Structural Design Calculation of G+11 Storied Office Building at Kolkata Supported on Pile Foundation

A Project Report Submitted in Partial Fulfilment

of the Requirements for the Degree of

BACHELOR OF TECHNOLOGY

IN

CIVIL ENGINEERING

By
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June, 2015

ABSTRACT

Structural engineers are facing the challenge of striving for the most efficient and economical design solution while ensuring that the final design of a building must be serviceable for its intended function, habitable for its occupants and safe over its design life-time. As our country is the fastest growing country across the globe and need of shelter with higher land cost in major cities like Kolkata, Mumbai, Delhi, etc where further horizontal expansion is not much possible due to space shortage, we are left with the solution of vertical expansion.

Engineers, designers and builders are trying to use different materials to their best advantage keeping in view the unique properties of each material. Structurally robust and aesthetically pleasing building are being constructed by combining the best properties of individual material. Meeting specific requirements of large span, building load, soil condition, time, flexibility & economy, etc, at the same time, high rise buildings are best suited solution. Also,Wind & Earthquake (EQ) engineering should be extended to the design of wind & earthquake sensitive tall buildings. This paper discusses the analysis & design procedure adopted for the evaluation of symmetric high rise multi-storey building (G+11) under effect of Wind and EQ. forces.

This report covers the design process in the following order: designing a building layout based on given grid plan, the calculation of the expected loads on the slabs, the design of slab depth and reinforcement, the estimation of beam and column sizes, the calculation of horizontal, vertical, seismic and wind loads along longer and shorter span, the design of beam reinforcement, the design of column reinforcement and finally, the design of pile cap. Additionally, the figures displaying building plan, details of slab reinforcement, staircase, beam, column and pile reinforcement – have been shown separately in A1 drawing sheets at the back of the report. The basic design of the office building includes 145 mm thick slabs throughout, 500 mm x 500 mm square columns and beams of dimension 325 mm x 650 mm, along both longer and shorter spans. The reinforcement is varied throughout the project depending on necessary loads and clauses, tables etc, from IS 456:2000, IS 1893 (part 1) : 2002, IS 875 (part 3): 1987, SP 16.

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APPROVAL SHEET

This project report entitled "**Design of G + 11 storied office building**" by ------------------- ATASI ROY MALAKAR ----------------------------------is approved for the partial fulfillment of the requirement for the degree of **Bachelor of Technology in Civil Engineering**.

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CERTIFICATE

It is certified that the work contained in the project titled " **Design of G + 11 storied office building**" by ATASI ROY MALAKAR has been carried out under my supervision and that this work has not been submitted elsewhere for a degree.

Tanushyama Banerji

Signature

(Asst. Prof. Tanushyama Banerjee)

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May, 2015

ACKNOWLEDGEMENT

This project document has been prepared by the inspiration received from Professor Tanushyama Banerjee of the Department of Civil Engineering, Calcutta Institute of Engineering and Management, under West Bengal University of Technology, who was also my project guide and supervisor. I acknowledge her for providing consistent support and guidance throughout the making of this project document.

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PROJECT

1. Structural Design calculation of G+11 storied office building at Kolkata supported on pile foundation.

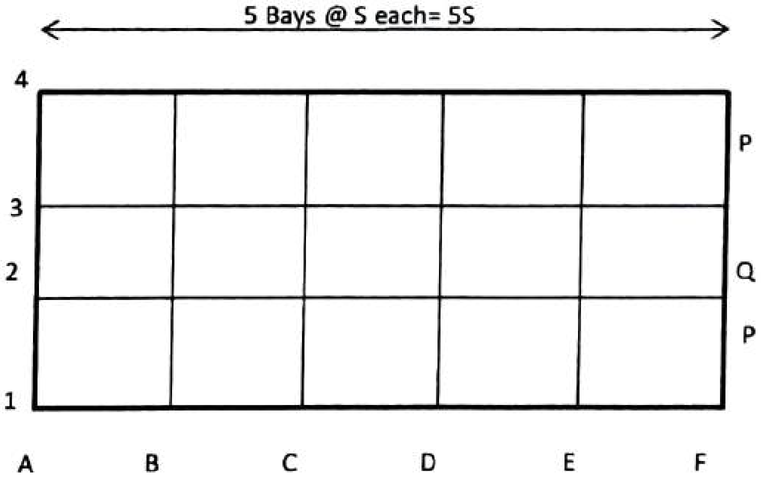
2. Grid Plan of the building is given.

3. Draw an architectural drawing showing:
   a) Ground Floor plan
   b) Typical floor plan
   c) Roof plan
   d) Structural plan
   e) Sectional elevation through staircase between any two floors.

4. Show the detail of reinforcement for:
   a) Typical floor slab
   b) One framed beam in shorter direction at fifth floor level.
   c) One framed beam in longer direction at fifth floor level.
   d) Typical column with pile and pile cap.

5. Design data given:
   a) Plinth height= 600 mm; Floor to floor height= 3.5 m
   b) Width of staircase= 1.50 m
   c) External wall thickness=230mm finish to 250 mm
   d) Internal wall thickness= 125 mm finish to 150 mm
   e) Size of lift block= 2m X 2 m
   f) Number of lift block and staircase= 2 each
   g) Height of stair room= 2.4 m
   h) Height of parapet wall=1 m
   i) Pile capacity= 500 kN for dia 400 mm
   j) Grade of concrete= M25
   k) Grade of steel= Fe 500

GRID PLAN OF G+11 STORIED OFFICE BUILDING



P= (4000+(last 2 digits of Uni Roll no. x 10) mm

Q= 0.75 P mm

S= 1000+P mm

▲ SLAB CALCULATIONS:

Let us assume a slab thickness of (overall depth) D = 145 mm from deflection point of view.

Therefore, effective depth (taking cover = 15 mm and 10 mm dia Fe-500 rods) is, $d = 145 - 15 - \frac{10}{2} = 125$ mm.

Floor to Floor distance is given as 3.5 m with floor finishing = 0.14 m.

Let Factor of safety = 1.5.

☐ LOAD CALCULATIONS:

(A) Dead Load:

i) Self weight of slab per m^2:

Unit weight = 25 kN/m^3

Self weight = $25 \times \frac{145}{1000} \times 1 = 3.625$ kN/m^2 — ①

ii) Unit weight of 40 mm thk base concrete for marble flooring = 23.5 kN/m^3.

So, weight = $23.5 \times \frac{40}{1000} \times 1 = 0.94$ kN/m^2 — ②

iii) Unit weight of 10 mm thk marble paving = 26.7 kN/m^3.

So, weight = $26.7 \times \frac{10}{1000} \times 1 = 0.267$ kN/m^2 — ③

iv) Unit weight of 13 mm thk cement plaster (ceiling) = 20.4 kN/m^3

So, weight = $20.4 \times \frac{13}{1000} \times 1 = 0.2652$ kN/m^2 — ④

v) Weight of light partition wall (wooden / glass) = 1 kN/m^2 — ⑤

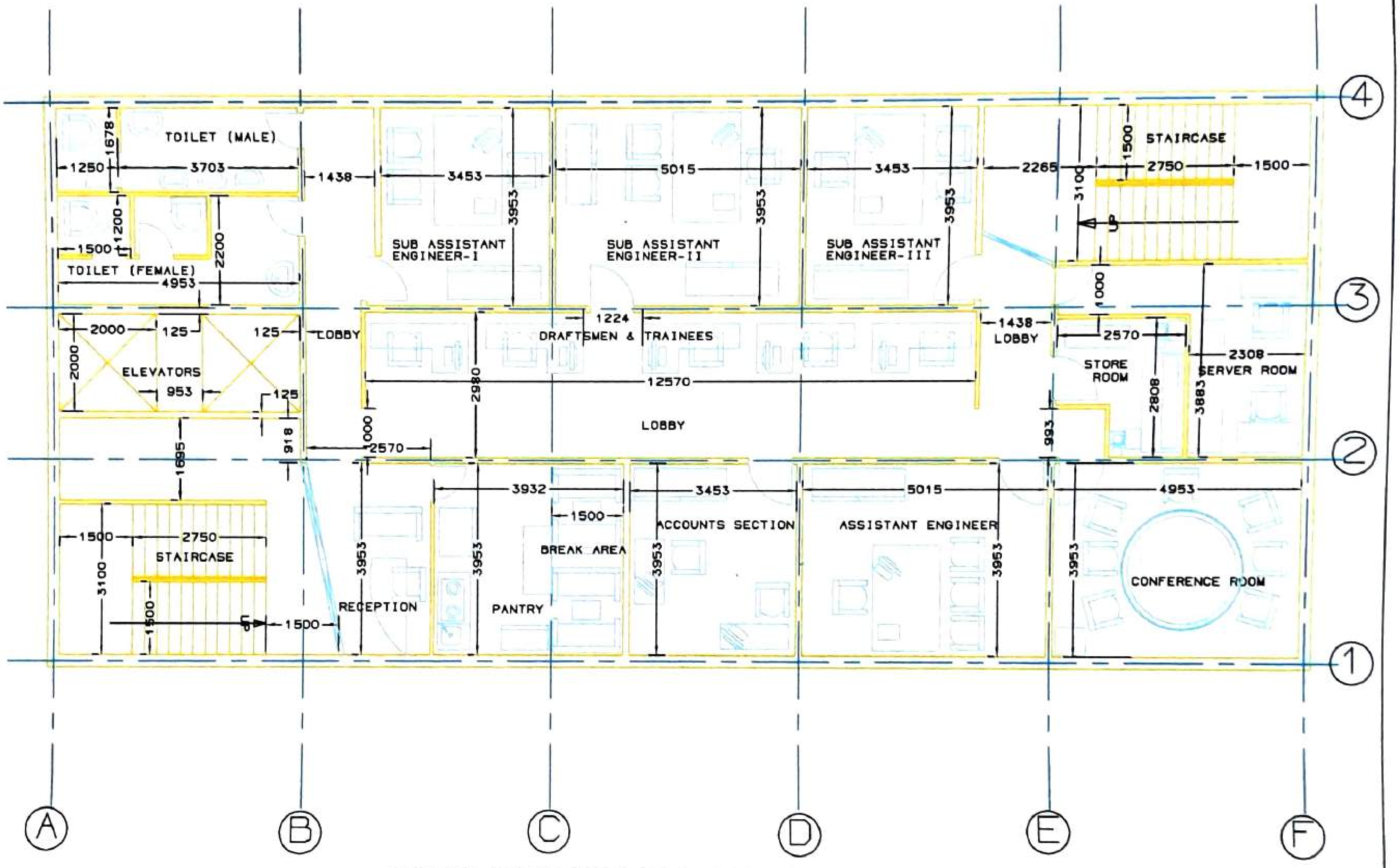
∴ Total Dead load = ① + ② + ③ + ④ + ⑤ = 6.0972 kN/m^2 — (A).

(B) Live Load:

Imposed load for office building (assuming room w/o separate storage) = 4 kN/m^2 — (B)

So, total load = Dead load + Live load = (A) + (B) = 10.0972 kN/m^2

(BY: ATASI ROY MALAKAR CVL-101 8th SEM)

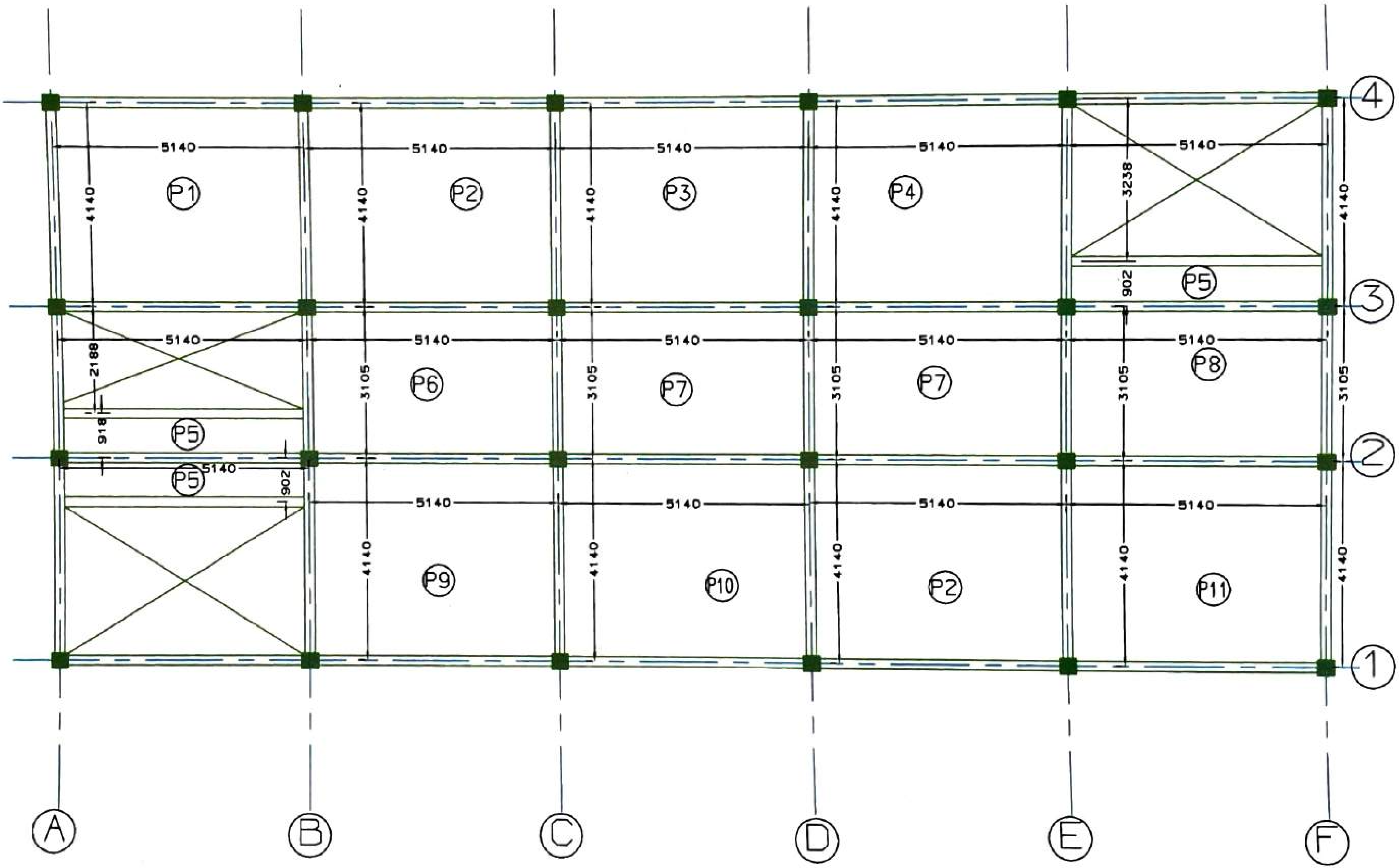


TYPICAL FLOOR PLAN OF G+11 OFFICE BUILDING

(ALL DIMENSIONS IN MM)
SCALE-1:100

1(a)

[BY: ATASI ROY MALAKAR CVL-101 8th SEM]



TYPICAL FLOOR STRUCTURAL PLAN OF G+11 OFFICE BUILDING

(ALL DIMENSIONS IN MM)
SCALE-1:100

1(b)

(c) Calculation of load coming from Brickwork in panel:

Unit weight of brickwork = 19.5 kN/m^2

Brickwork is present in panels marked P1, P2, P3, P5 & P7 (reference Dwg 2: Structural plan)

∴ Brickwork load for :-

i) Panel P1 → (5.14 × 4.14) m^2 of thickness = 0.075 m (brickwork thi

∴ Volume of brickwork = $\left(5.14 - \frac{0.25}{2} - \frac{0.125}{2}\right) \times 0.075 \times (3.5 - 0.14)$

= 1.248 m^3.

So, udl due to brickwork = $\frac{1.25 \times 19.5 \times 1.248}{5.14 \times 4.14}$ = 1.4296 kN/m^2

So, total load for this panel = 10.0972 + 1.4296 = 11.5268 kN/m^2

ii) Panel $P_2, P_4, P10$ → (5.14 × 4.14) sq m of thickness = 0.125 m.

∴ Volume of brickwork = $\left(4.14 - \frac{0.25}{2} - \frac{0.125}{2}\right) \times 0.125 \times (3.5 - 0.14)$

= 1.6600 m^3

So, udl due to brickwork = $\frac{1.25 \times 19.5 \times 1.66}{5.14 \times 4.14}$ = 1.9015 kN/m^2

So, total load for this panel = 10.0972 + 1.9015 = 11.9987 kN/m^2

iii) Panel P9 → (5.14 × 4.14) sq m of thickness = 0.075 m

∴ Volume of brickwork = $\left(4.14 - \frac{0.25}{2} - \frac{0.125}{2}\right) \times 0.075 \times (3.5 - 0.14)$

= 0.99603 m^3

So, udl due to brickwork = $\frac{1.25 \times 19.5 \times 0.996}{5.14 \times 4.14}$ = 1.1409 kN/m^2

So, total load for this panel = 9.5972 + 1.1409 = 11.2381 kN/m^2.

Now, for Fe-500 & M-25, $\frac{x_{umax}}{d} = 0.456$ & $R_{u_{max}} = 0.36 \times 0.456 \times 25 \times (1 - 0.416 \times 0.456)$

= 3.3256.

Also, $R_{umax} = \frac{M_{u,lim}}{bd^2}$ ∴ Required depth = $\sqrt{\frac{M}{bR_u}}$

Let us predict the critical/worst panels to be panel 1 (3 edges discontinuous) and panel 4 (maximum developed moment and maximum load).

Panel 1

Maximum moment, $M_u = 18.077$ kN-m $= R_u b d^2$ [where for M-25 & $\frac{x_{u\,max}}{d} = 0.456$,

for $b = 1000$, $d = \sqrt{\frac{18.077 \times 10^6}{3.325 \times 1000}} =$ $\quad$ $R_u = 0.36 \times 25 \times 0.456 \times (1 - 0.416 \times 0.48)$

$= 73.73$ mm < 125 m (d). $\quad$ $= 3.325$]

Hence okay.

For max moment, $M_u = 18.077$ kN-m, service load moment, $M = \frac{M_u}{1.5} = 12.051$ kN-m

Now, for $\frac{M_u}{bd^2} = 1.156959$ & for Fe-500, from SP6, pg-49, we get reinforcement percentage $P_t = 0.282\%$ or $\frac{A_{st}}{bd} \times 100 = 0.282$ or $A_{st} = 352.5\ mm^2$ (for M_{ux} at mid span). (reqd.)

Similarly we get, A_{st} for M_{ux} at edge $= 174\ mm^2$

A_{st} for M_{uy} at midspan $= 243.994\ mm^2$

A_{st} for M_{uy} at edge $= 328.023\ mm^2$.

Now $\frac{x_u}{d} = \frac{0.87 \times f_y \times A_{st}}{0.36 \times f_{ck} \times bd} \Rightarrow x_u = \frac{0.87 \times 500 \times 174}{0.36 \times 25 \times 1000} = 8.41$ mm

Modular ratio, $m = \frac{E_s}{E_c} = \frac{2 \times 10^5}{5000\sqrt{f_{ck}}} = 8$.

Now, $I_r = \frac{b x_u^3}{3} + m A_s (d - x_u)^2 = 19.1200 \times 10^6\ mm^4$.

$I_{gr} = \frac{bD^3}{12} = 254.0521 \times 10^6\ mm^4$.

Now from annex C of IS-456:2000, we get, $M_r = \frac{f_{cr} I_{gr}}{y_t} = \frac{0.7\sqrt{f_{ck}} \times I_{gr}}{D/2} =$

$= 12.26458 \times 10^6$ N-mm

So, $I_{eff} = I_r \left(\frac{1}{1.2 - \frac{M_r}{M} \cdot \frac{z}{d} \cdot \left(1 - \frac{x}{d}\right) \cdot \frac{b_w}{b}} \right)$

$= 68.5419 \times 10^6\ mm^4$, $\quad$ where $z = d - 0.42 x_u = 121.4678$ mm & for rectangular section, $\frac{b_w}{b} = 1$.

Therefore, $I_r < I_{eff} < I_{gr}$. Hence Ok.

Panel Mkd.	Edge Condition w/ load 'w' (KN/m²)	Lx (m)	Ly (m)	Ly/Lx	$\alpha_x(+)$	$\alpha_x(-)$	$\alpha_y(+)$	$\alpha_y(-)$
1	1 short edge continuous with udl = 11.527	4.14	5.14	1.2415 (Two-way)	0.061	0	0.043	0.05
2	1 long edge discontinuous with udl = 11.999	4.14	5.14	1.2415 (Two way)	0.041	0.054	0.028	0.03
3	1 long edge discontinuous with udl = 10.097	4.14	5.14	1.2415 (Two way)	0.041	0.054	0.028	0.03
4	2 adjacent edge discont. with udl = 11.999	4.14	5.14	1.2415 (Two way)	0.047	0.062	0.035	0.047
5	2 adjacent edge discont. with udl = 10.097	0.902	5.14	5.6984 (One way)				
6	1 short edge discont. with udl = 10.097	3.105	5.14	1.6554 (Two way)	0.047	0.062	0.028	0.037
7	All edges continuous with udl = 10.097	3.105	5.14	1.6554 (Two way)	0.044	0.058	0.024	0.032
8	1 short edge discont. with udl = 10.097	3.105	5.14	1.6554 (Two way)	0.047	0.062	0.028	0.037
9.	1 short edge discont. w/ udl = 11.238	4.14	5.14	1.2415 (Two way)	0.037	0.049	0.028	0.037
10	1 longer edge discont. with udl = 11.999	4.14	5.14	1.2415 (Two way)	0.041	0.054	0.028	0.037
11.	2 adjacent edge discont. with udl = 10.097	4.14	5.14	1.2415 (Two way)	0.047	0.062	0.035	0.047

Moment (kN-m) = $\alpha w l_x^2$ (two way); $\frac{wl^2}{8}$ (one way)

1.5Mx(+) = Mux(+)	1.5Mx(−) = Mux(−)	1.5My(+) = Muy(+)	1.5My(−) = Muy(−)	Depth (Check) Reqd. (mm)	(x direction) Dia of reinf (mm)	Ast reqd (Mux edge) (mm²)	(Top) Spacing reqd. (mm)	Spacing provided (mm)	Ast reqd (Mux mid span) mm²	(Bottom) Spacing reqd (mm)	Spacing provided (mm)	(y direction) Dia of reinf. (mm)	Ast reqd (Muy edge) (mm²)	Spacing reqd. (mm)	(Top) Spacing provided (mm)	Ast reqd (Muy mid span) mm²	Spacing reqd (mm)	(bottom) spacing provided (mm)
18.0771	0	12.7428	16.8917	73.787 (safe)	10mm	174.00	451.149	300	352.499	222.696	200	10mm	328.023	239.312	230	243.994	321.729	300
12.6476	16.6578	8.6374	11.4137	70.774 (safe)	Φ bar	323.220	242.869	200	242.094	324.254	280	Φ bar	217.587	360.775	230	174.00	451.149	300
10.6433	14.0180	7.2686	9.6049	64.924 (safe)	provided	269.557	291.218	200	202.391	387.863	280	provided	182.033	431.241	230	174.00	451.149	300
14.4985	19.1257	10.7967	14.4985	75.835 (safe)	at	374.332	209.707	200	279.249	281.111	280	at	279.249	281.111	230	205.411	382.160	300
$\frac{wl_x^2}{8}$ =	1.5403			21.521 (safe)	top	174.00	451.149	250	174.000	451.149	300	top	174.00	451.149	230	174.00	451.149	300
6.8630	9.0533	4.0886	5.4028	52.175 (safe)	and	174.00	451.149	200	174.000	451.149	280	and	174.00	451.149	230	174.00	451.149	300
6.4249	8.4692	3.5045	4.6727	50.464 (safe)	bottom	174.00	451.149	200	174.000	451.149	280	bottom	174.00	451.149	230	174.00	451.149	300
6.8630	9.0533	4.0886	5.4028	52.175 (safe)		174.00	451.149	250	174.000	451.149	300		174.00	451.149	230	174.00	451.149	300
10.6902	14.1573	8.0899	10.6902	65.246 (safe)		272.363	288.218	200	203.314	386.103	280		203.314	386.103	230	174.00	451.149	300
12.6476	16.6578	8.6374	11.4137	70.774 (safe)		323.220	242.869	200	242.094	324.254	280		217.587	360.775	230	174.00	451.149	300
12.2008	16.0958	9.0858	12.2009	69.567 (safe)		311.687	251.855	250	233.197	336.625	300		233.197	336.625	230	174.00	451.149	300

Deflection Calculations:-

$$\alpha = \frac{\left[M + \frac{M'}{2}\right]}{5000\sqrt{f_{ck}} \times I_{eff}} = \frac{12.051}{5000\sqrt{25} \times 68.87} = 6.9996 \times 10^{-6}$$

$$\beta = \frac{M'}{5000\sqrt{f_{ck}}\, I_{eff}} = 0$$

$$\text{Deflection} = \frac{5\alpha L_x^2 - 3\beta L_x^2}{48} = 12.4969\,mm$$

$$E_{ce} = \frac{5000\sqrt{f_{ck}}}{1+1.6} = 9615.3846\, N/mm^2$$

$$m' = \frac{2 \times 10^5}{E_{ce}} = 20.8$$

$$I_r = \frac{b x_u^3}{3} + m' A_{st} (d - x_u)^2 = 49.395 \times 10^6\, mm^4.$$

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M} \cdot \frac{z}{d} \cdot \left(1 - \frac{x_u}{d}\right) \frac{b_w}{b}} = 177.921 \times 10^6\, mm^4.$$

$$I_{gr} = \frac{bD^3}{12} = 254.052 \times 10^6\, mm^4.$$

$\therefore I_r < I_{eff} < I_{gr}$ Hence ok.

