab} = M = 47412.83 \text{ Nm}$$

$$\text{Ultimate moment} = M_u = 1.5 \times 47412.83 = 71119.245 \text{ Nm}$$

$$\text{Effective depth} = 400 - 60 = 340 \text{ mm (For base slab effective cover = 60 mm)}$$

$$\frac{M_u}{bd^2} = \frac{71119.245 \times 10^3}{1000 \times 340^2} = 0.615$$

Percentage of steel

$$p_t = 50\left[\frac{1 - \sqrt{1 - \frac{4.6}{20} \times 0.615}}{\frac{415}{20}}\right] = 0.177\%$$

Minimum percentage of steel when Fe 415 is used = 0.2%

$$A_{st} = \frac{0.2}{100} \times 1000 \times 340 = 680 \text{ mm}^2$$

$$\text{Spacing of 12 mm diameter bars} = \frac{113 \times 1000}{680} = 166 \text{ mm}$$

Provide 12 mm ϕ bars @ 160 mm c/c.

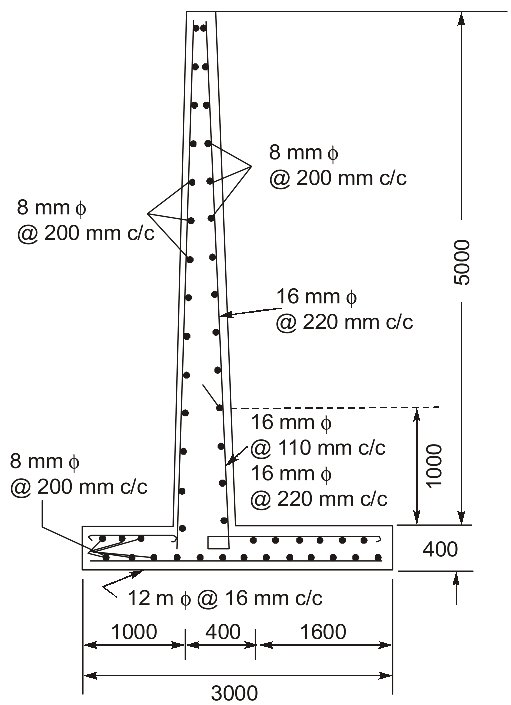
Design of the Heel Slab

The bending moment calculations for a 1 metre wide strip of the heel slab are shown in the table below

B.M. Calculations for a 1 metre wide strip of the heel slab

Load due to	Magnitude of the load (N)	Distance from *c* (m)	Moment about *c* (Nm)
Backing 1.6 × 5 × 18000 DL of heel slab 1.6 × 0.4 × 25000	144000 16000	0.8 0.8	115200 12800
			128000
Deduct for upward pressure *a b ih* 26367 × 1.6 *i g h* $\frac{1}{2}$ × 1.6 × 47075.2	42187.2 37660.16	0.8 $\frac{1.6}{3}$	33749.76 20085.42
Total deduction			53835.18
B.M. for heel slab			74164.82

B.M.for the heel slab = M = 74164.82 Nm



Ultimate moment M_u = 1.5 × 74164.82 = 111247.23 Nm

$$\frac{M_u}{bd^2} = \frac{111247.23 \times 10^3}{1000 \times 360^2} = 0.858$$

Percentage of steel,

$$p_t = 50\left[\frac{1-\sqrt{1-\frac{4.6 \times 0.858}{20}}}{\frac{415}{20}}\right] = 0.251\%$$

$$A_{st} = \frac{0.251}{100}(1000 \times 360) = 904 \text{ mm}^2$$

Spacing of 16 mm diameter bars = $\frac{201 \times 1000}{904}$ = 222 mm say 220 mm c/c

It is convenient to match the spacing of reinforcements of stem and heel slab.

Accordingly, we will provide,

16mm ϕ bars @ 220 mm c/c for the heel slab, and

16 mm ϕ bars @110 mm c/c for the stem.

