Structural Design Sessional
Group 7
Final Report

Group Members:

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Design Basis Report

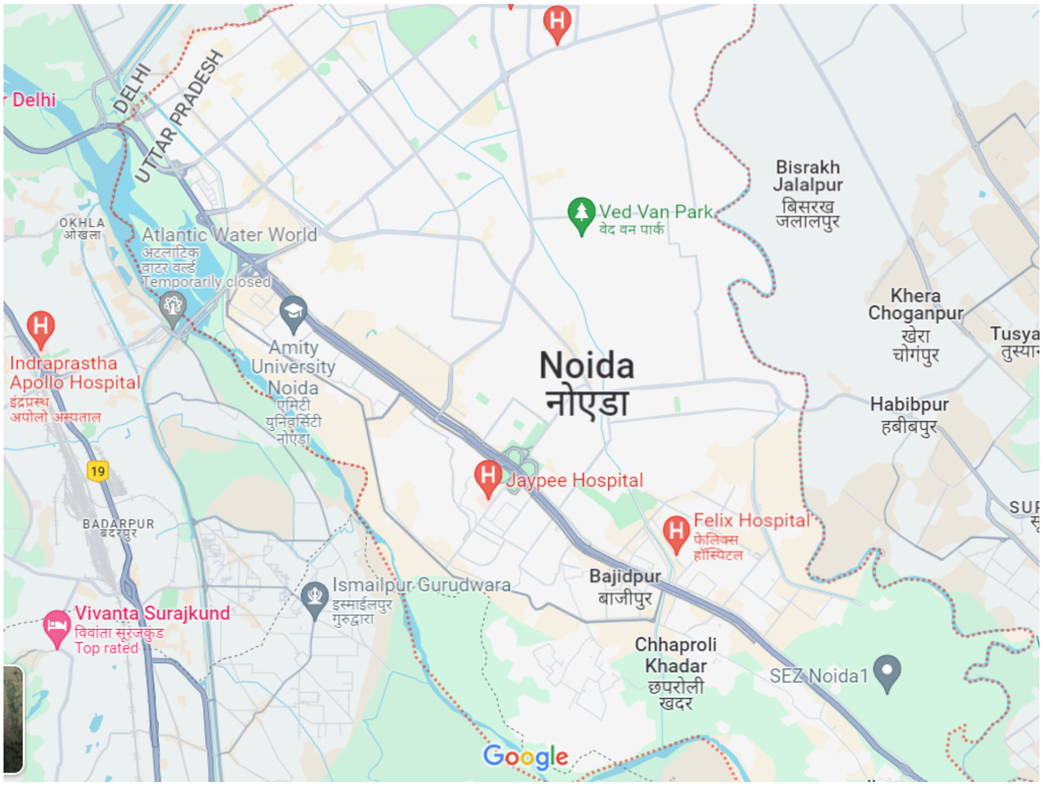
Problem Statement:

- To design an RCC Residential tower comprising four apartments in each floor which is located at Noida. Each Residential Tower is of type B+G+5+2*7
- Each floor will have four apartments of approximate area as (140±10) m2 approximately each, with common facilities
- The bearing capacity of the soil is 14 ton/m2
- Clear height of the basement and ground floor is 2.70 m and height of each floor is 3.25 m
- Analysis and design of the Towers should comply with the latest Indian Codes of Practice. The floor diaphragms are assumed to be rigid.
- Center-line dimensions will be followed for analysis.
- Seismic loads will be considered acting in the horizontal direction and not along the vertical direction.
- Anyotherparameter needed for the design can suitably be assumed with proper justification.

Materials Used:

- M30 grade of concrete for all RCC structural elements
- Fe500 for all the RCC reinforcements
- Fe410 for all the steel members

Features of Residential Tower:

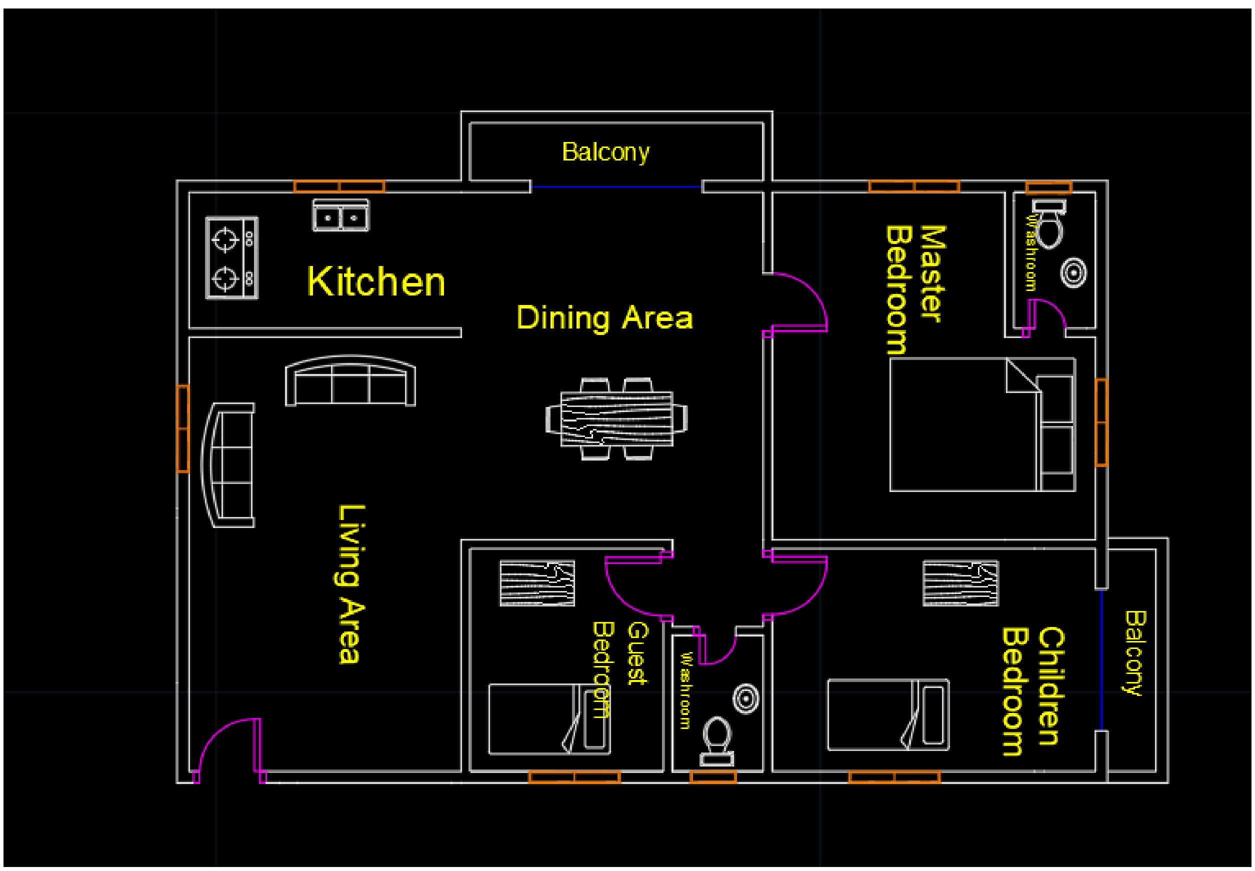


- ➢ The area of each apartment is 150 sq.metres
- ➢Common corridors are provided to access lifts and staircases from each flat.There are two lifts and two staircases.
- ➢This Residential Tower has parking in one ground floor and one basement
- ➢A separate ramp with some inclination is provided into the basement parking.
- ➢ There are 4 tanks provided on the terrace for proper water supply.

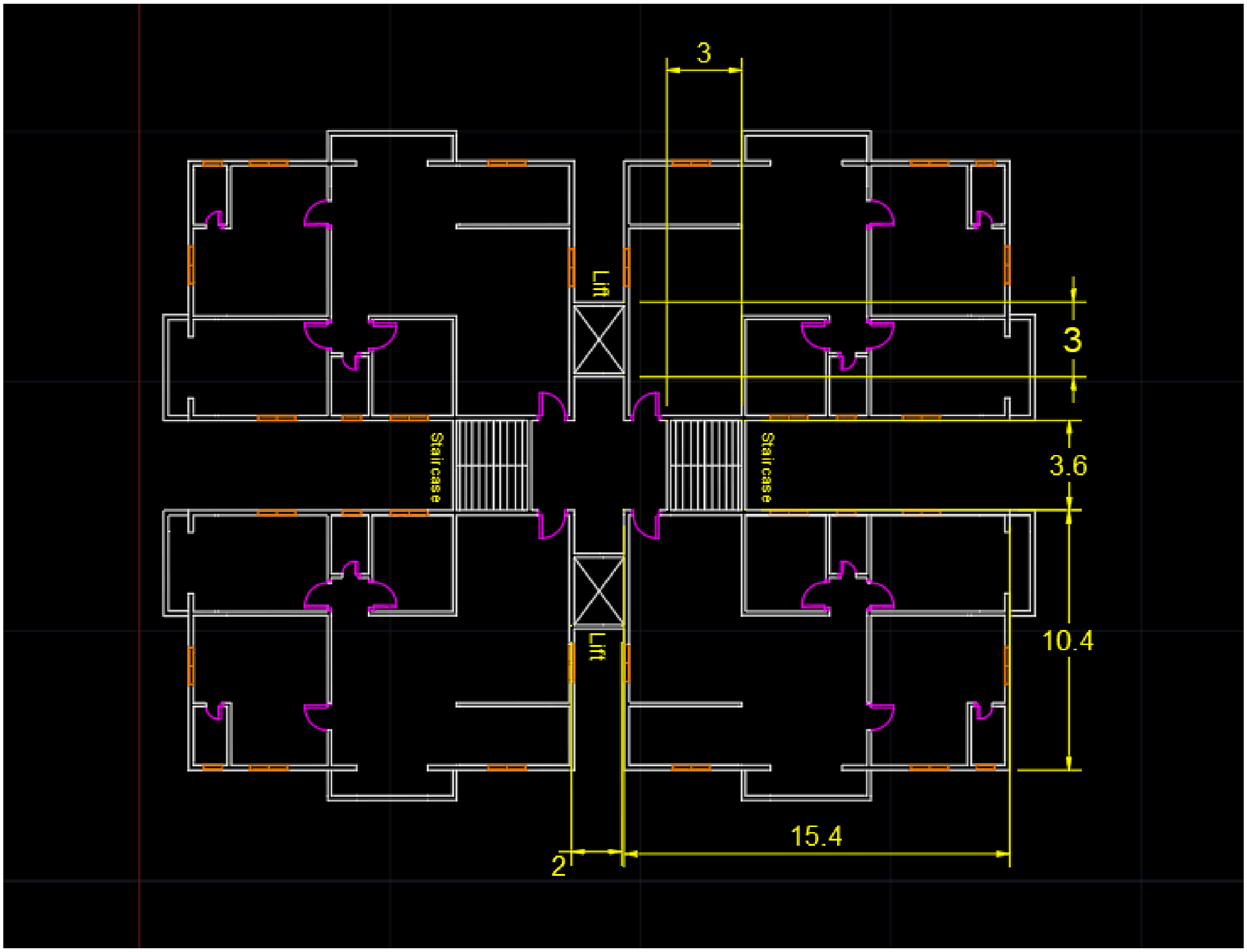
- ➢ This Residential tower has 4 apartments on each floor. Since there are 19 floors, a total of 76 apartments are present in this complex.
- ➢All 4 flats are similar in dimensions and plan
- ➢Each flat is designed as a 3 bedroom apartment with a living and a kitchen cum dining room. It has 1 common bathroom and a master bedroom with an attached bathroom along with one balcony and one utility space.

Plan of the Tower

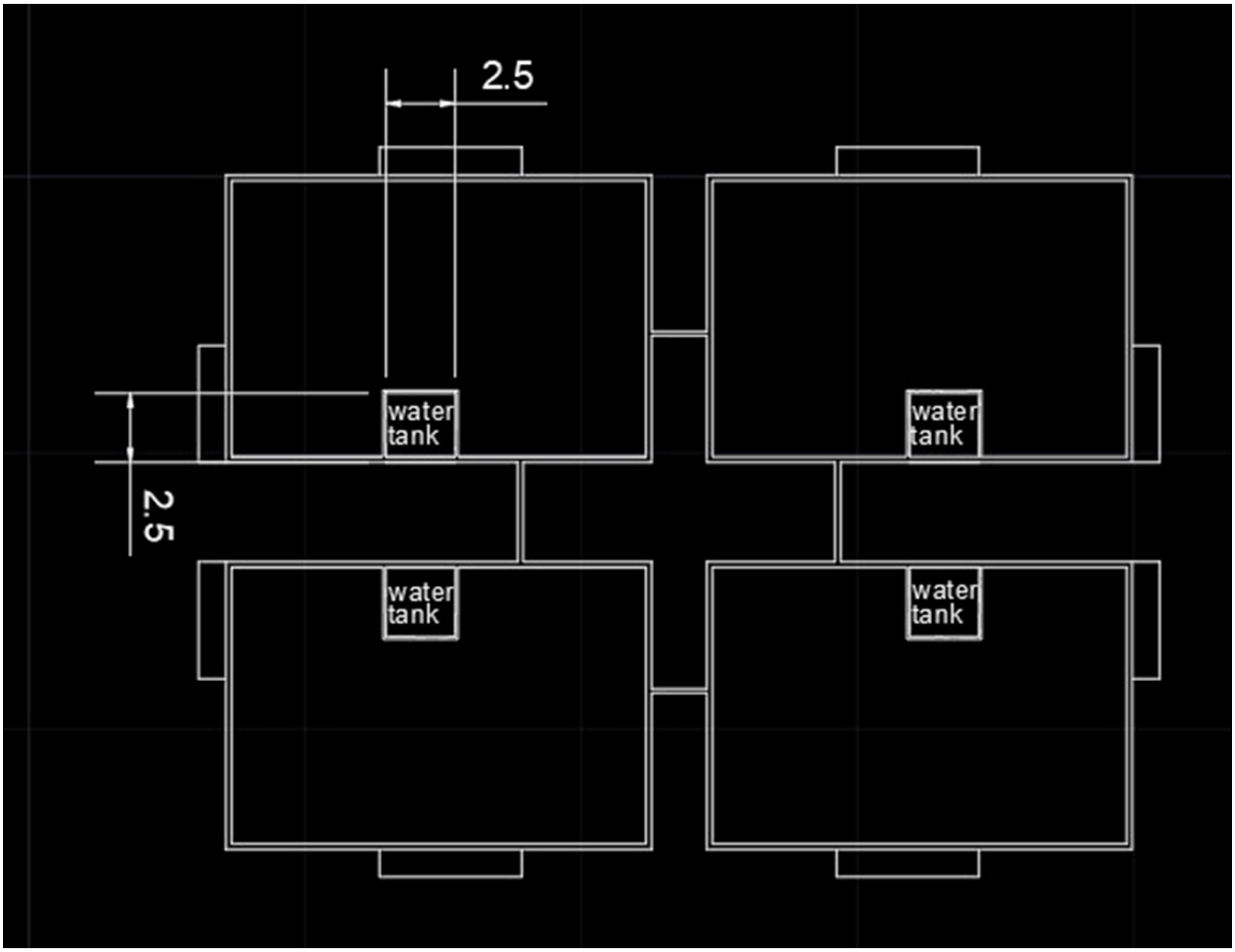
(for a typical single apartment)