$$\alpha = \frac{\left[M + \frac{M'}{2}\right]}{E_{ce} \times I_{eff}} = 7.04456 \times 10^{-6} \qquad \beta = \frac{M'}{E_{ce} \times I_{eff}} = 0.$$

$$\delta = \frac{5 \times \alpha l_x^2 - 3\beta L_x^2}{48} = 12.577\,mm$$

So elongation due to creep $= 12.577 - 12.4969 = 0.0803\,mm$

Deflection due to shrinkage $= \frac{K_3 K_4 \varepsilon_{cs}}{D} \times L_x^2$, where $K_3 = 0.086$

$= 1.1044\,mm$

$K_4 = 0.72\sqrt{P_{tmax}}$
$= 0.72\sqrt{0.253} = 0.3622$
$\varepsilon_{cs} = 0.0003$
$D = 145\,mm$, $L_x = 4140\,mm$

$\therefore$ Long term deflection $= 0.0803 + 1.1044$
$= 1.1847\,mm < \frac{4140}{350} = 11.83\,mm$

& Total deflection $= 12.4969 + 1.1044 + 1.1044 = 13.6816\,mm < \frac{4140}{250} = 16.56\,mm$ (Hence ok)

Deflection Calculations :-

$$\alpha = \frac{\left[M + \frac{M'}{2}\right]}{5000\sqrt{f_{ck}} \times I_{eff}} = \frac{12.051}{5000\sqrt{25} \times 68.87} = 6.9996 \times 10^{-6}$$

$$\beta = \frac{M'}{5000\sqrt{f_{ck}}\, I_{eff}} = 0.$$

$$\text{Deflection} = \frac{5\alpha L_x^2 - 3\beta L_x^2}{48} = 12.4969\,mm$$

$$E_{ce} = \frac{5000\sqrt{f_{ck}}}{1+1.6} = 9615.3846\, N/mm^2$$

$$m' = \frac{2 \times 10^5}{E_{ce}} = 20.8$$

$$I_r = \frac{b x_u^3}{3} + m' A_{st} (d - x_u)^2 = 49.395 \times 10^6\, mm^4.$$

$$I_{eff} = \frac{I_r}{1.2 - \frac{M_r}{M} \cdot \frac{z}{d} \cdot \left(1 - \frac{x_u}{d}\right) \cdot \frac{b_w}{b}} = 177.921 \times 10^6\, mm^4.$$

$$I_{gr} = \frac{bD^3}{12} = 254.052 \times 10^6\, mm^4.$$

$\therefore I_r < I_{eff} < I_{gr}$ Hence ok.

$$\alpha = \frac{\left[M + \frac{M'}{2}\right]}{E_{ce} \times I_{eff}} = 7.04456 \times 10^{-6} \qquad \beta = \frac{M'}{E_{ce} \times I_{eff}} = 0.$$

$$\delta = \frac{5 \times \alpha l_x^2 - 3\beta L_x^2}{48} = 12.577\,mm$$

So elongation due to creep = 12.577 − 12.4969 = 0.0803 mm

Deflection due to shrinkage = $\frac{K_3 K_4 \varepsilon_{cs}}{D} \times L_x^2$, where $K_3 = 0.086$

$= 1.1044$ mm

$K_4 = 0.72\sqrt{P_{tmax}}$
$= 0.72\sqrt{0.253} = 0.362$
$\varepsilon_{cs} = 0.0003$
$D = 145\,mm$, $L_x = 4140\,mm$

$\therefore$ Long term deflection = 0.0803 + 1.1044

$= 1.1847\,mm < \frac{4140}{350} = 11.83\,mm$

& Total deflection = 12.4969 + 1.1044 + 1.1044 = 13.6816 mm $< \frac{4140}{250} = 16.56\,mm$ (Hence ok)

Panel 4

Max moment, $M_u = 19.126$ kN-m $= R_u b d^2$, where R_u, as calculated before $= 3.325$

$\therefore$ for $b = 1000$, $d = \sqrt{\dfrac{19.126 \times 10^6}{3.325 \times 1000}} = 75.84$ mm < 125 mm (d).

Hence ok.

For max moment, $M_u = 19.126$ kN-m, service load moment, $M = \dfrac{M_u}{1.5} = 12.751$ kN-m

Now, for $\dfrac{M_u}{bd^2} = 1.224077$ & for Fe500, from SP-6, pg 49, we get reinforcement percentage $P_t = 0.299\%$, or $\dfrac{A_{st}}{bd} \times 100 = 0.299$ or $A_{st} = 374$ mm^2. (for M_{ux} at edge)

Similarly we get, A_{st} for M_{ux} at midspan $= 279.249$ mm^2.

for M_{uy} at edge $= 279.249$ mm^2

for M_{uy} at midspan $= 205.411$ mm^2.

Now, $\dfrac{x_u}{d} = \dfrac{0.87 \times f_y \times A_{st}}{0.36 f_{ck} b d} \Rightarrow x_u = 18.1$ mm, Modular ratio, $m = \dfrac{E_s}{E_c} = 8$.

$I_r = \dfrac{b x_u^3}{3} + m A_s (d - x_u)^2 = 36.1374 \times 10^6$ mm^4.

$I_{gr} = \dfrac{bD^3}{12} = 254.0521 \times 10^6$ mm^4.

Now from annex C of IS 456:2000, we get, $M_r = \dfrac{f_{cr} I_{gr}}{y_t} = \dfrac{0.7\sqrt{f_{ck}}\, I_{gr}}{D/2} = 12.2696$ kN-m

So, $I_{eff} = \dfrac{I_r}{\left[1.2 - \frac{M_r}{M} \times \frac{z}{d} \times \left(1 - \frac{x}{d}\right) \times \left(\frac{b_w}{b}\right)\right]} = 84.495 \times 10^6$ mm^4 where $z = d - 0.42 x_u = 117.39$ mm & $b_w/b = 1$ for rect. sec.

Therefore, $I_r < I_{eff} < I_{gr}$ Hence ok.

Deflection Calculations:–

$\alpha = \dfrac{M + M'/2}{5000\sqrt{f_{ck}} \times I_{eff}} = 7.594 \times 10^{-6}$ $\quad \beta = \dfrac{M'}{5000\sqrt{f_{ck}}\, I_{eff}} = 6.03625 \times 10^{-6}$.

Deflection $= \dfrac{5\alpha L_x^2 - 3\beta L_x^2}{48} = 7.0919$ mm.

$E_{ce} = \dfrac{5000\sqrt{f_{ck}}}{1 + 1.6} = 9615.3846$ N/mm^2 $\quad m' = 20.8$.

$\therefore I_r = \dfrac{b x_u^3}{3} + m' A_{st} (d - x_u)^2 = 90.7814 \times 10^6$ mm^4, $I_{eff} = \dfrac{I_r}{1.2 - \frac{M_r}{M} \frac{z}{d} \frac{b_w}{b} \left(1 - \frac{x_u}{d}\right)} = 212.2615 \times 10^6$ mm^4

$I_{gr} = \dfrac{bD^3}{12} = 254.052 \times 10^6$ mm^4. $\quad \therefore I_r < I_{eff} < I_{gr}$ Hence ok.

$\alpha = \dfrac{M + M'/2}{E_{ce} \times I_{eff}} = 7.859 \times 10^{-6}$; $\beta = \dfrac{M'}{E_{ce} I_{eff}} = 6.2474 \times 10^{-6}$ $\quad \therefore \delta = \dfrac{5\alpha L_x^2 - 3\beta L_x^2}{48} = 7.34$ mm

Creep elongation $= 7.34 - 7.0919 = 0.2481$ mm & shrinkage deflection $= \dfrac{k_3 k_4 \varepsilon_{cs} L_x^2}{D} = 1.201$ mm $\left[k_4 = 0.72\sqrt{P_t} = 0.72\sqrt{0.299} = 0.394\right]$

$\therefore$ Long term deflection $= 0.2481 + 1.201 = 1.449$ mm $< \dfrac{4140}{350} = 11.83$ mm

$\therefore$ Total deflection $= 7.0919 + 1.449$ mm $= 8.541$ mm $< 4140/250 = 16.56$ mm } Hence okay.

(BY: ATASI ROY MALAKAR CVL-101 8th SEM)

TYPICAL FLOOR SLAB LOAD DISTRIBUTION PLAN

(ALL DIMENSIONS IN MM)

SCALE-1:100

☐ BEAM CALCULATIONS:

Let us consider A2-B2-C2-D2-E2-F2 as the critical longer frame, and C1-C2-C3-C4 as the critical shorter frame.

▲ FEM CALCULATIONS: (for DL+LL)

↳ (A) For A_2-B2-C2-D2-E2-F2 ~

i) A2-B2 (passing between 2 P5 panels)

a) The udl intensity for Panel 5 is 10.097 kN/m^2.

Now udl acting on A2-B2 = $10.097 \times \frac{0.918}{2} = 4.634\ kN/m$

Reaction coefficient $= \frac{1}{2}$ ∴ $R_A = R_B = \frac{wL}{2} = \frac{4.634 \times 5.140}{2} = 11.91\ kN$

∴ FEM at ends → $M_A = M_B = \frac{wL^2}{12} = \frac{4.634 \times 5.140^2}{12} = -1.985\ kN\text{-}m$

at C, $M_C = \frac{4.634 \times 5.140^2}{24} = 0.9926\ kN\text{-}m$

b) Similar values of M_A, M_B & M_C from Panel 5 on the other side of A2-B2.

∴ Total, $M_A = M_B = 2 \times -1.985 = -3.97\ kN\text{-}m$ & $M_C = 2 \times 0.9926 = 1.985\ kN\text{-}m$.

ii) B2-C2 (Passing between P6 & P9)

a) The udl intensity for Panel P6 is 10.097 kN/m^2 (Trapezoidal loading)

∴ udl acting on B2-C2, $w_1 = 10.097 \times \frac{3.105}{2} = 15.68\ kN/m$

Now, $\alpha_1 = \left(\frac{3.105}{2}\right) \div 5.140 = 0.302$

Total Load, $W_1 = w_1 L(1-\alpha_1) = 5.140 \times (1-0.302) \times w_1 = 3.588 \times \frac{3.105}{2} \bar{w}$

$= 5.57 \times$ (Panel load intensity $(\bar{w})$) $= 56.255\ kN$

Reaction coefficient for trapezoidal loading, $r_B = \frac{1}{2}$ ∴ $R_A = R_B = \frac{W_1}{2} = 28.13\ kN$

B.M. coefficients, $K_A = K_B = \frac{1 - 2\alpha_1^2 + \alpha_1^3}{12(1-\alpha_1)} = 0.1009$

& $K_C = \frac{1 - 2\alpha_1^3}{24(1-\alpha_1)} = 0.0564$

∴ $M_A = M_B = -K_A WL = -K_B WL = 5.57 \times$ (Panel load intensity) $\times 5.140 \times (-0.1009)$

$= -2.889 \times$ Panel load intensity $= -29.175\ kN\text{-}m$

$M_C = +K_C WL = 0.0564 \times 5.14 \times 5.57$ (Panel load intensity)
$= 1.615 \times$ Panel load intensity $= 13.308$ kN-m.

b) The udl intensity for Panel 9 is 11.238 kN/m² (Trapezoidal loading)

$\therefore$ udl acting on B2-C2, $w_2 = 11.238 \times \frac{4.14}{2} = 23.263$ kN/m

Now, $\alpha_2 = \left(\frac{4.14}{2}\right) \div 5.14 = 0.403$

Total load, $W_2 = w_2 L (1-\alpha_2) = 5.14 \times (1-0.403) \times w_2$
$= 5.14 \times (1-0.403) \times \frac{4.14}{2} \times$ Panel load intensity
$= 6.352 \times$ Panel load intensity $= 71.384$ kN

Reaction coefficient for Trapezoidal loading, $r_B = \frac{1}{2}$

$\therefore R_A = R_B = \frac{W_2}{2} = 35.692$ kN

BM coefficients, $K_A = K_B = \frac{1-2\alpha_2^2+\alpha_2^3}{12(1-\alpha_2)} = 0.1034$

& $K_C = \frac{1-2\alpha_2^3}{24(1-\alpha_2)} = 0.0606$

$\therefore M_A = M_B = -K_A WL = -K_B WL = 6.352 \times$ Panel load intensity $\times 5.140 \times (-0.1034)$
$= -3.376 \times$ Panel load intensity $= -37.939$ kN-m

& $M_C = +K_C WL = 0.0606 \times 5.14 \times 6.352 \times$ Panel load intensity
$= 1.978 \times$ Panel load intensity $= 22.23$ kN-m

$\therefore$ Total $M_A = M_B = -29.175 - 37.939 = -67.114$ kN-m

& $M_C = 13.308 + 22.23 = 35.538$ kN-m.

Total $R_A = R_B = 28.13 + 35.692 = 63.82$ kN.

[Now, for C2-D2, D2-E2 & E2-F2 calculations are done similarly using the above derived formulae. Separate derivations have not been shown.]

iii) C2-D2 (Passing between panels P7 and P10).

a) Trapezoidal loading with panel udl intensity in P7 $= 10.097\ kN/m^2$

Dimensions and loading of Panel P7 is same as Panel P6.

Hence, $R_A = R_B = 28.13\ kN$, $M_A = M_B = -29.175\ kN\text{-}m$, $M_C = 13.308\ kN\text{-}m$

b) Trapezoidal loading with panel P10 udl intensity $= 11.999\ kN/m^2$

$\therefore W_2 = 6.352 \times 11.999 = 76.28\ kN$

$R_A = R_B = \frac{W_2}{2} = 38.11\ kN$,

$M_A = M_B = -3.376 \times 11.999 = -40.509\ kN\text{-}m$

& $M_C = 1.978 \times 11.999 = 23.73\ kN\text{-}m$

$\therefore$ Total $M_A = M_B = -29.175 - 40.509 = -69.684\ kN\text{-}m$

& $M_C = 13.308 + 23.73 = 37.038\ kN\text{-}m$

Total $R_A = R_B = 28.13 + 38.11 = 66.24\ kN$

iv) D2-E2 (Passing between panels P7 & P2).

a) Trapezoidal loading with panel udl intensity in $P7 = 10.097\ kN/m^2$

Dimensions and loading of Panel P7 is same as Panel P6.

Hence, $R_A = R_B = 28.13\ kN$, $M_A = M_B = -29.175\ kN\text{-}m$, $M_C = 13.308\ kN\text{-}m$

b) Trapezoidal loading with Panel P2 udl intensity $= 11.999\ kN/m^2$.

Hence dimension & loading of P2 being similar to P10, we have,

$R_A = R_B = 38.11\ kN$, $M_A = M_B = -40.509\ kN\text{-}m$ & $M_C = 23.73\ kN\text{-}m$.

$\therefore$ Total $M_A = M_B = -69.684\ kN\text{-}m$ & $M_C = 37.038\ kN\text{-}m$

Total $R_A = R_B = 28.13 + 38.11 = 66.24\ kN$.

v) E2-F2 (Passing between panels P8 & P11)

a) Trapezoidal loading with panel udl intensity in $P8 = 10.097\ kN/m^2$

Same as P6.

Hence, $R_A = R_B = 28.13\ kN$, $M_A = M_B = -29.175\ kN\text{-}m$, $M_C = 13.308\ kN\text{-}m$

b) Trapezoidal loading with panel P11 udl intensity = 10.097 kN/m²

$\therefore W_2 = 6.352 \times 10.097 = 64.136$ kN

$\therefore R_A = R_B = \frac{W_2}{2} = 32.068$ kN, $M_A = M_B = -3.376 \times 10.097$
$= -34.087$ kN-m

& $M_C = 1.978 \times 10.097 = 19.97$ kN-m

$\therefore$ Total $M_A = M_B = (-34.087) + (-29.175) = -63.262$ kN-m.
& $M_C = 13.308 + 19.97 = 33.278$ kN-m
& Total $R_A = R_B = 28.13 + 32.068 = 60.198$ kN.

↳ (B) For C1-C2-C3-C4 ~

i) C1-C2 (passing between P9 & P10).

a) The udl intensity for Panel P9 is 11.238 kN/m² (Triangular loading)

$\therefore$ udl acting on C1-C2, $w_1 = 11.238 \times \frac{4.14}{2} = 23.263$ kN/m

$\therefore$ Total Load, $W_1 = \frac{1}{2} \times 4.14 \times \frac{4.14}{2} \times$ Panel load intensity
$= 4.285 \times$ Panel load intensity $= 48.154$ kN.

Reaction coefficient $r_B = \frac{1}{2}$ $\therefore R_A = R_B = \frac{W_1}{2} = 24.077$ kN.

BM coefficients,
$K_A = K_B = \frac{5}{48}$ & $K_C = \frac{1}{16}$.

$\therefore M_A = M_B = -K_A W L = -\frac{5}{48} \times 4.14 \times 4.285 \times$ Panel load intensity
$= -1.848 \times$ Panel load intensity
$= -20.77$ kN-m

& $M_C = K_C W L = \frac{1}{16} \times 4.14 \times 4.285 \times$ Panel load intensity
$= 1.109 \times$ Panel load intensity $= 12.46$ kN-m

b) The udl intensity for Panel P10 is 11.999 kN/m (Triangular)

$\therefore$ Using above deductions, we have,

$W_2 = 4.285 \times 11.999 = 51.416$ kN $\therefore R_A = R_B = \frac{W_2}{2} = 25.71$ kN

& $M_A = M_B = -1.848 \times 11.999 = -22.17$ kN-m, $M_C = 1.109 \times 11.999 = 13.31$ kN-m

∴ Total $M_A = M_B = -20.77 - 22.17 = -42.94$ kN-m

& $M_c = 12.46 + 13.31 = 25.77$ kN-m

& Total $R_A = R_B = 24.08 + 25.71 = 49.79$ kN.

ii) $C_2 - C_3$. (Passing between P6 & P7)

a) The udl intensity for Panel P6 is 10.097 kN/m² (Triangular loading)

∴ Udl acting on $C_2 - C_3$, $w_1 = 10.097 \times \frac{3.105}{2} = 15.68$ kN/m

∴ Total load, $W_1 = \frac{1}{2} \times 3.105 \times \frac{3.105}{2} \times$ Panel load intensity

$= 2.41 \times$ Panel load intensity $= 24.34$ kN.

Reaction coefficient $r_B = \frac{1}{2}$ ∴ $R_A = R_B = \frac{W_1}{2} = 12.168$ kN.

BM coefficient, $K_A = K_B = \frac{5}{48}$, $K_c = \frac{1}{16}$.

∴ $M_A = M_B = -K_A W L = -\frac{5}{48} \times 3.105 \times 2.41 \times$ Panel load intensity

$= -0.779 \times$ Panel load intensity

$= -7.87$ kN-m

$M_c = K_c W L = \frac{1}{16} \times 3.105 \times 2.41 \times$ Panel load intensity

$= 0.468 \times$ Panel load intensity

$= 4.72$ kN-m

b) The udl intensity for Panel P7 is 10.097 kN/m² (Triangular loading)

∵ this case is similar in loading and dimension, we can directly conclude,

$R_A = R_B = 12.168$ kN, $M_A = M_B = -7.87$ kN-m, & $M_c = 4.72$ kN-m

∴ Total $M_A = M_B = 2 \times -7.87 = -15.74$ kN-m, $M_c = 2 \times 4.72 = 9.44$ kN-m

& Total $R_A = R_B = 2 \times 12.168 = 24.336$ kN.

iii) $C_3 - C_4$ (passing between P2 & P3)

This case is similar to $C_1 - C_2$ in loading & dimension combination.

∴ Total $M_A = M_B = -42.94$ kN-m, $M_c = 25.77$ kN-m

& Total $R_A = R_B = 49.79$ kN

FEM CALCULATIONS (for DL only)

Since moment and reaction forces are proportional to the magnitude of the load applied, hence we are going to apply Unitary method to calculate the corresponding values of moment M_A, M_B & M_C & reactions R_A & R_B for all the previous frame sections under the application of DL.

In short we will multiply corresponding value with $\left(\frac{DL}{DL+LL}\right)$.

↳(A) For A2-B2-C2-D2-E2-F2~

i) A2-B2 (passing between P5 panels on both sides).

$DL+LL = 10.097\ kN/m^2 \qquad DL = 10.097 - 4 = 6.097\ kN/m^2$

$\therefore$ Total $M_A = M_B = -3.97 \times \frac{6.097}{10.097} = -2.397$ kN-m,

Total $M_C = 1.985 \times \frac{6.097}{10.097} = 1.199$ kN-m

Total $R_A = R_B = 2 \times 11.91 \times \frac{6.097}{10.097} = 14.38$ kN.

ii) B2-C2 (between P6 & P9).

Total $M_A = M_B = -29.175 \times \frac{6.097}{10.097} - 37.939 \times \frac{7.238}{11.238} = -42.05$ kN-m

Total $M_C = 13.308 \times \frac{6.097}{10.097} + 22.23 \times \frac{7.238}{11.238} = 22.35$ kN-m

Total $R_A = R_B = 28.13 \times \frac{6.097}{10.097} + 35.92 \times \frac{7.238}{11.238} = 39.97$ kN.

iii) C2-D2 (between P7 & P10)

Total $M_A = M_B = -29.175 \times \frac{6.097}{10.097} - 40.509 \times \frac{7.999}{11.999} =$

$= -29.175 \times 0.604 - 40.509 \times 0.667 = -44.64$ kN-m

Total $M_C = 13.308 \times 0.604 + 23.73 \times 0.667 = 23.866$ kN-m

Total $R_A = R_B = 28.13 \times 0.604 + 38.11 \times 0.667 = 42.41$ kN.

iv) P2-E2 (between P7 & P2).

This case is similar to C2-D2.

Hence total $M_A = M_B = -44.64$ kN-m,

$M_c = 23.866$ kN-m

& Total $R_A = R_B = 42.41$ kN.

v) E2-F2 (between P8 & P11)

Total $M_A = M_B = -34.087 \times \frac{6.097}{10.097} - 29.175 \times \frac{6.097}{10.097}$

$= -63.262 \times 0.604 = -38.21$ kN-m

$M_c = 33.278 \times 0.604 = 20.10$ kN-m

$R_A = R_B = 60.198 \times 0.604 = 36.36$ kN.

↳ (A) for $C_1 - C_2 - C_3 - C_4$

i) $C_1 - C_2$ (between P9 & P10)

Total $M_A = M_B = -20.77 \times \frac{7.238}{11.238} - 22.17 \times \frac{7.999}{11.999}$.

$= -20.77 \times 0.644 - 22.17 \times 0.667 = -28.16$ kN-m

$M_c = 12.46 \times 0.644 + 13.31 \times 0.667 = 16.90$ kN-m

Total $R_A = R_B = 24.08 \times 0.644 + 25.71 \times 0.667 = 32.66$ kN.

ii) $C_2 - C_3$ (between P6 & P7)

Total $M_A = M_B = -15.74 \times \frac{6.097}{10.097} = -15.74 \times 0.604 = -9.507$ kN-m

$M_c = 9.44 \times 0.604 = 5.702$ kN-m

$R_A = R_B = 24.336 \times 0.604 = 14.70$ kN.

iii) $C_3 - C_4$ (between P_2 & P3)

This case is similar to $C_1 - C_2$ in loading & dimension combination,

∴ Total $M_A = M_B = -28.16$ kN-m

Total $M_c = 16.90$ kN-m

total $R_A = R_B = 32.66$ kN

DF CALCULATIONS :

For longer frame A2–B2–C2–D2–E2–F2,

Let depth of beam $= \frac{L}{8} = \frac{5140}{8} = 642.5 \simeq 650$ mm

width $= \frac{650}{2} = 325$ mm.

Self wt. of beam $= 0.325 \times 0.65 \times 25 = 5.281$ kN/m

Influence area of column C2, $A_F = 5.14 \times \frac{3.015+4.14}{2}$

$= 18.62\ m^2$.

There is no brickwork in C_2–C_3 and C_1–C_2, but considering brickwork even in those spaces, in addition to the existing ones in layout plan, for critical load consideration, we have, total brickwork load =

$$= 19.5 \times \left[0.15 \times (3.5-0.14) \times 5.14 + 0.15 \times (3.5-0.14) \times \frac{4.14 \times 3.105}{2}\right]$$

$= 86.12$ kN.

Slab Dead load $= 6.0972\ kN/m^2$

∴ Total load on column C2 at 5th Floor level = P

$$= A_f(DL+2.5) + A_f(DL+4) \times 7 + \text{Brickwork} \times 7 + \text{self wt. of beam} \times 8$$

$$= 18.62 \times (6.0972+2.5) + 18.62(6.0972+4) \times 7 + 86.12 \times 7 + \left(5.14 + \frac{4.14+3.105}{2}\right) \times 5.28$$

$= 2125.25$ kN

$P_u = 1.5 \times P = 3187.88$ kN.

For short column with only axial load,

$$P_u = 0.4 f_{ck} A_c + 0.67 f_y A_s$$

$\Rightarrow (0.4 \times 25 \times 0.99 + 0.67 \times 500 \times 0.01) A_g = 3187.88 \times 10^3$ N.

$\therefore A_g = 240595\ mm^2$

For short column, considering it is a square one,
we have $b = D = 490.5\ mm \simeq 500\ mm$.