Check for Sliding

Total horizontal pressure force per metre run of the wall

$$= P = C_p \frac{wH^2}{2} = \frac{1}{3} \times 18000 \times \frac{5.4^2}{2} = 87480\, N$$

Taking μ = 0.65,

Limiting friction = $\mu W = 0.65 \times 211500 = 137475$ N

Factor of safety against sliding = $\frac{\mu W}{p} = \frac{137475}{87480} = 1.57$

Solution : 10

Wall proportions

Thickness of the stem at the top = 200 mm

Thickness of the stem ot the bottom

Consider one metre run of the wall

Maximum bending moment per metre run of the wall = M

$$= C_p \frac{wh^3}{6} = \frac{1}{3} \times 18500 \times \frac{5.5^3}{6} = 170996.53 \text{ Nm}$$

Ultimate moment M_u = 1.5 × 170996.53 = 256494.79 Nm

$0.149 f_{ck} bd^2 = 0.149 \times 20 \times 1000\, d^2$

$= 256494.79 \times 10^3$

d = 293.4 mm

Effective cover to stem reinforcement = 40 mm

Overall thickness of the stem

= 293.4 + 40 = 333.4 mm

The thickness may be increased by 30% to 35% for an economical design.

Provide a thickness of 450 mm at the bottom of the stem.

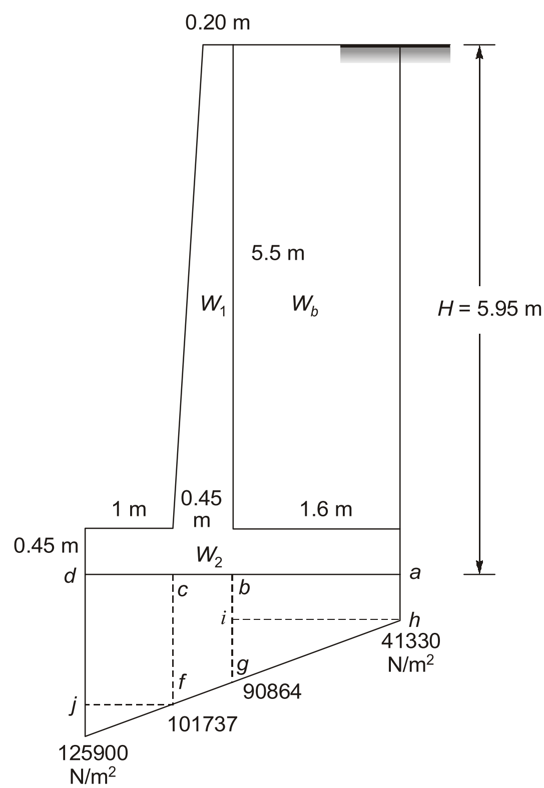
The base slab also will be made 450 mm thick.

Total height of the wall = H = 5.5 + 0.45 = 5.95 m

Width of the base slab, b = 0.5 H to 0.6 H

0.5 H = 0.5 × 5.95 = 2.975 m

0.6 × 5.95 = 3.570 m



Provide a base width of 3.50 m

$$\text{Toe projection} = \text{About one – third the base width}$$

$$= \frac{3.50}{3} = 1.17 \text{ m}$$

Provide a toe projection of 1 m.

Stability Calculation

See table below for Stability Calculations

Load due to	Magnitude of the load (N)	Distance from a (m)	Moment about a (Nm)
W_1 0.2 × 5.5 × 25000	27500	2.15	59125
$\frac{1}{2}$ × 0.25 × 5.5 × 25000	17187.5	$\frac{7}{3}$	40104.17
W_2 3.5 × 0.45 × 25000	39375	1.75	68906.25
W_b 2.05 × 5.5 × 18500	208587.5	1.025	213802.19
Moment of lateral pressure $C_p \frac{wH^3}{6} = \frac{1}{3} \times 18500 \times \frac{5.95^3}{6}$			216496.12
Total	292650		598433.73