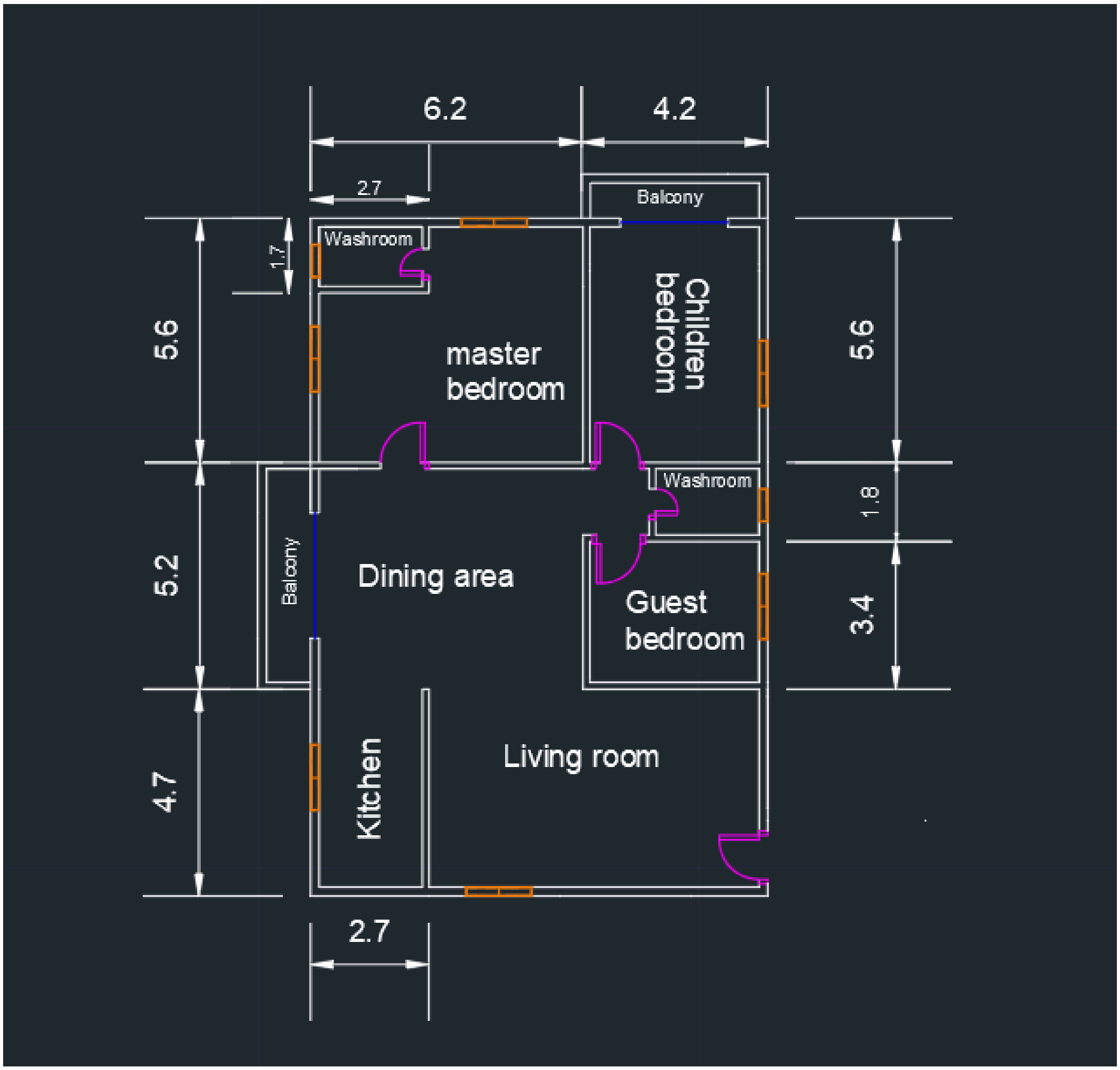
View of a Floor



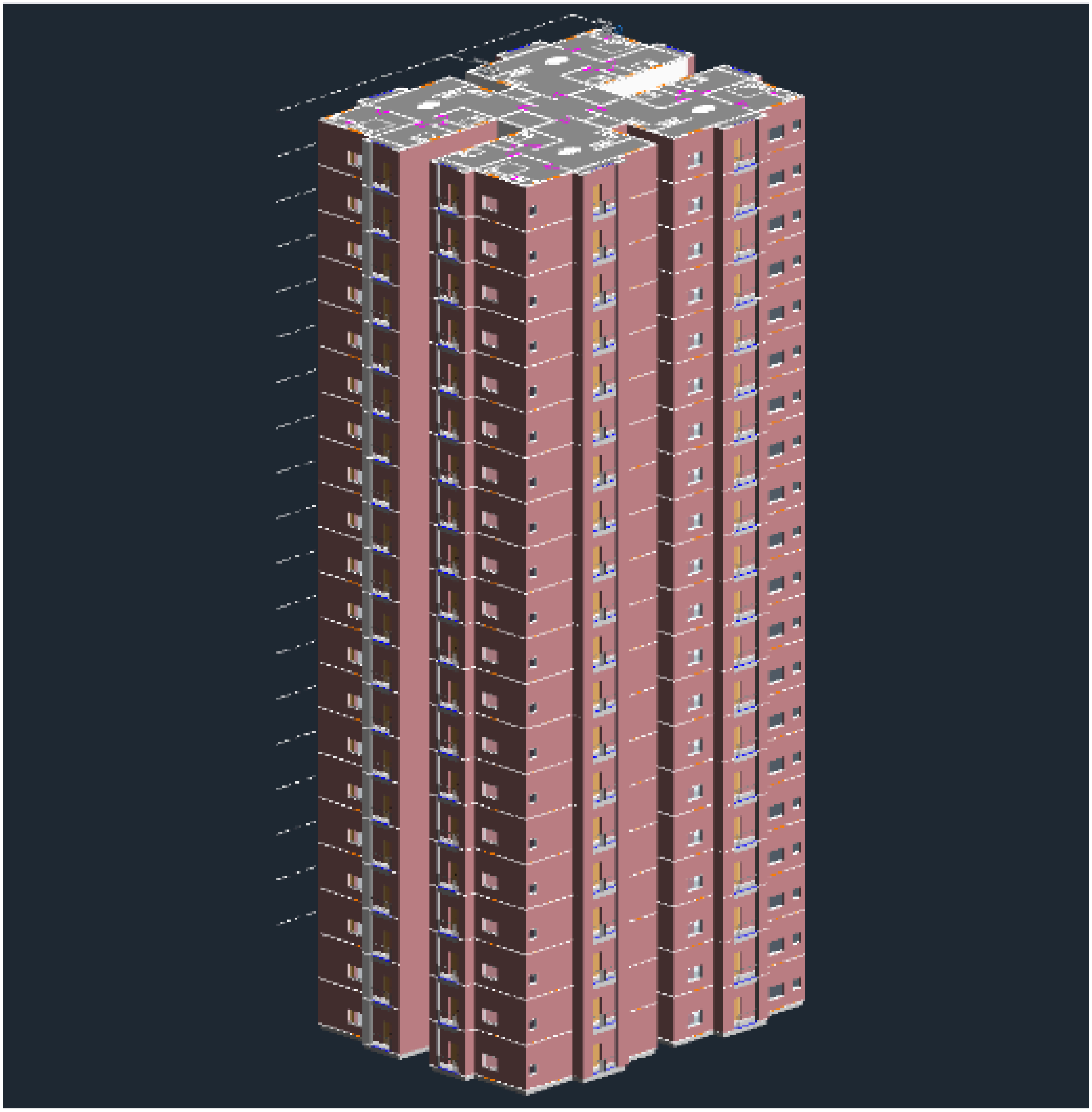
Roof Plan



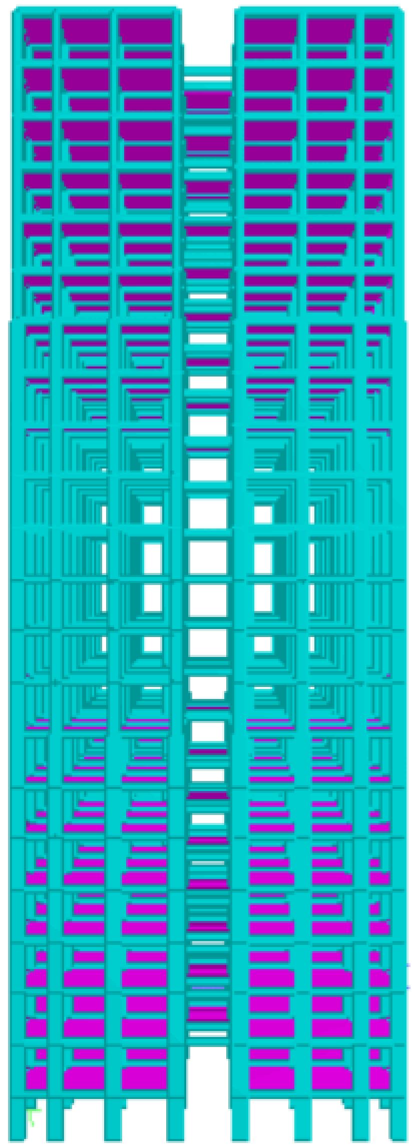
Dimensions of the Apartment(in m)



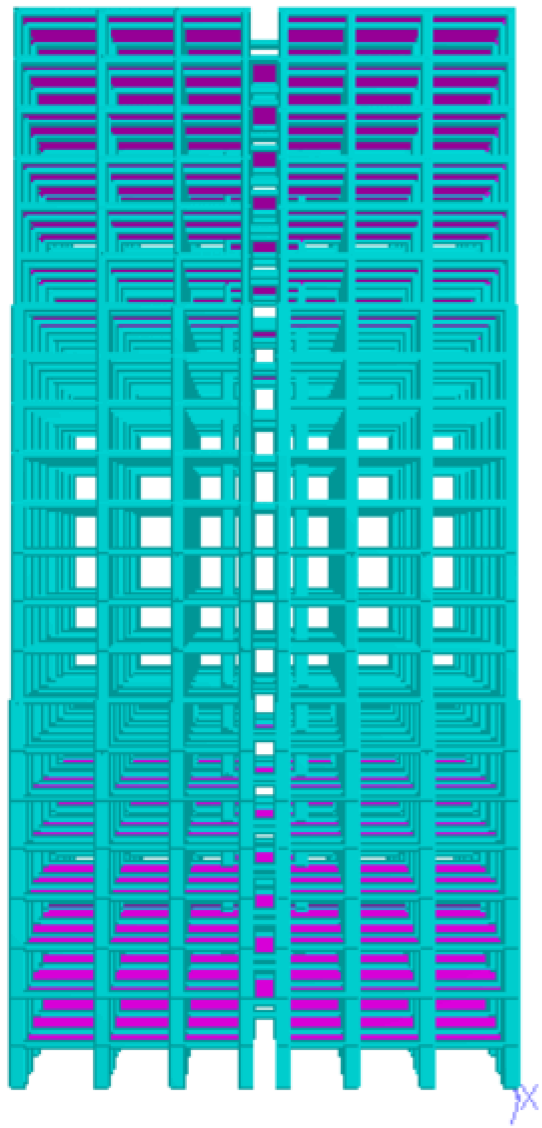
3D Revit Model



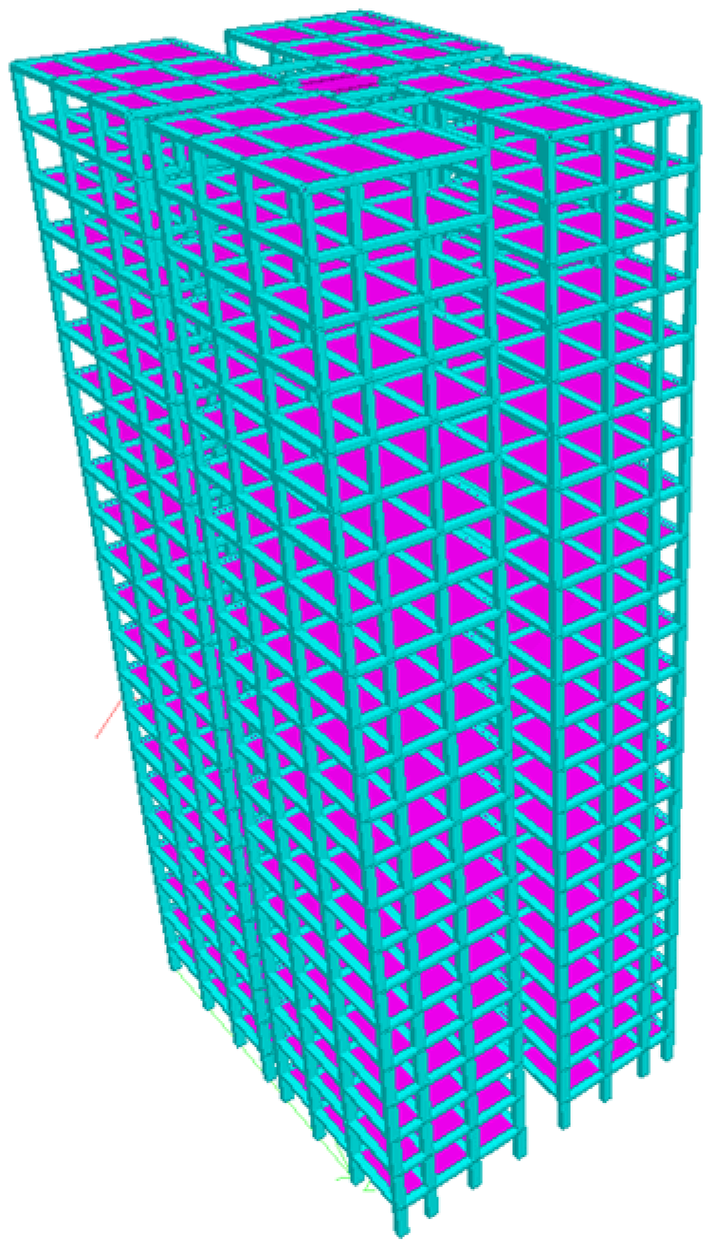
Front View



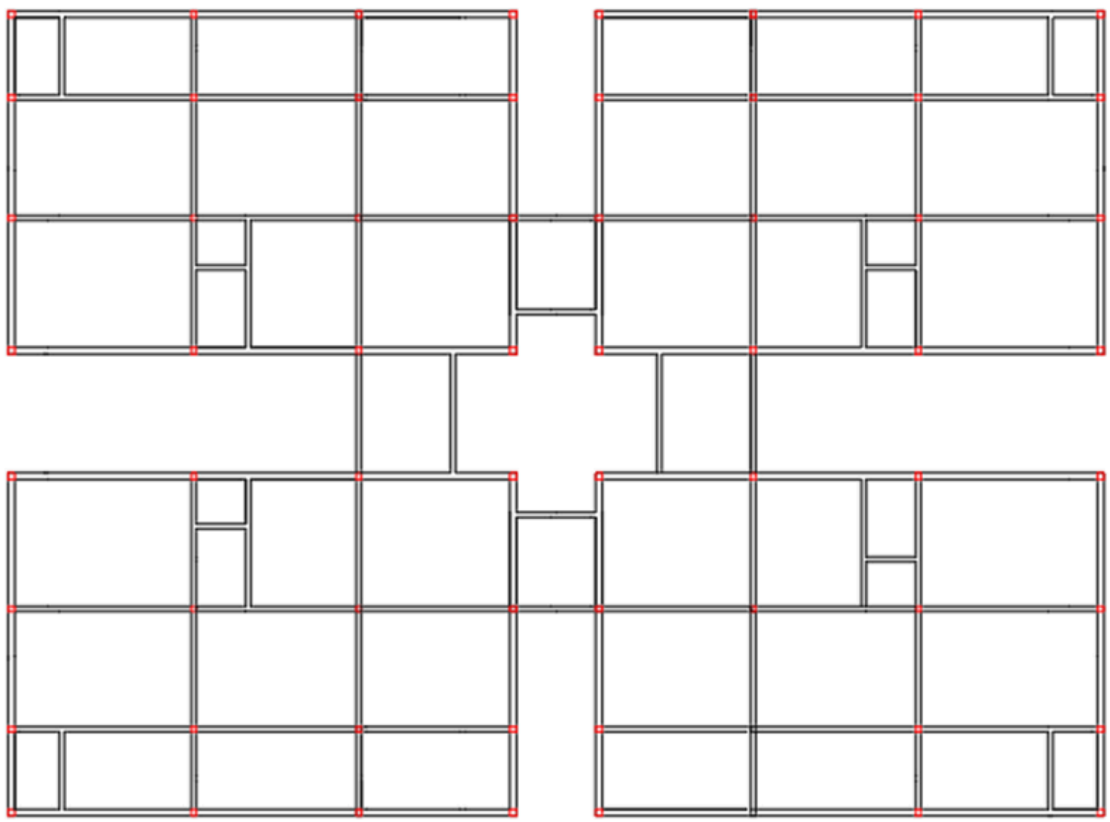
Side View



Isometric View



Beam column Layout for a floor



Column Dimensions

Column Shape : Square

800 mm * 800 mm (for first 8 floors)