$I_{beam} = \frac{650^3 \times 325}{12} = 7.44 \times 10^9\ mm^4$, $I_{column} = \frac{500^4}{12} = 5.208 \times 10^9$

i) For longer frame:

JOINT	MEMBER	LENGTH (MM)	MOM. OF INERTIA (mm^4)	$K = \frac{I}{L}$ (REL. STIFF) (mm^3)	$DF = \frac{K_i}{\Sigma K}$
A_5	A_5-A_6	3500	5.208×10^9	1.488×10^6	0.336
	A_5-B_5	5140	7.44×10^9	1.448×10^6	0.327
	A_5-A_4	3500	5.208×10^9	1.488×10^6	0.336
B_5	B_5-B_6	3500	5.208×10^9	1.488×10^6	0.253
	B_5-C_5	5140	7.44×10^9	1.448×10^6	0.246
	B_5-A_5	5140	7.44×10^9	1.448×10^6	0.246
	B_5-B_4	3500	5.208×10^9	1.488×10^6	0.253
C_5	C_5-C_6	3500	5.208×10^9	1.488×10^6	0.253
	C_5-D_5	5140	7.44×10^9	1.448×10^6	0.246
	C_5-B_5	5140	7.44×10^9	1.448×10^6	0.246
	C_5-C_4	3500	5.208×10^9	1.488×10^6	0.253
D_5	D_5-D_6	3500	5.208×10^9	1.488×10^6	0.253
	D_5-E_5	5140	7.44×10^9	1.448×10^6	0.246
	D_5-C_5	5140	7.44×10^9	1.448×10^6	0.246
	D_5-D_4	3500	5.208×10^9	1.488×10^6	0.253
E_5	E_5-E_6	3500	5.208×10^9	1.488×10^6	0.253
	E_5-F_5	5140	7.44×10^9	1.448×10^6	0.246
	E_5-D_5	5140	7.44×10^9	1.448×10^6	0.246
	E_5-E_4	3500	5.208×10^9	1.488×10^6	0.253
F_5	F_5-F_6	3500	5.208×10^9	1.488×10^6	0.336
	F_5-E_5	5140	7.44×10^9	1.448×10^6	0.327
	F_5-F_4	3500	5.208×10^9	1.488×10^6	0.336

ii) For shorter frame:

JOINT	MEMBER	LENGTH (MM)	MOM. OF INERTIA (mm^4)	$K = \frac{I}{L}$ (mm^3)	$DF = \frac{K_i}{\Sigma K}$
1_5	$1_5 - 1_6$	3500	5.208×10^9	1.488	0.312
	$1_5 - 2_5$	4140	7.44×10^9	1.797	0.376
	$1_5 - 1_4$	3500	5.208×10^9	1.488	0.312
2_5	$2_5 - 2_6$	3500	5.208×10^9	1.488	0.208
	$2_5 - 3_5$	3105	7.44×10^9	2.396	0.334
	$2_5 - 1_5$	4140	7.44×10^9	1.797	0.251
	$2_5 - 2_4$	3500	5.208×10^9	1.488	0.208
3_5	$3_5 - 3_6$	3500	5.208×10^9	1.488	0.208
	$3_5 - 4_5$	4140	7.44×10^9	1.797	0.251
	$3_5 - 2_5$	3105	7.44×10^9	2.396	0.334
	$3_5 - 3_4$	3500	5.208×10^9	1.488	0.208
4_5	$4_5 - 4_6$	3500	5.208×10^9	1.488	0.312
	$4_5 - 3_5$	4140	7.44×10^9	1.797	0.376
	$4_5 - 4_4$	3500	5.208×10^9	1.488	0.312

SUMMARY OF REACTIONS & MOMENTS FOR (DL+LL) & DL

i) DL+LL ii) DL

MEMBER	$R_A (=R_B)$ kN	$M_A (=M_B)$ kN-m	M_C (kN-m)	$R_A (=R_B)$ kN	$M_A (=M_B)$ kN-m	M_C (kN-m)
A2-B2	23.82	-3.97	1.985	14.38	-2.397	1.199
B2-C2	63.82	-67.114	35.538	39.97	-42.05	22.35
C2-D2	66.24	-69.684	37.038	42.41	-44.64	23.87
D2-E2	66.24	-69.684	37.038	42.41	-44.64	23.87
E2-F2	60.198	-63.262	33.278	36.36	-38.21	20.10
C1-C2	49.79	-42.94	25.77	32.66	-28.16	16.90
C2-C3	24.336	-15.74	9.44	14.70	-9.507	5.702
C3-C4	49.79	-42.94	25.77	32.66	-28.16	16.90

MOMENT DISTRIBUTION :

i) A2-B2-C2-D2-E2-F2 [→ a) DL+LL]

	A2	B2	B2	C2	C2	D2	D2	E2	E2	F2
DF	0.327	0.246	0.246	0.246	0.246	0.246	0.246	0.246	0.246	0.327
FEM	-3.97	3.97	-67.114	67.114	-69.684	69.684	-69.684	69.684	-63.262	63.262
Balance	1.29819	15.5334	15.5334	-0.6322	-0.6322	0	0	-1.5798	-1.5798	-20.687
C/O	7.7667	0.6491	-0.3161	7.7667	0	-0.3161	-0.7899	0	-10.3433	-0.7899
Balance	-2.5397	-0.0819	-0.0819	-1.9106	-1.9106	0.2721	0.2721	2.5445	2.5445	0.2583
C/O	0.04096	-1.2699	-0.9553	-0.0409	0.1360	0.9553	1.2722	0.1360	0.1292	1.2722
Balance	0.0134	0.5474	0.5474	-0.0234	-0.0234	-0.0780	-0.0780	-0.0652	-0.0652	-0.4160
TOTAL	2.5276	19.3481	-52.3865	72.2735	-72.1142	68.6067	-69.0076	70.7194	-72.5768	42.8999
Span Mt.	5.955		102.652		106.718		106.718		96.54	
Actual span mt	-2.45526		40.32198		36.36156		36.85849		38.80165	
Simple shear	23.82	23.82	63.82	63.82	66.24	66.24	66.24	66.24	60.198	60.198
Elastic shear	-4.2560	4.2560	-3.8691	3.8691	0.6324	-0.6324	-0.3330	0.3330	5.7737	-5.7737
Total shear	19.5640	28-0760	59.9509	67.6891	66.5223	65.5576	65.9070	66.5730	65.9717	54.4243

[→ b) DL]

	A2	B2	B2	C2	C2	D2	D2	E2	E2	F2
DF	0.327	0.246	0.246	0.246	0.246	0.246	0.246	0.246	0.246	0.327
FEM	-2.397	2.397	-42.05	42.05	-44.64	44.64	-44.64	44.64	-38.21	38.21
Balance	0.7838	9.7546	9.7546	0.6371	0.6371	0	0	-1.5818	-1.5818	-12.4947
C/O	4.8773	0.3919	0.3186	4.8773	0	0.3186	-0.7909	0	-6.2473	-0.7909
Balance	-1.5949	-0.1748	-0.1748	-1.1998	-1.1998	0.1162	0.1162	1.5368	1.5368	0.2586
C/O	-0.0874	-0.7974	-0.5999	-0.0874	0.0581	-0.5999	0.7684	0.0581	0.1293	0.7684
Balance	0.0286	0.3437	0.3437	0.0072	0.0072	-0.0414	-0.0414	-0.0461	-0.0461	-0.2513
TOTAL	1.6104	11.9151	-32.4077	46.2845	-45.1374	44.4334	-44.5877	44.6071	-44.4191	25.7002
	3.596		64.4		68.51		68.51		58.31	
Actual span mt	-1.55632		25.05391		23.72461		23.91261		2325036	
Simple shear	14.38	14.38	39.97	39.97	42.41	42.41	42.41	42.41	36.36	36.36
Elastic shear	-2.6314	2.6314	-2.6998	2.6998	0.1370	-0.1370	-0.0038	0.0038	3.6418	-3.6418
Total shear	11.7486	17.0114	32.2702	42.6698	42.5470	42.2730	42.4062	42.4138	40.018	32.7182

ii) C1-C2-C3-C4 [→ a) DL+LL]

	C1	C2	C2	C3	C3	C4
DF	0.376	0.251	0.334	0.334	0.251	0.376
FEM	-42.94	42.94	-15.74	15.74	-42.94	42.94
Balance	16.1454	-6.8272	-9.0848	9.0848	6.8272	-16.1454
c/o	-3.4136	8.0727	4.5424	-4.5424	-8.0727	3.4136
Balance	1.2835	-3.1664	-4.2134	4.2134	3.1664	-1.2835
c/o	-1.5832	0.6418	2.1067	-2.1067	0.6418	1.5832
Balance	0.5953	-0.6899	-0.9180	0.9180	0.6899	-0.5953
TOTAL	-29.9126	40.97101	-23.3071	23.3071	-40.971	29.9126
	68.71		25.18		68.71	
Actual Span Moment	33.2682		1.8729		33.2682	
Simple Shear	49.79	49.79	24.336	24.336	49.79	49.79
Elastic Shear	-2.6711	2.6711	0	0	2.6711	2.6711
Total Shear	47.1189	52.4611	24.336	24.336	52.4611	47.1189

[→ b) DL]

	C1	C2	C2	C3	C3	C4
DF	0.376	0.251	0.334	0.334	0.251	0.376
FEM	-28.16	28.16	-9.507	9.507	-28.16	28.16
Balance	10.5882	-4.6819	-6.2301	6.2301	4.6819	-10.5882
c/o	-2.34095	5.2941	3.1150	-3.1150	-5.2941	2.3410
Balance	0.88020	-2.11069	-2.8086	2.8086	2.11069	-0.8802
c/o	-1.0554	0.4410	1.4043	-1.4043	-0.4401	1.0554
Balance	0.3968	-0.4630	-0.6160	0.6160	0.4630	-0.3968
TOTAL	-19.6911	26.6386	-14.6424	14.6424	-26.6386	19.69113
	45.06		15.209		45.06	
Actual Span Moment	21.8951		0.5666		21.8951	
Simple Shear	32.66	32.66	14.7	14.7	32.66	32.66
Elastic shear	-1.67814	1.67814	0	0	1.67814	-1.67814
Total shear	30.9819	34.3381	14.7	14.7	34.3381	30.9819

▲ WIND LOAD CALCULATIONS:

From cl 6.3 of IS:875 (Part 3)-1987, we have the value of total wind load on a particular building or structure given as, $F = C_f\, A_e\, P_d$, where i) F is the force acting in a direction specified in respective tables,

ii) C_f is the force coefficient for the building, which differs for wind acting on different faces of the structure

iii) P_d is design wind pressure given as $P_d = 0.6\, V_z^2$, V_z, being design wind velocity at height 'z' from datum.

iv) A_e is Effective frontal area of structure given by $(b \times h)$.

Note:- For calculation of V_z, we have $V_z = K_1 K_2 K_3 V_b$

where K_1 = Risk coefficient for 50 years = 1,

K_2 = Coefficient of design wind speed variation with height, table 2 (IS 875, Pt-3)

K_3 = Topography factor for plain land = 1

& V_b = basic wind speed for Kolkata, Zone 2 = 50 m/s.

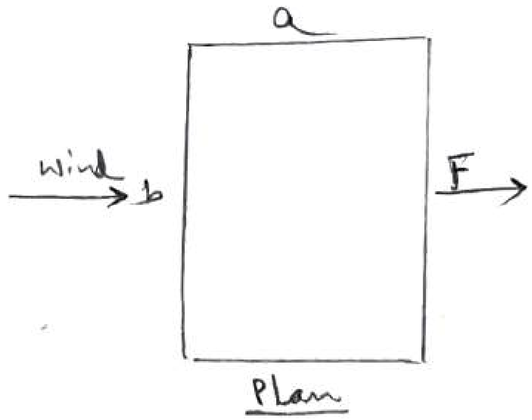
In short, $V_z = 1 \times K_2 \times 1 \times 50 = 50\, K_2$.

Now, following are the tables showing calculation of total wind load for shorter & longer frames $C_1-C_2-C_3-C_4$ and $A_2-B_2-C_2-D_2-E_2-F_2$ respectively:

[C_f for wind along shorter span & longer span

$h = 3.5 \times 12 + 1.5 + 0.6 = 44.1\,m$

i) For shorter span :-



$a = 2 \times 4.14 + 3.105 = 11.385\,m$

$b = 5.14 \times 5 = 25.7\,m$

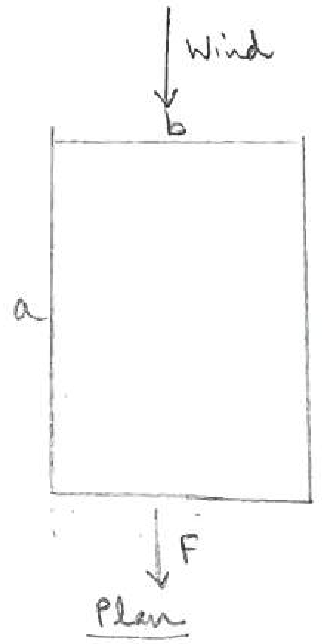
$\therefore \frac{a}{b} = \frac{11.385}{25.7} = 0.443$

$\frac{h}{b} = \frac{44.1}{25.7} = 1.716$

Now for $\frac{a}{b} = 0.443$ & $\frac{h}{b} = 1.716 \geq 1$

$C_f = 1.25$ (from Fig 4A of IS 875 Part 3)

(ii) For longer span



$a = 5 \times 5.14 = 25.7\,m$

$b = 2 \times 4.14 + 3.105 = 11.385\,m$

$\therefore \frac{a}{b} = 2.257$

$\frac{h}{b} = \frac{44.1}{11.385} = 3.874$

Now for $\frac{a}{b} = 2.257$ & $\frac{h}{b} = 3.874 \geq 1$

$C_f = 1.1$ (from Fig 4A)]

□ WIND LOAD ANALYSIS :-

Two tables have been made for wind load analysis

i) for shorter frame C1-C2-C3-C4

& ii) for longer span A2-B2-C2-D2-E2-F2

▲ For C1-C2-C3-C4 :-

$C_f = 1.25$

Fl. no (i)	Height (m)	K_2	$V_z = K_1K_2K_3K_4$ (m/s)	$P_z = 0.6 V_z^2 / 1000$ (kN/m²)	$A_e = h_e b_e$ (m²)	$F_z = C_f \cdot P_z \cdot A_e$ (kN)
1	5.6	0.88	44	1.1616	8.995	13.0607
2	9.1	0.88	44	1.1616	17.99	26.1215
3	12.6	0.9112	45.56	1.2454	17.99	28.0066
4	16.1	0.9488	47.44	1.3503	17.99	30.3656
5	19.6	0.9768	48.84	1.4312	17.99	32.1843
6	23.1	0.9955	49.775	1.4865	17.99	33.4284
7	26.6	1.013	50.65	1.5392	17.99	34.6140
8	30.1	1.0303	51.515	1.5923	17.99	35.8063
9	33.6	1.0408	52.04	1.6249	17.99	36.5399
10	37.1	1.0513	52.565	1.6578	17.99	37.2808
11	40.6	1.0618	53.09	1.6911	17.99	38.0293
12	44.1	1.0723	53.615	1.7247	14.135	30.4740

▲ For A2-B2-C2-D2-E2-F2 :-

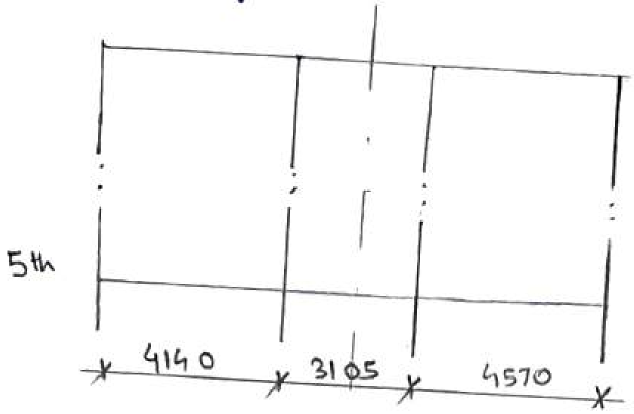
$C_f = 1.1$

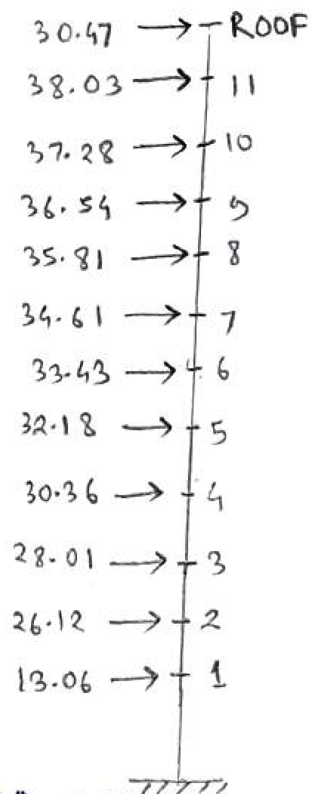
Fl. no. (i)	Height (m)	K_2	$V_z = K_1K_2K_3K_4$ (m/s)	$P_z = 0.6 V_z^2 / 1000$ (kN/m²)	$A_e = h_e b_e$ (m²)	$F_z = C_f P_z A_e$ (kN)
1	5.6	0.88	44.0	1.616	6.3394	8.1002
2	9.1	0.88	44	1.616	12.6788	16.2004
3	12.6	0.9112	45.56	1.2454	12.6788	17.3695
4	16.1	0.9488	47.44	1.3503	12.6788	18.8326
5	19.6	0.9768	48.84	1.4312	12.6788	19.9605
6	23.1	0.9955	49.775	1.4865	12.6788	20.7321
7	26.6	1.013	50.65	1.5392	12.6788	21.4674
8	30.1	1.0303	51.515	1.5923	12.6788	22.2069
9	33.6	1.0408	52.04	1.6249	12.6788	22.6618
10	37.1	1.0513	52.565	1.6578	12.6788	23.1214
11	40.6	1.0618	53.09	1.6911	12.6788	23.5855
12	44.1	1.0723	53.615	1.7247	9.9619	18.8998

A CANTILEVER METHOD ANALYSIS:

A) For shorter frame C1-C2-C3-C4:-

i) Considering 5th floor:-





☐ At mid-height between 5th & 6th Floors:-

Overturning moment

$= 33.43 \times \frac{3.5}{2} + 34.61 \times \frac{3}{2} \times 3.5$
$+ 35.81 \times \frac{5}{2} \times 3.5 + 36.54 \times \frac{7}{2} \times 3.5 + 37.28 \times \frac{9}{2} \times 3.5$
$+ 38.03 \times \frac{11}{2} \times 3.5 + 30.47 \times \frac{13}{2} \times 3.5 = 3013.5875$ kNm

Here $r_1 = -r_4 = 4.14 + \frac{3.105}{2} = 5.6925$ m
$r_2 = -r_3 = \frac{3.105}{2} = 1.5525$ m

$\therefore \Sigma r^2 = 2 \times (5.6925^2 + 1.5525^2) = 69.63$

$\therefore V_{1u} = \frac{3013.5875}{69.63} \times 5.6925 = 246.37$ kN $\Rightarrow V_{4u} = -246.37$ kN

& $V_{2u} = \frac{3013.5875}{69.63} \times 1.5525 = 67.19$ kN $\Rightarrow V_{3u} = -67.19$ kN

Now, $\Sigma H = 33.43 + 34.61 + 35.81 + 36.54 + 37.28 + 38.03 + 30.47$
$= 246.17$ kN

Let us assume $H_{1u} = H_{4u} = 246.17\alpha$
& $H_{2u} = H_{3u} = 246.17\beta$

□ At midheight between 4th & 5th floor :

Overturning moment = 3013.5875 + 246.17 × 3.5 + 32.18 × $\frac{3.5}{2}$

= 3931.4975 kNm

$\therefore V_{1l} = \frac{3931.4975}{69.63} \times 5.6925 = 321.41$ kN $\Rightarrow V_{4l} = -321.41$ kN

& $V_{2l} = \frac{3931.4975}{69.63} \times 1.5525 = 87.66$ kN $\Rightarrow V_{3l} = -87.66$ kN

$\Sigma H = 246.17 + 32.18 = 278.35$ kN.

Let us assume, $H_{1l} = H_{4l} = 278.35\alpha$

& $H_{2l} = H_{3l} = 278.35\beta$.

Now, taking moment about A,

$(278.35\alpha + 246.17\alpha) \times \frac{3.5}{2} = -246.37 \times \frac{4.14}{2} + 321.41 \times \frac{4.14}{2}$

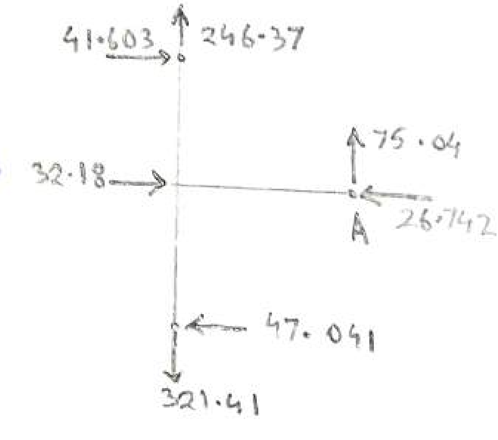
$\Rightarrow \alpha = 0.169$

$\therefore H_{1u} = 0.169 \times 246.17 = 41.603$ kN

$H_{1L} = 0.169 \times 278.35 = 47.041$ kN

$K_h = 41.603 + 32.18 - 47.041 = 26.742$ kN

$K_v = 321.41 - 246.37 = 75.04$ kN



Next, taking moment about B,

$(278.35\beta + 246.17\beta) \times \frac{3.5}{2} = 87.66 \times \frac{3.105}{2} - 67.19 \times \frac{3.105}{2}$
$+ 75.04 \times \frac{1}{2}(4.14 + 3.105)$

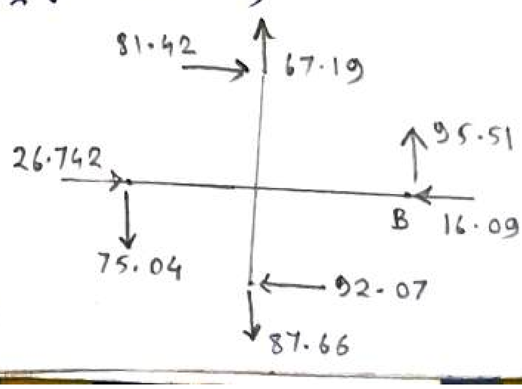
$\Rightarrow \beta = 0.331$

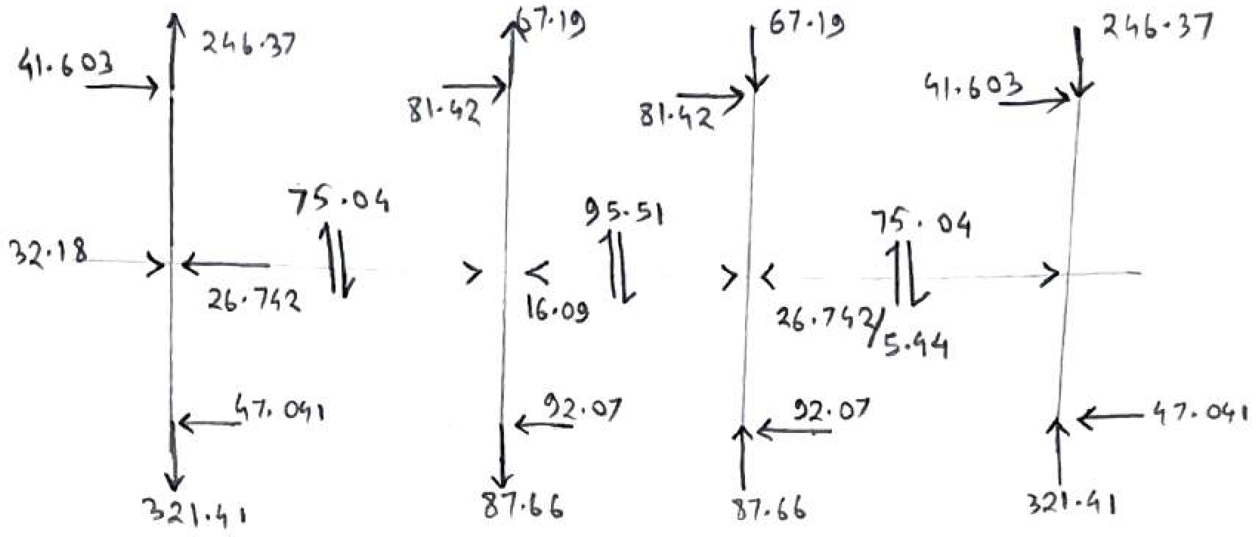
$\therefore H_{1u} = 0.331 \times 246.17 = 81.42$ kN

$H_{1L} = 0.331 \times 278.35 = 92.07$ kN

$K_h = 81.42 + 26.742 - 92.07 = 16.09$ kN

$K_v = 75.04 + 87.66 - 67.19 = 95.51$ kN





ii) Considering 1st floor :-

□ At mid-height between 1st & 2nd floors :-

Overturning moment $= 26.12\times\frac{3.5}{2} + 28.01\times3\times\frac{3.5}{2} + 30.36\times5\times\frac{3.5}{2} + 32.18\times7\times\frac{3.5}{2} + 33.43\times9\times\frac{3.5}{2} + 34.61\times11\times\frac{3.5}{2} + 35.81\times13\times\frac{3.5}{2} + 36.54\times15\times\frac{3.5}{2} + 37.28\times17\times\frac{3.5}{2} + 38.03\times19\times\frac{3.5}{2} + 30.47\times21\times\frac{3.5}{2} = 7312.585$ kN-m

$\therefore V_{1u} = \frac{7312.585}{69.63}\times5.6925 = 597.83$ kN $\Rightarrow V_{4u} = -597.83$ kN

& $V_{2u} = \frac{7312.585}{69.63}\times1.5525 = 163.04$ kN $\Rightarrow V_{3u} = -163.04$ kN

$\Sigma H = 26.12 + 28.01 + 30.36 + 32.18 + 33.43 + 34.61 + 35.81 + 36.54 + 37.28 + 38.03 + 30.47 = 362.84$ kN.