Distance of the point of application of the resultant force from the heel end a

$$= z = \frac{598433.73}{292650} = 2.045 \text{ m}$$

Eccentricity

$$e = z - \frac{b}{2} = 2.045 - 1.750 = 0.295$$

$$\frac{b}{6} = \frac{3.5}{6} = 0.583$$

$$\therefore \quad e < \frac{b}{6}$$

Extreme pressure intensity at the base

$$\frac{W}{b}\left[1 \pm \frac{6e}{b}\right] = \frac{292650}{3.5}\left[1 \pm \frac{6 \times 0.295}{3.5}\right] \text{N/m}^2$$

$$P_{max} = 25900 \text{ N/m}^2 \text{ and } p_{min} = 41330 \text{ N/m}^2$$

Safe bearing capacity of the soil = 200 kN/m^2 = 200000 N/m^2

Design of the stem

$$\text{Maximum B.M} = M = 170996.53 \text{ Nm}$$

$$\text{Ultimate moment} = M_u = 1.5 \times 170996.53 = 256494.79 \text{ Nm}$$

Effective depth

$$d = 450 - 40 = 410 \text{ mm}$$

$$\frac{M_u}{bd^2} = \frac{256494.79 \times 10^3}{1000 \times 410^2} = 1.526$$

Percentage of steel $$P_t = 50\left[\frac{1-\sqrt{1-\frac{4.6\times1.526}{20}}}{\frac{250}{20}}\right] = 0.778\%$$

$$A_{st} = \frac{0.778}{100}\times(1000\times310) = 2412\text{ mm}^2$$

$$\text{Spacing of 18 mm diameter bars} = \frac{254\times1000}{2412} = 105\text{ mm}$$

Provide 18 mm ϕ bars @ 100 mm c/c

$$\text{Distribution steel} = \frac{0.15}{100}\times1000\times450 = 675\text{ mm}^2$$

$$\text{Spacing of 8 mm diameter bars} = \frac{50\times1000}{675} = 74\text{ mm say 70 mm}$$

Provide 8 mm ϕ bars @ 140 mm c/c near each face.

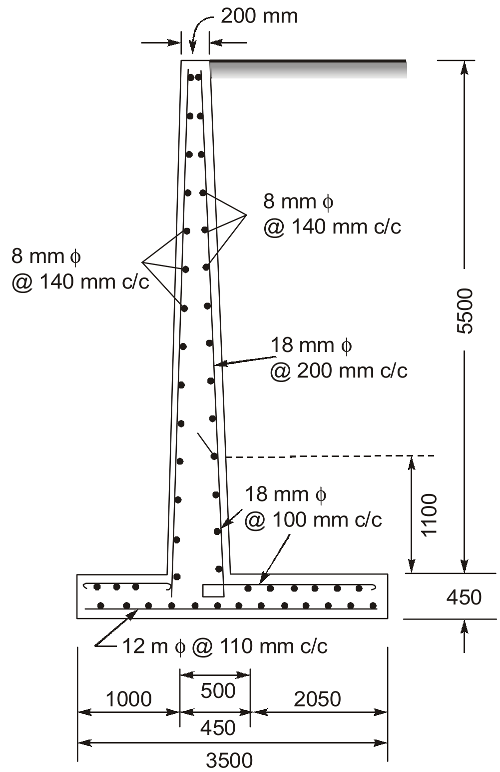
Design of the toe slab

The bending moment calculations for a 1 metre wide strip of the toe slab are shown in the table below.

B.M. Calculations for a 1 metre wide strip of the toe slab

Load due to	**Magnitude of the load (N)**	**Distance from *c* (m)**	**Moment about *c* (Nm)**
Upward pressure *c d j f* 101737 × 1101737 $j f e \frac{1}{2} \times 1 \times 24163$	 12081.5	 0.5 $\frac{2}{3}$	 50868.50 8054.33
			58922.83
Deduct for self weight of toe slab 1 × 0.45 × 25000	11250	0.5	5625
B.M. for toe slab			53297.83

B.M.for the toe slab = M = 53287.83 Nm



Ultimate moment $M_u = 1.5 \times 53287.83 = 79946.745$ Nm

Effective depth $= d = 450 - 60 = 390$ mm

[Effective cover to reinforcement for base slab = 60 mm]

$$\frac{M_u}{bd^2} = \frac{79946.745 \times 10^3}{1000 \times 390^2} = 0.526$$

Percentage of steel,

$$p_t = 50\left[\frac{1-\sqrt{1-\frac{4.6 \times 0.526}{20}}}{\frac{250}{20}}\right] = 0.25\%$$

$$A_{st} = \frac{0.25}{100}(1000 \times 360) = 975 \text{ mm}^2$$

Spacing of 12 mm diameter bars

$$= \frac{113 \times 1000}{975} = 115 \text{ mm}$$

Provide 12 mm ϕ bars @ 110 mm c/c.