650 mm * 650 mm (for the next 8 floors)

470 mm * 470 mm (till roof)

Beam Dimensions

For all primary beams : 400mm * 400mm

Varieties of possible load combinations in software for the Analysis of structural system

Load combination	DL factor	LL factor	EL/WL factor
Seismic +X	0	0	1
Seismic -X	0	0	1
Seismic +Z	0	0	1
Seismic -Z	0	0	1
DL	1	0	0
LL	0	1	0
WL +X	0	0	1
WL -X	0	0	1
WL +Z	0	0	1
WL -Z	0	0	1
DL+LL	1.5	1.5	0
DL+LL+S +X	1.2	1.2	1.2
DL+LL+S -X	1.2	1.2	1.2
DL+LL+S +Z	1.2	1.2	1.2
DL+LL+S –Z	1.2	1.2	1.2

Load combination	DL factor	LL factor	EL/WL factor
DL+LL +WL+X	1.2	1.2	1.2
DL+LL +WL-X	1.2	1.2	1.2
DL+LL +WL+Z	1.2	1.2	1.2
DL+LL +WL-Z	1.2	1.2	1.2
DL+LL -WL+X	1.2	1.2	1.2
DL+LL -WL-X	1.2	1.2	1.2
DL+LL -WL+Z	1.2	1.2	1.2
DL+LL -WL-Z	1.2	1.2	1.2
DL+LL+S +X	1.2	1.2	1.2
DL+LL+S -X	1.2	1.2	1.2
DL+LL+S +Z	1.2	1.2	1.2
DL+LL+S –Z	1.2	1.2	1.2

Dead Load

Unit Weight of Concrete = 30 kN/m^2
Floor finish = 30*0.125 = 3.75 kN/m^2

1.	Concrete Slab	30*0.15 = 4.5 kN/m
2.	Beam	25*0.4*0.4 = 4 kN/m And for each apartment 25*4 = 100 kN/m
3.	Column	1) 25*0.8*0.8=16 kN/m 2) 25*0.65*0.65=10.5 kN/m 3) 25*0.47*0.47=5.5 kN/m
4.	Wall load(Member load)	18 kN/m

(Load Definitions and values as per IS: 875 Part 1
Parameters used from IS:875 Part – 1)

Live Load

Live Loads	kN/m2
Bedroom	2
Toilet & Bathroom	2
Kitchen	2
Dining cum Living Room	2
Utility room	2
Study room	2

Live Loads	kN/m^2
Store room	2
Staircase	3
Common spaces	3
Verandah	3
Balcony	3

Wind Load

Wind Data	Value	Reference
Wind Load	Intensity varies with height	Refer IS:875, pt 3, Sec 5.2
Base Velocity	47 m/s	Refer IS:875, pt 3, Sec 5.2
Terrain Category	4	Refer IS:875, pt 3, Sec 5.3.2.2

Design factors

Design Factors	Value	Reference
Risk Coefficient Factor, k1	1.00	Refer IS:875, pt 3, Sec 5.3.1, Table 1)
Terrain & Height Factor, k2	Varies with height	Refer IS:875, pt 3, Sec 5.3.2.2,Table-2)
Topography Factor, k3	1.00	Refer IS:875, pt 3, Sec 5.3.3.1
Wind directionality factor, Kd	0.9	Cl 2.2.1
Area Averaging factor, Ka	0.9	Cl 7.2.2
Combination Factor, Kc	0.9	

As per IS 857 Part 3 -2015

Basic wind speed (Vb) = 47 m/s
Probability factor (k1) = 1
Topography Factor (k3) = 1
Importance factor for cyclonic region (K4) = 1
Wind directionality factor (Kd) =0.9
Area averaging factor (Ka) = 0.9
Combination factor (Kc) = 0.9

Design wind speed Vz = Vb*k1*k2*k3*k4
= 47*k2 m/s (varies with height)
[Refer IS:875, pt 3, Sec 5.3]

Design Wind Pressure Pz = 0.6*(Vz)^2
= 0.132*k2^2 kN/m^2
[Refer IS:875, pt 3, Sec 5.4]

Wind Pressure P = kd*ka*kc*Pz
= 0.966*k2^2 kN/m^2

Therefore, **Required Wind Pressure Pd** = max(0.7*Pz, P) kN/m^2

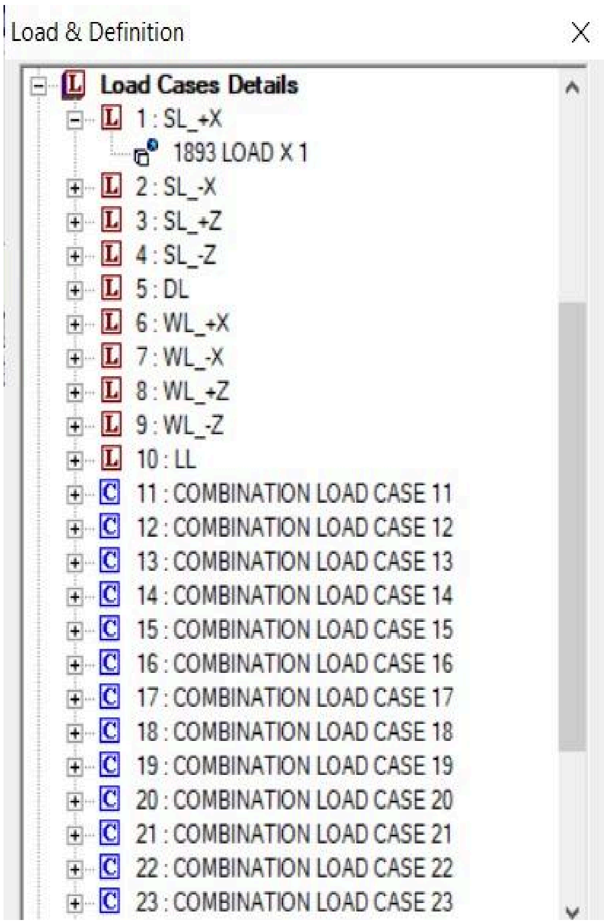
Variation of Pz with height

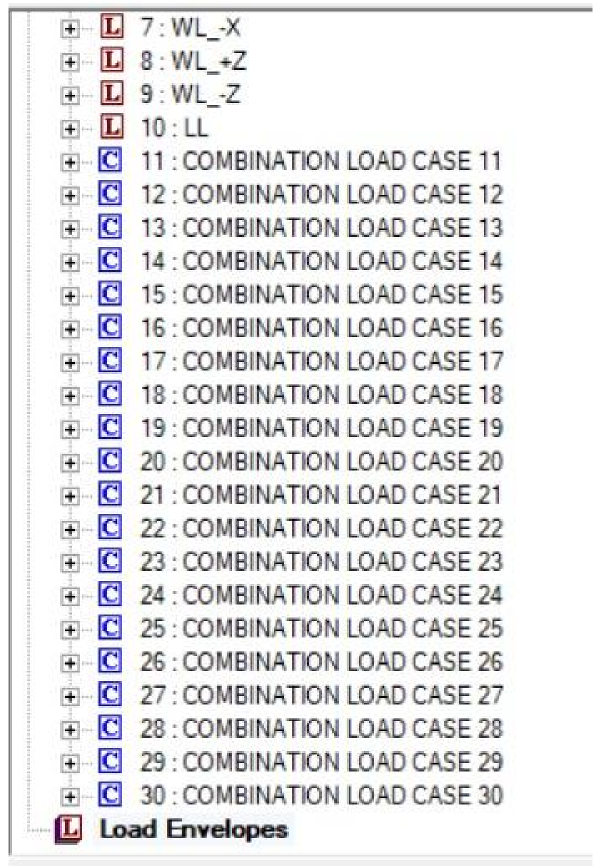
Height (in m.)	K2	Vz (in m/s)	Pz (in kN/m^2)	0.7 * Pz	P	max(0.7*Pz, P)
2.70	0.800000	37.600000	0.848256	0.5937792	0.61824	0.61824
5.95	0.800000	37.600000	0.848256	0.5937792	0.61824	0.61824
9.20	0.800000	37.600000	0.848256	0.5937792	0.61824	0.61824
12.45	0.800000	37.600000	0.848256	0.5937792	0.61824	0.61824
15.70	0.800000	37.600000	0.848256	0.5937792	0.61824	0.61824
18.95	0.800000	37.600000	0.848256	0.5937792	0.61824	0.61824
22.20	0.837400	39.357800	0.929421853	0.650595297	0.677396642	0.677396642
25.45	0.892650	41.954550	1.056110559	0.739277392	0.769732006	0.769732006
28.70	0.947900	44.551300	1.190890999	0.833623699	0.86796492	0.86796492
31.95	0.982675	46.185725	1.279872716	0.895910901	0.93281805	0.93281805
35.20	1.003800	47.178600	1.335492179	0.934844525	0.973355549	0.973355549
38.45	1.024925	48.171475	1.392294602	0.974606222	1.014755233	1.014755233
41.70	1.046050	49.164350	1.450279987	1.015195991	1.057017102	1.057017102
44.95	1.067175	50.157225	1.509448332	1.056613832	1.100141156	1.100141156
48.20	1.088300	51.150100	1.569799638	1.098859747	1.144127396	1.144127396
51.45	1.102900	51.836300	1.612201199	1.128540839	1.175031204	1.175031204
54.70	1.109400	52.141800	1.631260384	1.141882269	1.188922236	1.188922236
57.95	1.115900	52.447300	1.650431566	1.155302096	1.202894894	1.202894894
61.20	1.122400	52.752800	1.669714745	1.168800321	1.21694918	1.21694918
64.45	1.128900	53.058300	1.689109919	1.182376944	1.231085093	1.231085093
67.70	1.135400	53.363800	1.70861709	1.196031963	1.245302633	1.245302633
70.95	1.141900	53.669300	1.728236257	1.20976538	1.259601799	1.259601799

Seismic Load

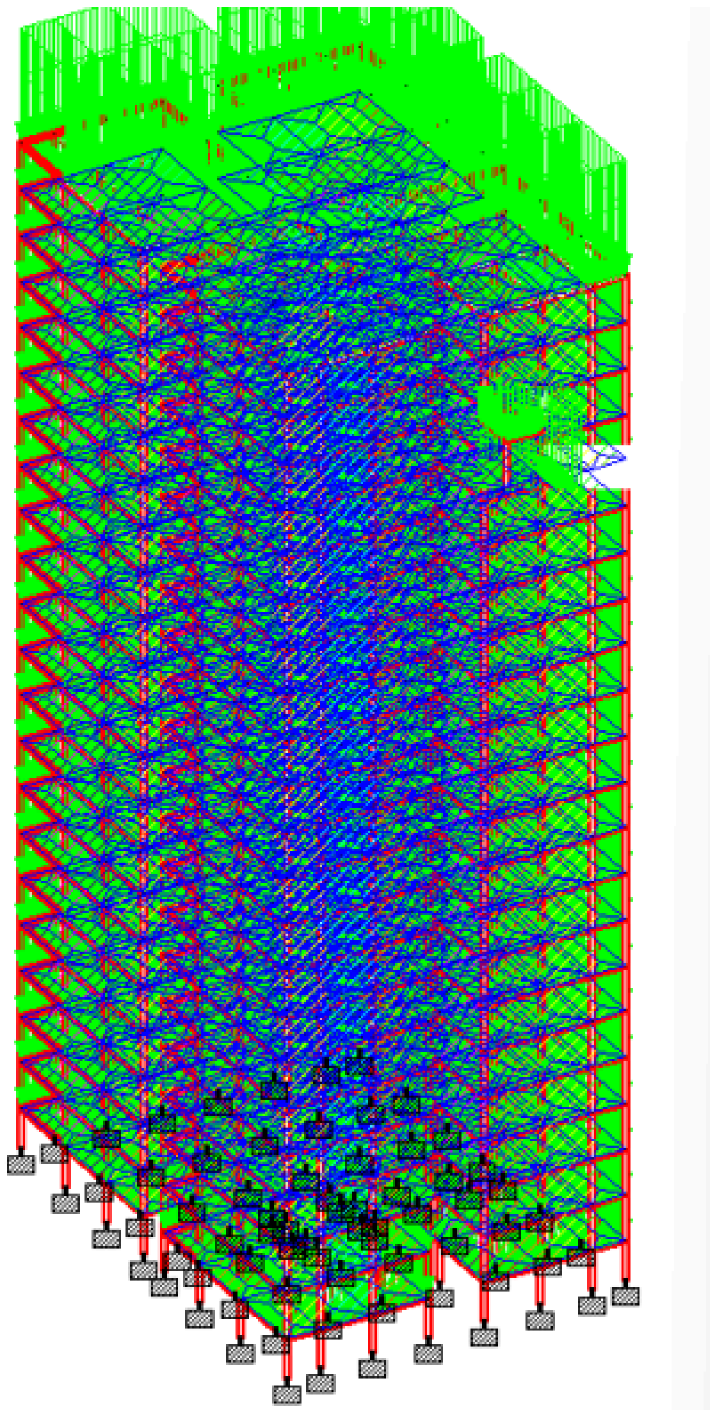
Seismic Parameter	Value
Seismic Zone of Noida	**V**
Zone factor	0.36
Soil Type	Medium
Structure Type	RC Framed Building