□ At mid-height between Ground floor & 1st floor :-

Overturning moment $= 7312.585 + 362.84\times\frac{1}{2}(3.5+5.6) + 13.06\times\frac{5.6}{2}$
$= 9000.1$ kN-m

$V_{1l} = \frac{9000.1}{69.63}\times5.6925 = 735.79$ kN $\Rightarrow V_{4l} = -735.79$ kN

& $V_{2l} = \frac{9000.1}{69.63}\times1.5525 = 200.67$ kN $\Rightarrow V_{3l} = -200.67$ kN

$\Sigma H = 362.84 + 13.06 = 375.9$ kN.

Similarly proceeding as above, $\alpha = 0.1692$, $\beta = 0.3308$.

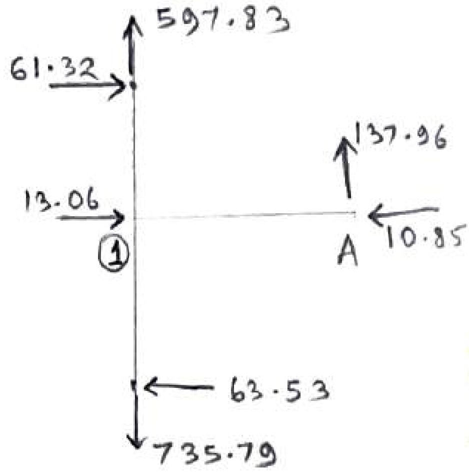
At ①

$\therefore H_{1u} = 0.169 \times 362.84 = 61.32\text{ kN}$

$H_{1L} = 0.169 \times 375.9\text{ kN} = 63.53\text{ kN}$

$K_h = 61.32 + 13.06 - 63.53 = 10.85\text{ kN}$

$K_v = 735.79 - 597.83 = 137.96\text{ kN}$



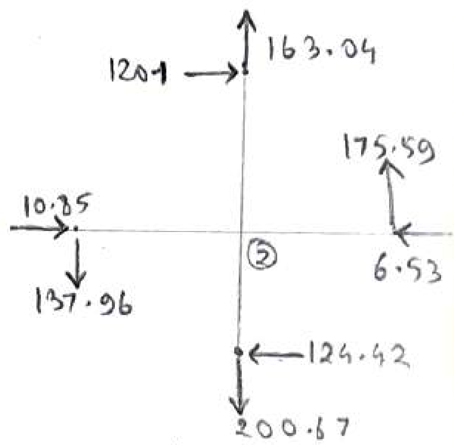
At ②

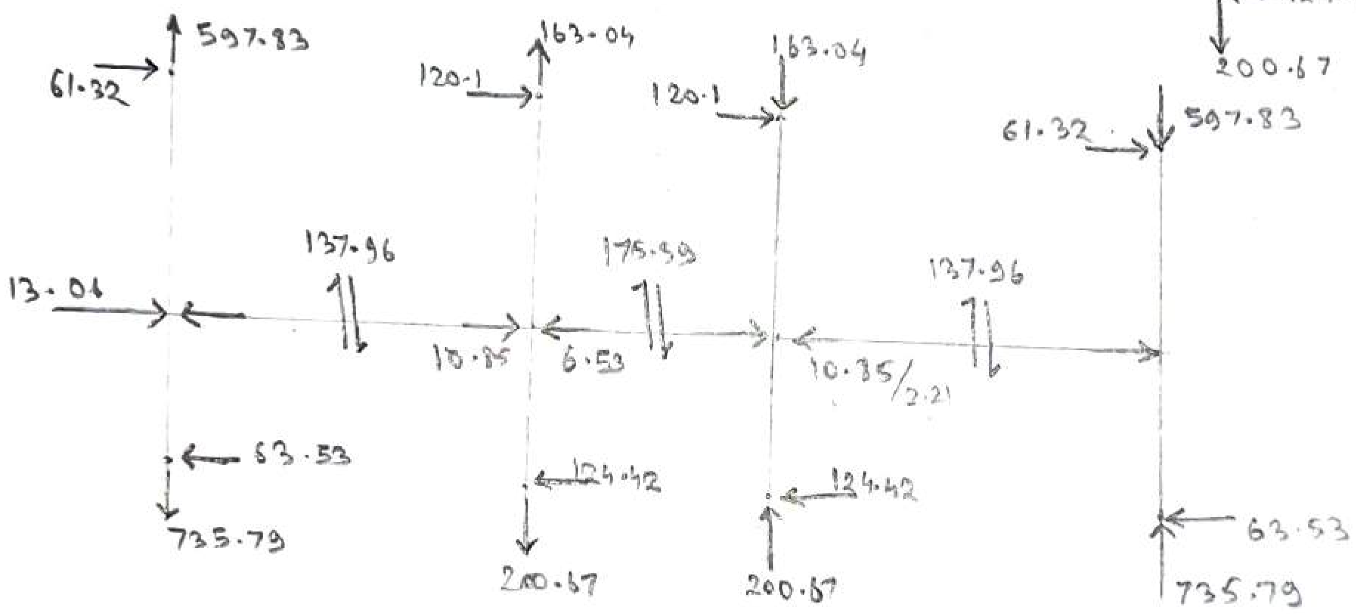
& $H_{1u} = 0.331 \times 362.84 = 120.1\text{ kN}$

$H_{1L} = 0.331 \times 375.9 = 124.42\text{ kN}$

$K_h = 120.1 + 10.85 - 124.42 = 6.53\text{ kN}$

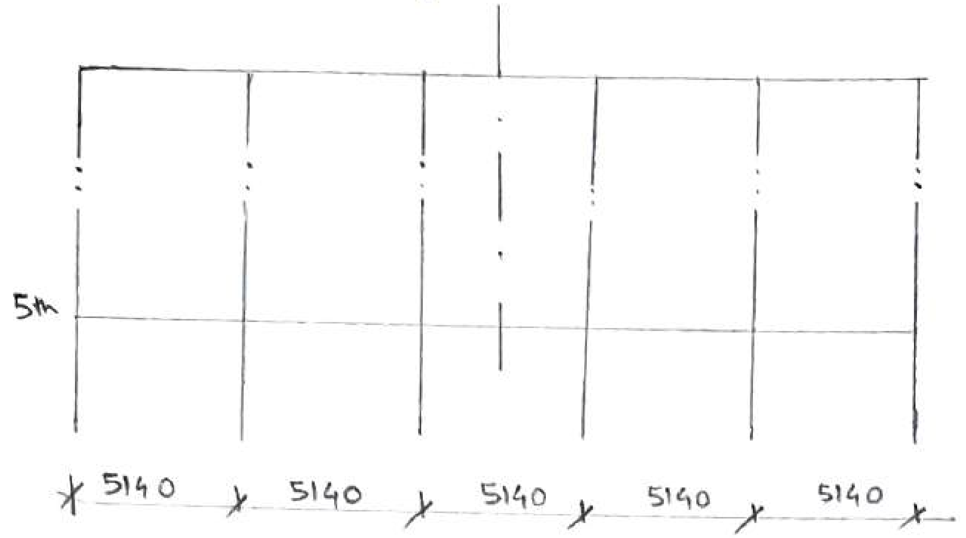
$K_v = -163.04 + 137.96 + 200.67 = 175.59\text{ kN}$

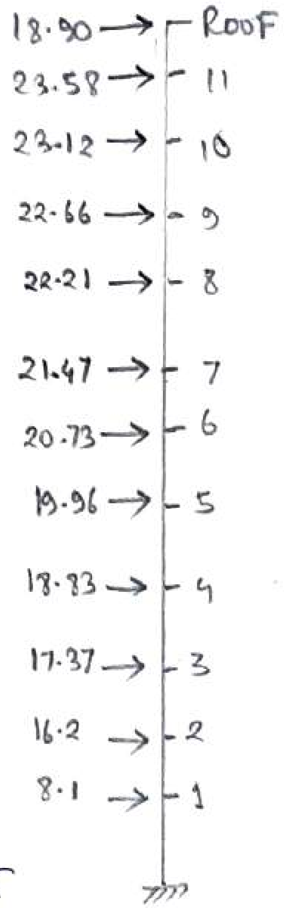




B> For longer frame A2-B2-C2-D2-E2-F2 :-

i) Considering 5th floor :-





□ At mid-height between 5th & 6th floors :-

Overturning moment $= 20.73 \times \frac{3.5}{2} + 21.47 \times 3 \times \frac{3.5}{2} + 22.21 \times 5 \times \frac{3.5}{2} + 22.66 \times 7 \times \frac{3.5}{2} + 23.12 \times 9 \times \frac{3.5}{2} + 23.58 \times 11 \times \frac{3.5}{2} + 18.9 \times 13 \times \frac{3.5}{2} = 1868.95$ kNm

Here, $r_1 = -r_6 = 2 \times 5.14 + \frac{5.14}{2} = 12.85$ m

$r_2 = -r_5 = 5.14 + \frac{5.14}{2} = 7.71$ m

$r_3 = -r_4 = \frac{5.14}{2} = 2.57$ m

$\therefore \Sigma r^2 = 2 \times \{12.85^2 + 7.71^2 + 2.57^2\} = 462.34 \, m^2$

$\therefore V_{1u} = \frac{1868.95}{462.34} \times 12.85 = 51.94$ kN $\Rightarrow V_{6u} = -51.94$ kN

$V_{2u} = \frac{1868.95}{462.34} \times 7.71 = 31.17$ kN $\Rightarrow V_{5u} = -31.17$ kN

$V_{3u} = \frac{1868.95}{462.34} \times 2.57 = 10.39$ kN $\Rightarrow V_{4u} = -10.39$ kN

$\Sigma H = 20.73 + 21.47 + 22.21 + 22.66 + 23.12 + 23.58 + 18.9 = 152.67$ kN

Let us assume, $H_{1u} = H_{6u} = 152.67\alpha$ kN, $H_{2u} = H_{5u} = 152.67\beta$ kN

& $H_{3u} = H_{4u} = 152.67\gamma$ kN.

□ At mid-height between 4th & 5th floors:–

Overturning moment = $1868.95 + 152.67 \times 3.5 + 19.96 \times \frac{3.5}{2}$

$= 2438.22$ kN-m

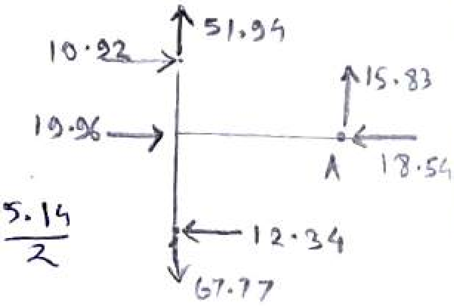
$\therefore V_{1l} = \frac{2438.22}{462.34} \times 12.85 = 67.77$ kN $\Rightarrow V_{6l} = -67.77$ kN

$V_{2l} = \frac{2438.22}{462.34} \times 7.71 = 40.66$ kN $\Rightarrow V_{5l} = -40.66$ kN

$V_{3l} = \frac{2438.22}{462.34} \times 2.57 = 13.55$ kN $\Rightarrow V_{4l} = -13.55$ kN.

$\Sigma H = 152.67 + 19.96 = 172.63$ kN.

Let us assume $H_{1l} = H_{6l} = 172.63\alpha$, $H_{2l} = H_{5l} = 172.63\beta$ & $H_{3l} = H_{4l} = 172.63\gamma$.



Taking Moment about A,

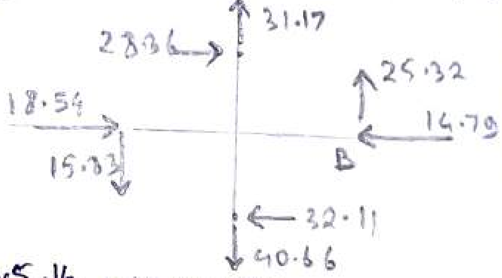
$(172.63\alpha + 152.67\alpha) \times \frac{3.5}{2} = (67.77 - 51.94) \times \frac{5.14}{2}$

$\Rightarrow \alpha = 0.0715$

$\therefore H_{1u} = 152.67 \times 0.0715 = 10.92$ kN, & $H_{1l} = 172.63 \times 0.0715 = 12.34$ kN

$K_h = 19.96 + 10.92 - 12.34 = 18.54$ kN

$K_v = 67.77 - 51.94 = 15.83$ kN.



Taking moment about B,

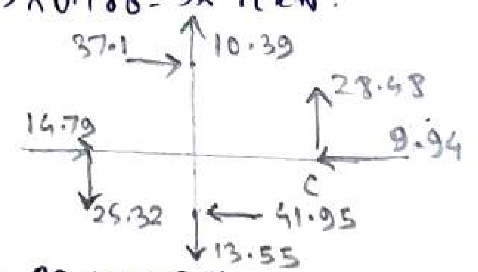
$(172.63\beta + 152.67\beta) \times \frac{3.5}{2} = (40.66 - 31.17) \times \frac{5.14}{2} + 15.83 \times 5.14$

$\Rightarrow \beta = 0.186$

$\therefore H_{2u} = 152.67 \times 0.186 = 28.36$ kN & $H_{2l} = 172.63 \times 0.186 = 32.11$ kN.

$\therefore K_h = -32.11 + 28.36 + 18.54 = 14.79$ kN

$K_v = 40.66 - 31.17 + 15.83 = 25.32$ kN



Taking moment about C,

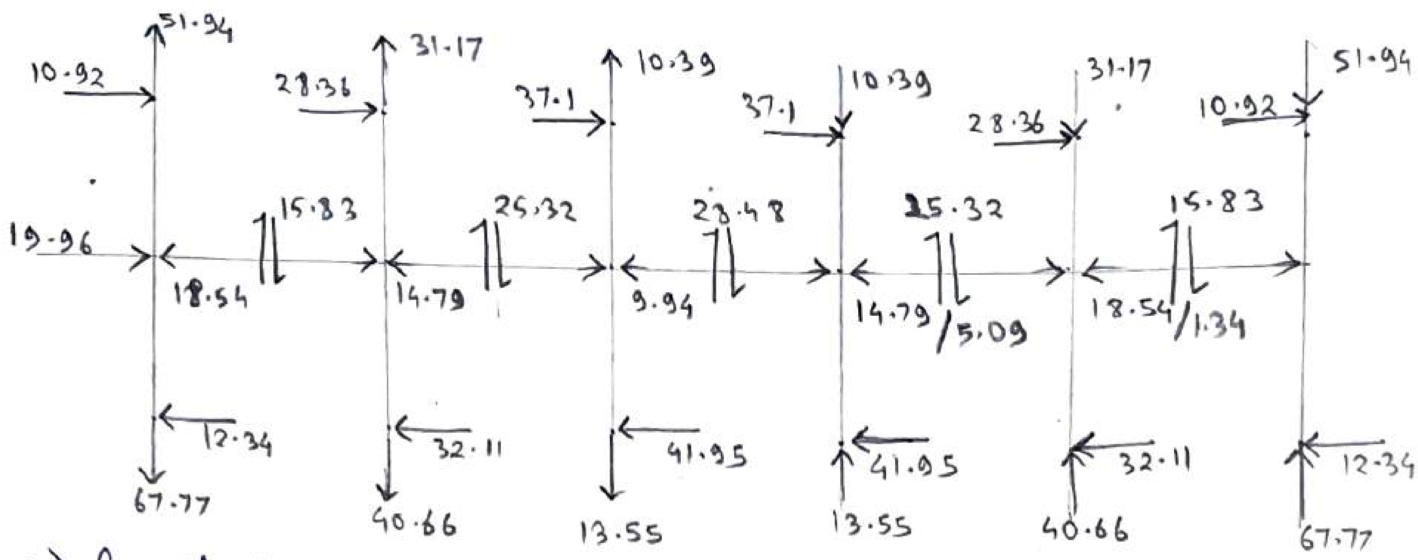
$(172.63\gamma + 152.67\gamma) \times \frac{3.5}{2} = (13.55 - 10.39) \times \frac{5.14}{2} + 25.32 \times 5.14$

$\Rightarrow \gamma = 0.243$

$\therefore H_{3u} = 152.67 \times 0.243 = 37.1$ kN & $H_{3l} = 172.63 \times 0.243 = 41.95$ kN.

$\therefore K_h = 37.1 + 14.79 - 41.95 = 9.94$ kN

$K_v = 13.55 - 10.39 + 25.32 = 28.48$ kN.



ii) Considering 1st floor:-

□ At midheight between 1st & 2nd floors:-

Overturning moment $= 16.2 \times \frac{3.5}{2} + 17.37 \times 3 \times \frac{3.5}{2} + 18.83 \times 5 \times \frac{3.5}{2}$
$+ 19.96 \times 7 \times \frac{3.5}{2} + 20.73 \times 9 \times \frac{3.5}{2} + 21.47 \times 11 \times \frac{3.5}{2} + 22.21 \times 13 \times \frac{3.5}{2}$
$+ 22.66 \times 15 \times \frac{3.5}{2} + 23.12 \times 17 \times \frac{3.5}{2} + 23.58 \times \frac{19 \times 3.5}{2}$
$+ 18.9 \times 21 \times \frac{3.5}{2} = 4535.14$ kN-m

$\therefore V_{1u} = \frac{4535.14}{462.34} \times 12.85 = 126.05$ kN $\Rightarrow V_{6u} = -126.05$ kN

$V_{2u} = \frac{4535.14}{462.34} \times 7.71 = 75.63$ kN $\Rightarrow V_{5u} = -75.63$ kN

$V_{3u} = \frac{4535.14}{462.34} \times 2.57 = 25.21$ kN $\Rightarrow V_{4u} = -25.21$ kN.

$\Sigma H = 16.2 + 17.37 + 18.83 + 19.96 + 20.73 + 21.47 + 22.21 + 18.9 + 22.66$
$+ 23.12 + 23.58 = 225.03$ kN.

□ At midheight between Ground floor & 1st floors:-

Overturning moment $= 4535.14 + 225.03 \times \frac{1}{2}(3.5 + 5.6) + 8.1 \times \frac{5.6}{2}$
$= 5581.71$ kN-m

$V_{1l} = \frac{5581.71}{462.34} \times 12.85 = 155.13$ kN $\Rightarrow V_{6l} = -155.13$ kN

$V_{2l} = \frac{5581.71}{462.34} \times 7.71 = 93.08$ kN $\Rightarrow V_{5l} = -93.08$ kN

$V_{3l} = \frac{5581.71}{462.34} \times 2.57 = 31.03$ kN $\Rightarrow V_{4l} = -31.03$ kN.

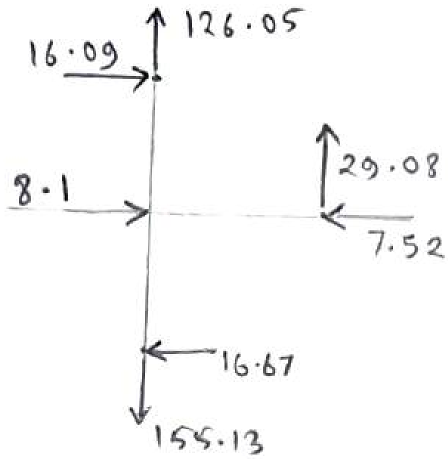
$\Sigma H = 225.03 + 8.1 = 233.13\ kN$

Now, proceeding as before, $\alpha = 0.0715$, $\beta = 0.186$, $\gamma = 0.243$

$\therefore H_{1u} = 0.0715 \times 225.03 = 16.09\ kN$, $H_{1L} = 0.0715 \times 233.13 = 16.67\ kN$

$K_h = 16.09 + 8.1 - 16.67 = 7.52\ kN$

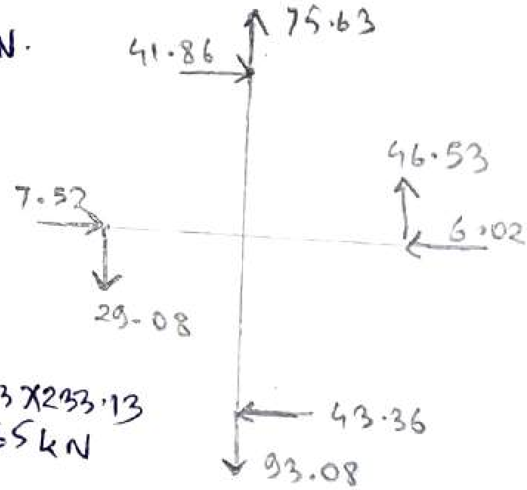
$K_v = 155.13 - 126.05 = 29.08\ kN$



$\therefore H_{2u} = 0.186 \times 225.03 = 41.86\ kN$, $H_{2L} = 0.186 \times 233.13 = 43.36\ kN$.

$K_h = 41.86 + 7.52 - 43.36 = 6.02\ kN$

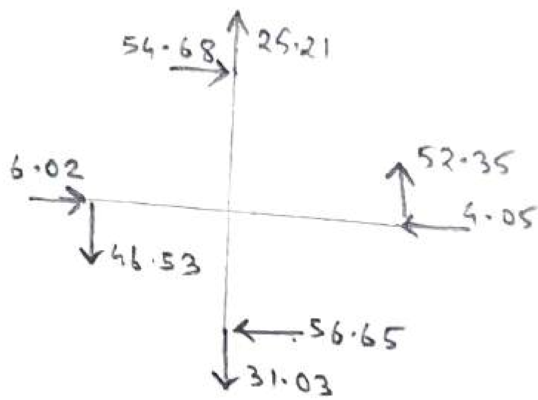
$K_v = 93.08 - 75.63 + 29.08 = 46.53\ kN$.

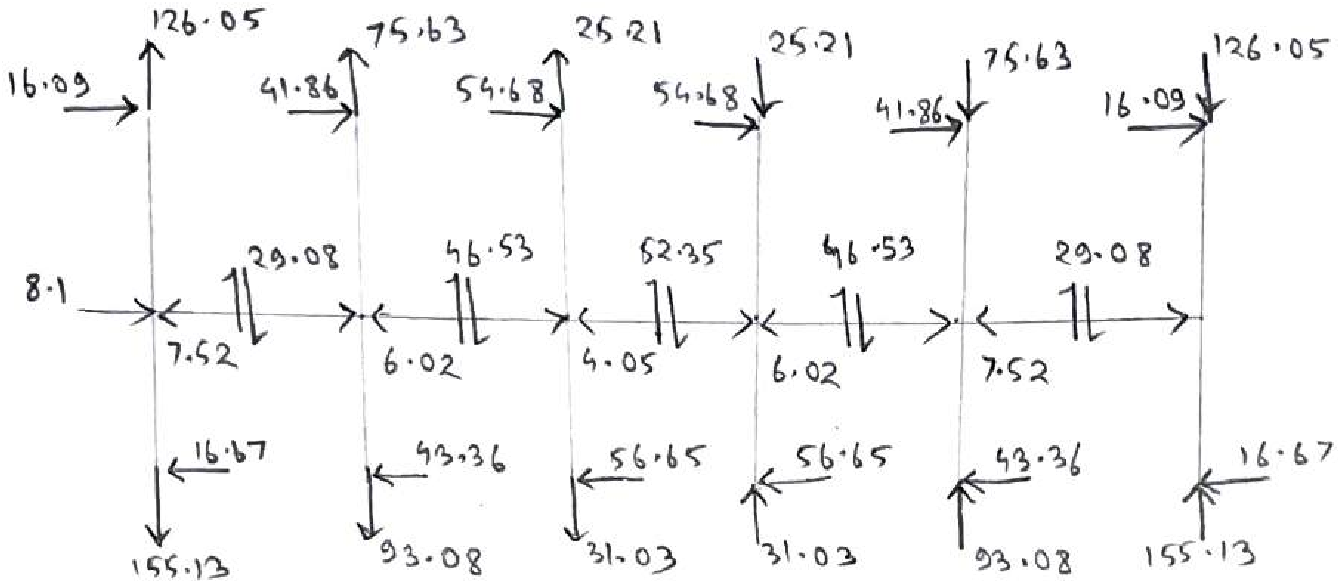


& $H_{3u} = 0.243 \times 225.03 = 54.68\ kN$, $H_{3L} = 0.243 \times 233.13 = 56.65\ kN$

$K_h = 54.68 + 6.02 - 56.65 = 4.05\ kN$

& $K_v = 31.03 - 25.21 + 46.53 = 52.35\ kN$




126.05
75.63
25.21
25.21
75.63
126.05
16.09
41.86
54.68
54.68
41.86
16.09
8.1
29.08
46.53
52.35
46.53
29.08
7.52
6.02
4.05
6.02
7.52
16.67
43.36
56.65
56.65
43.36
16.67
155.13
93.08
31.03
31.03
93.08
155.13

☐ DESIGN OF WATER TANK :-

Area of office-floor per typical floor available

$= 4.953 \times 3.953 + 1.438 \times 4.028 \times 2 + 5.015 \times 3.953 + 3.453 \times 3.953$
$+ 1.438 \times 3.105 + 3.883 \times 4.953 + 1.695 \times 4.953 + 1 \times 2.98$
$+ 12.57 \times 2.98 + 2.57 \times 3.953 + 3.932 \times 3.953 + 3.453 \times 3.953$
$+ 3.953 \times 5.015 + 4.953 \times 3.953 = 215.9\ m^2 = 2323.93 \simeq 2324\ ft^2$

Now assuming 2 persons per 100 ft^2, total number of persons in the building $= 11 \times \frac{2324}{100} \times 2 = 511.28 \simeq 515$

Assuming average daily demand = 45 Lcpd

Requirement of water per day $= 45 \times 515 = 23175\ l = 23.175\ m^3$

Assuming pumping is done twice a day,

Volume of reservoir $= \frac{23.175}{2} = 11.5875\ m^3$

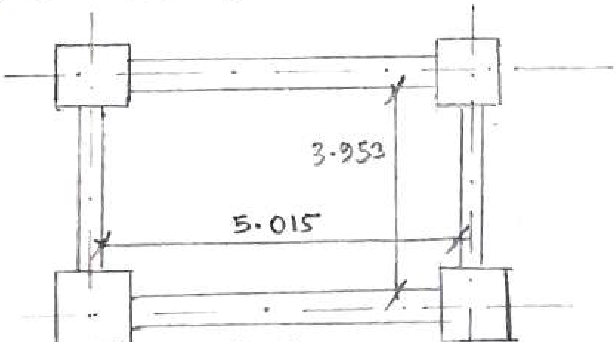
Let us provide internal dimension of the tank as 5.015 m X 3.953 m

$\therefore$ Area $= 5.015 \times 3.953 = 19.824\ m^2$

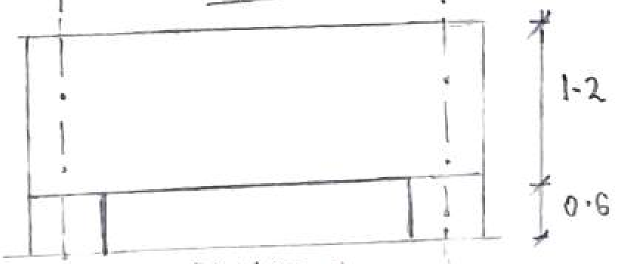
$\therefore$ Height $= \frac{11.5875}{19.824} = 0.585\ m$

Thickness of bottom & top slab is 150 mm with free board 300 mm.