Design of the heel slab

The bending moment calculations for a 1 metre wide strip of the heel slab are shown in the table below.

B.M. Calculations for a 1 metre wide strip of the heel slab

Load due to	Magnitude of the load (N)	Distance from *c* (m)	Moment about *c* (Nm)
Backing 2.05 × 5.5 × 18500 DL of heel slab 2.05 × 0.45 × 25000	208587.5 23062.5	1.025 1.025	213802.19 23639.06
			237441.25
Deduct for upward pressure *a b ih* 41330 × 2.05 *i g h* $\frac{1}{2}$ × 2.05 × 49534	84726.5 50772.35	1.025 $\frac{2.05}{3}$	86844.66 34694.44
Total deduction			121539.10
B.M. for heel slab			115902.15

B.M for the heel slab = 115902.15 Nm

Ultimate moment $M_u = 1.5 \times 115902.15 = 173853.23$ Nm

$$\frac{M_u}{bd^2} = \frac{173853.23 \times 10^3}{1000 \times 390^2} = 1.143$$

Percentage of steel
$$p_1 = 50\left[\frac{1-\sqrt{1-\frac{4.6 \times 1.143}{20}}}{\frac{250}{20}}\right] = 0.566\%$$

$$A_{st} = \frac{0.566}{100}(1000 \times 390) = 2207 \text{ mm}^2$$

Spacing of 18 mm diameter bars $= \frac{254 \times 1000}{2207} = 115$ mm

In order the spacing of these bars may match with the spacing of stem reinforcement, we will therefore

Provide 18 mm ϕ bars @ 100 mm c/c

Check for sliding

Total horizontal pressure force per metre run ol the wall

$$= P = C_p \frac{wH^2}{2} = \frac{1}{3} \times 18500 \times \frac{5.95^2}{2} = 109157.71$$

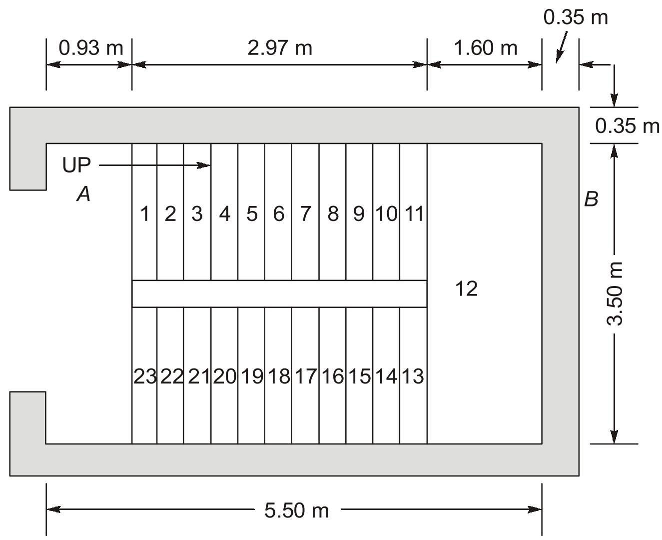
Taking $\mu = 0.65$,

Limiting friction $= \mu W = 0.65 \times 292650 = 190222.5\ N$

Factor of safety against sliding $= \frac{\mu W}{P} = \frac{190222.5}{109157.71} = 1.74$

Solution : 11

It is proposed to provide two flights for the stair way.



Hence, the height of each flight $= \dfrac{3.75}{2} = 1.875$ metre

Assuming 150 mms risers number of risers required

$$= \frac{187.5}{15} = 12$$

Hence, the actual rise of each riser $= \dfrac{1875}{12} = 156.2$ mm

Number of treads in each flight = number of risers – 1

= 12 – 1 = 11 treads.