Load Case Details

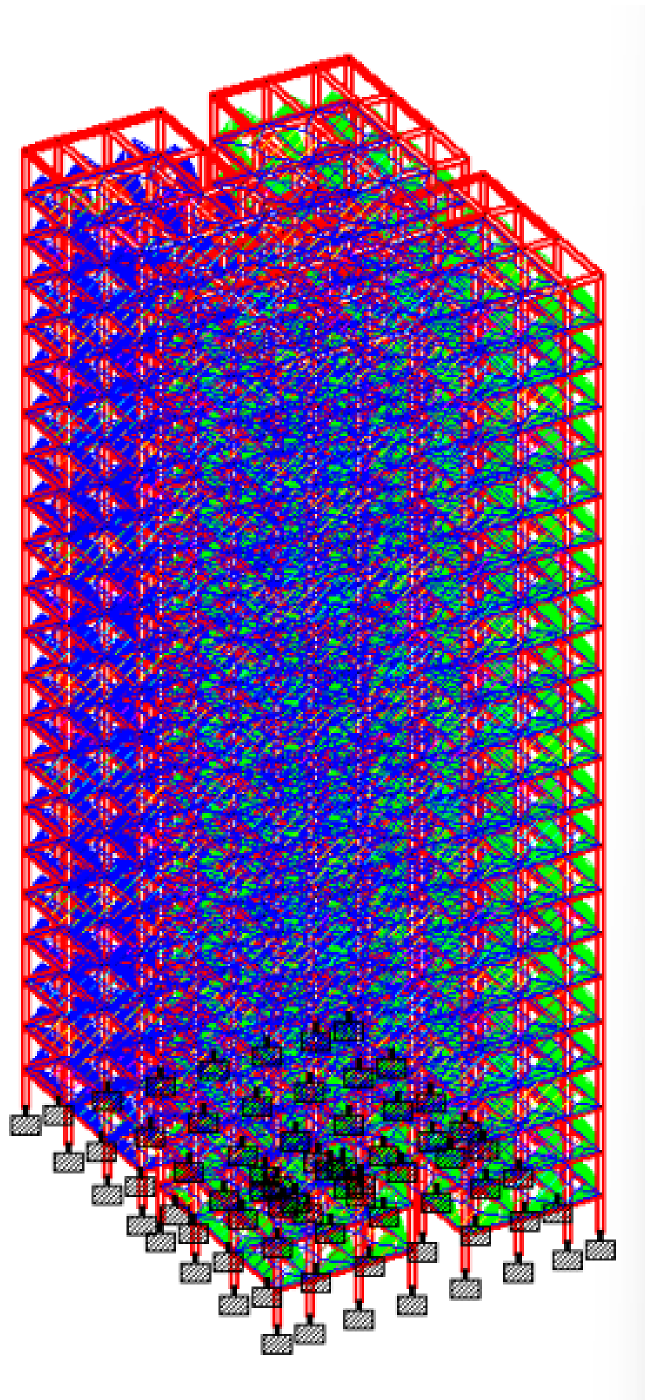




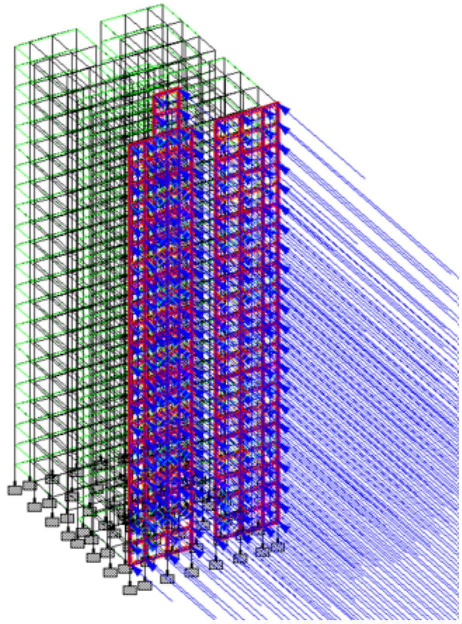
Dead Load Profile



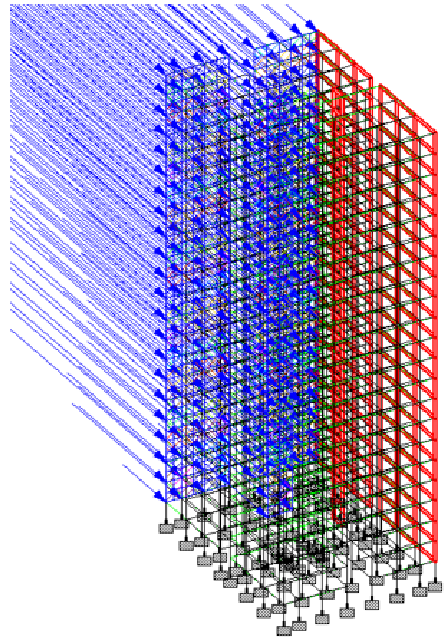
Live Load Profile



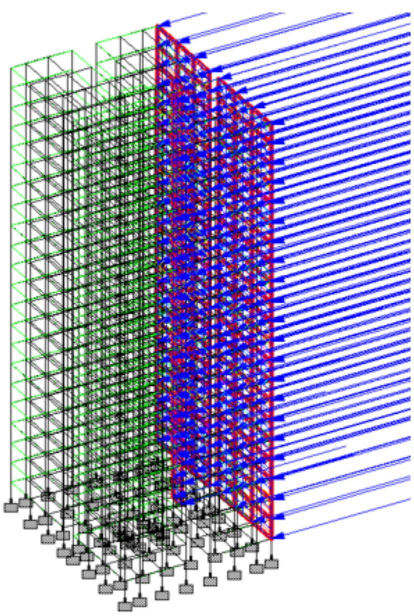
Wind Load Profiles



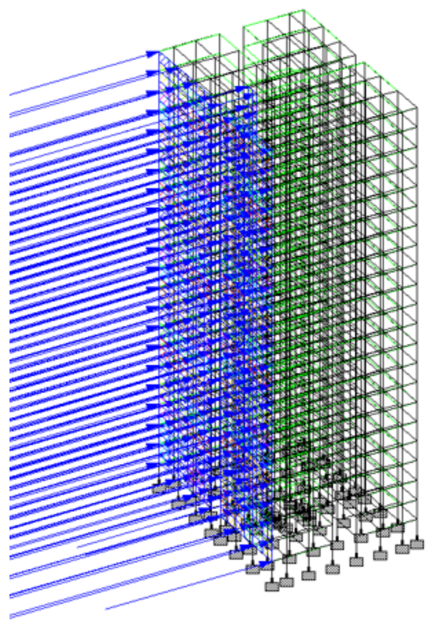
-Z direction



+Z direction

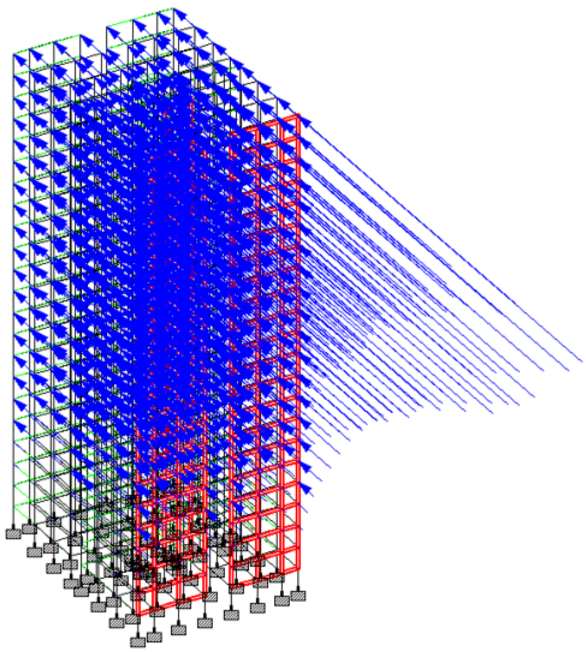


-X direction

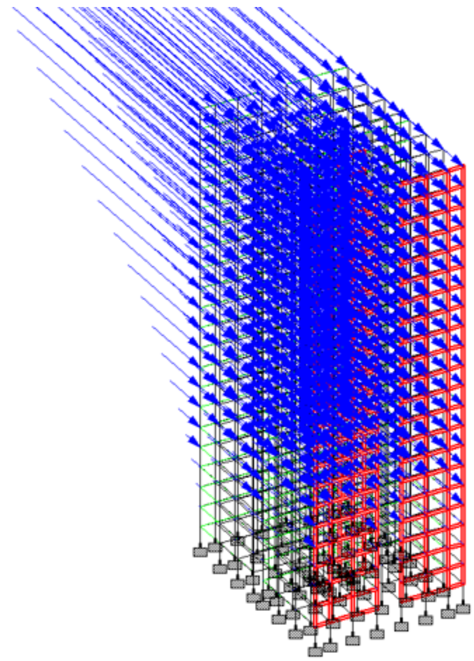


+X direction

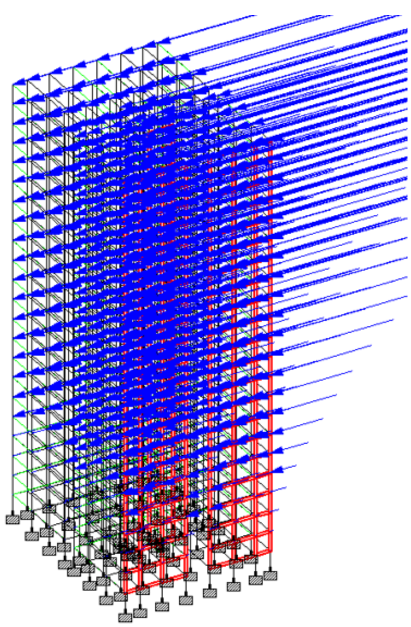
Seismic Load Profiles



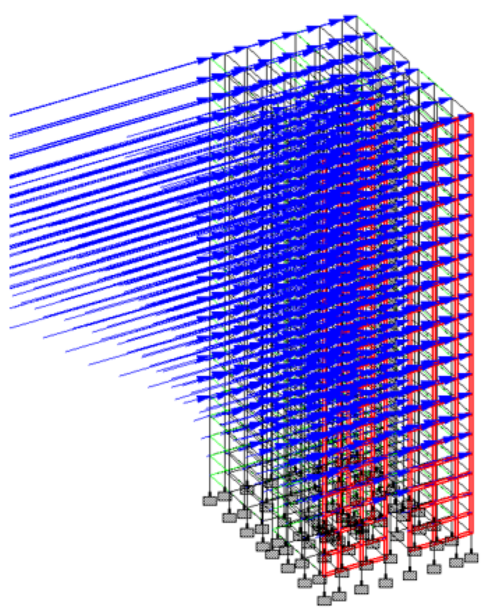
-Z direction



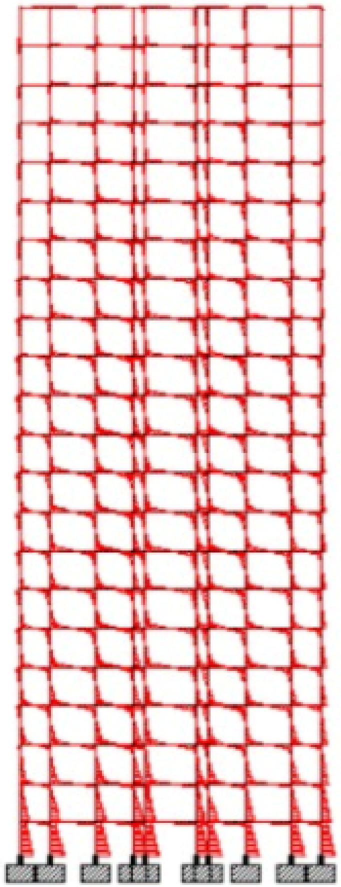
+Z direction



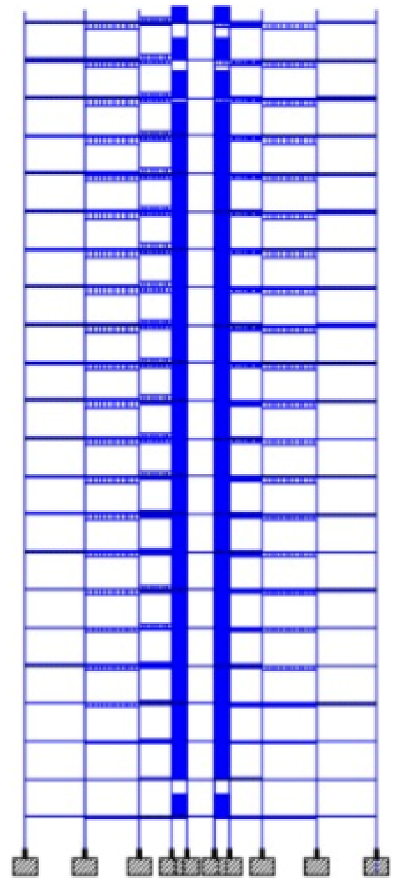
-X direction



+X direction



Bending Moment diagram
for Entire Structure



Shear Force diagram
for entire structure

Design Calculation using STAAD results

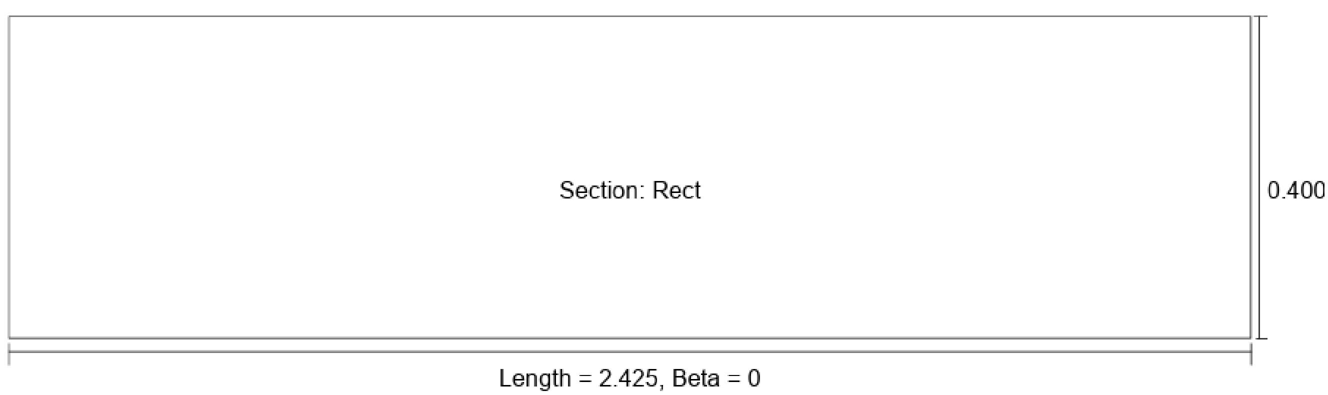
Beam Design:

(for a specified beam, here all are for beam no.749)

Geometry of Beam:

STAAD.Pro Query Geometry

Beam no. 749

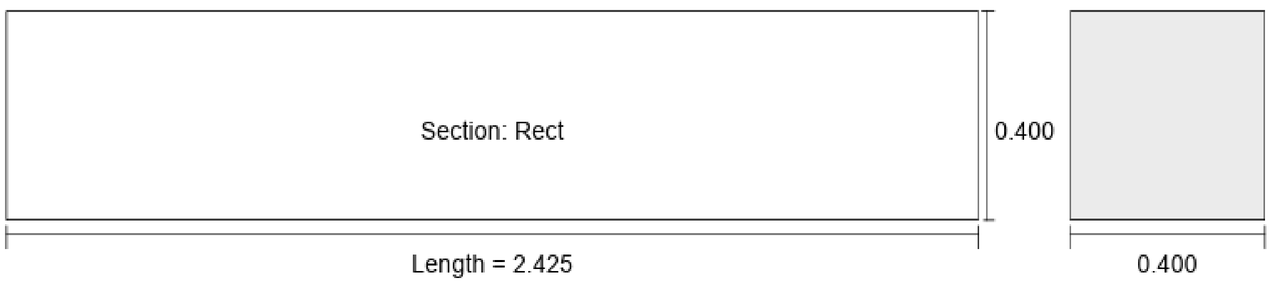


Node	X-Coord (m)	Y-Coord (m)	Z-Coord (m)
297	0.115000	12.450000	5.640000
298	2.540000	12.450000	5.640000

Property of the Beam:

STAAD.Pro Query Property

Beam no. 749



Unit : kN - m

Physical Properties

Ax	0.160	Ix	0.004
Ay	0.160	Iy	0.002
Az	0.160	Iz	0.002
Depth	0.400	Width	0.400

Material Properties

Elasticity(kN/mm2)	21.718	Density(kg/m3)	23.562
Poisson	0.170	Alpha	10 E-6

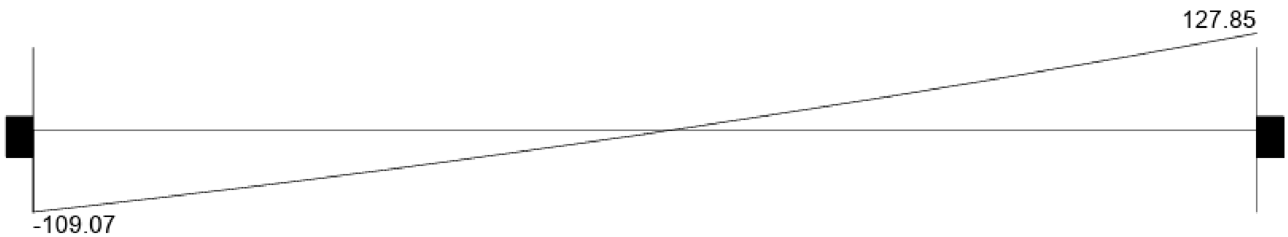
Bending and Shear Results:

(at maximum load combination)

STAAD.Pro Query   Bending and Shear Results

Bending about Z for Beam 749

Load Case: 24:COMBINATION LOAD CASE 24

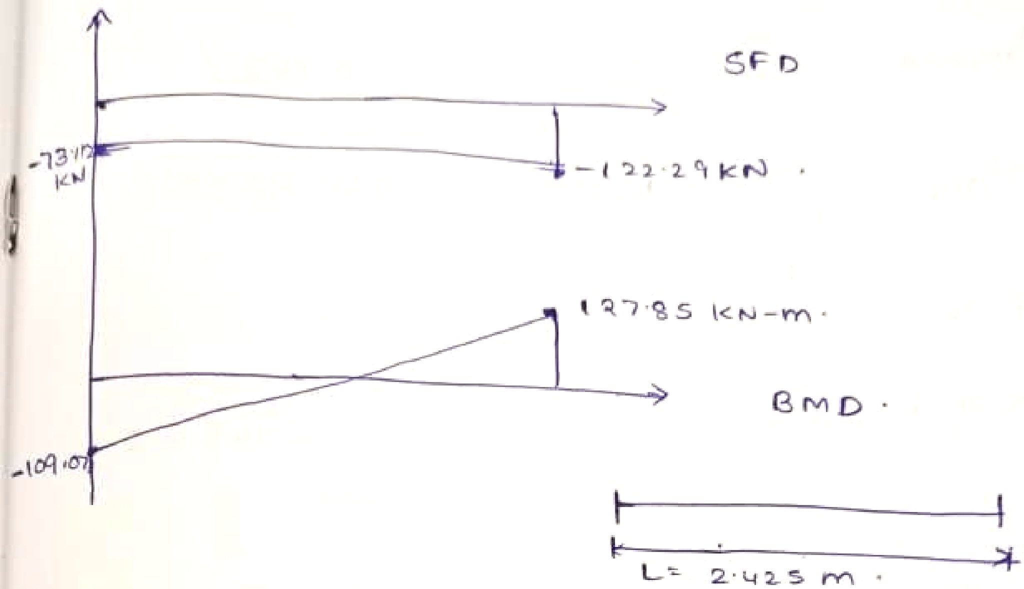


Dist.m	Fy(kN)	Mz(kN-m)
0.000000	-73.1120	-109.0710
0.202083	-77.2102	-93.8822
0.404167	-81.3084	-77.8652
0.606250	-85.4066	-61.0201
0.808333	-89.5048	-43.3467
1.010417	-93.6030	-24.8452
1.212500	-97.7013	-5.5155
1.414583	-101.7995	14.6424
1.616667	-105.8977	35.6285
1.818750	-109.9959	57.4427
2.020833	-114.0941	80.0851
2.222917	-118.1923	103.5557
2.425000	-122.2905	127.8545

Manual Calculations

Design of selected beam

Beam : 749.



Depth = 0.4 m

Width = 0.4 m.

Assumptions
tie bar dia = 8 mm
clear cover = 25 mm
eff cover = 29 mm
d = D-eff = 371 mm

Design for flexure

$f_{ck} = 30$ MPa

$f_y = 500$ MPa (main reinforcement)

$\frac{x_u}{d} = 0.46$ (for balanced section) [From IS 456 : 2000]

For our section

$$\frac{x_u}{d} = 1.22 - \left((1.2)^2 - \frac{6.68}{f_{ck}} \times \frac{M_u}{bd^2}\right)^{0.5}$$

$$= 1.22 - \left((1.2)^2 - \frac{6.68}{30\times10^6} \times \frac{127.85\times10^3}{0.4\times(0.371)^2}\right)^{0.5}$$

$$= 1.22 - 0.9606$$

$$= 0.2593 < 0.46.$$

As $\frac{x_u}{d} < \left(\frac{x_u}{d}\right)_{balanced}$.

Here, beam is in the under-reinforced regime.

$$(p_t)_{balanced} = 41.38 \times 0.46 \times \frac{30}{500} = 1.142\%$$

$$(p_t)_{required} = 41.38 \times 0.2593 \times \frac{30}{500} = 0.643\%$$

$$(A_{st})_{req} = \frac{0.643}{100} \times 400 \times 371 = 954.212\ mm^2$$

If we use 16 mm dia bars.

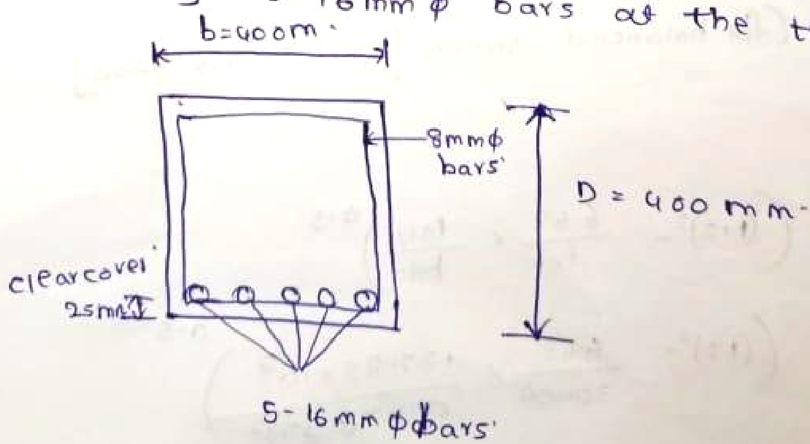
$$\text{No. of bars required} = \frac{(A_{st})_{require}}{\frac{\pi}{4} \times (16)^2} = 4.748 \approx 5.$$

$$A_t \text{ for } 5-16\,mm \text{ bars} = 5 \times \frac{\pi}{4} \times (16)^2 = \cancel{62.8}\ 1004.8.$$

$$p_t \text{ for } 5-16\,mm \text{ bars} = \frac{1004.8}{400 \times 371} \times 100 = 0.677 < p_{t_{bal}}$$

Hence section remains in under reinforced regime after 5-16mm ϕ bars

→ Providing 5-16mm ϕ bars at the tension side of the beam



Design for shear.

$V_u = 122.29\,kN$

$$\tau_v = \frac{V_u}{bd} = \frac{122.29}{400 \times 371} = 8.24 \times 10^{-4}\ kN/mm^2$$

$$= 8.24 \times 10^{-1}\ N/mm^2$$

$$= 0.824\ N/mm^2$$

$(\tau_c)_{max} = 3.5\ N/mm^2$ [IS 456:2000 Table 20]

As, $\tau_v < (\tau_c)_{max}$

, $0.824 < 3.5$ (safe)

$\tau_c =$ M30

$0.75 - 0.59$
$0.5 - 0.5$
$0.64 - ?$

$$\frac{0.75 - 0.64}{0.75 - 0.5} = \frac{0.59 - x}{0.59 - 0.5}$$

$\tau_c = 0.5504$ (via interpolation)

[Table 19 of IS456:2000]

$\tau_v > \tau_c$

Reinforcement is required.

Using 8mm ϕ bars with 2legs

$$A_{sv} = 2 \times \frac{\pi}{4}(\phi^2) = 2 \times \frac{\pi}{4} \times 8^2 = 100.53096\,mm^2$$

$$S_v = \frac{A_{sv}(0.87)f_y}{b \times 0.4} = \frac{100.53096 \times 0.87 \times 410}{400 \times 0.4} = 224.121\,mm$$

$0.75 \times d$ (278.25)

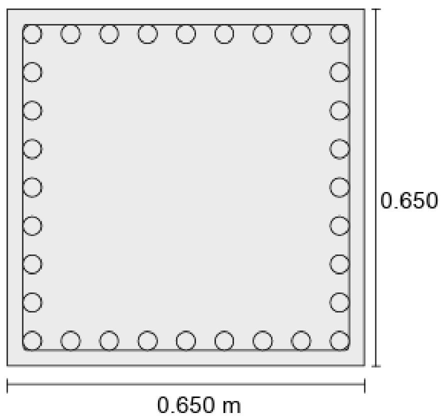
Providing 8mm ϕ bars - 200 c/c.

Concrete Design of Beam:

STAAD.Pro Query   Concrete Design

Beam no. 1954

Design Code: IS-456



Design Load

Load	6
Location	End 1
Pu(Kns)	-47.200001
Mz(Kns-Mt)	34.060001
My(Kns-Mt)	0.260000

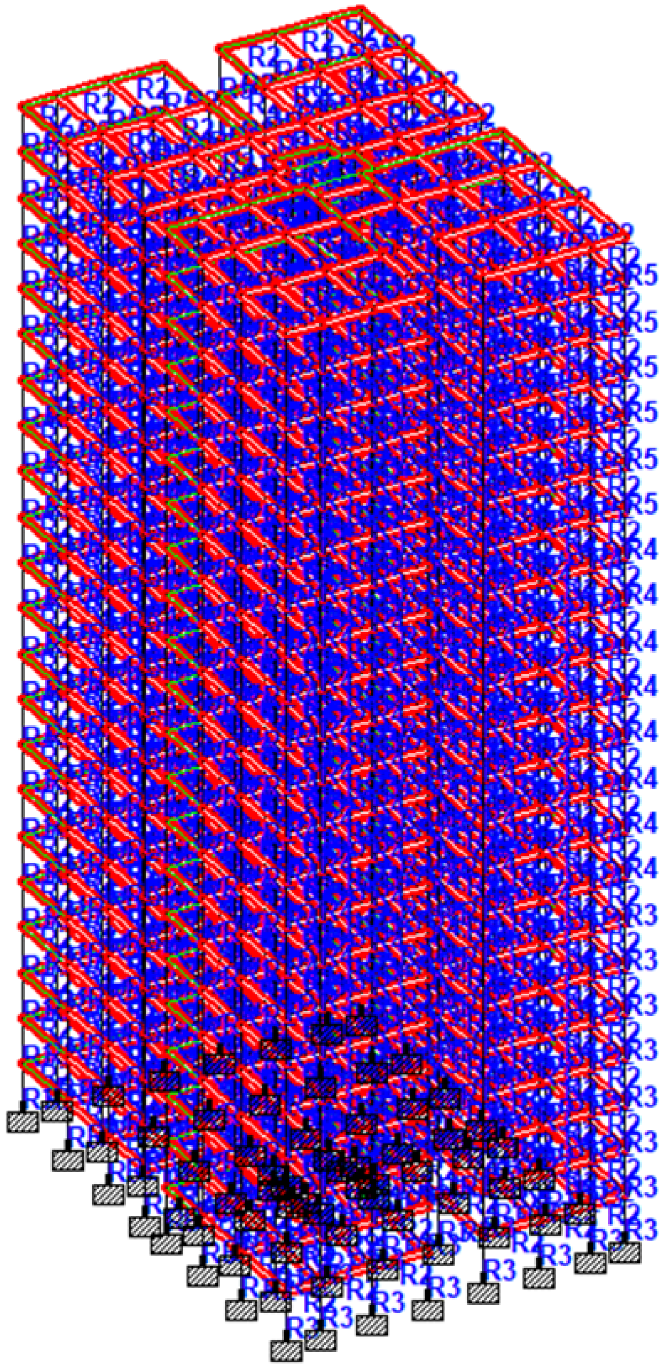
Design Results

Fy(Mpa)	415
Fc(Mpa)	30
As Reqd(mm²)	3380.000000
As (%)	0.857000
Bar Size	12
Bar No	32

Grouping of Beams:

Details	Group 1	Group 2	Group 3	Group 4
Section	2425.0 mm	3650.0 mm	3925.0 mm	5525.0 mm
Cross Section Details	400 x 400 mm^2	400 x 400 mm^2	400 x 400 mm^2	400 x 400 mm^2
Top Reinforcement	#4-20 Φ	#4-20 Φ	#4-20 Φ	#4-20 Φ
Bottom Reinforcement	#4-20 Φ	#4-20 Φ	#4-20 Φ	#4-20 Φ
Shear Reinforcement	2 L- 8 mm dia rectangular ties @175 mm c/c	2 L- 8 mm dia rectangular ties @175 mm c/c	2 L- 8 mm dia rectangular ties @175 mm c/c	2 L- 8 mm dia rectangular ties @175 mm c/c

Details	Group 5	Group 6	Group 7
Section	5000.0 mm	2500.0 mm	4000.0 mm
Cross Section Details	400 x 400 mm^2	400 x 400 mm^2	400 x 400 mm^2
Top Reinforcement	#4-20 Φ	#4-20 Φ	#4-20 Φ
Bottom Reinforcement	#4-20 Φ	#4-20 Φ	#4-20 Φ
Shear Reinforcement	2 L- 8 mm dia rectangular ties @175 mm c/c	2 L- 8 mm dia rectangular ties @175 mm c/c	2 L- 8 mm dia rectangular ties @175 mm c/c



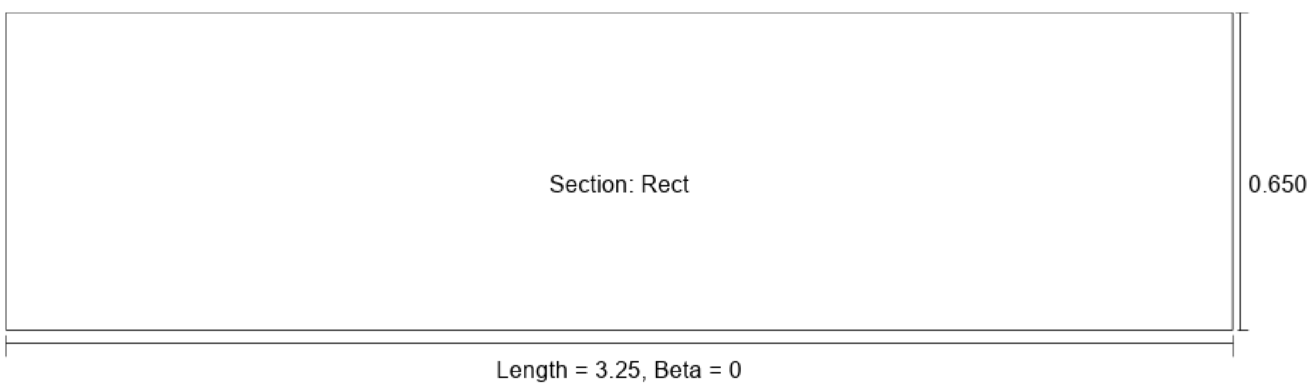
Column Design:

(for a specified column, here all are for beam no. 1954)

Geometry of Column

STAAD.Pro Query Geometry

Beam no. 1954

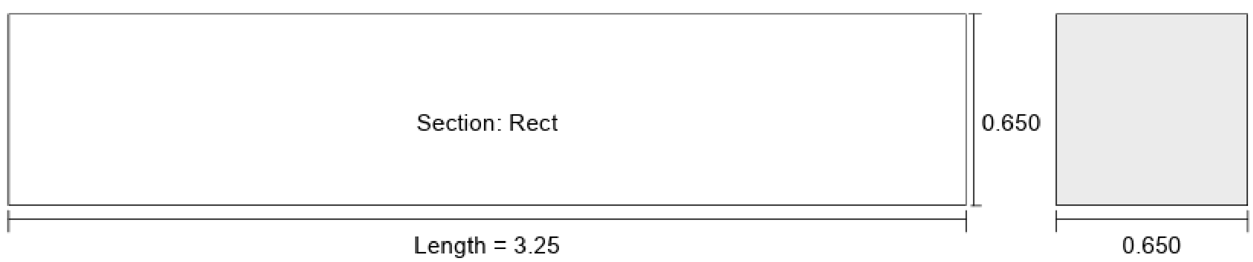


Node	X-Coord (m)	Y-Coord (m)	Z-Coord (m)
722	2.540000	31.950001	0.115000
794	2.540000	35.200001	0.115000

Property of the Column:

STAAD.Pro Query Property

Beam no. 1954



Unit : kN - m

Physical Properties

Ax	0.422	Ix	0.025
Ay	0.422	Iy	0.015
Az	0.422	Iz	0.015
Depth	0.650	Width	0.650

Material Properties

Elasticity(kN/mm2)	21.718	Density(kg/m3)	23.562
Poisson	0.170	Alpha	10 E-6

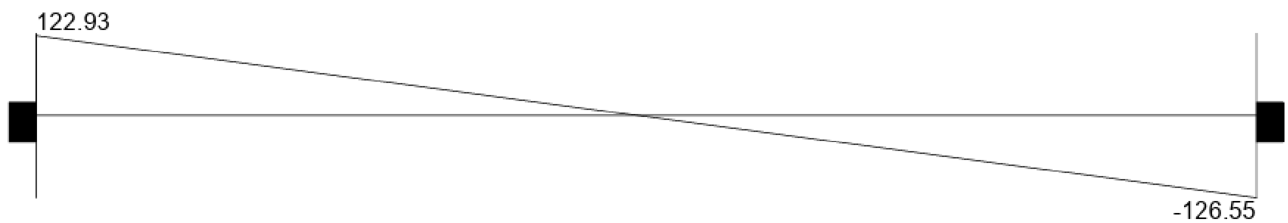
Bending and Shear Results:

(at maximum load combination)

STAAD.Pro Query   Bending and Shear Results

Bending about Z for Beam 1954

Load Case: 24:COMBINATION LOAD CASE 24



Dist.m	Fy(kN)	Mz(kN-m)
0.000000	76.7631	122.9306
0.270833	76.7631	102.1406
0.541667	76.7631	81.3506
0.812500	76.7631	60.5606
1.083333	76.7631	39.7706
1.354167	76.7631	18.9806
1.625000	76.7631	-1.8094
1.895833	76.7631	-22.5994
2.166667	76.7631	-43.3894
2.437500	76.7631	-64.1794
2.708333	76.7631	-84.9694
2.979167	76.7631	-105.7594
3.250000	76.7631	-126.5494

Design of selected column:-

Design loads → $M_z = 34.06$ kN-m

$M_y = 0.26$ kN.m

$P_u = 47.2$ kN

Total length of the column = 3.25m = 3250mm

width (b) = 650mm

Depth (D) = 650mm

$l_{eff} = 0.65(L) = 2.1125$m (As the column is restrained from both the ends).

Check for the short column:-

Slenderness ratio (λ) = $\frac{L_{eff}}{b} = \frac{2.1125}{0.65} = 3.25 < 12$ (short column).

$(e_{min})_z = \frac{3250}{500} + \frac{650}{30} = 28.17$mm

$D/20 = \frac{650}{20} = 32.5$ mm

So, $e_z = 32.5$mm

$(e_{min})_y = \frac{3250}{500} + \frac{650}{30} = 28.17$mm

$b/20 = \frac{650}{20} = 32.5$mm

So, $e_y = 32.5$mm

$M_{uz}' = P_u(e_z) = (47.2)\left(\frac{32.5}{1000}\right) = 1.534$ kN.m

$M_{uy}' = P_u(e_y) = (47.2)\left(\frac{32.5}{1000}\right) = 1.534$ kN-m

$M_{uz} = 34.06$ kN-m, $M_{uy} = 0.26$ kN-m

Since, $M_{uz} > M_{uz}'$ and $M_{uy}' > M_{uy}$.

Design moments are $M_{uz} = 34.06$ kN-m

$M_{uy}' = 1.534$ kN-m

Assuming, $P_t = 0.85\%$

$d' = 90mm$

$$\frac{P_t}{f_{ck}} = \frac{0.85}{30} = 0.0284.$$

$$\frac{P_u}{f_{ck}bd} = \frac{47.2 \times 10^3}{(30)(650)^2} = 3.7239 \times 10^{-3}$$

$$\frac{d'}{D} = \frac{90}{650} = 0.14$$

$$\frac{d'}{B} = \frac{90}{650} = 0.14$$

Assuming 12mm dia. bars, $p_t = 0.85\% \rightarrow$ (1)

Checking for biaxial moment on columns,

$$\left(\frac{34.06}{M_{uz1}}\right)^{\alpha} + \left(\frac{2.834}{M_{uy1}}\right)^{\alpha}$$

(M_{uz1}, M_{uy1} are obtained via interaction diagram in SP16)

α can be obtained by:

$$A_{st} = 0.0085 (650)^2$$

$$= 3591.25 mm^2$$

$$A_g = (650)^2$$

$$= 422500 mm^2$$

$$A_{cc} = A_g - A_{st}$$

$$= 422500 - 3591.25$$

$$= 418908.75 mm^2$$

$$P_{uz} = 0.45 f_{ck} A_{cc} + 0.75 A_{sc} f_y$$

$$= (0.45)(30)(418908.75) + (0.75)(3591.25)(415)$$

$$= 6773044.688N$$

$$= 6773.044kN$$

$$\frac{P_u}{P_{ux}} = \frac{47.2}{6773.044}$$

$$= 6.96 \times 10^{-3}$$

So, $\alpha = 1$

For Biaxial moment,

$$\left(\frac{34.06}{M_{ux1}}\right)^1 + \left(\frac{1.534}{M_{uy1}}\right)^1$$

$$\simeq 0.06751 < 1$$

So permissible.

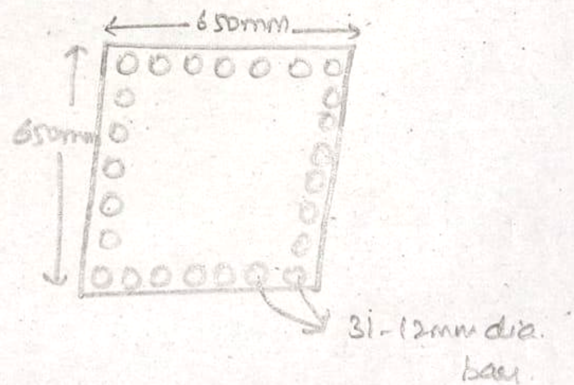
By using the assumption (1)

No. of 12 mm dia. bars required $= \frac{A_{st}}{\pi D^2/4}$

$$= \frac{3591.25}{\pi (12)^2/4}$$

$$= 31.75 \simeq \underline{31}.$$

Using 31 bars of 12mm dia.



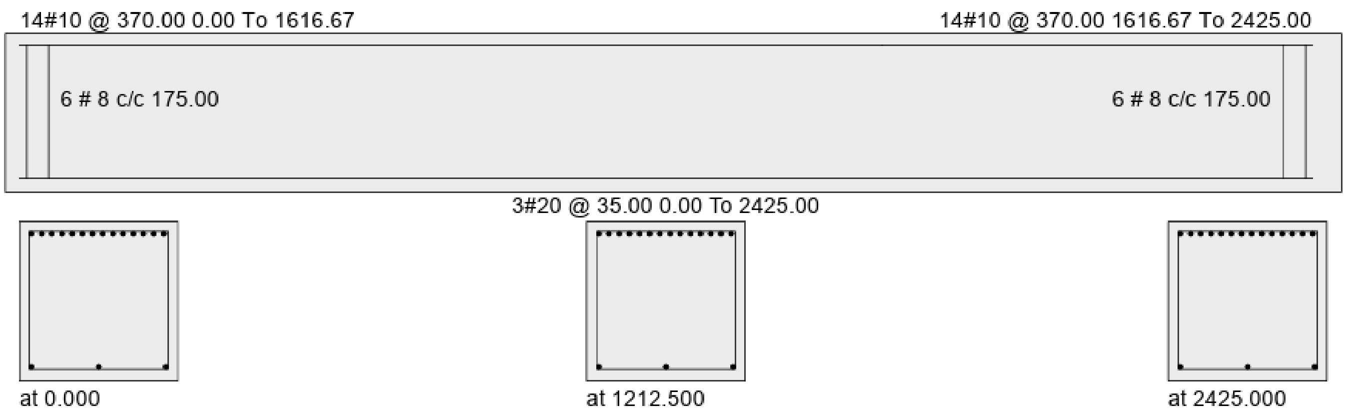
Concrete Design of Column:

(showcased for only group 2 column)

STAAD.Pro Query   Concrete Design

Beam no. 749

Design Code: IS-456



Design Load

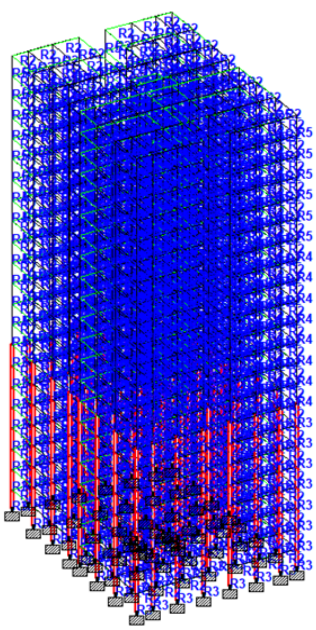
Mz(Kn Met)	Dist.et	Load
109.220001	0.000000	28
-123.269997	0.000000	26
-123.919998	2.400000	24

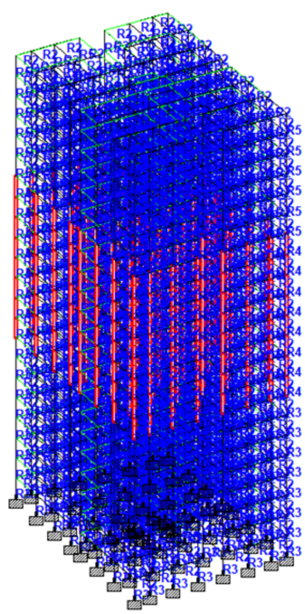
Design Parameter

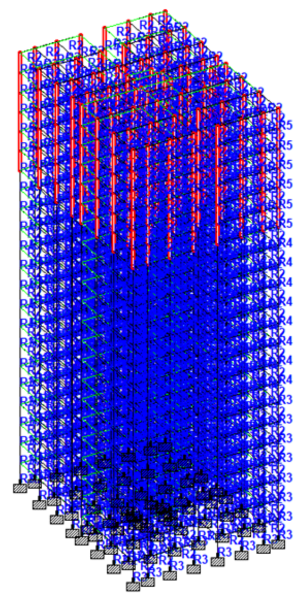
Fy(Mpa)	415.000000
Fc(Mpa)	30.000000
Depth(m)	0.400000
Width(m)	0.400000
Length(m)	2.425000

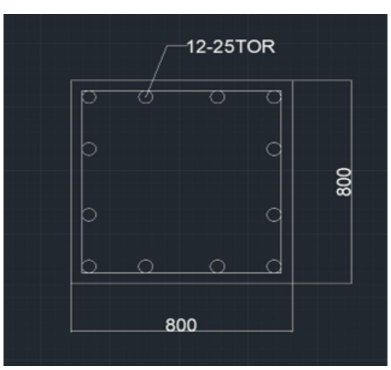
Grouping of Columns:

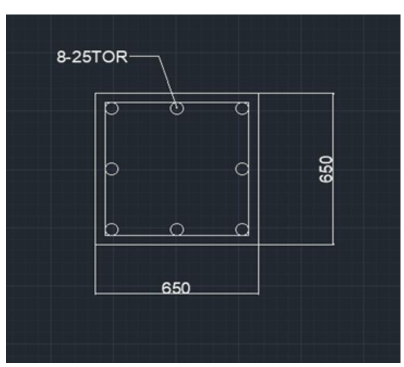
Details	Group 1	Group 2	Group 3
Floor Level	B+G+6	7-15	16-19
Cross Section details	800 x 800 mm^2	650 x 650 mm^2	470 x 470 mm^2
Reinforcement details	#12-25 Φ	#8-25 Φ	#4-25 Φ
Tie bar details	8 mm diameter rectangular ties @ 190 mm c/c	8 mm diameter rectangular ties @ 190 mm c/c	8 mm diameter rectangular ties @ 190 mm c/c

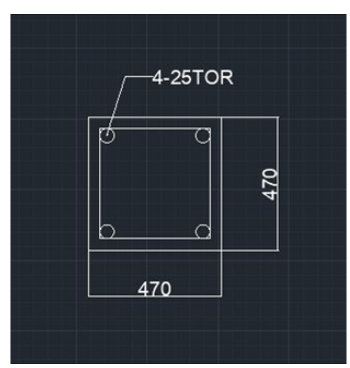




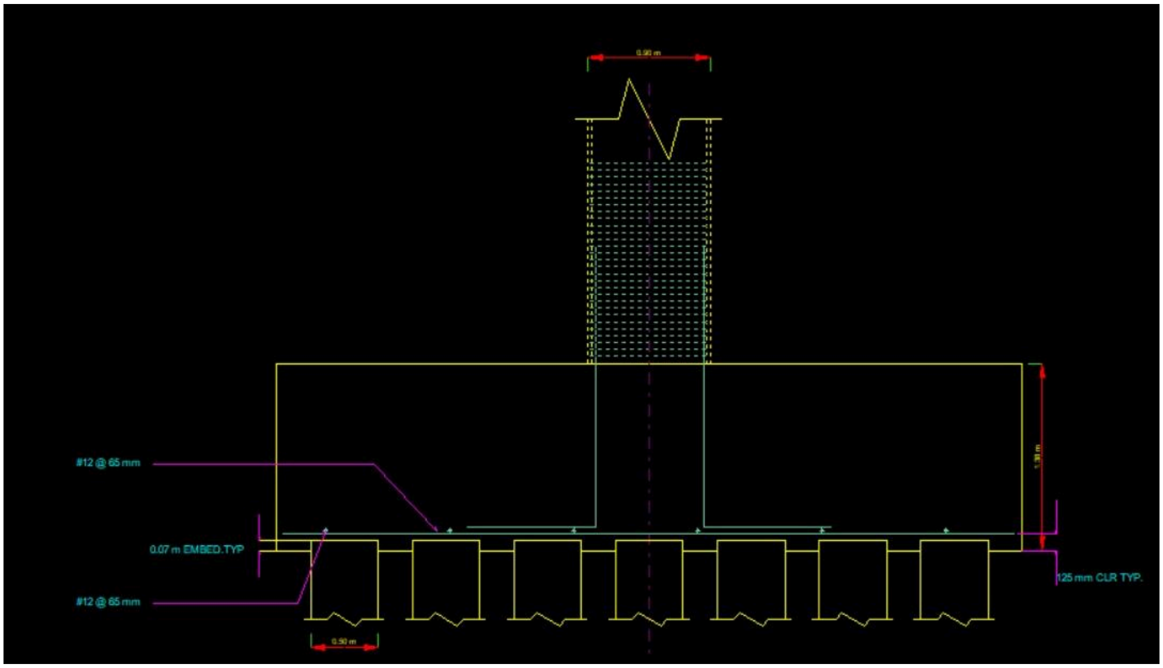


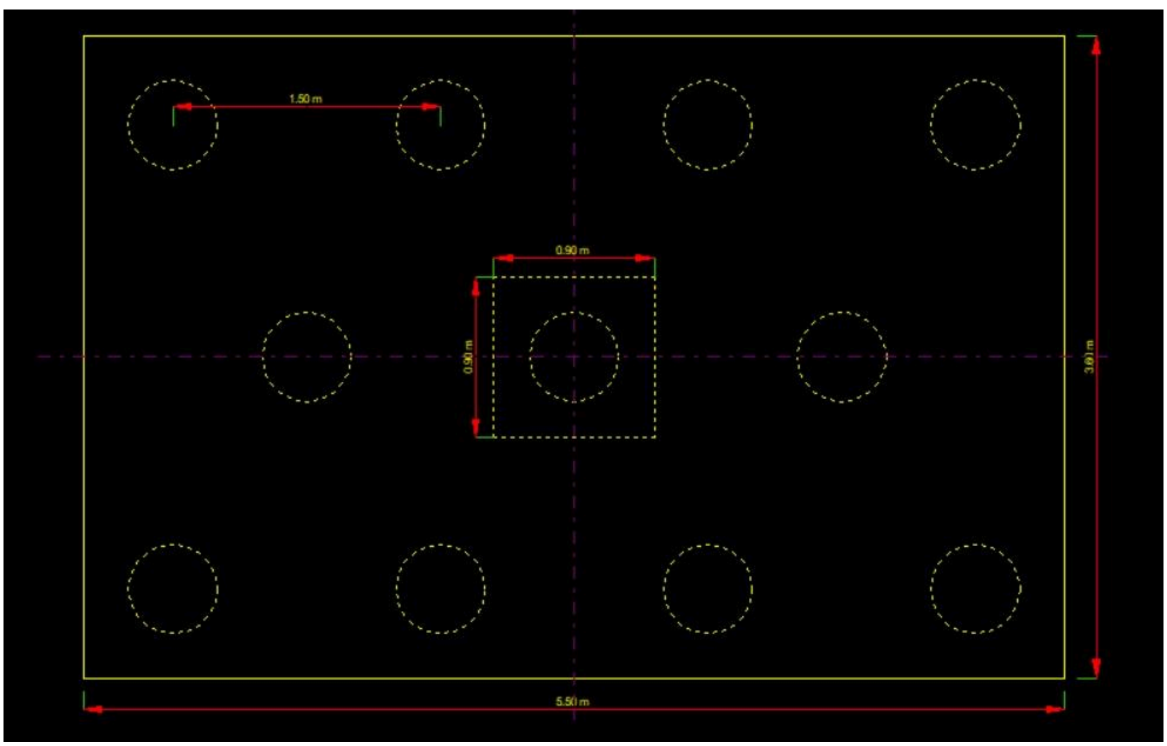






Design and Detailing of Foundation





PILE ARRANGEMENT

Column Dimensions

Column Shape : Rectangular

Column Length - X (Pl) : 0.900 m

Column Width - Z (Pw) : 0.900 m

Pedestal

Include Pedestal? No

Pedestal Shape : N/A

Pedestal Height (Ph) : N/A

Pedestal Length - X (Pl) : N/A

Pedestal Width - Z (Pw) : N/A

Pile Cap Geometrical Data

Pile Cap Length $\mathbf{P_{CL}}$ = 5.500 m

Pile Cap Width $\mathbf{P_{CW}}$ = 3.598 m

Initial Pile Cap Thickness $\mathbf{t_I}$ = 0.300 m

Pile Geometrical Data

Pile spacing $\mathbf{P_s}$ = 1.500 m

Pile Edge distance **e** = 0.500 m

Pile Diameter $\mathbf{d_p}$ = 0.500 m

Pile Capacities

Axial Capacity $\mathbf{P_P}$ = 500.000 kN

Lateral Capacity $\mathbf{P_L}$ = 100.000 kN

Uplift Capacity $\mathbf{P_U}$ = 300.000 kN

Material Properties

Concrete $\mathbf{f'_c}$ = 30000.005 kN/m^2

Reinforcement $\mathbf{f_y}$ = 415000.070 kN/m^2

Concrete Cover

Bottom Clear Cover $\mathbf{CC_B}$ = 0.050 m

Side Clear Cover $\mathbf{CC_S}$ = 0.050 m

Pile in Pile Cap $\mathbf{PC_P}$ = 0.075 m

Loading applied at top of cap

Load Case	F_x (kN)	F_y (kN)	F_z (kN)	M_x (kNm)	M_y (kNm)	M_z (kNm)
1	40.778	885.335	1.783	2.783	0.000	-186.206
2	-40.778	-885.335	-1.783	-2.783	0.000	186.206
3	2.610	413.472	38.231	200.691	0.000	-4.530
4	-2.610	-413.472	-38.231	-200.691	0.000	4.530
5	-0.343	-4577.757	-11.732	-8.702	0.000	0.624
6	30.966	492.461	-1.178	-8.663	0.000	-134.804
7	-23.062	-436.365	-2.790	-9.673	0.000	103.377
8	0.269	147.920	19.564	98.721	0.000	1.856
9	-2.887	-168.120	-17.313	-89.346	0.000	9.105
10	-1.194	-384.382	-1.719	0.016	0.000	2.839
101	3.749	-4926.242	-15.168	-17.647	0.000	-17.002
102	2.999	-3940.994	-12.135	-14.118	0.000	-13.602
201	5.623	-7389.364	-22.752	-26.471	0.000	-25.503
202	3.599	-4729.192	-14.561	-16.941	0.000	-16.322
203	4.499	-5911.491	-18.202	-21.177	0.000	-20.403
204	3.374	-4433.618	-13.651	-15.883	0.000	-15.302

Pile Cap size (in investigated direction) **H =** 5.500 m

Pile Cap size (in investigated perpendicular direction) **B =** 3.598 m

PILE CAP DESIGN CALCULATION

Pile Reactions

Total pile number **N** = 11

	Arrangement		Reaction		
Pile No.	X (m)	Y (m)	Axial (kN)	Lateral (kN)	Uplift (kN)
1	-2.250	-1.299	-686.191	2.131	0.000
2	-2.250	1.299	-679.782	2.131	0.000
3	-1.500	0.000	-683.742	2.131	0.000
4	-0.750	-1.299	-687.701	2.131	0.000
5	-0.750	1.299	-681.293	2.131	0.000
6	0.000	0.000	-685.252	2.131	0.000
7	0.750	-1.299	-689.212	2.131	0.000
8	0.750	1.299	-682.804	2.131	0.000
9	1.500	0.000	-686.763	2.131	0.000
10	2.250	-1.299	-690.722	2.131	0.000
11	2.250	1.299	-684.314	2.131	0.000

Reinforcement Calculation

Maximum bar size allowed along length # 40

Maximum bar size allowed along width # 32

Bending Moment At Critical Section = -3607.705 kNm (Along Length)

Bending Moment At Critical Section = -2337.955 kNm (Along Width)

Pile Cap Thickness **t** = 1.383 m

Selected bar size along length # 16

Selected bar size along width # 12

Selected bar spacing along length = 84.93 mm

Selected bar spacing along width = 68.20 mm

Pile Cap Thickness Check

Calculated Thickness (t) = 1.383 m

Check for Moment (Along Length)

Critical load case for thickness is reported only when required thickness is more than the given minimum thickness

Critical Load Case : 201

Pile No.	Moment along x_1-x_1(kNm)	Moment along x_2-x_2(kNm)
1	-1235.120	0.000
2	-1223.586	0.000
3	-717.916	0.000
4	-206.307	0.000
5	-204.384	0.000
6	0.000	0.000
7	0.000	-206.760
8	0.000	-204.837
9	0.000	-721.088
10	0.000	-1243.278
11	0.000	-1231.743

Effective Depth(d_{ef}) = $h_{cap} - \left(p_{id} + cc + 0.5 \times d_b\right)$ = 1.252 m

Depth of neutral axis for balanced section(x_u) $\dfrac{700 \times d_{eff}}{1100 + 0.87 \times f_y}$ = 0.600 m

As Per IS 456 2000 ANNEX G,G-1.1 C

Ultimate moment of resistance(M_{ulim}) = $0.36 \times f_c \times b \times X_u \times \left(d_{eff} - 0.416 \times X_u\right)$ = 23365.968 kNm

We observed M_u <= M_{ulim} hence singly reinforced and under reinforced section can be used

Check for Moment (Along Width)

Critical load case for thickness is reported only when required thickness is more than the given minimum thickness

Critical Load Case : 201

Pile No.	Moment along y_1-y_1(kNm)	Moment along y_2-y_2(kNm)
1	-582.565	0.000
2	0.000	-577.125
3	0.000	0.000
4	-583.848	0.000
5	0.000	-578.407
6	0.000	0.000
7	-585.130	0.000
8	0.000	-579.690
9	0.000	0.000
10	-586.413	0.000
11	0.000	-580.972

Governing moment (M_u) = -2337.955 kNm

We assume singly reinforced and under reinforcement section

Effective Depth(d_{ef}) = $h_{cap} - \left(p_{id} + cc + 0.5 \times d_b\right)$ = 1.252 m

Depth of neutral axis for balanced section(x_u) $\dfrac{700 \times d_{eff}}{1100 + 0.87 \times f_y}$ = 0.600 m

As Per IS 456 2000 ANNEX G,G-1.1 C

Ultimate moment of resistance(M_{ulim}) = $0.36 \times f_c \times b \times X_u \times \left(d_{eff} - 0.416 \times X_u\right)$ = 35717.849 kNm

We observed M_u <= M_{ulim} hence singly reinforced and under reinforced section can be used

Check for One Way Shear (Along Length)

Pile No.	Shear Force x_1-x_1(kN)	Shear Force x_2-x_2(kN)
1	-417.111	0.000
2	-413.266	0.000
3	-39.902	0.000
4	0.000	0.000
5	0.000	0.000
6	0.000	0.000
7	0.000	0.000
8	0.000	0.000
9	0.000	-40.076
10	0.000	-419.830
11	0.000	-415.985
TOTAL	-870.279	-875.891

Design Shear Force for One-Way Action V_u = -875.891 kN

As Per IS 456 2000 ANNEX B,B-5.1 and Clause No 34.2.4.2

Design Shear Stress (T_v) = $\frac{V_u}{B \times d}$ = -194.440 kN/m^2

Allowable Shear Stress (T_c) = $\frac{0.85 \times \sqrt{0.8 \times f_c}}{6 \times \beta} \times \left(\sqrt{1 + 5 \times \beta} - 1\right)$ = 277.474 kN/m^2

Where Beta = $\max\left(\frac{0.8 \times f_c}{6.89 \times pt}, 1\right)$ = 26.278

and percentage of steel required (p_t) = $\frac{A_{st}}{B \times d} \times 100$ = 0.133

Here $T_v <= T_c$ Hence safe

Check for One Way Shear (Along Width)

Pile No.	Shear Force y_1-y_1(kN)	Shear Force y_2-y_2(kN)
1	0.000	0.000
2	0.000	0.000
3	0.000	0.000
4	0.000	0.000
5	0.000	0.000
6	0.000	0.000
7	0.000	0.000
8	0.000	0.000
9	0.000	0.000
10	0.000	0.000
11	0.000	0.000
TOTAL	0.000	0.000

Design Shear force (V_u)		= 0.000	kN

As Per IS 456 2000 ANNEX B,B-5.1 and Clause No 34.2.4.2

Design Shear Stress (T_v) =	$\frac{V_u}{B \times d}$	= 0.000	kN/m^2
Allowable Shear Stress (T_c) =	$\frac{0.85 \times \sqrt{0.8 \times f_c}}{6 \times \beta} \times \left(\sqrt{1 + 5 \times \beta} - 1\right)$	= 277.474	kN/m^2
Where Beta =	$\max\left(\frac{0.8 \times f_c}{6.89 \times pt}, 1\right)$	= 26.278	
and percentage of steel required (p_t) =	$\frac{A_{st}}{B \times d} \times 100$	= 0.133	
Here	$T_v <= T_c$	Hence safe	

Check for Two Way Shear (Along Length)

Pile No.	Two-way Shear at column face (kN)
1	-417.111
2	-413.266
3	-415.642
4	-395.444
5	-391.807
6	0.000
7	-396.302
8	-392.665
9	-417.455
10	-419.830
11	-415.985
TOTAL	-4075.508

Design Two-Way Shear force		= -4075.508	kN

As Per IS 456 2000 Clause 31.6.2.1

Two Way Shear Stress(T_v) =	$\frac{V_t}{b_0 \times d_{eff}}$	= -378.160	kN/m^2
Where,perimeter of critical section(b_0) =	$2 \times (b + h + 2 \times d)$	= 8.608	m

As Per IS 456 2000 Clause 31.6.3.1

Allowable shear stress =	$K_s \times t_c$	= 1369.307	kN/m^2
Where,k_s =	$\min[(0.5 + \beta), 1]$	= 1.000	
Ratio of shorter to longer dimension(B_c)		= 1.000	
and,T_c =	$0.25 \times \sqrt{f_c} \times b \times d$	= 1369.307	kN/m^2

$T_v < K_s T_c$ hence Safe

Calculation of Maximum Bar Size

Along Length

Selected maximum bar size = 40 mm

Bar diameter corresponding to max bar size(d_b) =40.000 mm

As Per IS 456 2000 Clause No 26.2.1

Development Length(l_d) = $\frac{0.87 \times d_b \times f_y}{4 \times \tau_{bd}}$ = 1.471 m

Allowable Length(l_{db}) = $0.5 \times (B - b) - C_g$ = 2.250 m

$l_{db} > l_d$ hence, safe

Along Width

Selected maximum bar size = 32 mm

Bar diameter corresponding to max bar size(d_b) =32.000 mm

As Per IS 456 2000 Clause No 26.2.1

Development Length(l_d) = $\frac{0.87 \times d_b \times f_y}{4 \times \tau_{bd}}$ = 1.177 m

Allowable Length(l_{db}) = $0.5 \times (H - h) - Cr$ = 1.299 m

$l_{db} > l_d$ hence, safe

Selection of Bottom and Top Reinforcement

Top reinforcement is provided same as bottom reinforcement

Along Length

Critical Load Case : 201

As Per IS 456 2000 Clause 26.5.2.1

Minimum Area of Steel (A_{stmin}) = $0.12\% \times B \times h_{cap}$ = 5945.335 mm2

As Per IS 456 2000 ANNEX G,G-1.1 b

Area of steel required (A_{sq}) = $0.5 \times \left(\frac{f_c}{f_y}\right) \times \left(1 - \sqrt{1 - \frac{4.5977 \times M_u}{f_c \times b \times d \times d}}\right) \times b \times d$ = 8327.142 mm2

Area of steel provided (A_{st}) = = 8327.142 mm2

$A_{stmin} <= A_{st}$ Steel area is accepted

Minimum spacing allowed (S_{min}) = 40 + d_b = 56 mm

Selected spacing (S) = 84.93 mm

S_{min} <= S <= 450 mm and selected bar size < selected maximum bar size... The reinforcement is accepted.

Along Width

Critical Load Case : 201

As Per IS 456 2000 Clause 26.5.2.1

Minimum Area of Steel (A_{stmin}) $0.12\% \times B \times h_{cap}$ = 9061.800 mm2

As Per IS 456 2000 ANNEX G,G-1.1 b

Area of steel required (A_{sq})= $0.5 \times \left(\frac{f_c}{f_y}\right) \times \left(1 - \sqrt{1 - \frac{4.5977 \times M_u}{f_c \times b \times d \times d}}\right) \times b \times d$ = 9061.800 mm2

Area of steel provided (A_{st}) = = 9061.800 mm2

$A_{stmin} <= A_{st}$ Steel area is accepted

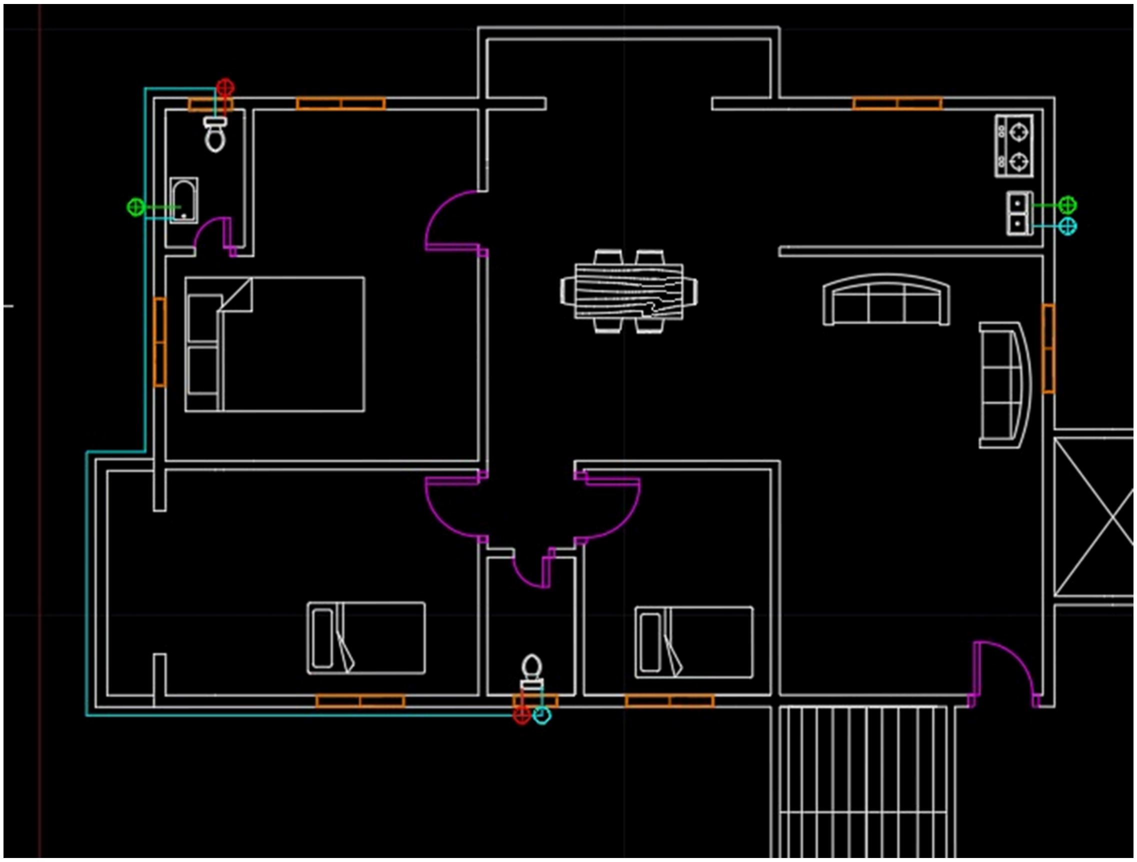
Minimum spacing allowed (S_{min}) = 40 + d_b = 52.00 mm

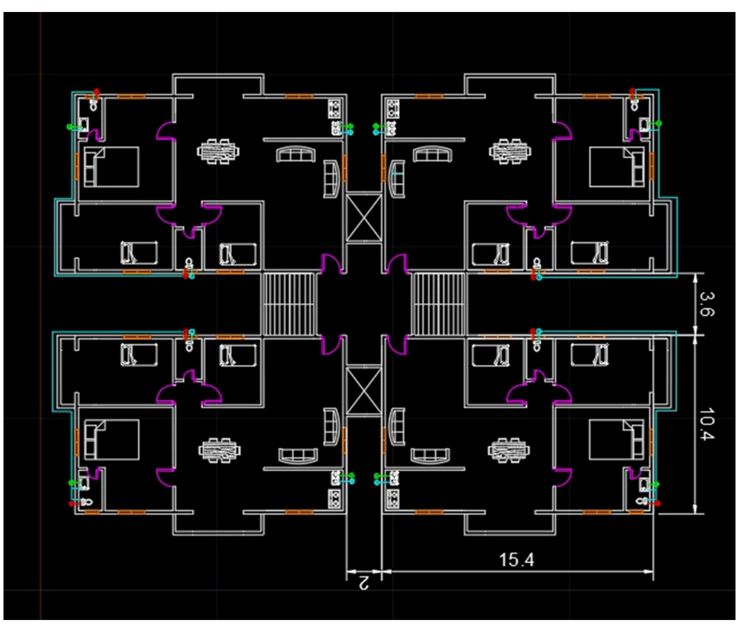
Selected spacing (S) = 68.20 mm

S_{min} <= S <= 450 mm and selected bar size < selected maximum bar size... The reinforcement is accepted.

Plumbing and Water Supply Network

For a single flat





(Plumbing layout for a entire floor)

- Blue coloured pipes are inlet pipes.
- Red and green coloured pipes are outlet pipes.
- Red pipes are into the septic tank.
- Green pipes are for gray water and sewage.
- Plumbing was designed in this way for the aesthetic purpose of the front view so that all the water coming from the pipes will be collected at the same place.

Calculations:

(done with suitable assumptions)

Water supply design for FTA:

Water demand = 135 LPCD

For total apartment: There are 4 houses in one floor

Total Population = 76*4 = 304 (Considering the average family size as 4)

Total water demand = 304 * 135 = 41040 litres/day

Assuming that water supply is available in the street main all through 24 hours,

Average rate of supply = 48000 litres /day,

i.e., Assuming 5 hours pumping into the overhead storage tank, with 8 motors of 1200 litres/hr capacity

Rate of pumping = 8x1200 = 9600 litres/hr

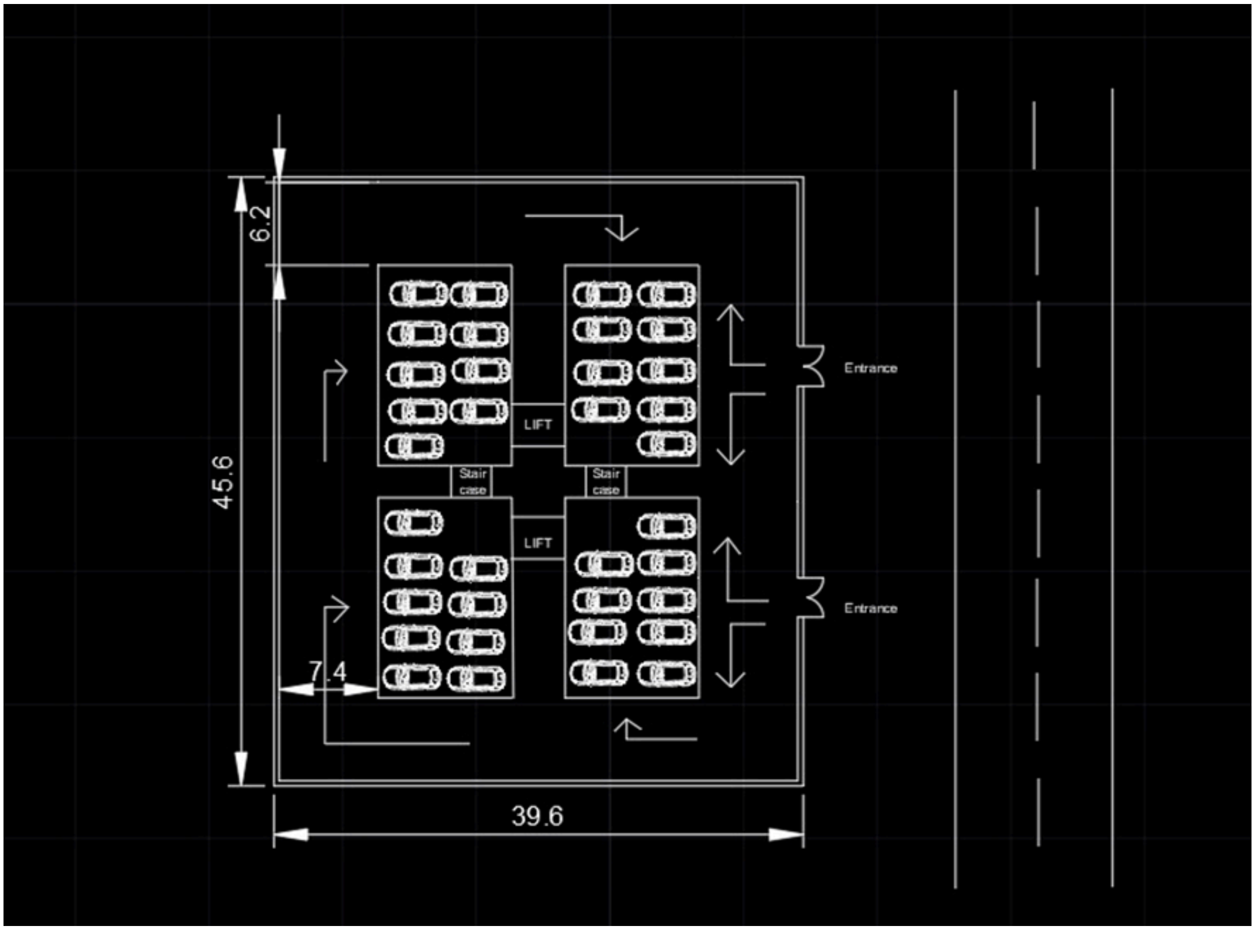
Assuming that the pattern of pumping is **2.5 hours in the morning and 2.5 hours in the evening**

Each tank capacity = 48000/4 = 12000 litres = 12 m^3

Tank dimensions considered = 2.5m * 2.5m * 2m = 12.5 m^3

(refer roof plan attached)

Road Network Details



- Separate gates for entry and exit are provided for smooth regulation of vehicle traffic and pedestrians.
- A 3m wide entry road is provided for ample space considerations.
- These entry gates are connected to the 7m wide main road
- We assume that most of the cars are parked in the basement and remaining cars and bikes are allotted space in the ground floor

Quantity and Cost Estimation

* Columns :

- No. of columns in each floor = 64
- Calculation of volume of columns :

  - Column Group-1 (B+G+6 Floors) :- Size : 0.8 m × 0.8 m.

    Height : B+G+6 floors
    = 2.7 + 2.7 + 6×3.25
    = 24.9 m.

    Volume = 0.8×0.8×24.9
    = 15.936 m^3

  - Column Group-2 (Floor 7 to 15) :-

    Size : 0.65 m × 0.65 m

    Height : 9×3.25
    = 29.25 m.

    Volume : 0.65×0.65×29.25 m^3
    = 12.358 m^3

  - Column Group-3 (Floor 16 to 19) :

    Size = 0.47×0.47 m

    Height = 4×3.25
    = 13 m

    Volume = 0.47×0.47×13
    = 2.8717 m^3

Volume of a column = 15.936 + 12.358 + 2.8717
= 31.1657 m^3

Total column volume = 64 × 31.1657
= 1994.6048 m^3

Volume of concrete = $1994.6048 \times \frac{99.17}{100}$ = 1978.053 m^3

Volume of steel = 16.555 m^3

Beams :

- Beam size : 0.4m × 0.4 m
- Calculation of volume of beams :

- Length of beams per apartment
= (4.7×4) + (5.2×4) + (5.6×4) + (2.7×5) + (3.5×4) + (4.2×5)
= 110.5 m

- Volume of beams per apartment = 110.5 × 0.4 × 0.4
= 17.68 m^3

- Volume per floor = 17.68 × 4 = 70.72 m^3

- Total volume of beams = 70.72 × 21 = 1485.12 m^3
- Volume of concrete = 1485.12 × 0.99357 = 1475.571 m^3
- Volume of steel = 9.549 m^3

* Slabs :

- Floor area of single apartment $= (4.7+5.2+5.6)(2.7+3.5+4.2)$

$= 15.5 \times 10.4 = 161.2 \ m^2$

- Floor area $= 4 \times 161.2 = 644.8 \ m^2$

- Thickness of slab $= 150$ mm.

- Volume of slabs per floor $= 644.8 \times 0.15$

$= 96.72 \ m^3$

- Total volume for B+G+19 floors $= 96.72 \times 21$

$= \underline{2031.12} \ m^3$

- Volume of concrete $= 2031.12 \times 0.9921$

$= 2015.074 \ m^3$

- Volume of steel $= 2031.12 \times 0.0079$

$= 16.04585 \ m^3$

* **Bricks Estimation** :

Brick size : 190 mm x 90 mm x 90 mm

- Calculation for top 19 Floors :

  - length of wall for single apartment

    $= 2(4.7+5.2+5.6+6.2+4.2)+(4.7+4.2+3.4+6.2+5.6$

    $= 75.9\ m$

  - length of wall ~~along~~ for a floor : 4×75.9

    $= 303.6\ m$

  - No. of bricks along length : $\frac{303.6}{0.19} = 1598$

  - Height of wall $= 19 \times 3.25 = 61.75\ m$

  - No. of bricks along height $= \frac{61.75}{0.09} = 687$

  - Total bricks $= 1598 \times 687$

    $= 10,97,826$

- Calculation for basement & ground floor :

  considering outer walls only -

  - length of wall $= 4(4.7+5.2+5.6+6.2+4.2)$

    $= 103.6\ m$

- height of basement + ground floor = 2.7 + 2.7
= 5.4 m

- bricks along length = $\frac{103.6}{0.19}$ = 546
- bricks along height = $\frac{5.4}{0.09}$ = 60

- Total bricks = 564×60 = 33840

• Calculation for lift covering :

- Lift size = 2m × 3m.
- required column = 2(2+3) = 10 m
- height = B+G+19 Floors
= 5.4 + 61.75
= 67.15 m

- Bricks along length = $\frac{10}{0.19}$ = 53

- Bricks along height = $\frac{67.15}{0.09}$ = 747

- Total Bricks = 747×53 = 39591

• Total No. of Bricks = 1097826 + 33840 + 39591
= <u>11,71,257</u>

- Total bricks = 11,71,257

- Total concrete volume = 1978.053 + 1475.571 + 2015.074 = 5468.698 m^3

- Total steel volume = 16.555 + 9.549 + 16.04585 = 42.15 m^3

Total weight of steel = 7800 × 42.15

= 328768.83 kg

Considering 10% wastage ⇒

Bricks required = 1288382.7

Concrete required = 6015.5678 m^3

Steel required = 361645.713 kg

✱ Cost Estimation :

Assuming : cost of one brick = Rs. 10

Rate of steel bars = Rs 70 / kg

Rate of M20 concrete = Rs 4800 / m^3

∴ costs ⇒ Bricks = 1,28,83,827

Steel = 2,53,15,199.91

Concrete = 2,88,74,725.44

Total = Rs 6,70,73752.35 ≈ Rs 6.71 Cr