$\therefore$ Total height $= 0.585 + 2 \times 0.15 + 0.3 = 1.185\ m \simeq 1.2\ m$



PLAN



ELEVATION

∴ Outer dimension of water tank = $(5.015 + 0.15 \times 2) \times (3.953 + 0.15 \times 2)$

i.e. $5.315\,m \times 4.253\,m$

▲ Calculation of weight of water tank :-

i) Concrete slabs = $2 \times [(3.953 \times 5.015 \times 0.15) + \{1.2 \times (3.953 - 0.35) \times 0.15\} + \{1.2 \times (5.015 - 0.35) \times 0.15\}] \times 25$

= **223.09** kN

ii) Columns = $4 \times (0.5 \times 0.5) \times (1.2 + 0.6) \times 25 = 45$ kN

iii) Brickwork = $1.3 \times 3.953 \times 0.15 \times 19 = 14.65$ KN.

iv) Cement plaster = $2 \times [(5.015 \times 3.953) + (1.3 \times 3.953) + (1.3 \times 3.953) + (1.3 \times 5.015)] \times 0.013 \times 20.4$

= 18.86 kN.

v) Weight of water = $11.5875 \times 10 = 115.88$ kN.

∴ Total weight = $223.09 + 45 + 14.65 + 18.86 + 115.88$

= 417.48 kN.

□ LIFT MACHINE ROOM :-

Internal dimension of lift machine room = $4.64\,m \times 2.605\,m$

Let us provide, thickness of floor slab = 180 mm

& thickness of roof slab = 100 mm

i) Self weight of slab = $4.64 \times 2.605 \times (0.18 + 0.1) \times 25 = 84.61$ kN

ii) Column = $4 \times 4.1 \times 0.5 \times 0.5 \times 25 = 102.5$ kN.

iii) Beams = $2 \times (4.64 + 2.605) \times 0.65 \times 0.325 \times 25 = 76.52$ kN

iv) Side walls = $2 \times (4.64 + 2.605) \times 2.6 \times 0.25 \times 19 = 186.11$ kN

v) Floor finish = $4.64 \times 2.605 \times 0.04 \times 23.5 = 11.36$ kN

vi) Plaster = $[4.64 \times 2.605 \times 0.013 + 2(4.64 + 2.605) \times 2.6 \times 0.02] \times 20.4$

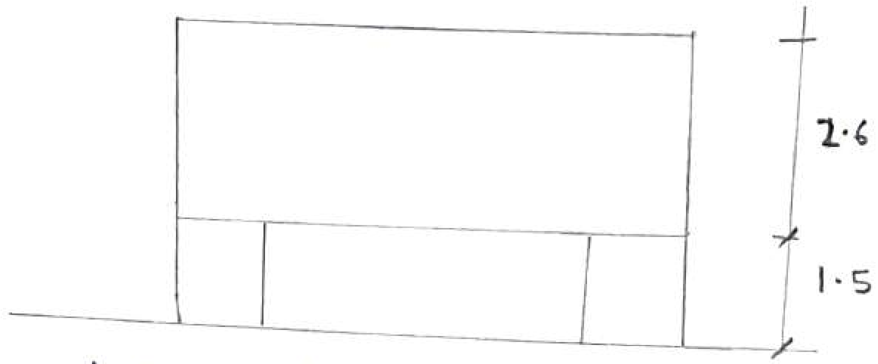
= 18.58 kN.

vii) Line load = 4.64 × 2.605 × 10 = 120.872 kN.

viii) Weight of passengers = 2 × 5.66 = 11.32 kN

[8 persons per lift @ 5.66 kN]

∴ Total load = 84.61 + 102.5 + 76.52 + 186.11 + 11.36 + 18.58 + 120.872 + 11.32 = 611.872 kN.



Development length for M-25 & Fe-500,

$L_d = 49\phi = 49 \times 12 = 588$ mm

∴ Provide development length, L_d = 600 mm

☐ CALCULATION OF WEIGHT FROM STAIRCASE :-

Total weight of staircase (Floor to Floor)

= weight of going + weight of landing

Dead load on going = 7.914 kN/m²

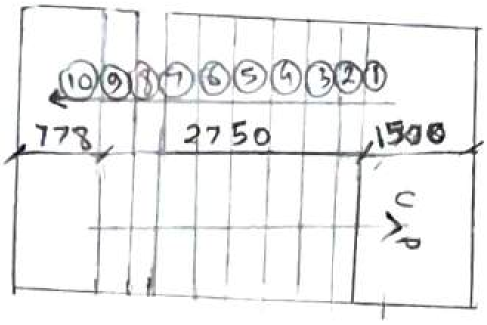
Weight of going = 7.914 × (1.5 × 2 × 3.0) = 69.295 kN

Dead load on landing = 4.962 kN/m²

Weight of landing = 4.962 × 2 × 1.5 × 3.1 = 48.546 kN

∴ Total weight = 69.295 + 48.546 = 117.84 kN.

DEAD LOAD CALCULATION OF STAIRCASE:



Floor to Floor height = 3.5m

Height of flight = $\frac{3.5}{2}$ = 1.75 m

Assume tread = 250 mm & riser = 160 mm

No. of risers = 11.

Hence no. of tread = 11−1 = 10.

Thickness of waist slab = 125 mm

∴ Overall depth considering 10 mm φ bar used and 15 mm clear cover. = 150 mm

wt. of waist slab = $t \times \sqrt{1+\frac{R^2}{T^2}} \times 25 = 0.15 \times \sqrt{1+\frac{0.160^2}{0.25^2}} \times 25$

= 4.452 kN/m

wt. of step/m width = $\frac{1}{2} \times 0.16 \times 25$ = 2 kN/m

10 mm thk marbled flooring @ 0.257 kN/m

10 mm thk base concrete for marble flooring = 0.04 × 23.5
= 0.94 kN/m

13 mm thick plaster @ 0.265 kN/m.

Total DL = 7.914 kN/m

Landing:-

Self wt. of slab = (0.14 × 1 × 25) = 3.50 kN/m

Total DL = 3.5 + 0.94 + 0.265 + 0.257 = 4.962 kN/m

☐ DESIGN OF STAIRCASE :

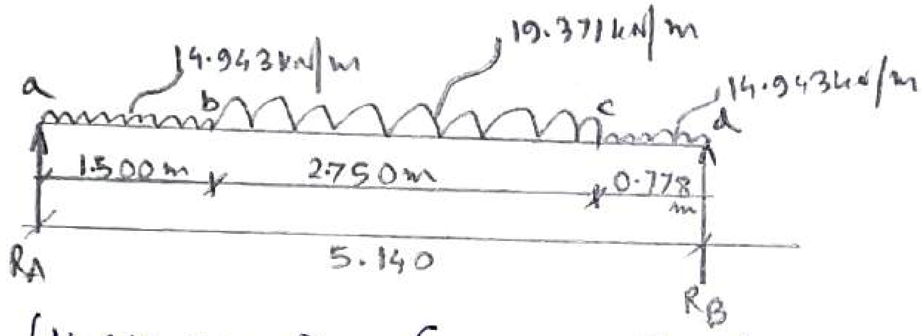
Floor to floor height = 3.5m

Live load = 5kN/m per m width of stair case

$\therefore$ Total DL+LL = w = 4.962 + 5 = 9.962 kN/m
Factored load = w_u = 1.5 × 9.962 = 14.943 kN/m } For landing

$\therefore$ Total DL+LL = w = 7.914 + 5 = 12.914 kN/m
Factored load = w_u = 1.5 × 12.914 = 19.371 kN/m } For stairs

△ Load Diagram :-



$R_A + R_B = (14.943 \times 1.500) + (19.371 \times 2.75) + (14.943 \times 0.778)$
= 87.314 kN.

Taking moment about A,

$R_B \times 5.140 = 14.943 \times \frac{1.5^2}{2} + 19.371 \times 2.75 \times \left(\frac{2.75}{2} + 1.5\right)$
$+ 14.943 \times 0.778 \times \left(\frac{0.778}{2} + 4.25\right) = 223.89$ kNm

$\therefore$ RB = 43.559 kN $\quad \therefore R_A = 43.751$ kN

Let us assume, max. BM occurs at portion bc.

V_x at bc = $R_A - (14.943 \times 1.5) - (19.371 \times x)$
= 21.336 kN at x = 0 m
= −31.934 kN at x = 2.75 m

We know, BM is max. where shear forces changes sign.

M_x at bc = $R_A x - 14.943 \times 1.5 \times \left(x - \frac{1.5}{2}\right) - 19.371 \times \frac{(x-1.5)^2}{2}$

$\frac{dM}{dx} = R_A - (14.943 \times 1.5) - 19.371 \times (x - 1.5) = 0 \Rightarrow x = 2.601$ m from left support

$\therefore$ Max BM $= R_A \times 2.601 - 14.943 \times 1.5 \times \left(2.601 - \frac{1.5}{2}\right) - 10.371 \frac{(2.601-1.5)^2}{2}$

$= 60.566$ kN-m

Now, for $f_{ck} = 25\ N/mm^2$ & $f_y = 500\ N/mm^2$,

$M_u = 0.133\ f_{ck} b d^2 = 0.133 \times 25 \times 1000 \times 150^2$

$= 74.8125$ kN-m > 60.566 kN-m

Hence, section can be designed as singly reinforced.

▲ Area of reinforcement →

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

$\Rightarrow 60.566 \times 10^6 = 0.87 \times 500 \times A_{st} \times 150 \left(1 - \frac{A_{st}}{1000 \times 150} \times \frac{500}{25}\right)$

$\therefore A_{st} - A_{st}^2 \times \frac{1}{7500} - 928.214 = 0$

$\Rightarrow A_{st}^2 - 7500 A_{st} + 6961605 = 0 \Rightarrow A_{st} = \frac{7500 \pm \sqrt{7500^2 - 4 \times 6961605}}{2}$

$= 1085.25\ mm^2$

Using 10 mm ϕ bars, $A_\phi = \frac{\pi}{4} \times 10^2 = 78.54\ mm^2$

Spacing $= \frac{78.54 \times 1000}{1085.25} = 72.37$ mm

Provide 10 mm ϕ bars @ 70mm c/c

• Distribution steel $= 0.12\%$ of area $= \frac{0.12}{100} \times 1000 \times 150 = 180\ mm^2$

$\therefore$ Spacing of 10ϕ bars $= \frac{78.54}{180} \times 1000 = 436.33$ mm

$\therefore$ Provide 10 mm ϕ bars @ 300 mm c/c

• Development length = (Cl. 26.2.1.1 IS 456-2000)

$L_d = \frac{\phi (0.87 f_y)}{4 \tau_{bd}} = \frac{10 \times 0.87 \times 500}{4 \times 1.4 \times 1.6} = 951.56$ mm

$\therefore$ Provide 960 mm development length.

□ CALCULATION OF TOTAL GRAVITY LOAD :-

A Roof :-

(i) Slab :-

Total load = 4.565 × [(4.953 × 3.953) × 2 + (1.438 × 3.953) × 2 + (3.453 × 3.953) × 2 + (3.953 × 5.015) × 3 + (2.98 × 4.953) + (2.98 × 5.015) × 3 + (0.918 × 4.953) × 3 + (1.5 × 3.953) + (3.453 × 3.953)] = 1041.225 kN.

ii) Beam :-

Total load = 5.281 × [4 × 25.7 + 6 × 11.385] + (3 × 5.281 × 4.14) + (5.281 × 5.14 × 3) = 1050.65 kN

iii) Water tank :-

Total load = 417.48 kN

iv) Lift-machine room :-

Total load = 611.87 kN

v) Parapet wall :-

Total load = 0.25 × 1 × 19 × 2 × (25.7 + 11.385)

= 352.31 kN.

vi) Staircase :-

Total load = $2 \times \frac{117.84}{2}$ = 117.84 kN

vi) Stair room :-

(a) Columns = $4 \times 0.5^2 \times 2.4 \times 25$ = 60 kN

(b) Slab = 0.1 × 4.14 × 5.14 × 25 = 53.199 kN

(c) Wall = (2 × 5.14 + 4.14) × 2.4 × 0.25 × 19 = 164.388 kN

Total load = 2 × (60 + 53.199 + 164.388) = 555.174 kN.

∴ Total load from roof = 1041.225 + 1050.66 + 417.48 + 611.87 + 352.31 + 117.84 + 555.175 = **4146.56 kN**

▲ Typical Floor :-

i) Total load Slab :- $= 6.0972 \times \frac{1041.225}{4.565} = 1390.7\ kN$

ii) Beam :-

Total load = 1050.66 kN

iii) Columns :-

Total load $= (4 \times 6) \times 0.5^2 \times 3.5 \times 25 = 525\ kN$

iv) Brickwork :-

Total load = wt. of 250 mm thk wall + 150 mm thk wall + 75 mm thk wall

$= [[0.25 \times (3.5 - 0.15) \times 74.17] + [0.15 \times (3.5 - 0.15) \times 69.744] + \{0.075 \times (3.5 - 0.15) \times 28.725]] \times 19$

$= 1983.24\ kN$

v) Staircase :-

Total load = 117.84 kN

∴ Total load from typical floor = 1390.7 + 1050.66 + 525 + 1983.24 + 117.84 = **5067.44 kN**

▲ Ground floor :-

i) Tie beam :-

Total load $= 0.325 \times 0.65 \times 2 \times (25.7 + 11.385) \times 25$

$= 391.71\ kN$

ii) Columns :-

Total load = $4 \times 6 \times 0.5^2 \times 5.6 \times 25$ = 840 kN

iii) Brickwork :-

Total load = $[[0.25 \times (3.5-0.15) \times 75.688] + [0.15 \times (3.5-0.15) \times 14.7]] \times 19$ = 1344.734 kN

iv) Staircase :-

Total load = 117.84 kN.

∴ Total load from ground floor = 391.71 + 840 + 1344.73 + 117.84
= **2694.283 kN**

Hence, Total Gravity load,

W = 4146.56 + 5067.44 + 2694.283 = **62582.68 kN**

◻ CALCULATION FOR DESIGN BASE SHEAR:

As per cl. no. 7.6.1 of IS-1893 (Part 1) : 2002, Fundamental Natural Time Period for RC frame building is given by :-

$T_a = 0.075\, h^{0.75}$

$= 0.075 \times 44.1^{0.75}$ where, h = Height of the building, = 44.1 m

$= 1.2835$ s

Cl. no. 6.4.2 gives Design Horizontal Seismic Coefficient, A_h as,

$A_h = \frac{Z I}{2R} \frac{S_a}{g}$

$= \frac{0.16 \times 1}{2 \times 3} \times 1.0596$

$= 0.02826$

where, Z = Zone factor = 0.16 [table 2] (Kolkata is in Zone III)

I = Importance factor = 1.0 [table 6]

R = Response reduction factor = 3.0 [table 7, OMRF]

$\frac{S_a}{g}$ = Avg. response accn. coeff = 1.0596 (Fig. 2) [For medium soil with T_a = 1.2835 s]

∴ Design Seismic Base shear [Cl. 7.5.3],

$V_B = A_h W$ where, W = seismic weight of building = 62582.68 kN

= 1768.59 kN & A_h = 0.02826

☐ DESIGN LATERAL FORCE AT DIFFERENT FLOORS ALONG LONGER & SHORTER FRAME :-

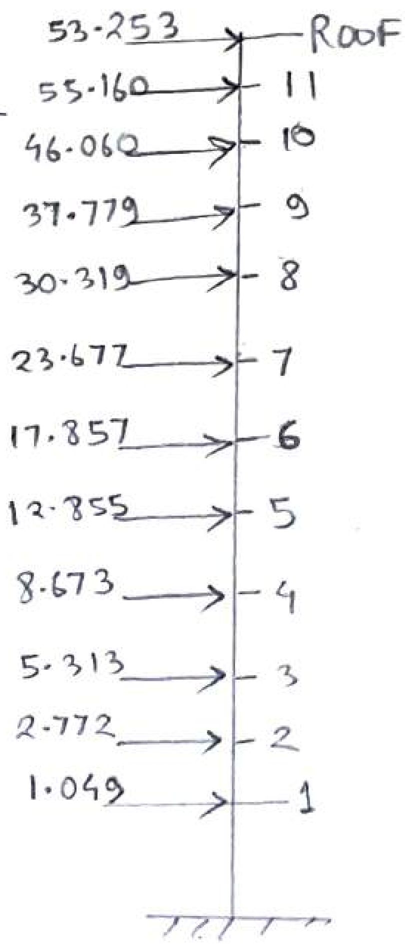
Floor No.	Floor Weight (W_i) (kN)	Height (h_i) (m)	$W_i h_i^2$	Q_i	Force on Longer Frame (kN) ($Q_i/4$)	Force on shorter frame (kN) ($Q_i/6$)
1	5067.44	5.6	158914.92	6.296	1.574	1.049
2	5067.44	9.1	419634.71	16.63	4.158	2.772
3	5067.44	12.6	804506.77	31.88	7.970	5.313
4	5067.44	16.1	1313531.12	52.04	13.010	8.673
5	5067.44	19.6	1946707.75	77.13	19.282	12.855
6	5067.44	23.1	2704036.66	107.14	26.785	17.857
7	5067.44	26.6	3585517.85	142.06	35.515	23.677
8	5067.44	30.1	4591151.31	181.91	45.478	30.319
9	5067.44	33.6	5720937.06	226.67	56.668	37.779
10	5067.44	37.1	6974875.09	276.36	69.090	46.060
11	5067.44	40.6	8352965.40	330.96	82.740	55.160
Roof	4146.56	44.1	8064271.35	319.52	79.880	53.253

where, Design lateral force, $Q_i = V_B \times \dfrac{W_i h_i^2}{\Sigma W_i h_i^2}$, where, $\Sigma W_i h_i^2$ = 44637050 kN.

& V_B = 1768.59 kN.

☐ FOR SHORTER FRAME [C1-C2-C3-C4] :

▲ Considering 5th Floor:-



i) At mid-height between 5th - 6th floor :-

Overturning moment =

$$= 17.857 \times \frac{3.5}{2} + 23.677 \times 3 \times \frac{3.5}{2} + 30.319 \times 5 \times \frac{3.5}{2} + 37.779 \times 7 \times \frac{3.5}{2} + 46.060 \times 9 \times \frac{3.5}{2} + 55.160 \times 11 \times \frac{3.5}{2} + 53.253 \times 13 \times \frac{3.5}{2}$$

$$= 3882.42 \text{ kN-m}$$

Hence $r_1 = -r_4 = 5.6925$ m

$r_2 = -r_3 = 1.5525$ m

$\Sigma r^2 = 69.63 \text{ m}^2$

$\therefore V_{1u} = \frac{3882.42}{69.63} \times 5.6925 = 317.402$ kN $\quad \therefore V_{4u} = -317.402$ kN

& $V_{2u} = \frac{3882.42}{69.63} \times 1.5525 = 86.564$ kN $\quad \therefore V_{3u} = -86.564$ kN

Now, $\Sigma H = 17.857 + 23.677 + 30.319 + 37.779 + 46.060 + 55.160 + 53.253 = 264.105$ kN

Same as before, for shorter span, $\alpha = 0.169$

$\beta = 0.331$

$\therefore H_{1u} = 0.169 \times 264.105 = 44.634$ kN

& $H_{2u} = 0.331 \times 264.105 = 87.419$ kN.

ii) At mid-height between 4th & 5th floor:-

Overturning moment = $3882.42 + 264.105 \times 3.5 + 12.855 \times \frac{3.5}{2}$

$= 4829.28$ kN-m

$\therefore V_{1l} = \frac{4829.28}{69.63} \times 5.6925 = 394.81$ kN $\Rightarrow V_{4l} = -394.81$ kN

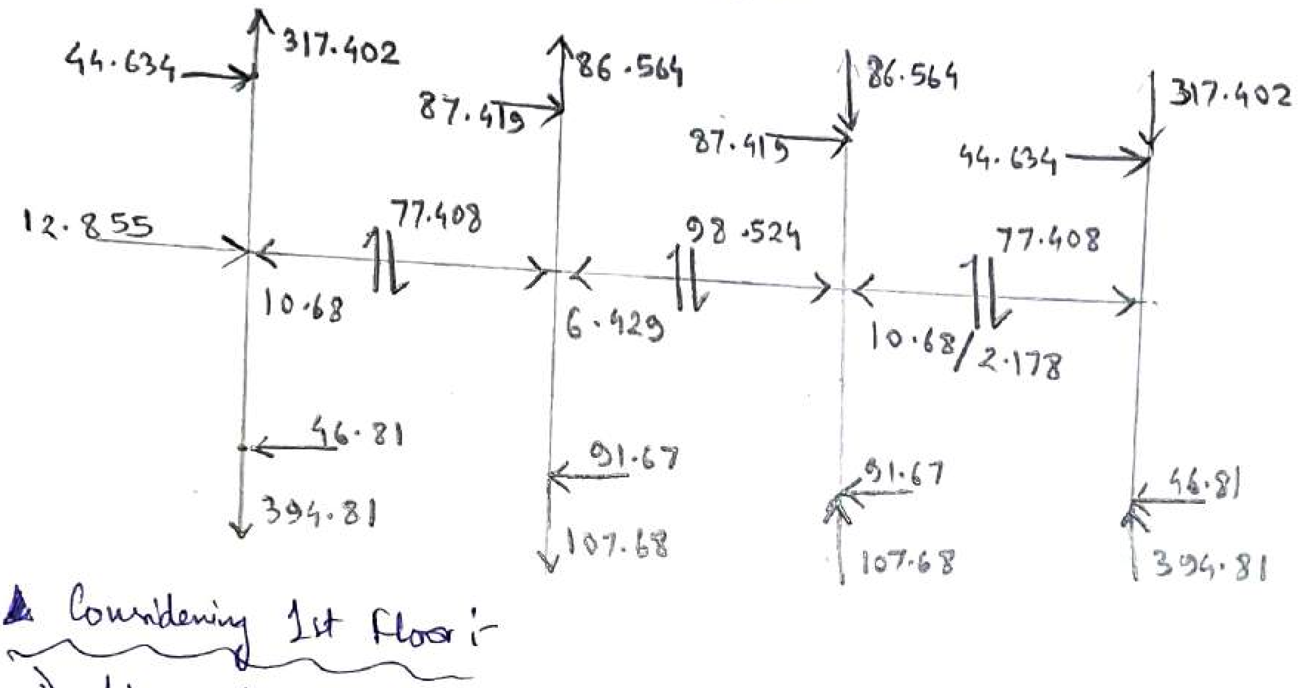
& $V_{2l} = \frac{4829.28}{69.63} \times 1.5525 = 107.68$ kN $\Rightarrow V_{3l} = -107.68$ kN.

$\Sigma H = 264.105 + 12.855 = 276.96$ kN

As, $\alpha = 0.169$, & $\beta = 0.331$.

$\therefore H_{1l} = 0.169 \times 276.96 = 46.81$ kN

$H_{2l} = 0.331 \times 276.96 = 91.67$ kN.