Let the width of the stair be 1600 mm.

Let the tread of the steps be 270 mm.

Figure shows the arrangement of the stairs in plan.

Design of the flight *AB*. Let the bearing for the flight be 150 mm.

Effective Horizontal Span $= 2.97 + 1.60 + \dfrac{0.15}{2} = 4.645$ metres

Let the thickness of the waist be 220 mm. (This can be assumed at 40 mm to 50 mm per metre run of horizontal span).

Loads

Dead load of 220 mm waist = 25×220 = 5500 N/metre^2

Ceiling finish (12.5 mm) thick = 24×12.5 = 300 N/m^2

Total = 5800 N/m^2

Corresponding load per sq metre on plan

$$= \frac{\sqrt{R^2 + T^2}}{T} \times 5800 = \frac{\sqrt{156.2^2 + 270^2}}{270} \times 5800 \text{ N/m}^2$$

$$= 6700 \text{ N/m}^2$$

Hence the actual load per sqm of plan area will consist of the following

Waist and ceiling $= 6700 \text{ N/m}^2$

Dead load of steps $\left(\frac{156.2}{2} \text{mm average}\right) = 78.1 \times 25 = 1950 \text{ N/m}^2$

Top finish (12.5 mm thick) $= 12.5 \times 24 = 300 \text{ N/m}^2$

Live Load $= 3000 \text{ N/m}^2$

Total $= 11950 \text{ N/m}^2$

Maximum bending moment per metre width of stairs

$$= \frac{11950 \times 4.645^2}{8} = 32229 \text{ Nm}$$

Ultimate moment $M_u = 1.5 \times 32229 = 48343.5 \text{ Nm}$

$0.149\, f_{ck}\, bd^2 = 0.149 \times 20 \times 1000 \times d^2 = 48343.5 \times 10^3$

$\therefore \quad d = 128 \text{ mm}$

Effective depth available, using 12 mm diameter bars

$$= 220 - 21 = 199 \text{ mm}$$

$$\frac{M_u}{bd^2} = \frac{48343.5 \times 10^3}{1000 \times 199^2} = 1.22$$

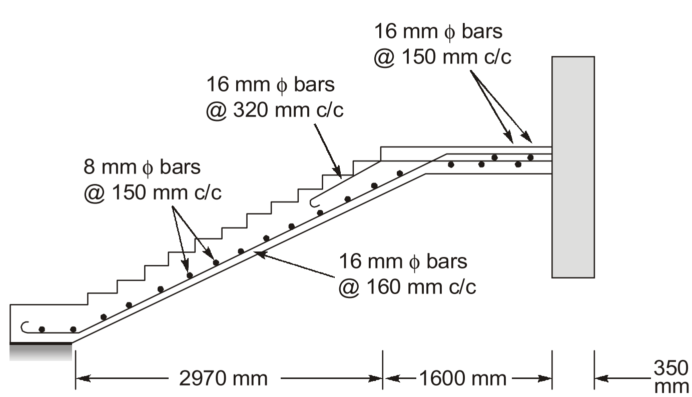
Percentage of steel $= P_t = 50\left[\dfrac{1 - \sqrt{1 - \dfrac{4.6 \times 1.22}{20}}}{\dfrac{250}{20}}\right] = 0.61\%$

$$A_{st} = \frac{0.61}{100} = (1000 \times 200) = 1220 \text{ mm}^2$$

Spacing of 12 mm diameter bars = 92 mm say 90 mm

Or alternatively, Spacing of 16 mm diameter bars $= \dfrac{201 \times 1000}{1220} = 164$ mm say 160 mm

Provide 16 mm ϕ bars @ 160 mm c/c



$$\text{Distribution steel} = \frac{0.15}{100}(1000 \times 220) = 330 \text{ mm}^2$$

$$\text{Spacing of 8 mm diameter bars} = \frac{50 \times 1000}{330} = 151 \text{mm}$$

Provide 8 mm ϕ bars @ 150 mm c/c.