Considering 1st Floor:-

i) At mid-height between 1st & 2nd floors:-

Overturning moment = $2.772 \times \frac{3.5}{2} + 5.313 \times 3 \times \frac{3.5}{2} + 8.673 \times 5 \times \frac{3.5}{2} + 12.855 \times 7 \times \frac{3.5}{2} + 17.857 \times 9 \times \frac{3.5}{2} + 23.677 \times 11 \times \frac{3.5}{2} + 30.319 \times 13 \times \frac{3.5}{2} + 37.779 \times 15 \times \frac{3.5}{2} + 46.060 \times 17 \times \frac{3.5}{2}$ $55.16 \times 19 \times \frac{3.5}{2} + 53.253 \times 21 \times \frac{3.5}{2} = 7845.996$ kN-m

$V_{1u} = \frac{7845.996}{69.63} \times 5.6925 = 641.44$ kN $\Rightarrow V_{4u} = -641.44$ kN

$V_{2u} = \frac{7845.996}{69.63} \times 1.5525 = 174.938$ kN $\Rightarrow V_{3u} = -174.938$ kN

$\Sigma H = 2.772 + 5.313 + 8.673 + 12.855 + 17.857 + 23.677 + 30.319 + 37.779 + 46.06 + 55.16 + 53.253 = 293.718$ kN

$H_{1u} = \underset{(\alpha)}{0.169} \times 293.718 = 49.64$ kN

$H_{2u} = \underset{(\beta)}{0.331} \times 293.718 = 97.22$ kN

ii) At mid-height between Ground floor & 1st floor :-

Overturning moment $= 7845.996 + 293.718 \times \frac{1}{2}(3.5 + 5.6) + 1.049 \times \frac{5.6}{2} = 9185.35$ kN-m

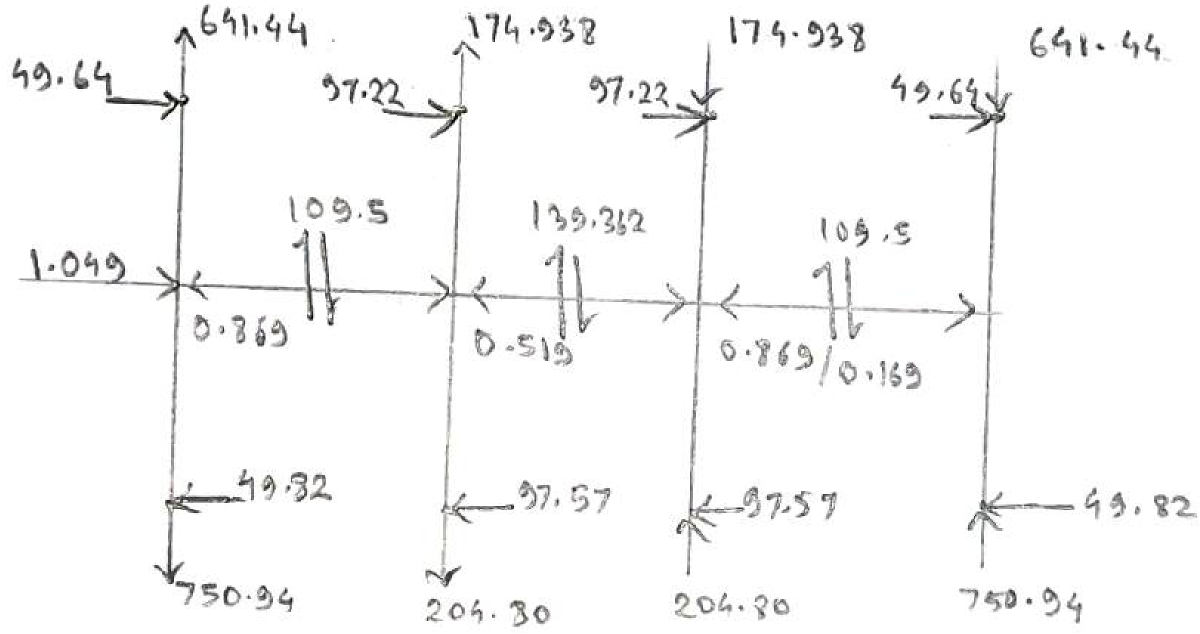
$V_{1l} = \frac{9185.35}{69.63} \times 5.6925 = 750.94$ kN $\Rightarrow V_{4l} = -750.94$ kN

$V_{2l} = \frac{9185.35}{69.63} \times 1.5525 = 204.80$ kN $\Rightarrow V_{3l} = -204.80$ kN

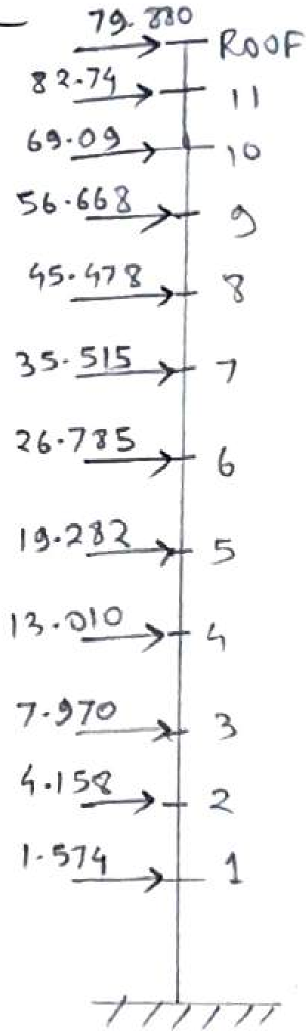
$\Sigma H = 293.718 + 1.049 = 294.767$ kN

$H_{1l} = \underset{(\alpha)}{0.169} \times 294.767 = 49.82$ kN

$H_{2l} = \underset{(\beta)}{0.331} \times 294.767 = 97.57$ kN



□ FOR LONGER FRAME (A2-B2-C2-D2-E2-F2):–



Considering 5th floor:–

i) At midheight between 5th–6th Floor:–

Overturning moment = $26.785 \times \frac{3.5}{2}$
$+ 35.515 \times 3 \times \frac{3.5}{2} + 45.478 \times 5 \times \frac{3.5}{2}$
$+ 56.668 \times 7 \times \frac{3.5}{2} + 69.09 \times 9 \times \frac{3.5}{2}$
$+ 82.74 \times 11 \times \frac{3.5}{2} + 79.880 \times 13 \times \frac{3.5}{2}$
$= 5823.626$ kN-m

Here, $r_1 = -r_6 = 12.85$ m
$r_2 = -r_5 = 7.71$ m.
$r_3 = -r_4 = 2.57$ m

$\therefore \Sigma r^2 = 462.34\ m^2$

$\therefore V_1 = \frac{5823.626}{462.34} \times 12.85 = 161.858$ kN, $V_6 = -161.858$ kN

$V_2 = \frac{5823.626}{462.34} \times 7.71 = 97.115$ kN, $V_5 = -97.115$ kN

$V_3 = \frac{5823.626}{462.34} \times 2.57 = 32.372$ kN, $V_4 = -32.372$ kN

$\Sigma H = 26.785 + 35.515 + 45.478 + 56.668 + 69.09 + 82.74 + 79.88$
$= 396.156$ kN

Same as before, $\alpha = 0.0715$, $\beta = 0.186$ & $\gamma = 0.243$

$\therefore H_{1u} = 0.0715 \times 396.156 = 28.325$ kN

$H_{2u} = 0.186 \times 396.156 = 73.685$ kN

$H_{3u} = 0.243 \times 396.156 = 96.266$ kN

ii) At midheight between 4th & 5th floors :-

Overturning moment $= 5823.626 + 396.156 \times 3.5 + 19.282 \times \frac{3.5}{2}$
$= 7243.86$ kN-m

$V_{1l} = \frac{7243.86}{462.34} \times 12.85 = 201.33$ kN $\Rightarrow V_{6l} = -201.33$ kN.

$V_{2l} = \frac{7243.86}{462.34} \times 7.71 = 120.80$ kN $\Rightarrow V_{5l} = -120.80$ kN

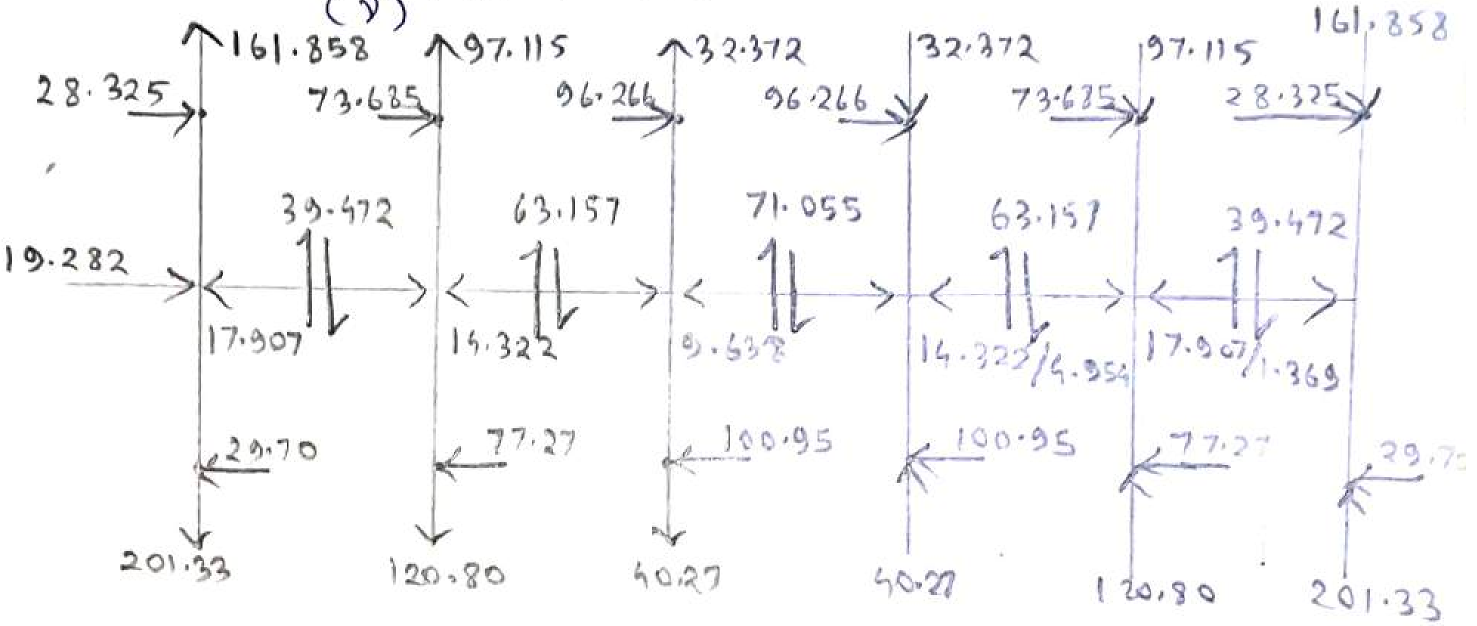
$V_{3l} = \frac{7243.86}{462.34} \times 2.57 = 40.27$ kN $\Rightarrow V_{4l} = -40.27$ kN.

$\Sigma H = 396.156 + 19.282 = 415.44$ kN.

$\therefore H_{1l} = 0.0715 \times 415.44 = 29.70$ kN
(α)

$H_{2l} = 0.186 \times 415.44 = 77.27$ kN
(β)

$H_{3l} = 0.243 \times 415.44 = 100.95$ kN
(γ)



▲ Considering 1st floor :-

i) At midheight between 1st & 2nd floors :-

Overturning moment $= 4.158 \times \frac{3.5}{2} + 7.970 \times 3 \times \frac{3.5}{2} + 5 \times \frac{3.5}{2} \times 13.01 + 7 \times \frac{3.5}{2} \times 19.28$
$+ 9 \times \frac{3.5}{2} \times 26.785 + 11 \times \frac{3.5}{2} \times 35.515 + 13 \times \frac{3.5}{2} \times 45.478 + 15 \times \frac{3.5}{2}$
$\times 56.668 + 17 \times \frac{3.5}{2} \times 69.09 + 19 \times \frac{3.5}{2} \times 82.74 + 21 \times \frac{3.5}{2} \times 79.880$
$= 11768.97$ kN-m

$V_{1u} = \frac{11768.97}{462.34} \times 12.85 = 327.10\,kN$, $\therefore V_{6u} = -327.10\,kN$

$V_{2u} = \frac{11768.97}{462.34} \times 7.71 = 196.26\,kN$ $\therefore V_{5u} = -196.26\,kN$

$V_{3u} = \frac{11768.97}{462.34} \times 2.57 = 65.42\,kN$, $\therefore V_{4u} = -65.42\,kN$

$\Sigma H = 4.158 + 7.970 + 13.01 + 19.282 + 26.785 + 35.515 + 45.478 + 56.668 + 69.09 + 82.74 + 79.88 = 440.576\,kN$

$H_{1u} = \underset{(\alpha)}{0.0715} \times 440.576 = 31.50\,kN$

$H_{2u} = \underset{(\beta)}{0.186} \times 440.576 = 81.95\,kN$

$H_{3u} = \underset{(\gamma)}{0.243} \times 440.576 = 107.06\,kN$.

ii) At midheight between Ground floor & 1st floor :-

Overturning moment $= 11768.97 + 440.576 \times \frac{1}{2}(3.5 + 5.6) + 1.574 \times \frac{5.6}{2}$
$= 13777.99\,kN$.

$V_{1l} = \frac{13777.99}{462.34} \times 12.85 = 382.94\,kN$, $\Rightarrow V_{6l} = -382.94\,kN$

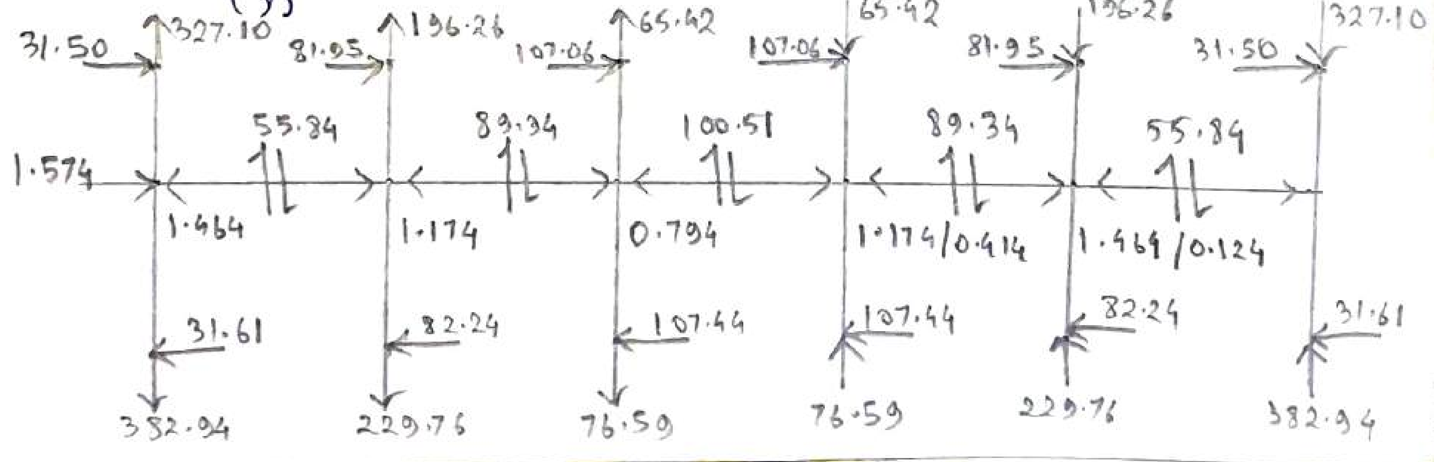
$V_{2l} = \frac{13,777.99}{462.34} \times 7.71 = 229.76\,kN \Rightarrow V_{5l} = -229.76\,kN$

$V_{3l} = \frac{13,777.99}{462.34} \times 2.57 = 76.59\,kN \Rightarrow V_{4l} = -76.59\,kN$

$\Sigma H = 440.576 + 1.574 = 442.15\,kN$.

$\therefore H_{1l} = \underset{(\alpha)}{0.0715} \times 442.15 = 31.61\,kN$, $H_{2l} = \underset{(\beta)}{0.186} \times 442.15 = 82.24\,kN$

& $H_{3l} = \underset{(\gamma)}{0.243} \times 442.15 = 107.44\,kN$.



BEAM CALCULATIONS:

Load Combination	Max BM at Mid Span (kN-m): Middle of End Span	Max BM at Mid Span (kN-m): Middle of Interior span	Max BM at Support (kN-m): End Support	Max BM at Support (kN-m): Next to End Support	Max BM at Support (kN-m): Interior Support	Maximum SF (kN): End Support	Maximum SF (kN): Next to End Supp.	Maximum SF (kN): Interior Support
	(i) For Longer Span Beam							
DL	23.25	25.05	-25.70	-44.61	-46.28	32.72	42.41	42.67
DL+LL	38.80	40.32	-42.90	-72.58	-72.27	54.42	66.57	67.69
WL	–	–	±40.68	±65.07	±73.19	15.83	25.32	28.48
SL	–	–	±101.44	±162.31	±182.61	39.472	63.157	71.055
1.5(DL+LL)	58.2	60.48	-64.35	-108.87	-108.405	81.63	99.855	101.535
1.5(DL+WL)	34.875	37.575	22.47 -99.57	30.69 -164.52	40.365 -179.205	72.825	101.595	106.725
1.5(DL+SL)	34.875	37.575	113.61 -190.71	176.55 -310.38	204.495 -343.335	108.39	158.35	170.59
1.2(DL+LL+WL)	46.56	48.384	-2.664 -100.296	-9.012 -165.18	1.104 -174.552	84.3	110.27	115.404
1.2(DL+LL+SL)	46.56	48.384	70.248 -173.208	107.68 -281.87	132.408 -305.86	112.67	155.67	166.49
Critical Design Value	58.2	60.48	113.61 -190.71	176.55 -310.38	204.495 -343.335	112.67	158.35	170.59
	(ii) For Shorter Span Beam							
DL	21.895	0.57	-19.691	-26.639	–	30.98	34.338	–
DL+LL	33.268	1.873	-29.913	-40.971	–	2.6711	2.6711	–
WL	–	–	±155.33	±148.28	–	75.09	95.51	–
SL	–	–	±160.24	±152.95	–	77.408	98.524	–
1.5(DL+LL)	49.902	2.8095	-44.87	-61.456	–	4.01	4.01	–
1.5(DL+WL)	32.842	0.855	203.46 -262.53	182.46 -262.38	– –	159.03	194.772	–
1.5(DL+SL)	32.842	0.855	210.82 -269.90	189.47 -269.38	– –	162.58	199.29	–
1.2(DL+LL+WL)	39.922	2.248	150.50 -222.29	128.77 -227.10	– –	93.25	117.82	–
1.2(DL+LL+SL)	39.922	2.248	156.39 -228.18	134.37 -232.70	– –	96.09	121.43	–
Critical Design Value	49.902	2.8095	210.82 -269.90	189.47 -269.38	– –	162.58	199.29	–

◻ DESIGN OF LONGER SPAN BEAM:

Maximum support moment = − 343.335 kN-m

= +204.495 kN-m

Maximum span moment = 60.48 kN-m

Maximum Shear force = 170.59 kN.

Assumed total depth of beam, D = 650 mm,

width of beam, b = 325 mm (D : b = 2 : 1)

Let us provide 30 mm nominal cover and reinforcement diameter

= 20 mm

∴ Effective depth = $d = 650 - 30 - \frac{20}{2} = 610$ mm.

Design constant, $R_u = 3.325$.

Considering singly reinforced section, required effective depth,

$$d_{reqd} = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{343.335 \times 10^6}{3.325 \times 325}} = 563.66 \text{ mm} < 610 \text{ mm}$$

∴ Depth is sufficient, hence the section can be designed as a singly reinforced section.

Limiting depth of N.A. = $x_{umax} = \frac{700}{1100 + 0.87 f_y} \times d = 0.456 \times d$

$$\therefore \frac{x_{umax}}{d} = 0.456$$

Let d : b = 1.75 : 1.

Now, $M_u = 0.36 f_{ck} \frac{x_{umax}}{d}\left(1 - 0.416 \frac{x_{umax}}{d}\right) bd^2$

$\Rightarrow 343.335 \times 10^6 = R_u bd^2 \Rightarrow d = 565.36$ mm, $b = 323.06$ mm

≃ 570 mm, ≃ 325 mm

$$\therefore A_{st} = \frac{M_u}{0.87 f_y (d - 0.416 x_{umax})} = \frac{343.335 \times 10^6}{0.87 \times 500 (570 - 0.416 \times 0.456 \times 570)}$$

$= 1708.86 \text{ mm}^2$

Minimum area of reinforcement = $\frac{0.85 \times b \times d}{f_y} = \frac{0.85 \times 325 \times 570}{500}$

$= 314.925 \text{ mm}^2 < 1708.86 \text{ mm}^2$

Hence ok.

Let 20 mm Φ bars be provided.

∴ No. of bars required = $\frac{1708.86}{\frac{\pi}{4} \times 20^2} = 5.439 \simeq 6$

Provide 6-20 Φ bars.

☐ DESIGN OF SHORTER SPAN BEAM:

Maximum Support moment = -269.90 kN-m

= 210.82 kN-m

Mid span moment = 49.902 kN-m

Maximum shear force = 199.29 kN.

Assumed total depth, D = 650 mm, width, b = 325 mm

Let us provide 30 mm nominal cover & reinforcement dia = 20 mm.

∴ $d = 650 - 30 - \frac{20}{2} = 610$ mm

$R_u = 3.325$.

∴ $d_{reqd} = \sqrt{\frac{M_u}{R_u b}} = \sqrt{\frac{269.90 \times 10^6}{3.325 \times 325}} = 499.76 < 610$ mm

∴ Depth is sufficient, hence the section can be designed as a singly reinforced beam. Let d ≃ 525 mm, b = 300 mm

$A_{st} = \frac{M_u}{0.87 f_y (d - 0.416 x_{umax})} = \frac{269.90 \times 10^6}{0.87 \times 500 (525 - 0.416 \times 0.456 \times 300)}$

$= 1325.51$ mm²

Min. area of reinforcement = $\frac{0.85 bd}{f_y} = \frac{0.85 \times 300 \times 525}{500} = 267.75$ mm²

< 1325.51 mm²

Hence ok

Let 20 mm Φ bars be provided

∴ No. of bars required = $\frac{1325.51}{\frac{\pi}{4} \times 20^2} = 4.22 \simeq 5$

Provide 5-20 Φ bars.

▲ Check against positive moment at support :—

i) Longer span beam →

Positive support moment $= 204.495 \, kN\text{-}m$

Here 6-20 ϕ bars are curtailed down to 4-20 ϕ bars at 0.15 L or develop length ($L_d = 49\phi$), whichever is more.

Here, $A_{st} = 4 \times \frac{\pi}{4} \times 20^2 = 1256.64 \, mm^2$,

$d = 650 - 30 - \frac{20}{2} = 610 \, mm$

$\therefore$ Moment capacity, $M_u = 0.87 \, f_y \, A_{st} \, d \left(1 - \frac{A_{st}}{bd} \cdot \frac{f_y}{f_{ck}}\right)$

$= 0.87 \times 500 \times 1256.64 \times 610 \times \left(1 - \frac{1256.64}{325 \times 610} \times \frac{500}{25}\right)$

$= 291.18 \, kN\text{-}m > 204.495 \, kN\text{-}m$

Hence safe.

ii) Shorter span beam →

Positive support moment $= 210.82 \, kNm$

Here 5-20 ϕ bars are curtailed down to 3-20 ϕ bars at 0.15 L or develop length ($L_d = 49\phi$), whichever is more.

Here, $A_{st} = 3 \times \frac{\pi}{4} \times 20^2 = 942.48 \, mm^2$.

$d = 650 - 30 - \frac{20}{2} = 610 \, mm$

$\therefore$ Moment capacity, $M_u = 0.87 \, f_y \, A_{st} \, d \left(1 - \frac{A_{st}}{bd} \cdot \frac{f_y}{f_{ck}}\right)$.

$= 0.87 \times 500 \times 942.48 \times 610 \times \left(1 - \frac{942.48}{325 \times 610} \times \frac{500}{25}\right)$

$= 226.31 \, kN\text{-}m > 210.82 \, kNm$

Hence safe

Check against span moment :—

(a) Longer Span beam →

The beam acts as a T-beam in the span. So, the moment capacity is to be calculated as a singly reinforced T-beam.

As per clause no. 23.1.2 of IS 456-2000 effective flange width of the beam is given as,

$$b_f = \frac{L_o}{6} + b_w + 6D_f$$

$$= \text{Width of web} + \frac{1}{2} \times \left(\begin{array}{l}\text{Sum of the clear distances to the} \\ \text{adjacent beam on either side}\end{array}\right)$$

whichever is less.

· Here L_o may be taken as $0.7 \times \text{span} = 0.7L$.

$$\therefore b_f = \frac{0.7 \times 5.14}{6} + 0.325 + 6 \times 0.145 = 1.795\ m$$

$$= 0.325 + \frac{1}{2} \times (3.105 + 4.14 - 2 \times 0.325) = 3.6225\ m$$

$$\therefore b_f = 1.795\ m$$

Now, depth of neutral axis, $x_u = \dfrac{0.87 \times 500 \times 6 \times \frac{\pi}{4} \times 20^2}{0.36 \times 25 \times 1795}$

$$= 50.76\ mm < 145\ mm.$$

Therefore, neutral axis lies within the flange.

∴ Moment capacity of the T-beam is given as,

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

$$= 0.87 \times 500 \times 6 \times \frac{\pi}{4} \times 20^2 \times 610 \times \left(1 - \frac{500}{25} \times \frac{6 \times \frac{\pi}{4} \times 20^2}{1795 \times 610}\right)$$

$$= 482.95\ kNm > 60.48\ kN\text{-}m$$

(Max. span moment)

Hence the section is also safe under mid span moment.

(ii) Shorter Span beam →

As stated before,

$b_f = \frac{l_0}{6} + b_w + 6D_f$

$= b_w + \frac{1}{2} \times$ (Sum of the clear distances to the adjacent beam on either side.)