Solution : 12

1. Computation of design constants

For Fe 415 steel, $f_y = 415$ N/mm^2, For M20 concrete, $f_{ck} = 20$ N/mm^2

$$\frac{x_{u,max}}{d} \text{ (For Fe 415 steel)} = \frac{700}{1100 + 0.87 \times 415} = 0.479$$

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

$$= 0.36 \times 20 \times 0.479(1 - 0.416 \times 0.479) = 2.761$$

2. Computation of step dimensions

$R = 150$ mm; $T = 250$ mm; $b = \sqrt{R^2 + T^2} = \sqrt{150^2 + 250^2} = 292$ mm

Let us keep waist thickness = 80 mm

$$D = 80 + \frac{RT}{b} = 80 + \frac{150 \times 250}{292} = 208 \text{ mm}$$

Hence the effective depth of equivalent beam = $D/2$ = 104 mm

Width b = 292 mm; Span L = 1.25 m

3. Computation of loading and B.M. Each step spans horizontally

$$\text{Dead load of each step per meter} = \frac{1}{2} \times \frac{150}{1000} \times \frac{250}{1000} \times 25000 \;\; \Omega \;\; 469 \text{ N/m}$$

$$\text{Dead load of waist slab} = \frac{80 \times 292}{10^6} \times 25000 = 584 \text{ N/m}$$

Load of finishing = 70 N/m (say)

Total = 1123 N/m

Live load @ 3000 N/m^2 = 250/1000 × 3000 × 1 = 750 N/m

Total w = 1873 N/m

$$\therefore \quad w_u = 1.5 \text{ w} = 1.5 \times 1873 \;\Omega\; 2810 \text{ N/m}$$

$$M_u = \frac{w_u L^2}{8} = \frac{2810(1.25)^2}{8} = 548.8 \text{ N-m} = 54.88 \times 10^4 \text{ N-mm}$$

4. Computation of effective depth

$$d = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{54.88 \times 10^4}{2.761 \times 292}} = 26.1 \text{mm}$$

But available d = 104 mm

5. Computation of steel reinforcement

Since available d is more than that required from B.M., we have an under-reinforced section, for which

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

$$= \frac{0.5 \times 20}{415}\left[1 - \sqrt{1 - \frac{4.6 \times 54.88 \times 10^4}{20 \times 292(104)^2}}\right] 292 \times 104 = 14.8 \text{ mm}^2$$

However, provided minimum steel in the form of one bar of 8 mm diameter per step giving A_{st} = 50.3 mm^2. Provide distribution reinforcement in the form of 8 mm φ bars @ 450 mm c/c. The reinforcement is arranged as shown.