$\therefore b_f = \frac{0.7 \times 3.105}{6} + 0.325 + 6 \times 0.145 = 1.557$ m

$= 0.325 + \frac{1}{2} \times (2 \times 5.14 - 2 \times 0.325) = 5.14$ m

$\therefore b_f = 1.557$ m $= 1557$ mm

$\therefore$ Depth of NA, $x_u = \frac{0.87 \times 500 \times 5 \times \frac{\pi}{4} \times 20^2}{0.36 \times 25 \times 1557} = 48.76$ mm < 145 mm

Therefore, neutral axis lies well within the flange.

$\therefore$ Moment capacity of T-beam is given as,

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

$= 0.87 \times 500 \times 5 \times \frac{\pi}{4} \times 20^2 \times 610 \times \left(1 - \frac{500}{25} \times \frac{5 \times \frac{\pi}{4} \times 20^2}{1557 \times 610}\right)$

$= 403.02$ kN-m > 49.902 kN-m.

Hence the section is safe under mid-span moment.

Now maximum shear force $= 199.29$ kN.

$\therefore$ Shear stress $= \tau_v = \frac{199.29 \times 10^3}{325 \times 610} = 1.005$ N/mm^2

Percentage of tensile reinforcement, $P_t = \frac{942.48}{325 \times 610} \times 100 = 0.475\%$

From Table 19 of IS 456:2000, for $P_t = 0.475\%$, design shear strength,

$\tau_c = 0.477$ N/mm^2

$\therefore$ Net shear force $= 199.29 - 0.477 \times 325 \times 610 \times 10^{-3} = 104.725$ kN

Provide 8 mm ϕ 2-legged stirrups, $A_{sv} = 2 \times 50.3 = 100.6$ mm^2

$\therefore$ Spacing reqd. $= S_v = \frac{0.87 \times 500 \times 100.6 \times 610}{104.725 \times 10^3} = 254.90$ mm (∴ Provide 8mm ϕ 2legged stirrups @ 250 c/c)

☐ COLUMN CALCULATIONS:

▲ LOAD CALCULATION:

Column C2 is the column under consideration.

From previous calculation,

Influence area, $A_F = 18.62\ m^2$

DL of slab $= 6.0972\ kN/m^2$.

Brickwork load $= 19.5\left[0.15 \times (3.5-0.14) \times 5.14 + 0.15 \times (3.5-0.14) \times \frac{4.14 \times 3.105}{2}\right]$

$= 86.12\ kN$.

Beam self wt $= 5.281\ kN/m$

Column self wt $= 0.5^2 \times 3.6 \times 25 = 22.50\ kN$

Dead load at 5th floor :-

$DL_5 = 18.62 \times 6.0972 \times 8 + 86.12 \times 7 + 22.50 \times 7 + 5.281 \times \left(5.14 + \frac{4.14+3.105}{2}\right) \times 8 = 2008.15\ kN.$

Dead load at ground floor,

$DL_G = 18.62 \times 6.0972 \times 12 + 86.12 \times 11 + 22.50 \times 11 + 22.50 \times \frac{5.6}{2.9} + 5.281 \times \left(5.14 + \frac{4.14+3.105}{2}\right) \times 12 = 2590.33\ kN$

As per clause 3.2.1 of IS 875 pt II,

Reduction in live load = 40% for no. of floors above 5 and below 10

= 50% for no. of floor above > 10

∴ Live load at 5th floor,

$LL_5 = 0.5 \times [18.62 \times 2.5 + 18.62 \times 4 \times 7] = 283.96\ kN$

& Live load at ground floor,

$LL_G = 0.5 \times [18.62 \times 2.5 + 18.62 \times 4 \times 11] = 432.915\ kN$

For 5th floor,

DL: $M_x = \frac{1}{2} \times (26.6386 - 14.6424) = 5.998$ kN-m

$M_y = \frac{1}{2} \times (46.2845 - 45.1374) = 0.574$ kN-m

DL+LL: $M_x = \frac{1}{2}(40.97101 - 23.3071) = 8.832$ kN-m

$M_y = \frac{1}{2}(72.2735 - 72.1142) = 0.0796$ kN-m

for Ground floor,

At 1st Floor level, DF of upper column = 0.280 } Along y direction
DF of lower column = 0.175

DF of upper column = 0.225 } Along x-direction
DF of lower column = 0.141

DL: $M_x = \frac{0.141}{0.225+0.141}(26.6386 - 14.6424) = 4.621$ kN-m

$M_y = \frac{0.175}{0.280+0.175}(46.2845 - 45.1374) = 0.442$ kN-m

DL+LL: $M_x = \frac{0.141}{0.225+0.141}(40.97101 - 23.3071) = 6.805$ kN-m

$M_y = \frac{0.175}{0.280+0.175}(72.2735 - 72.1142) = 0.0612$ kN-m

LOAD COMBINATION FOR COLUMN AT GROUND & 5th FLOOR:

LOAD COMBINATION	AXIAL LOAD (P) (KN)	Mx (KN-m)	My (KN-m)
	GROUND FLOOR		
DEAD LOAD	2590.33	4.621	0.442
DEAD LOAD + LIVE LOAD	3023.245	6.805	0.0612
WIND LOAD → Short(x)	200.67	348.38	—
→ Long (y)	31.03	—	158.62
SEISMIC LOAD → Short(x)	204.80	273.196	—
→ Long (y)	76.59	—	300.832
1.5 (DL+LL)	4534.868	10.208	0.0918
1.5(DL+WL) → Short(x)	4186.500	529.502	0.663
→ Long (y)	3932.04	6.9315	238.593
1.5(DL+SL) → Short(x)	4192.695	416.726	0.663
→ Long (y)	4000.380	6.9315	451.911
1.2 (DL+LL+WL) → Short(x)	3868.692	426.222	0.0734
→ Long (y)	3665.13	8.166	190.417
1.2(DL+LL+SL) → Short(x)	3873.648	336.001	0.0734
→ Long (y)	3719.80	8.166	361.072
	5TH FLOOR		
DEAD LOAD	2008.15	5.998	0.574
DEAD LOAD + LIVE LOAD	2292.11	8.832	0.0796
WIND LOAD → Short(x)	67.19	142.485	—
→ Long (y)	10.39	—	64.925
SEISMIC LOAD → Short(x)	86.564	152.983	—
→ Long (y)	32.372	—	168.466
1.5(DL+LL)	3438.165	13.248	0.1194
1.5 (DL+WL) → Short(x)	3113.01	222.724	0.861
→ Long (y)	3027.81	8.997	98.248
1.5 (DL+SL) → Short(x)	3142.071	238.472	0.861
→ Long (y)	3060.783	8.997	253.56
1.2 (DL+LL+WL) → Short(x)	2831.16	181.580	0.0955
→ Long (y)	2763.00	10.598	78.006
1.2 (DL+LL+SL) → Short(x)	2854.409	194.178	0.0955
→ Long (y)	2789.378	10.598	202.255

☐ DESIGN OF GROUND FLOOR COLUMN:

The check for biaxial bending under critical load combination is shown here:-

Critical combination (Highest moment in x or y direction)

→ 1.5 (DL+WL) along x direction (short)

$\therefore P_u = 4186.500$ kN, $M_{xu} = 529.502$ kN-m, $M_{yu} = 0.663$ kN-m

Column dimensions b = 500 mm, D = 500 mm.

Let us provide nominal cover of 50mm & reinforcement dia of 28 mm.

$\therefore d' = 50 + \frac{28}{2} = 64$ mm

$\therefore \frac{d'}{D} = \frac{64}{500} = 0.128$

Let us provide steel reinforcement in the column, $p_t = 3.75\%$

$\left(\frac{d'}{D} = 0.128, \frac{P_u}{f_{ck} bD} = 0.6698, \frac{p_t}{f_{ck}} = 0.150\right)$ (chart 48,49, sp-16)

We get, $\frac{M_u}{f_{ck} bD^2} = 0.173 \Rightarrow M_u = 0.173 \times 25 \times 500 \times 500^2$
$= 540.625$ kN-m

As it is a square column,

$M_{ux_1} = M_{uy_1} = 540.625$ kN-m

Now from clause no. 39.6 from IS 456 : 2000,

$P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_{sc}$, where, $A_{sc} = 3.75\%$ of A_g
$= 0.0375 A_g$

& A_g = Gross sectional area
$= 0.5^2 = 250000$ mm²

$A_c = A_g - A_{sc} = 0.9625 A_g$

$\therefore P_{uz} = 0.45 \times 25 \times 0.9625 \times 250000 + 0.75 \times 500 \times 0.0375 \times 250000$
$= 6222.66$ kN

Now, $\frac{P_u}{P_{uz}} = \frac{4186.500}{6222.66} = 0.673$

For values of $\frac{P_u}{P_{uz}}$ between 0.2 to 0.8, the value of α_n vary linearly between 1.0 to 2.0

$$\alpha_n = 1 + \frac{2-1}{0.8-0.2} \times (0.673 - 0.2) = 1.788$$

$$\therefore \left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha_n} = \left(\frac{529.502}{540.625}\right)^{1.788} + \left(\frac{0.663}{540.625}\right)^{1.788}$$

$$= 0.9635 \leq 1.0$$

Hence the column is safe under combined axial load & biaxial loading.

∴ Percentage of steel reinforcement, $P_t = 3.75\%$

$$\therefore A_{sc} = \frac{3.75}{100} \times 500 \times 500 = 9375\ mm^2$$

Cross-sectional area for each 28 mm dia bar, $\frac{\pi}{4} \times 28^2$

$$= 615.75\ mm^2$$

$$\therefore \text{No. of bars required} = \frac{9375}{615.75} = 15.225 \simeq 16$$

∴ Provide 16 no. 28 mm dia bar equally distributed on each side

☐ DESIGN OF FIFTH FLOOR COLUMN:

Critical combination: 1.5(DL+SL) in x direction (short)

where, $P_u = 3142.071$, $M_{ux} = 238.472$ & $M_{uy} = 0.861$

Column dimension: b = 500 mm, D = 500 mm.

Let us provide nominal cover of 50 mm & reinforcement dia of 20 mm

$$\therefore d' = 50 + \frac{20}{2} = 60\ mm$$

$$\therefore \frac{d'}{D} = \frac{60}{500} = 0.12$$

Let us provide steel reinforcement in the column, $P_t = 3.25\%$

$$\frac{P_t}{f_{ck}} = 0.130, \quad \frac{P_u}{f_{ck}bD} = 0.503, \text{ (Ref. Chart 48 \& 49 of SP 16)}$$

$$\frac{M_u}{f_{ck}bD^2} = 0.15 \Rightarrow M_u = 0.15 \times 25 \times 500 \times 500^2 = 468.75\ kN\text{-}m$$

As it is a square column, $M_{ux_1} = M_{uy_1} = 468.75$ kN-m

$\therefore P_{uz} = 0.45 f_{ck} A_c + 0.75 f_y A_g$, where $A_{sc} = 3.25\%$ of $A_g = 0.0325 A_g$, $A_c = A_g - A_{sc} = 0.9675 A_g$

$\therefore P_{uz} = 0.45 \times 25 \times 0.9675 \times 250000 + 0.75 \times 500 \times 0.0325 \times 250000$

$= 5767.97\ kN$

Now, $\frac{P_u}{P_{uz}} = \frac{3142.071}{5767.97} = 0.545$

For $\frac{P_u}{P_{uz}} = 0.545$, $\alpha_n = 1 + \frac{2-1}{0.8-0.2}(0.545 - 0.2) = 1.574$

$\therefore \left(\frac{M_{ux}}{M_{ux_1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy_1}}\right)^{\alpha_n} = \left(\frac{238.472}{468.75}\right)^{1.574} + \left(\frac{0.861}{468.75}\right)^{1.574} = 0.345$

≤ 1.0

Hence the column is safe under combined axial load & biaxial bending.

Percentage of steel reinforcement = p_t = 3.25%

$A_{sc} = \frac{3.25}{100} \times 500 \times 500 = 4983.5\ mm^2$

Cross-sectional area for each 20mm dia bar $= \frac{\pi}{4} \times 20^2 = 314.159\ mm^2$

$\therefore$ No. of bars required $= \frac{4983.5}{314.159} = 15.863 \simeq 16$

$\therefore$ Provide 16 nos. 20mm dia bar equally distributed on each side

Also provide three lateral ties of 8mm Φ dia @250 mm c/c

From Table 4.4 of SP-34, the lap length for 20mm Φ bar of $f_y = 500\ N/mm^2$ is given as 777 mm for M25 grade of concrete.

Let us provide a lap length of 800 mm at 5th floor.

☐ CHECK FOR BI-AXIAL BENDING AGAINST EACH LOAD COMBINATION

DATA:

P_u (KN)	M_{ux} (KN-m)	M_{uy} (KN-m)	$\frac{P_u}{f_{ck} bD}$	$\frac{M_u}{f_{ck} bD^2}$	M_{ux1} or M_{uy1}	$\frac{P_u}{P_{uz}}$	α_n	$\left(\frac{M_{ux}}{M_{ux1}}\right)^{\alpha_n} + \left(\frac{M_{uy}}{M_{uy1}}\right)^{\alpha_n} \le 1$	Remarks
GROUND FLOOR, $P_t = 3.75\%$, $\frac{P_t}{f_{ck}} = 0.150$, $\frac{d'}{d} = 0.128$, $P_{uz} = 6222.66$ kN									
4534.87	10.208	0.098	0.726	0.120	375	0.73	1.9	0.001	Safe
4186.50	529.502	0.663	0.670	0.173	540.62	0.67	1.8	0.964	Safe
3932.04	6.9315	238.59	0.629	0.140	437.50	0.63	1.7	0.353	Safe
4192.69	416.726	0.663	0.671	0.150	468.75	0.67	1.8	0.810	Safe
4000.38	6.9315	451.911	0.640	0.147	459.38	0.64	1.7	0.973	Safe
3868.69	426.222	0.0734	0.620	0.140	437.50	0.62	1.7	0.956	Safe
3665.13	8.166	190.417	0.586	0.150	468.75	0.59	1.6	0.228	Safe
3873.65	336.001	0.0734	0.619	0.140	437.50	0.62	1.7	0.638	Safe
3719.80	8.166	361.072	0.595	0.150	468.75	0.60	1.7	0.649	Safe
5th FLOOR, $P_t = 3.25\%$, $\frac{P_t}{f_{ck}} = 0.130$, $\frac{d'}{D} = 0.120$, $P_{uz} = 5767.97$ kN									
3438.16	13.248	0.1194	0.550	0.140	437.5	0.60	1.7	0.003	Safe
3113.01	222.724	0.861	0.498	0.150	468.75	0.54	1.6	0.312	Safe
3027.81	8.997	98.248	0.484	0.149	465.62	0.52	1.5	0.093	Safe
3142.071	238.472	0.861	0.500	0.150	468.75	0.54	1.6	0.345	Safe
3060.78	8.997	253.56	0.490	0.150	468.75	0.53	1.6	0.388	Safe
2831.16	181.58	0.0955	0.453	0.170	531.25	0.49	1.5	0.203	Safe
2763.00	10.598	78.006	0.442	0.160	500	0.48	1.5	0.069	Safe
2854.41	194.178	0.0955	0.457	0.170	531.25	0.49	1.5	0.223	Safe
2789.38	10.598	202.26	0.446	0.170	531.25	0.48	1.5	0.244	Safe

(Pile Foundation Calculation) Load Combination for Piles:

Load Combination	Axial Load (P) (kN)	Mx (kN-m)	My (kN-m)
Dead load	2590.33	4.621	0.442
Dead load + live load	3023.245	6.805	0.0612
Wind load → Short (x)	200.67	348.38	—
→ Long (y)	31.03	—	158.62
Seismic load → Short (x)	204.80	273.196	—
→ Long (y)	76.59	—	300.832
DL + LL	3023.245	6.805	0.0612
DL + WL → Short (x)	2791	353.00	0.442
→ Long (y)	2621.36	4.621	159.062
DL + SL → Short (x)	2795.13	277.817	0.442
→ Long (y)	2666.92	4.621	301.274
DL + LL + WL → Short (x)	3223.91	355.185	0.0612
→ Long (y)	3054.275	6.805	158.68
DL + LL + SL → Short (x)	3228.04	280.00	0.0612
→ Long (y)	3099.83	6.805	300.893

Design of Pile Foundation:

Assumed Pile Capacity = 500 kN, Pile diameter = 400 mm.

Among various load combinations, maximum axial load on pile group = 3228.04 kN.

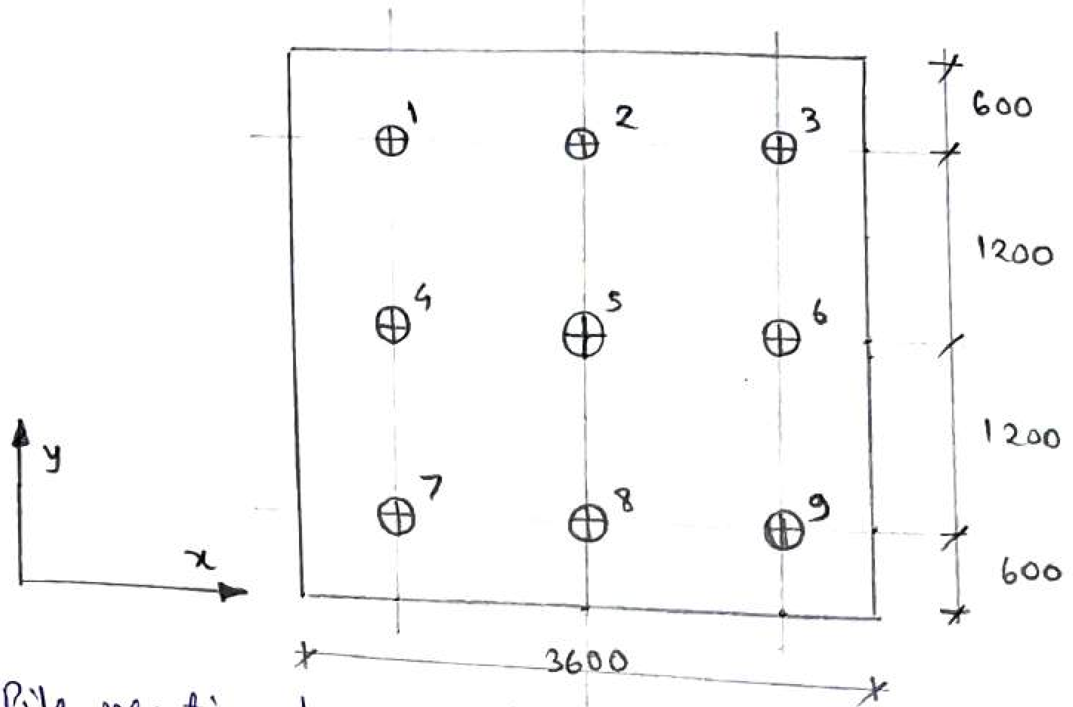
Hence no. of piles required = $\frac{3228.04}{500} = 6.456$

∴ Let us provide 9 nos. of piles as shown in the following page.

Minimum edge distance = 1.5 × Pile diameter = 1.5 × 400 = 600 mm

c/c distance between piles = 3 × Pile diameter = 3 × 400 = 1200 mm.

∴ Length of square pile cap = 2 × 600 + 2 × 1200 = 3600 mm.



Pile reaction of each pile is calculated against every load combination in the following table.

Reaction of Pile 3 under load combination (DL+LL+LLR) short (x) is shown as an example here for detailed calculations:-

$$P = 3223.91 \text{ kN}, \quad M_x = 355.185 \text{ kN-m}, \quad M_y = 0.0612 \text{ kN-m}$$

Origin being at the centre of the pile cap, for pile 3,

$$x = 1.2 m, \; y = 1.2 m$$

Now $\Sigma x^2 = \Sigma y^2 = 6 \times 1.2^2 = 8.64 \; m^2$

$\therefore$ Pile reaction $= \dfrac{P}{n} \pm \dfrac{M_x y}{\Sigma y^2} \pm \dfrac{M_y x}{\Sigma x^2}$

$$= \frac{3223.91}{9} + \frac{355.185 \times 1.2}{8.64} + \frac{0.0612 \times 1.2}{8.64}$$

$$= 407.55 \text{ kN} < \text{Pile capacity} = 500 \text{ kN}.$$

$\therefore$ Hence Safe.

It is also to be noted that no pile should at all be in tension. If the pile reaction is found to be negative, the arrangement of piles must be redesigned.

□ DETERMINATION OF PILE REACTION FOR EACH LOAD COMBINATION

$\Sigma x^2 = \Sigma y^2 = 8.64$ ← Pile reaction kN < 500 kN →

Axial load (P) (kN)	Mx (kN-m)	My (kN-m)	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Pile 9
3023.245	6.805	0.0612	336.85	336.86	336.87	335.91	335.92	335.92	334.96	334.97	334.98
2791.00	353.00	0.442	359.08	359.14	359.20	310.05	310.11	310.17	261.02	261.08	261.14
2621.36	4.621	159.062	269.81	291.90	313.99	269.17	291.26	313.35	268.53	290.62	312.71
2795.13	277.817	0.442	349.09	349.16	349.22	310.51	310.57	310.63	271.92	271.98	272.05
2666.92	4.621	301.276	255.12	296.97	338.81	254.48	296.32	338.17	253.84	295.68	337.53
3223.91	355.185	0.0612	407.54	407.54	407.55	358.20	358.21	358.22	308.87	308.88	308.89
3054.275	6.805	158.68	318.27	340.31	362.35	317.32	339.36	361.40	316.38	338.42	360.46
3228.04	280.00	0.0612	397.55	397.56	397.57	358.66	358.67	358.68	319.77	319.78	319.79
3099.83	6.805	300.893	303.58	345.37	387.16	302.64	344.43	386.22	301.69	343.48	385.27
Remark →			Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe

Note: Pile cap length is 3600 x 3600 mm, but for columns A2, B2, C2, D2, E2, F2, A3, B3, C3, D3, E3 & F3, the pile cap overlaps. The only solutions are :–

i) To proceed with combined footing in those areas,

ii) To increase dimension between those columns

iii) To decrease the pile radius and increase concrete strength and number of bars.

Here we proceed with option number (iii). We decrease the radius of piles to 300 mm for those areas and increase concrete strength from M25 to M30 and increase the numbers of bars.

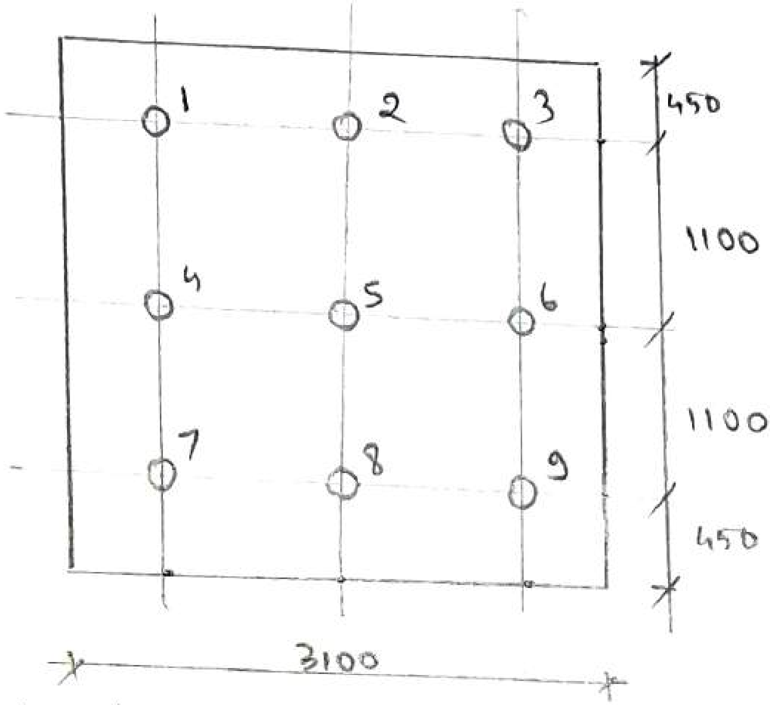
□ REVISED DESIGN OF PILE FOUNDATION:

Assumed Pile Capacity = 500 kN, Pile diameter = 300 mm

Among various load combinations, maximum axial load on pile group = 3228.04 kN.

Hence no. of piles required = $\frac{3228.04}{500} = 6.456$

Let us provide 9 nos. of piles as shown in the following figure :-



Minimum edge distance = 1.5 × pile diameter = 1.5 × 300 = 450 mm

Minimum distance between piles = 3 × pile diameter = 3 × 300 = 900 mm

Assuming spacing between piles = 1100 mm

Length of the square pile cap = 2×450 + 2×1100 = 3100 mm

Pile reaction of each pile has been calculated against every load combination in the following table.

Only Reaction of pile 3 under load combination (DL+LL+HL) short (x-direction) will be shown in detail here.