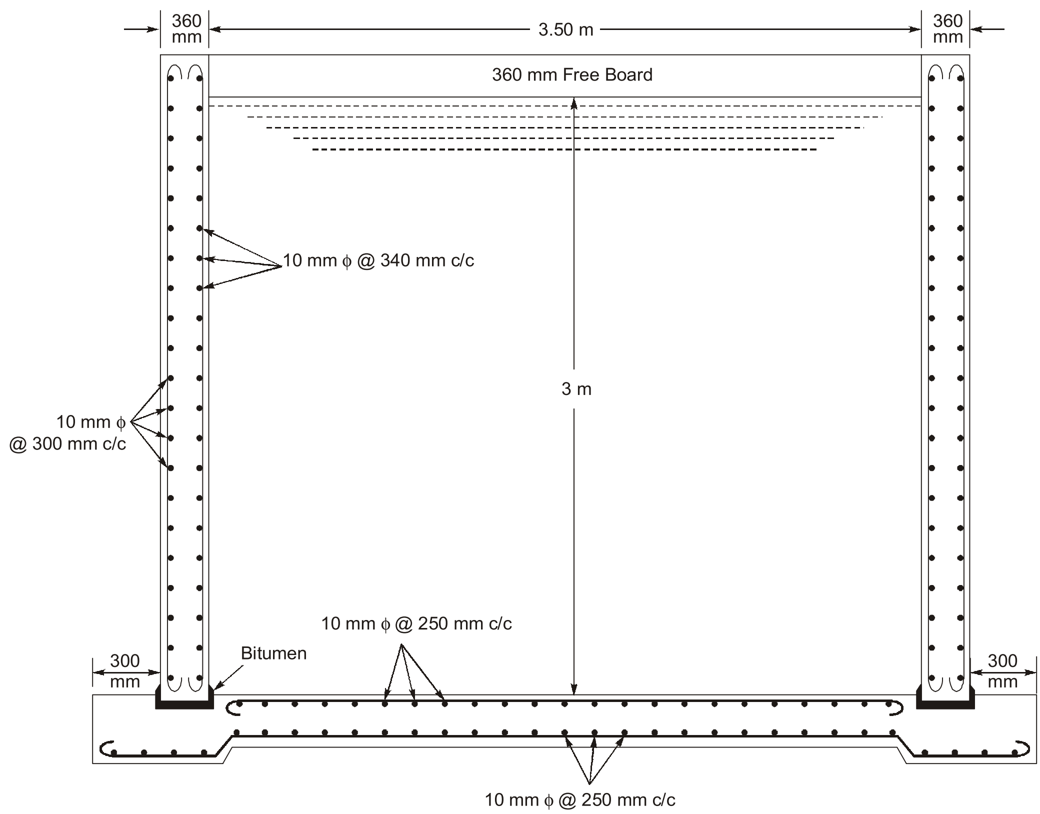
Solution : 13

Thickness of wall. This may not be less than

(i) 150 mm

(ii) 30 mm per m depth + 50 mm = (30 × 30) + 50 = 140 mm

Provide a thickness of 150.



Circular tank with a flexible connection of the wall with the base

Consider the bottom 1 metre height of the wall. Pressure intensity corresponding to the centre of the bottom 1 metre height of wall.

$$= p = wh = 9810 \times 2.5 = 24525 \text{ N/m}^2$$

Hoop tension $= T = \frac{pD}{2} = \frac{24525 \times 3.5}{2} = 42919 \text{ N}$

Steel required for 1 m height $= \frac{42919}{115} = 373.2 \text{ mm}^2$

Minimum steel required $= 0.3\%$ of gross area $= \dfrac{0.3}{100} \times 150 = 450\ \text{mm}^2$

Spacing of 10 mm ϕ pars $= \dfrac{79 \times 1000}{450} = 175$ mm say 170 mm c/c

If the steel is provided near both the faces, the spacing of the bars will be 340 mm c/c.

Let us provide 10 mm ϕ bars @ 300 mm c/c

Actual area of steel provided $= \dfrac{79 \times 1000}{150} = 526.7\ \text{mm}^2$

Check for tensile stress in concrete

Tensile stress in concrete $= \dfrac{T}{A + (m-1)A_{st}}$

$$= \frac{42919}{(1000 \times 150) + (13.33 - 1)526.7} = 0.27\ \text{N/mm}^2$$

Vertical distribution steel This shall be at least 0.3% of gross area

$$= \frac{0.3}{100} \times 150 \times 1000 = 150\ \text{mm}^2 \text{ per m}$$

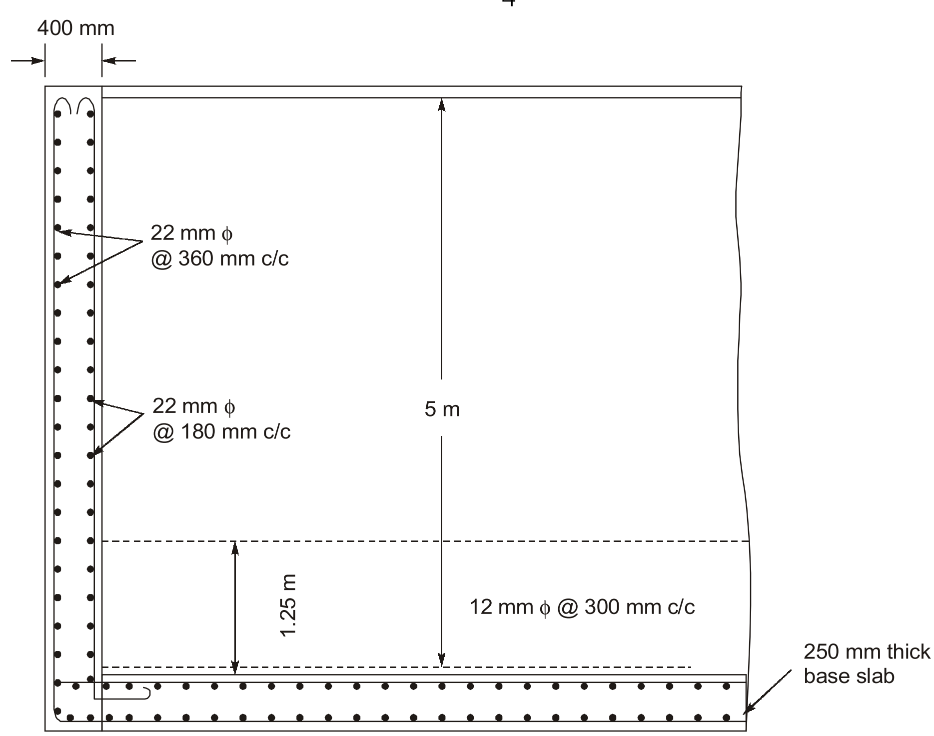
Provide 10 mm ϕ bars @ 300 c/c near each face.

Base slab. This will be a 150 mm thick slab with a top mech and a bottom mesh of steel with 10 mm ϕ bars at 250 mm c/c.

Figure above shows the details of reinforcement.

Solution : 14

It will be assumed that the walls cantilever up from the floor for the bottom $\dfrac{H}{4}$ or 1 metre whichever is greater. In our case the cantilevering height will be $\dfrac{5}{4} = 1.25$ metres, from the base.



Maximum ring tension at this lever per metre height

$$= 9810(5 - 1.25) \times \frac{30}{2} \text{N} = 551813 \text{ N}$$

$$\text{Steel for ring tension} = \frac{551813}{115} = 4799 \text{ mm}^2$$

$$\text{Spacing of 22 mm } \phi \text{ bars} = \frac{380 \times 1000}{4799}$$

Since the reinforcement is provided near both the faces, the spacing of the bars near each face will be 150 mm c/c.

$$\therefore \quad \text{Steel provided per metre height} = \frac{380 \times 1000}{75} = 5067 \text{ mm}^2$$

Thickness of wall. This shall be not less than the following:

(i) 150 mm

(ii) 30 mm metre depth + 50 mm = 30 × 5 + 50 = 200 mm

(iii) Requirement to limit the tensile stress on the equivalent concrete area to 1.2 Nmm^2.

$$1.2 = \frac{551813}{1000t + (13.33 - 1)5067}, \quad \therefore \quad t = 397 \text{ mm}$$

Provide a thickness of 400 mm

$$\text{Distribution steel} = \frac{0.3}{100} \times 400 \times 1000 = 1200 \text{ mm}^2$$

$$\therefore \quad \text{Spacing of 12 mm diameter bars} = \frac{113 \times 1000}{1200} = 94 \text{ mm say 90 mm}$$

Since this steel is provided near both the faces the spacing of 22 mm diameter bars near each face will be 180 mm c/c.

Design of the bottom 1.25 m cantilevering part

$$\text{Water pressure at the bottom} = 9810 \times 5 = 49050 \text{ N/metre}^2$$

∴ Maximum cantilevering bending moment

$$= \frac{1}{2} \times 49050 \times 1.25 \times \frac{1.25}{3} \text{Nm} = 12773 \text{ Nm}$$

If 12 mm ϕ bars be provided at a cover of 25 mm, the effective depth available

$$= 400 - 25 - 22 - 6 = 347 \text{ mm}$$

$$A_{st} = \frac{12773 \times 1000}{115 \times 0.85 \times 347} \text{mm}^2 = 378 \text{ mm}^2$$

Available vertical steel near water face as part of distribution steel

$$= \frac{1200}{2} = 600 \text{ mm}^2$$

The base slab may be made 250 mm thick. Nominal reinforcement of a top mesh and a bottom mesh of steel with 12 mm ϕ @ 300 mm c/c may be provided.

■■■■