□ DETERMINATION OF PILE REACTION FOR EACH LOAD COMBINATION

$\Sigma x^2 = \Sigma y^2 = 7.26$

Axial load (P) (KN)	Mx (KN-m)	My (KN-m)	Pile Reaction (kN) (< 500 kN)								
			Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Pile 9
3023.245	6.805	0.061	336.47	336.47	336.48	335.91	335.92	335.92	335.36	335.36	335.37
2791.00	353.00	0.442	363.53	363.60	363.66	310.04	310.11	310.18	256.56	256.63	256.69
2621.360	4.621	159.062	267.86	291.96	316.06	267.16	291.26	315.36	266.46	290.56	314.66
2795.130	277.817	0.442	352.60	352.66	352.73	310.50	310.57	310.64	268.41	268.48	268.54
2666.920	4.621	301.274	251.38	297.02	342.17	250.68	296.32	341.57	249.98	295.62	341.27
3223.910	355.185	0.061	412.02	412.03	412.04	358.20	358.21	358.22	304.39	304.40	304.41
3054.275	6.805	158.68	316.35	340.40	364.44	315.32	339.36	363.41	314.29	338.33	362.38
3228.040	280.00	0.061	401.09	401.10	401.10	358.662	358.67	358.68	316.24	316.25	316.26
3099.830	6.805	300.893	299.87	345.46	391.05	298.84	344.43	390.02	297.80	343.39	388.98
			Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe	Safe

For load combination (DL + LL + WL) short (x), $P = 3223.91$ kN,

$M_x = 355.185$ kN-m, $M_y = 0.0612$ kN-m

Origin being at the centre of the pile cap for pile 3,

$x = 1.1$ m, $y = 1.1$ m

Now, $\Sigma x^2 = \Sigma y^2 = 6 \times 1.1^2 = 7.26$ m^2

$$\therefore \text{Pile reaction} = \frac{P}{n} \pm \frac{M_x y}{\Sigma y^2} \pm \frac{M_y x}{\Sigma x^2} = \frac{3223.91}{9} + \frac{355.185 \times 1.1}{7.26}$$

$$+ \frac{0.0612 \times 1.1}{7.26} = 412.04 \text{ kN} < 500 \text{ kN (Pile capacity)}$$

∴ Hence Safe

It is also to be noted that no pile should at all be in tension. If the pile reaction is found to be negative, the piles need to be rearranged.

□ DESIGN OF PILE CAP:

As it can be seen from the previous table, maximum pile reaction occurs at pile 1, 2 & 3 under the load combination of DL+LL+WL (short).

Pile reaction: Pile 1 = 412.02 kN, Pile 2 = 412.03 kN, Pile 3 = 412.04 kN.

▲ Calculation of Depth →

Moment at column face = $(412.02 + 412.03 + 412.04) \times \left(1.1 - \frac{0.5}{2}\right)$

$= 1050.676$ kN-m

Effective depth, $d = \sqrt{\frac{1.2 \times 1050.676}{3.325 \times 3100}} = 349.742$ mm

Let us provide a total depth, D = 1200 mm

∴ Effective depth = 1200 − 75 − 75 = 1050 mm

▲ One-way shear check →

Shear force at a distance d (= 1050 mm) from column face =

$412.02 + 412.03 + 412.04 = 1236.09$ kN.

∴ shear stress, $\tau_v = \frac{1236.09 \times 10^3 \times 1.2}{3100 \times 1050} = 0.456$ N/mm²

Assumed steel reinforcement, $P_t = 0.25\%$

From table 19 of IS 456:2000,

Design shear strength of concrete, $\tau_c = 0.370$ N/mm²

Here, $\tau_v > \tau_c$. Hence shear reinforcement is required.

∴ Net shear force, $V_{us} = (1.2 \times 1236.09 \times 10^3) - 0.370 \times 3100 \times 1200$

$= 106.908$ kN

Providing 8 mm Φ bars as vertical stirrups,

∴ Spacing required = $\frac{0.87 \times 500 \times \frac{\pi}{4} \times 8^2 \times 1050}{106.908 \times 10^3} = 214.75$ mm

∴ Provide 8 mm Φ bars as vertical stirrups @ 200 mm c/c

Two way shear check →

Perimeter of shear force at a distance d/2 from the column face $= 4 \times (0.5 + 1.05) = 6.2\ m$

Punching shear from the piles around the column =

412.02 + 412.03 + 412.04 + 358.20 + 358.21 + 358.22 + 304.39 + 304.40 + 304.41 = 3223.92 kN

∴ Shear stress, $\tau_v = \frac{1.2 \times 3223.92 \times 10^3}{6.2 \times 10^3 \times 1050} = 0.594\ N/mm^2$

Allowable shear stress, $\tau_c = 0.25\sqrt{f_{ck}} = 0.25 \times \sqrt{30}$

$= 1.369\ N/mm^2$

Now, $K_s = (1+\beta_c) = 1+1 = 2$, Adopt max $K_s = 1$.

∴ $K_s \tau_c = 1 \times 1.369 = 1.369\ N/mm^2 > 0.594\ N/mm^2$

Hence safe.

Steel Reinforcement →

$$\frac{M_u}{bd^2} = \frac{1.2 \times 1050.676 \times 10^6}{3100 \times 1050^2} = 0.369$$

Now from table 4 of SP-16, for $\frac{M_u}{bd^2} = 0.369$, $P_t = 0.086\%$

But, previously we assumed, $P_t = 0.25\%$

∴ $A_{st} = \frac{0.25}{100} \times 3100 \times 1050 = 8137.5\ mm^2$

Providing 25 mm Ø bar as main reinforcement,

Area of each bar $= \frac{\pi}{4} \times 25^2 = 490.87\ mm^2$

∴ No. of bars required $= \frac{8137.5}{490.87} = 16.58 \simeq 20$

Available width $= 3100 - 75 \times 2 = 2950$ mm.

∴ Required spacing $= \frac{2950}{20} = 147.5$ mm.

∴ Provide 25 mm Ø bars @ 140 mm c/c as main reinforcement in both direction.

Also provide 12 mm Ø bars @ 140 mm c/c as top reinforcement in both directions.

As per clause no. 26.5.1.3 of IS 456:2000, side reinforcement is to be provided when depth exceeds 750 mm

Side face reinforcement $= \frac{0.1}{100} \times 3100 \times 1100 = 3410\ mm^2$

To be distributed on two faces $= \frac{3410}{2} = 1705\ mm^2$.

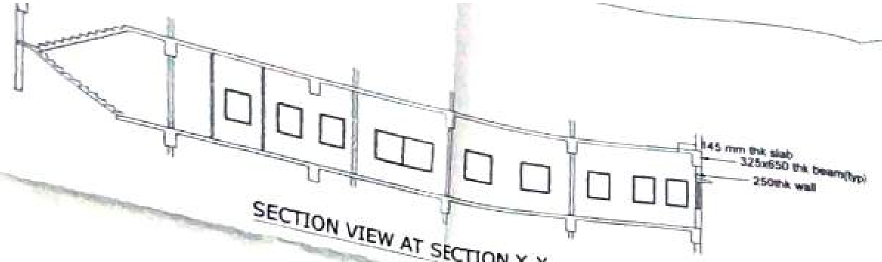
Providing 12 mm ϕ bars no. of bars required $= \frac{1705}{\frac{\pi}{4} \times 12^2} = 15.08 \simeq 16$

$\therefore$ Spacing required $= \frac{1100-150-75}{16} = 54.69$ mm

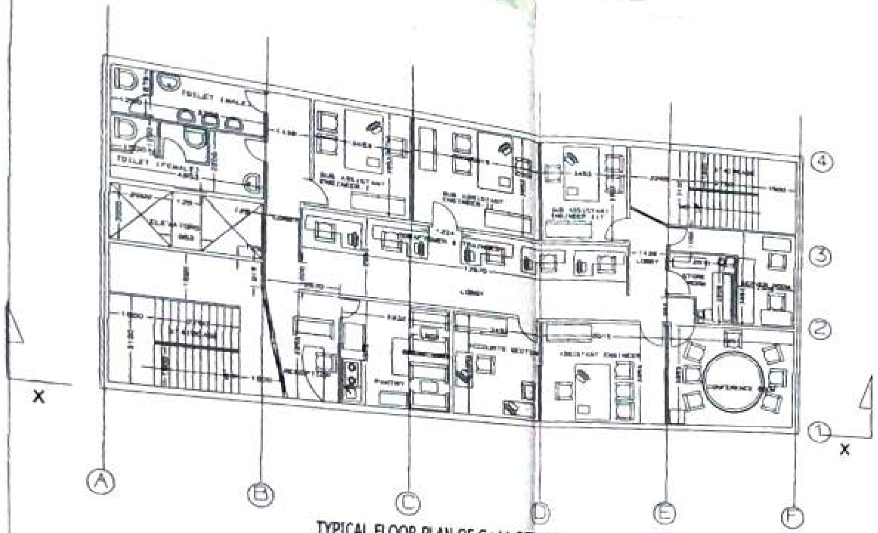
$\therefore$ Provide 12 mm ϕ bars @ 50 mm c/c as side face reinforcement on each face.

REFERENCES

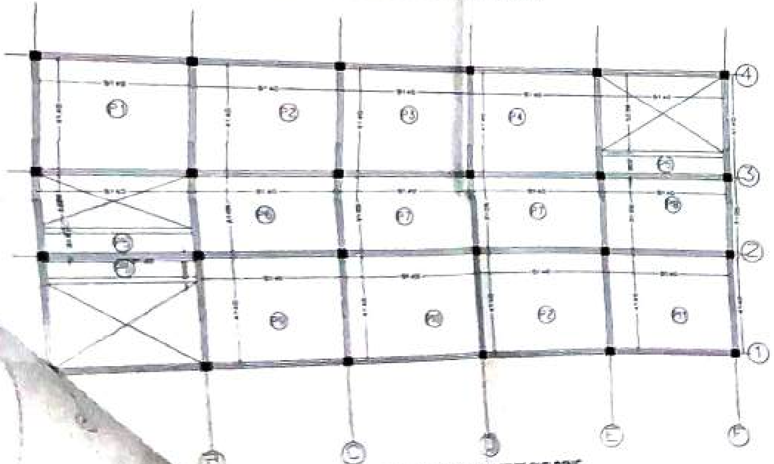
1. IS:456 -2000
2. IS:875 (part 3) - 1987
3. IS: 1893 (part 1) - 2002
4. SP-16
5. SP-34
6. RCC Designs (Reinforced Concrete Structures)- Punmia, Jain, Jain



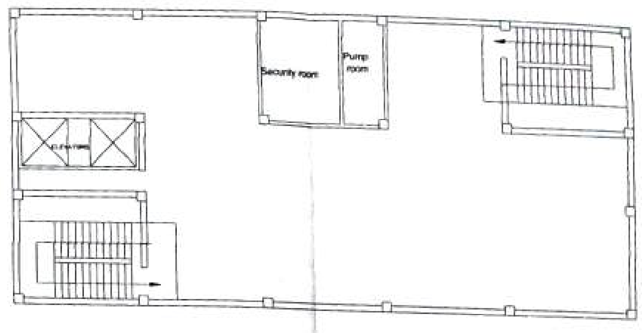
SECTION VIEW AT SECTION X-X



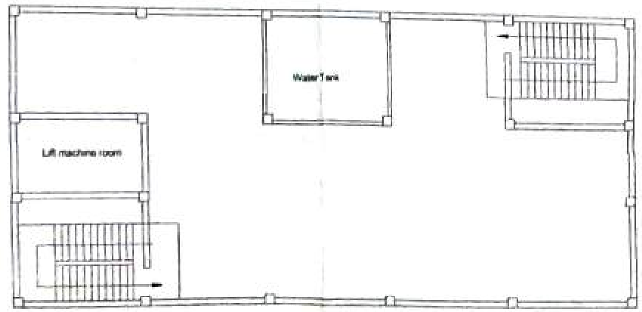
TYPICAL FLOOR PLAN OF G+11 OFFICE BUILDING



TYPICAL FLOOR STRUCTURAL PLAN OF G+11 OFFICE BUILDING



GROUND FLOOR PLAN OF G+11 OFFICE BUILDING



ROOF PLAN OF G+11 OFFICE BUILDING

NOTES

1) ALL DIMENSIONS ARE IN MM
2) GRADE OF CONCRETE MIX SHALL BE M25

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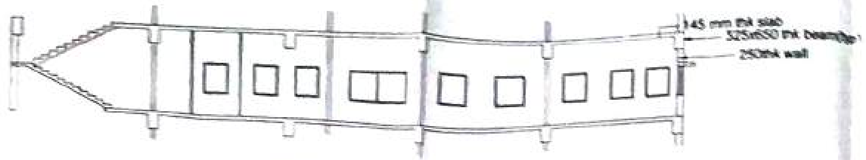
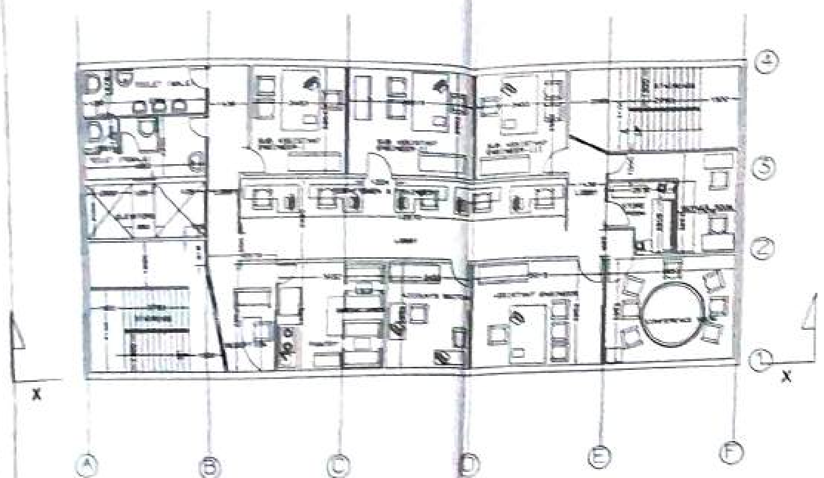
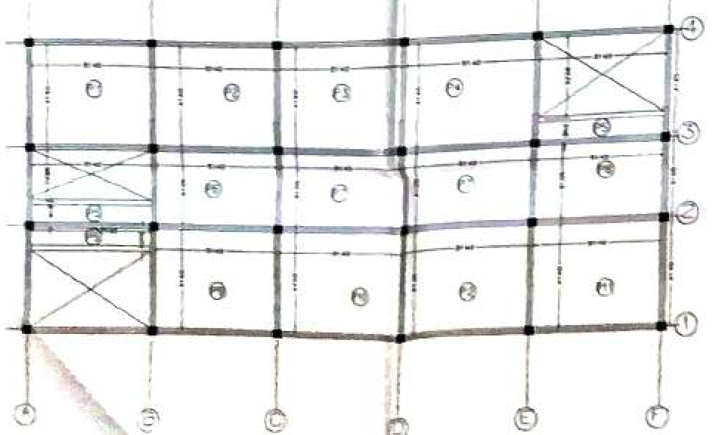


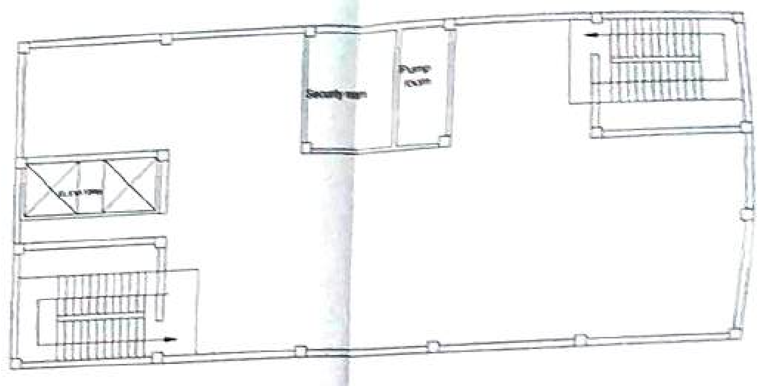
SECTION VIEW AT SECTION X-X



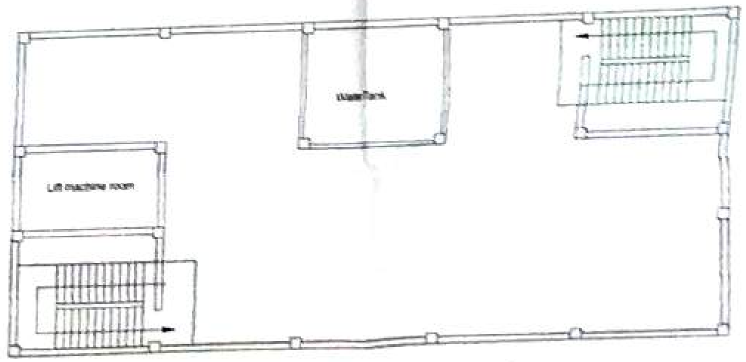
TYPICAL FLOOR PLAN OF G+11 OFFICE BUILDING



TYPICAL FLOOR STRUCTURAL PLAN OF G+11 OFFICE BUILDING



GROUND FLOOR PLAN OF G+11 OFFICE BUILDING



ROOF PLAN OF G+11 OFFICE BUILDING

NOTES

1) ALL DIMENSIONS ARE IN mm

2) GRADE OF CONCRETE MIX SHALL BE M25

NAME ATASI ROY MALAKAR

UNIVERSITY ROLL NO. [illegible]

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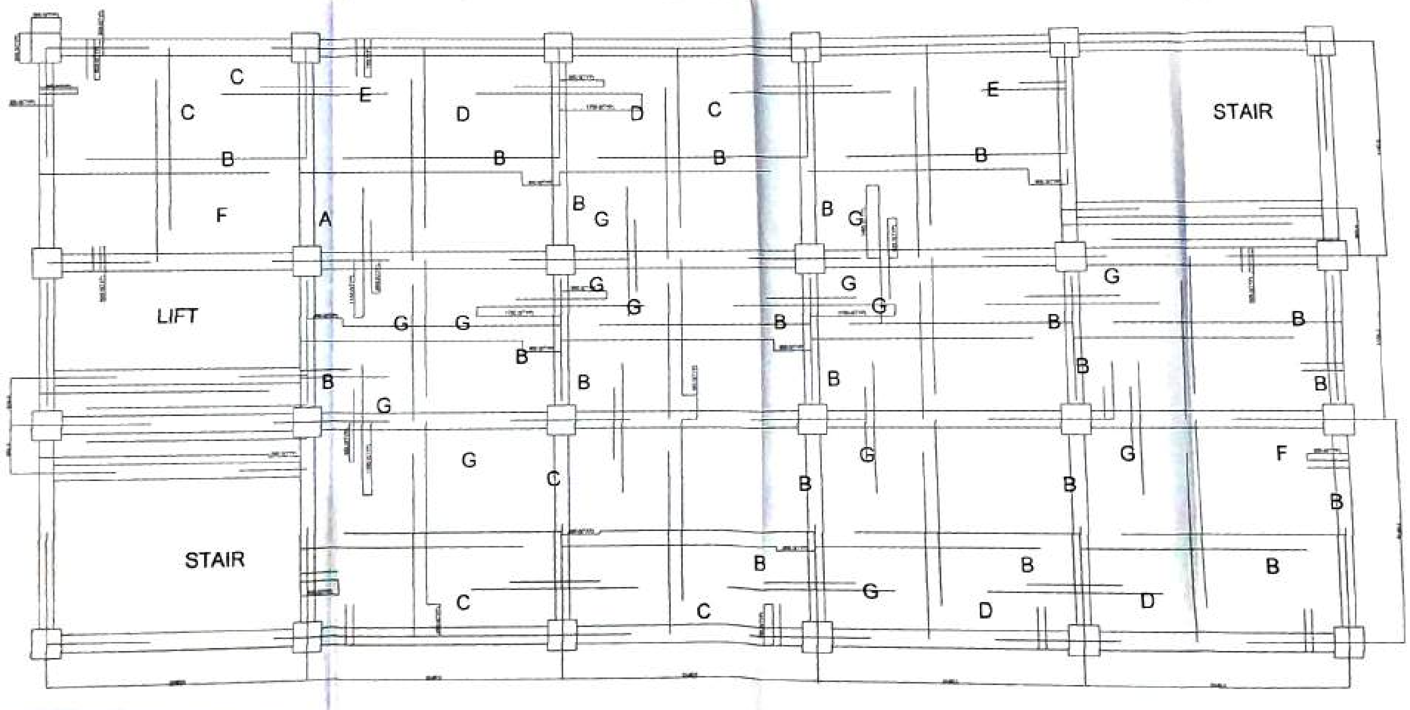

REINFORCEMENT DETAIL OF SLAB

NOTES:

ALL DIMENSION ARE IN MM

FOR DETAILS OF BEAMS & COL REFER TO SHEET 3.

FOR DETAILS OF G.A REFER TO SHEET 1

BARS OF FE500 IS USED
GRADE OF CONCRETE :M20

LEGEND:

A =10MM DIABAR @200 C/C
B =10MM DIABAR @300 C/C
C =8MM DIA BAR @270 C/C
D =10MM DIABAR @200 C/C
E =10MM DIABAR @170 C/C
F =10MM DIABAR @200 C/C
G =10MM DIA BAR @300 C/C
H =10MM DIABAR @200 C/C
I =8MM DIA BAR @270 C/C

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DETAIL REINFORCE OF TYP SLAB OF G+11 OFFICE STOREY BUILDING CALCUTTA INSTITUTE OF ENGINEERING AND MANAGEMENT	
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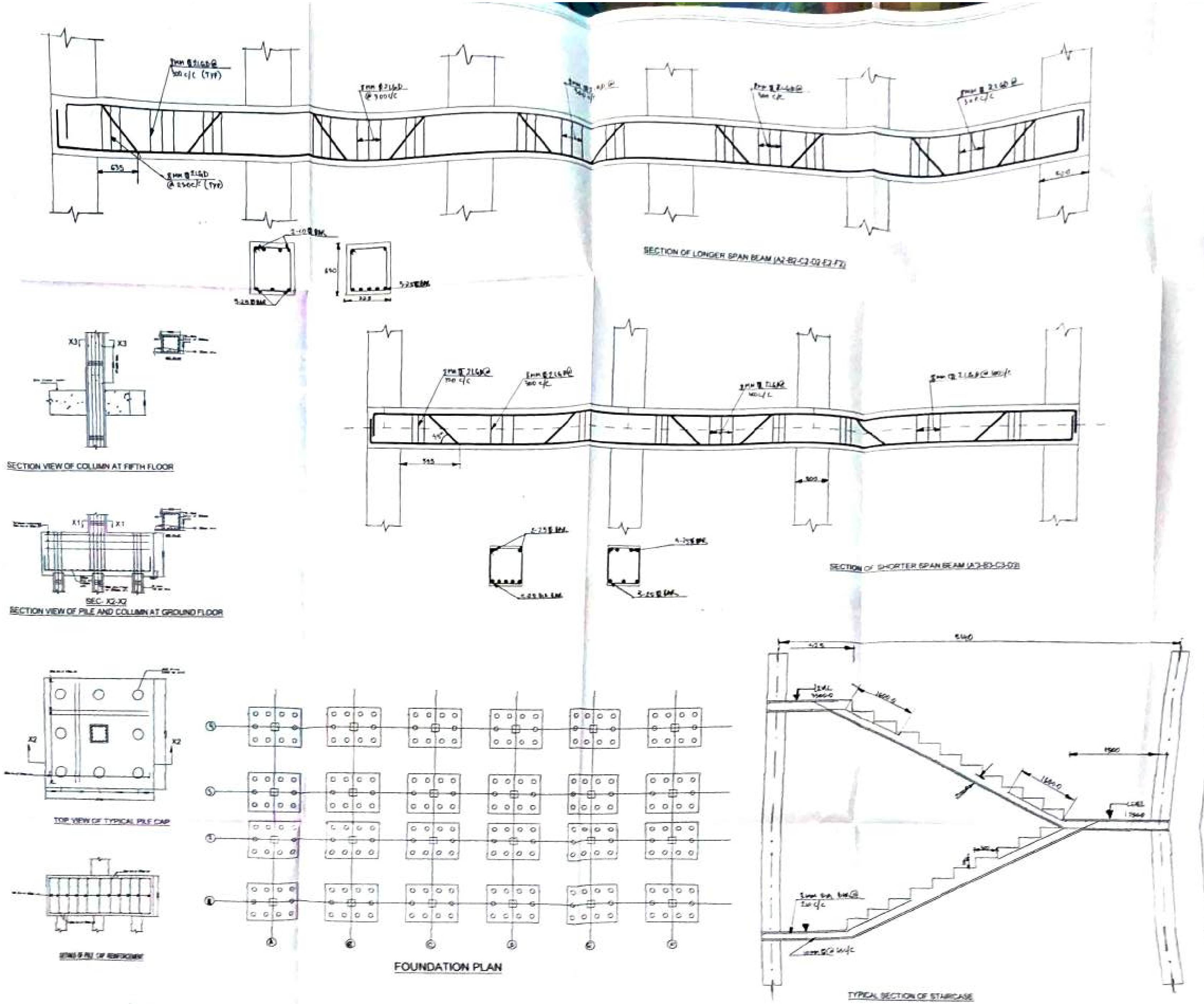

GENERAL NOTES

1) ALL DIMENSIONS ARE IN MM
2) GRADE OF CONCRETE MIX SHALL BE M25
3) REINFORCEMENT SHALL BE HIGH STRENGTH DEFORMED BARS OF GRADE FE-500
4) CLEAR COVER TO REINFORCEMENT IN FOUNDATION IS 120 MM
5) CLEAR COVER TO REINFORCEMENT IN COLUMN SHALL BE 50MM
6) CLEAR COVER TO REINFORCEMENT IN STAIRCASE SHALL BE 20MM
7) LAP LENGTH IN REINFORCING BARS SHALL CONFORM TO CLAUSE 25.2.5.1 OF IS456-2000

SCHEDULE OF COLUMN REINFORCEMENT

1) 18-28MM Ø BARS UPTO 5TH FLOOR
2) 18-20MM Ø BARS FROM 5TH FLOOR TO ROOF

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DETAIL DRAWING SECTION OF BEAMS,COLUMN, PILE CAP & STAIRS
CALCUTTA INSTITUTE OF ENGINEERING AND MANAGEMENT
REMARKS: SIGN: