Design Sessional Project Report (CE39004)

Indian Institute of Technology Kharagpur-721302

Department Of Civil Engineering

GROUP NO :6

Project
G+B+17 RESIDENTIAL TOWER LOCATED AT GUWAHATI

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DESIGN BASIS REPORT

Problem Statement

1. The campus is located at Guwahati for even group numbers.

2. Each Residential Tower will be of (B+G+5+2*Group Number) storey

3. Each floor will have four apartments of approximate area as (140±10) m2 approximately each, with common facilities.

4. The bearing capacity of the soil for towers with is 20 ton/m2.

5. Clear height of the, basement and ground floor is 2.70 m

6. Height of each floor is 3.25 m.

7. Analysis and design of the Towers should comply with the latest Indian Codes of Practice.

8. For all RCC structural elements, M30 grade of concrete and Fe500 steel will be used. For steel members, Fe410 grade will be used.

9. The floor diaphragms are assumed to be rigid.

10. Centre-line dimensions will be followed for analysis.

11. Seismic loads will be considered acting in the horizontal direction and not along the vertical direction.

12. Any other parameter needed for the design can suitably be assumed with proper justification.

Materials Considered

- ❖ M25 grade concrete for all structural members.
- ❖ Fe500 for all the RCC reinforcements.
- ❖ Fe410 for all the steel members.
- ❖ The bearing capacity of the soil for towers with is 20 ton/m2.

Location

Guwahati (Seismic Zone V)

- Guwahati, the largest city in the Indian state of Assam, is situated in a seismically active region.
- The city falls within Seismic Zone V, a high-risk area for earthquakes, according to the seismic zoning map of India.

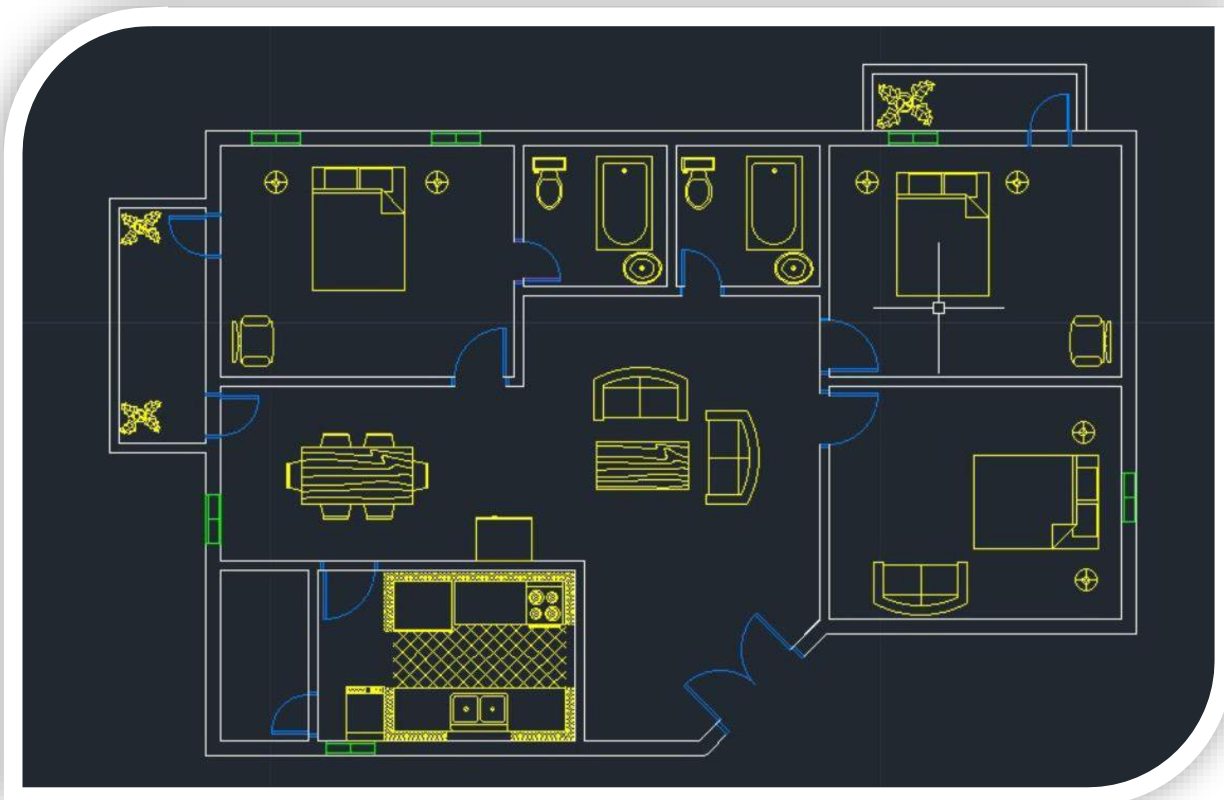
Geometry and Floor Area

H - shaped tower with 4 apartments per floor with floor area of 147.5 m^2

Of a single apartment.

Plan Of Tower

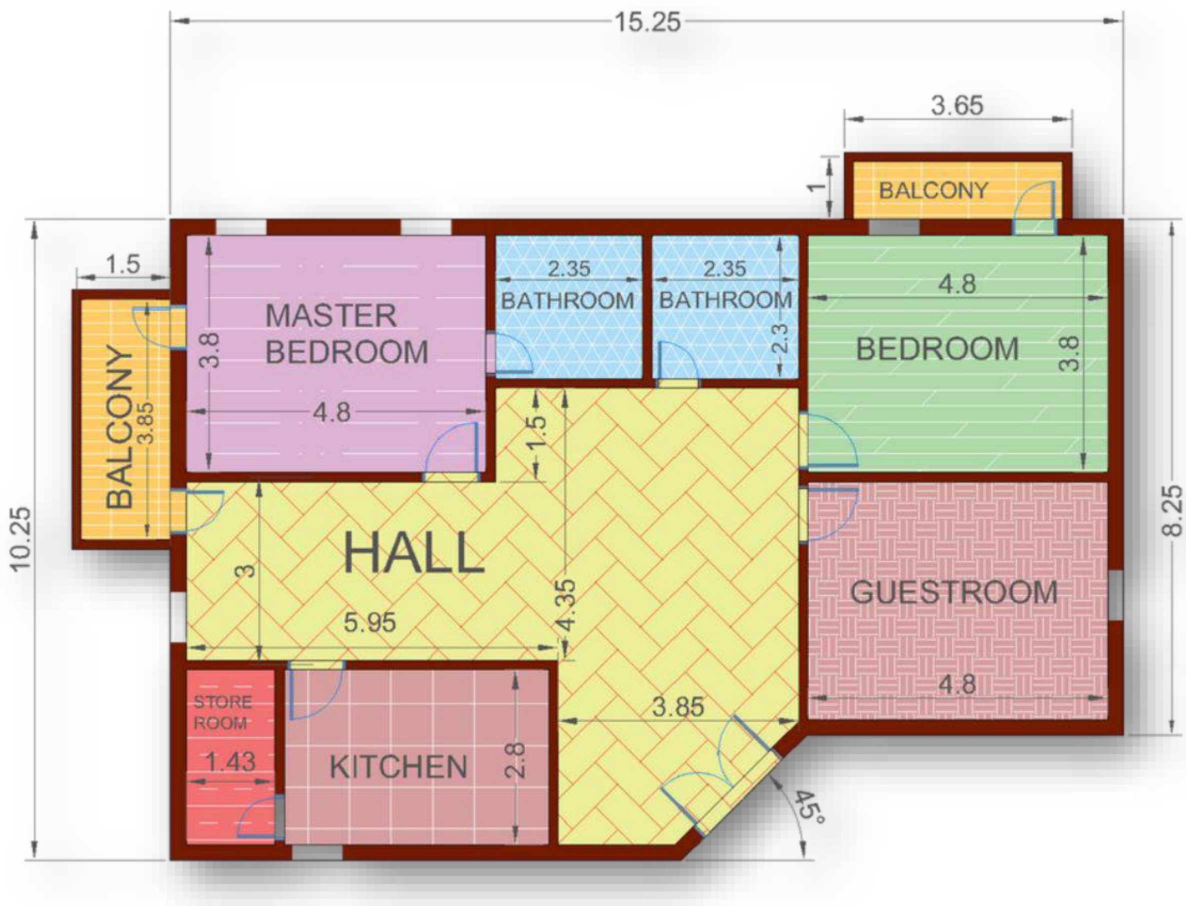
Apartment GAD



Each Apartment Consists of

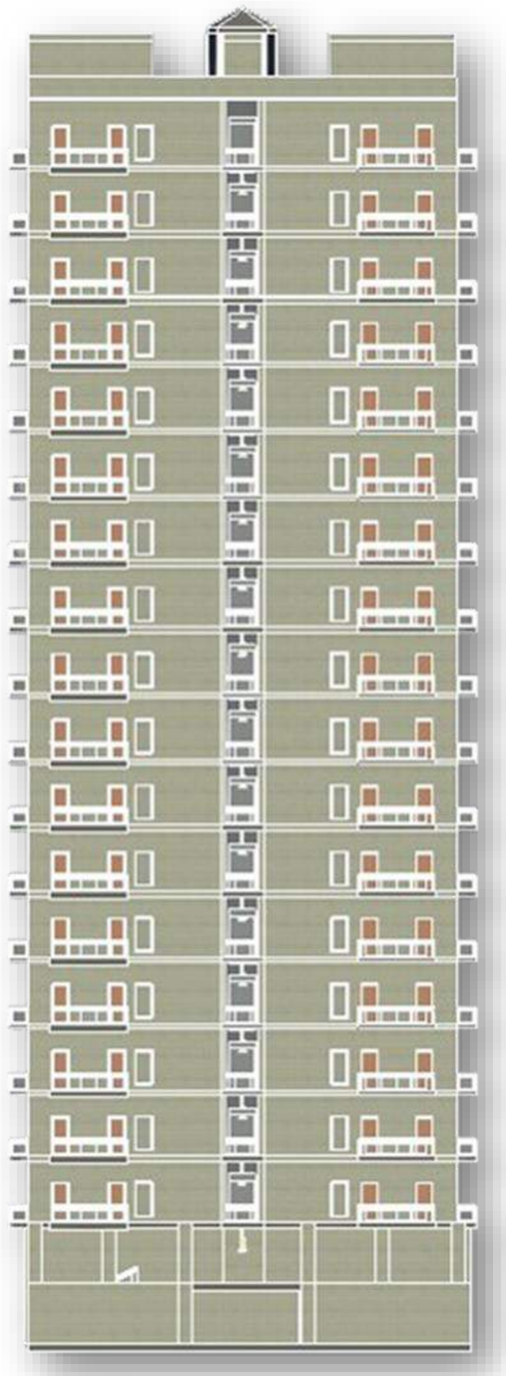
1 Master BedRoom	1 Bed Room
1 Kitchen	2 Bathroom
1 GuestRoom	1 StoreRoom
1 Hall	2 Balcony

Apartment GAD with Dimensions

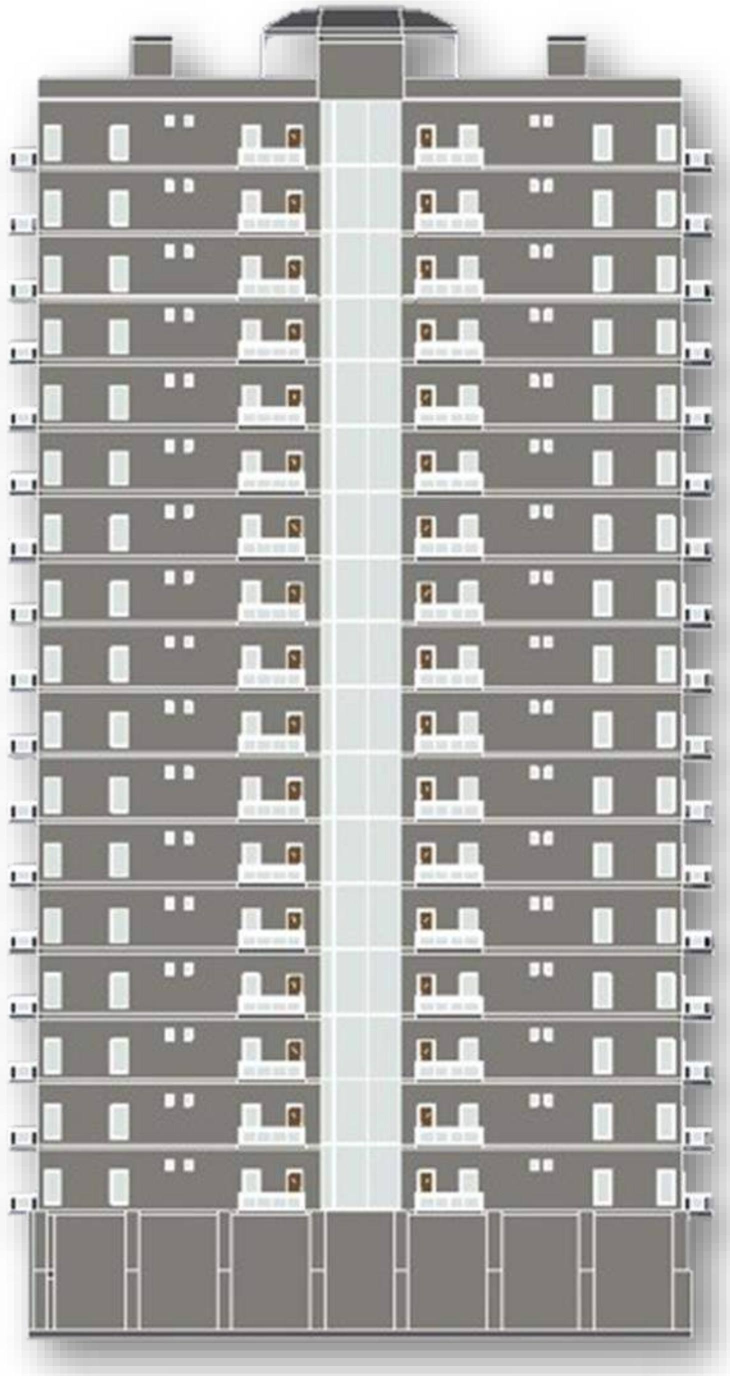


FRONT,SIDE AND ISOMETRIC VIEW

Front View



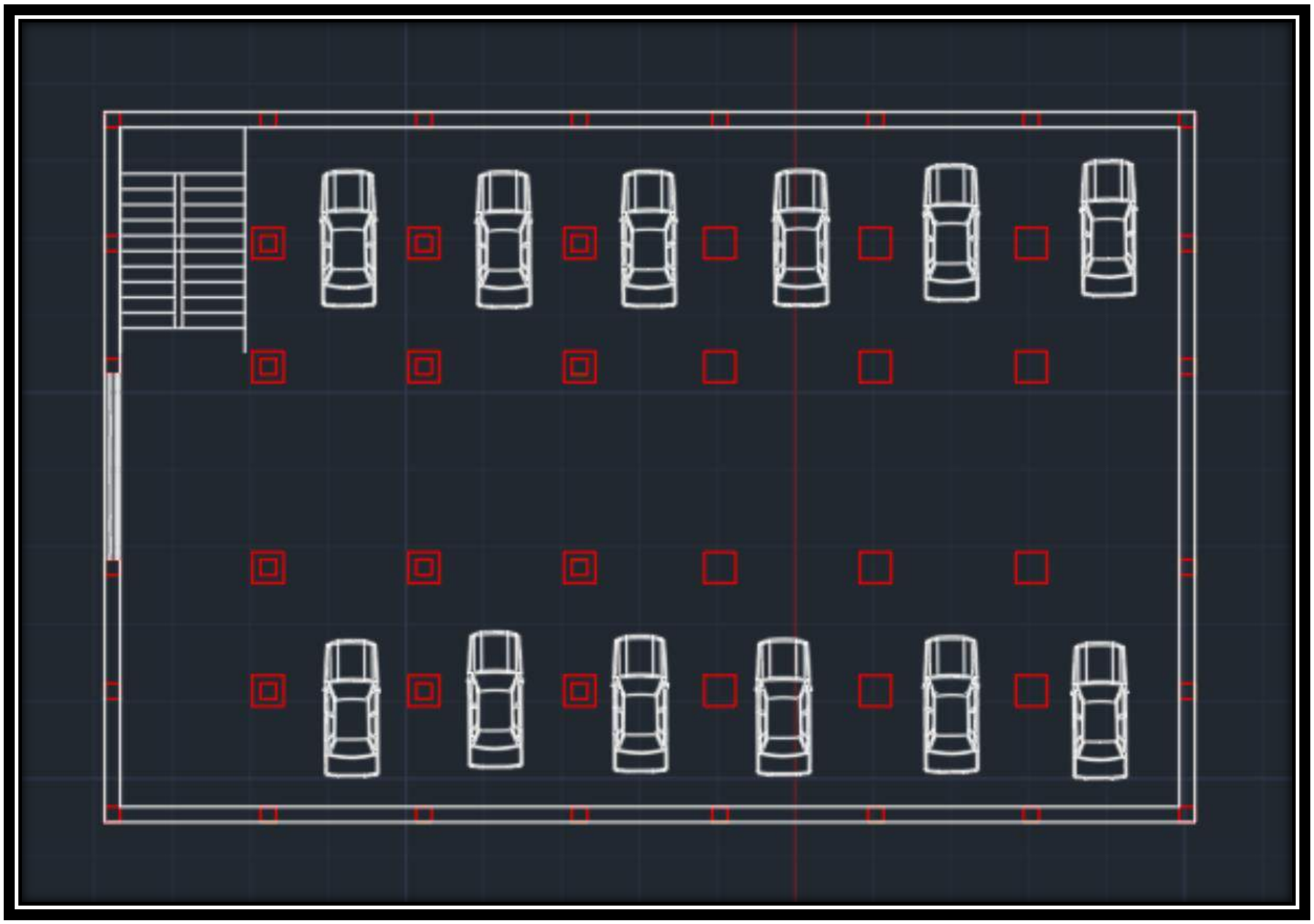
Side view



Isometric View

Floor Plan

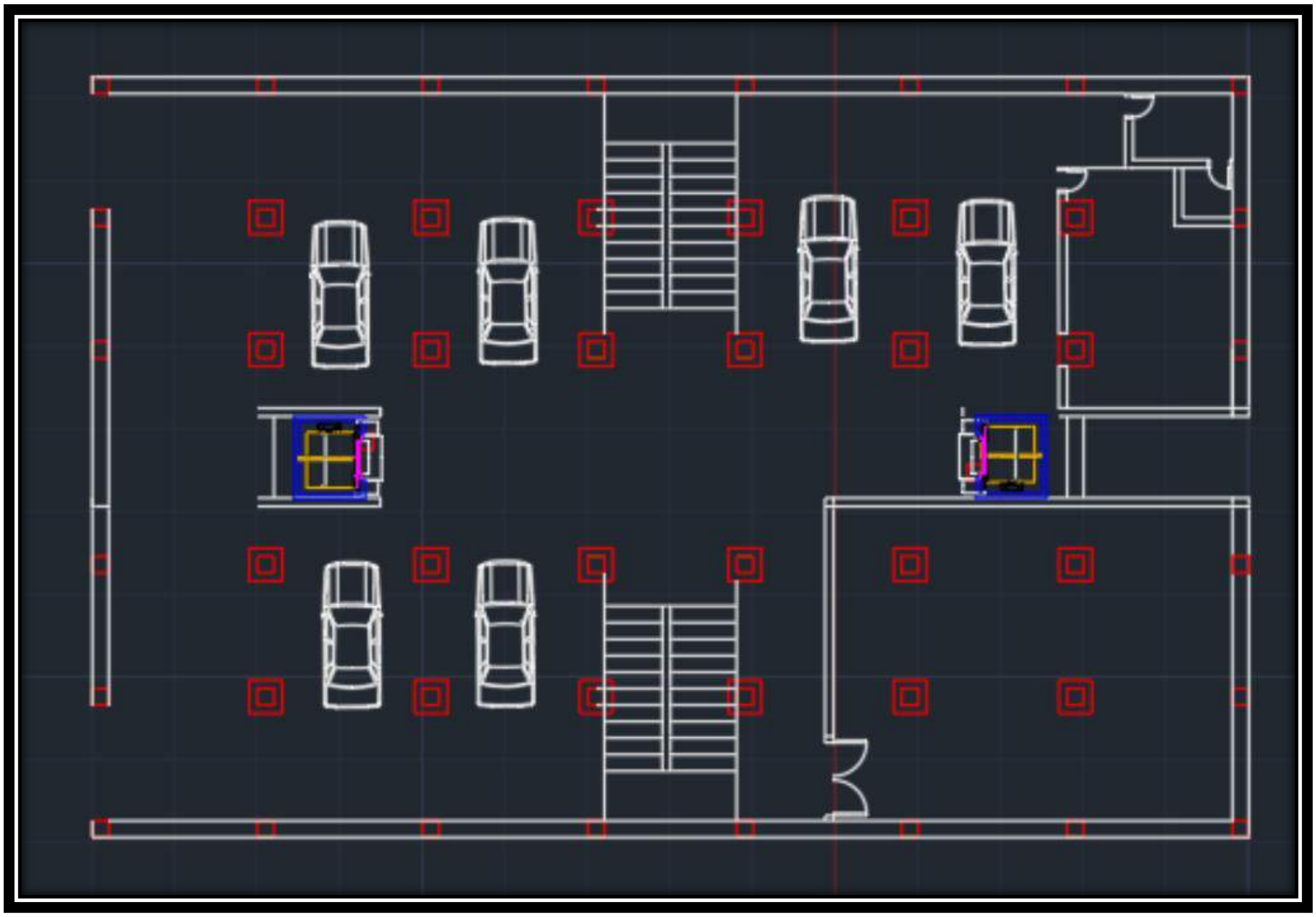
Basement Plan



Basement Plans:

- One Entry/Exit Gate for Vehicles.
- One staircase to Ground Floor.

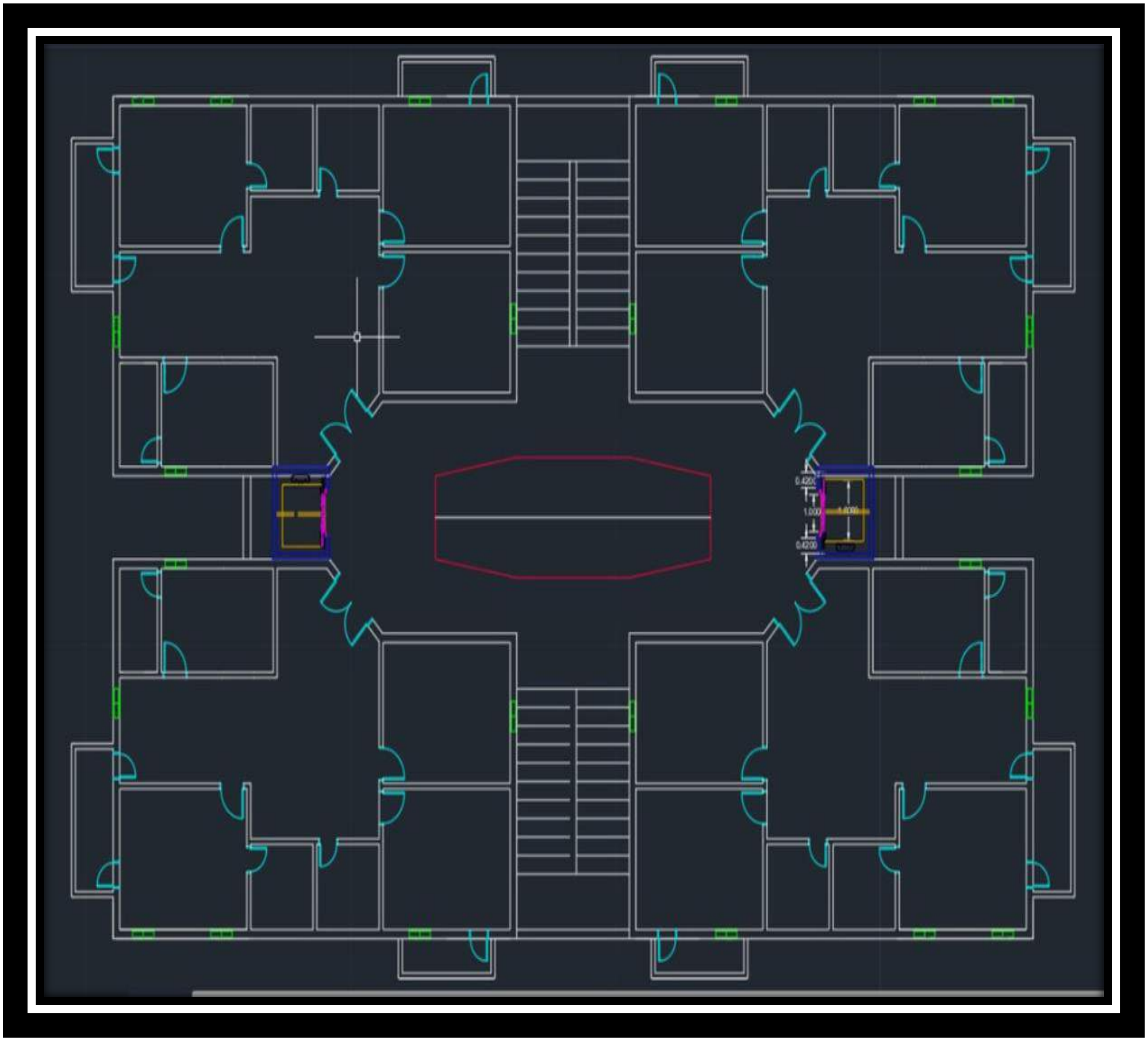
Ground Floor Plan



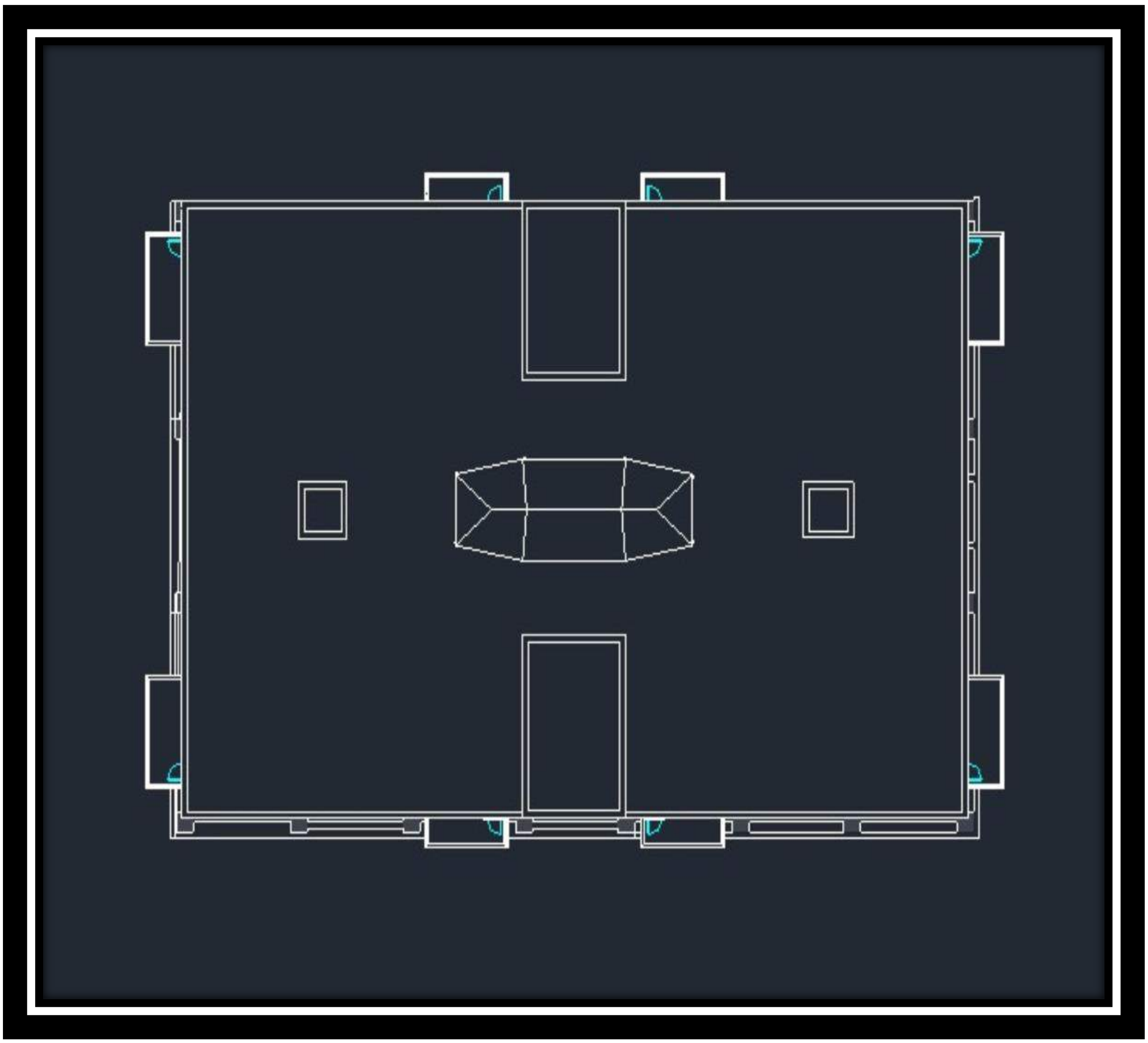
Ground Floor Plan:

- ❖ Two Entrance Gates
- ❖ Two Entry/Exit Gate for Vehicles .
- ❖ Two staircase and two elevators are there .
- ❖ One Guard Office.

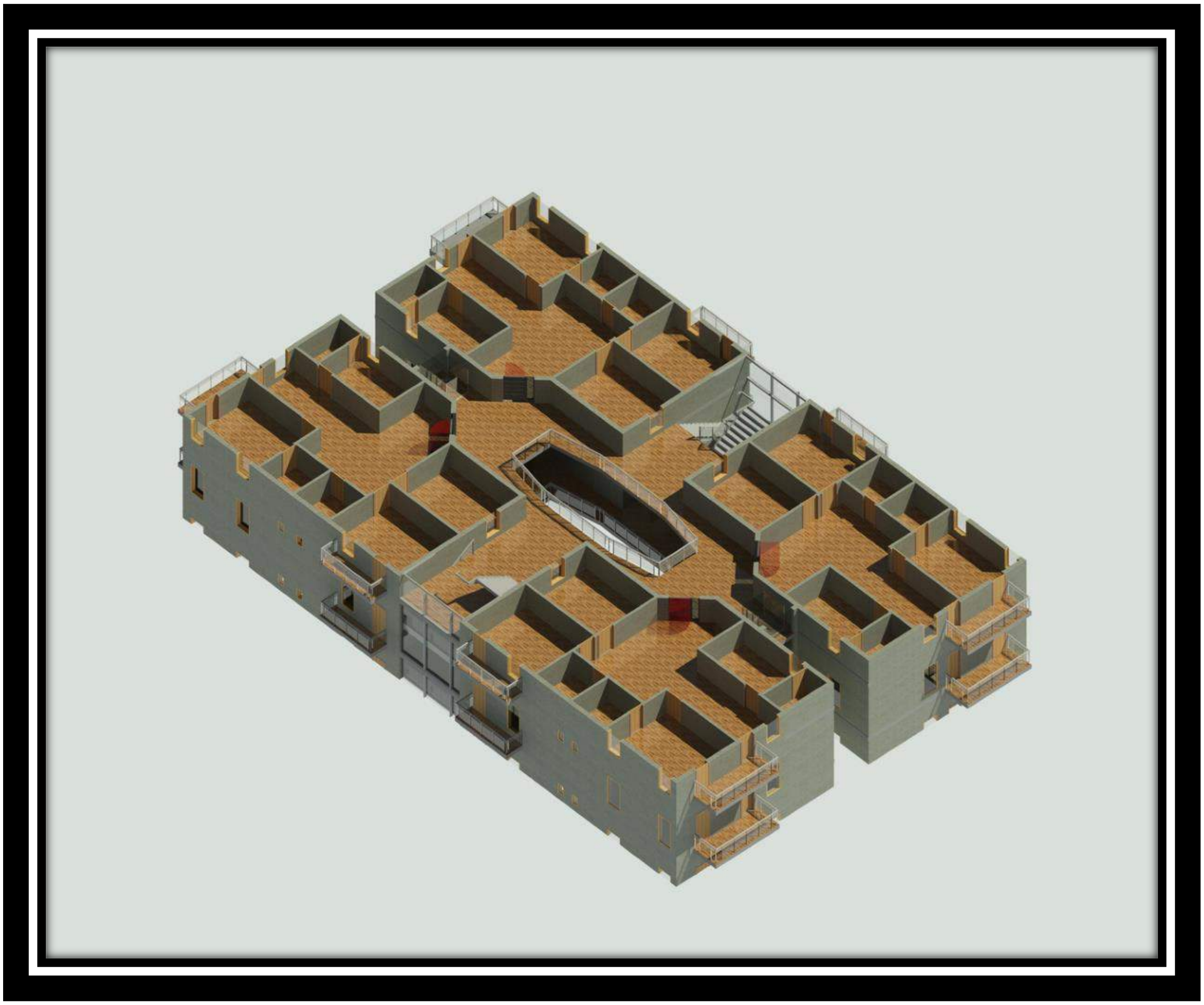
Typical Floor Plan



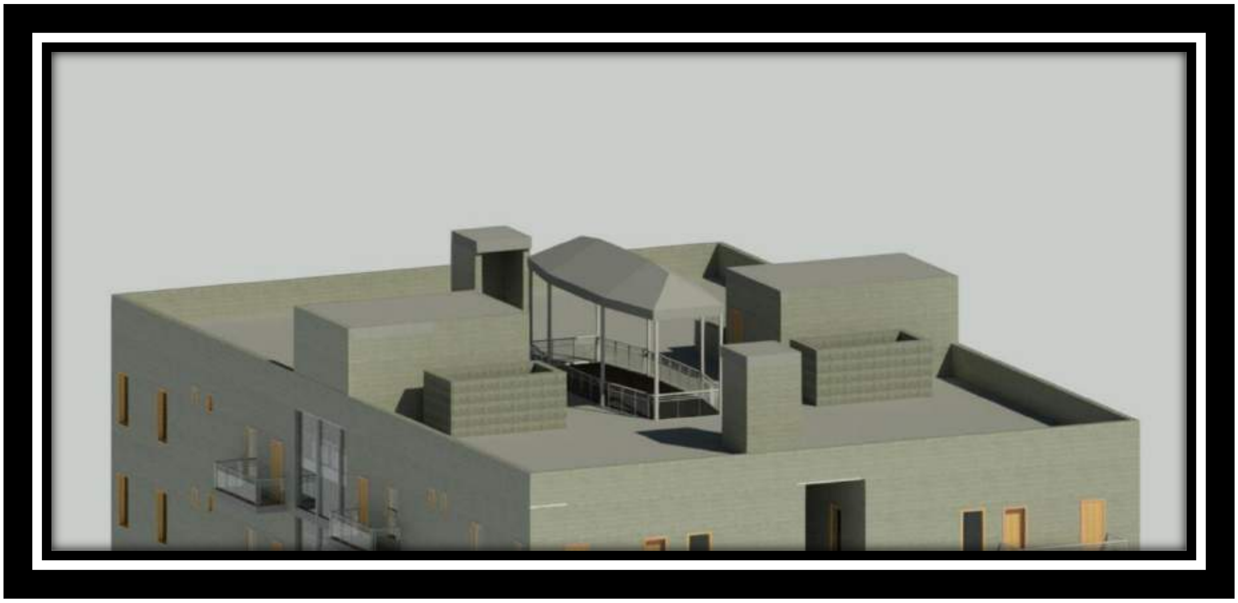
Roof Plan



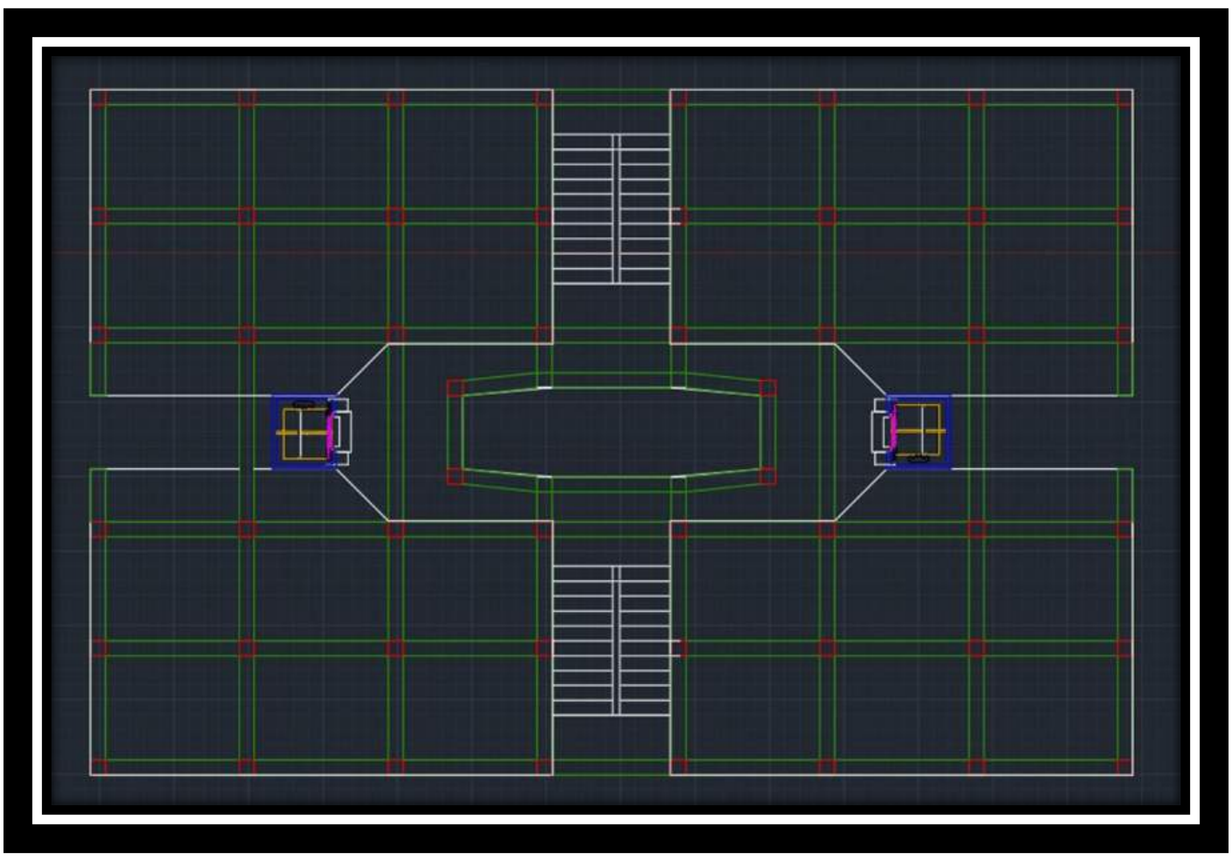
Typical 3D Floor Plan



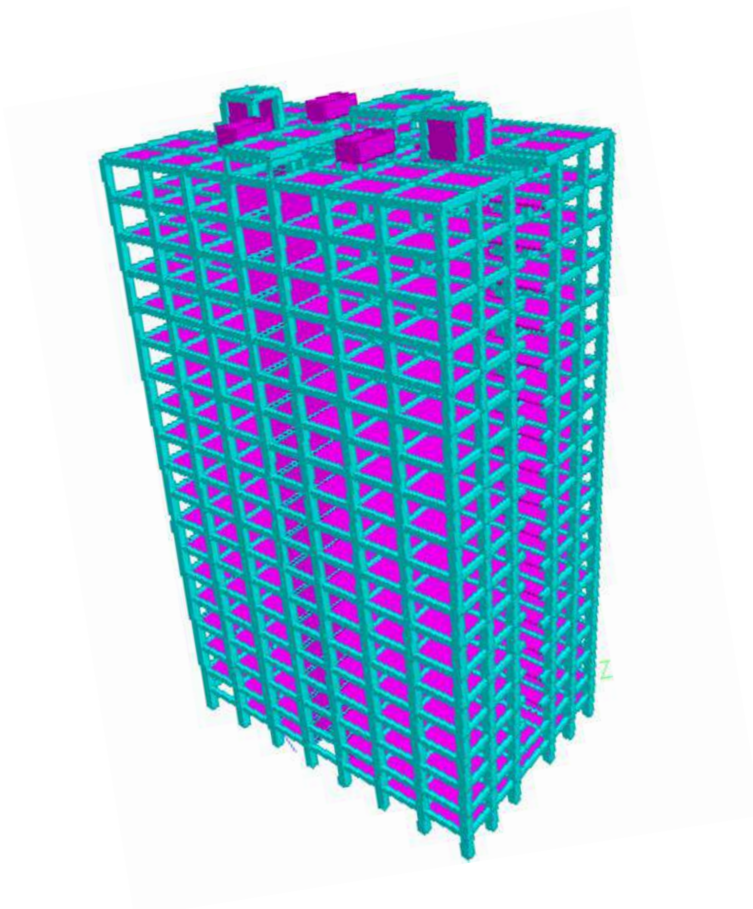
Typical 3D Roof Plan

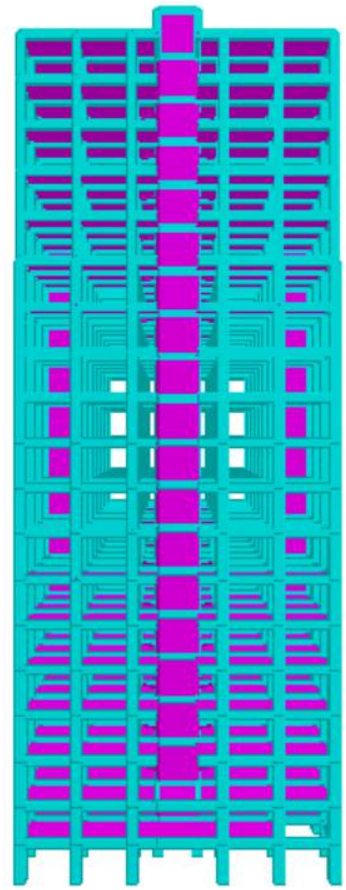


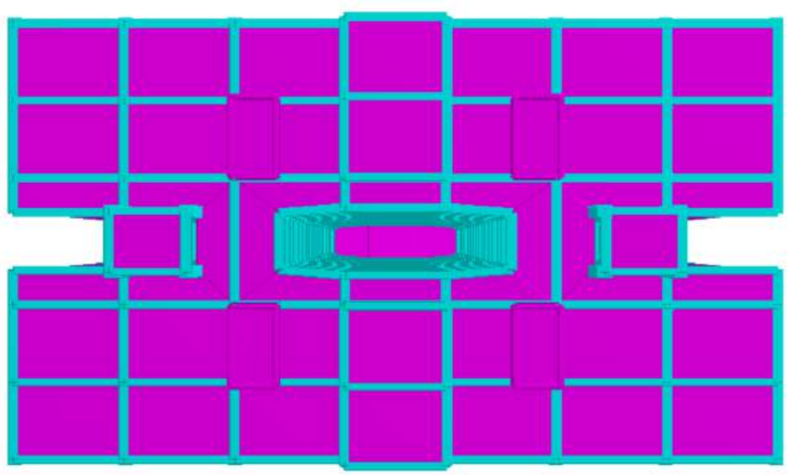
<u>***Beam Column Layout***</u>



Structure

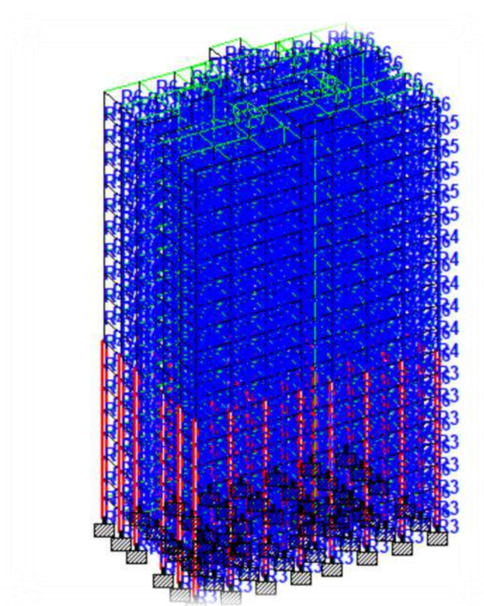




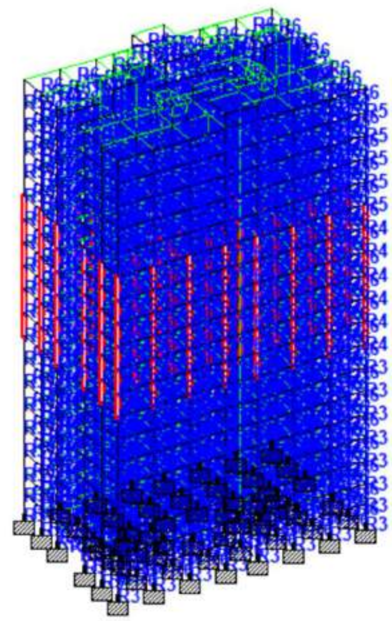


Column Grouping

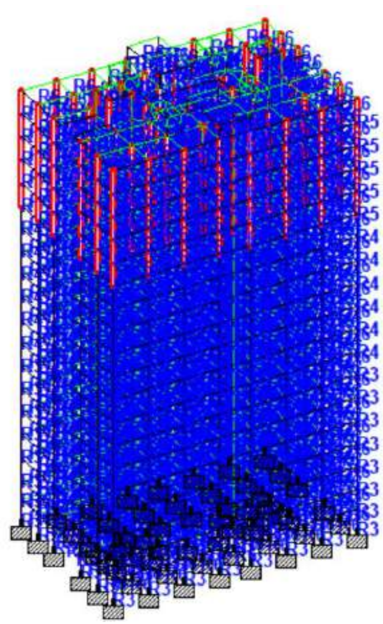
1) 800mm x 800mm



2) 700mm x 700mm

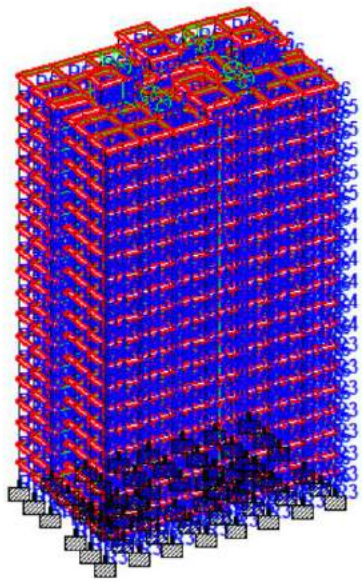


3) 600mm x 600mm



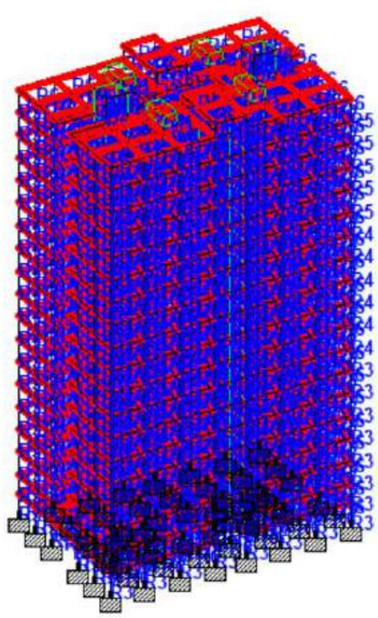
Beams

Size- 600mm x 400mm



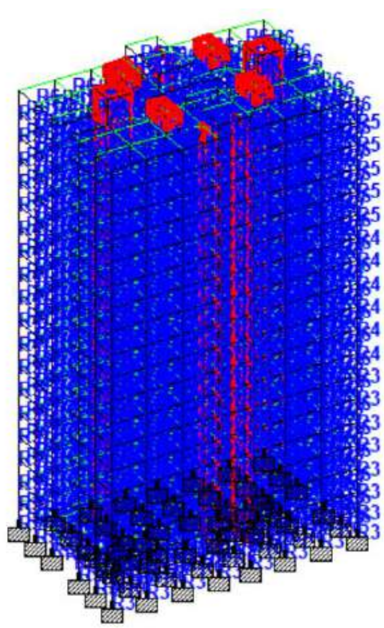
Slabs

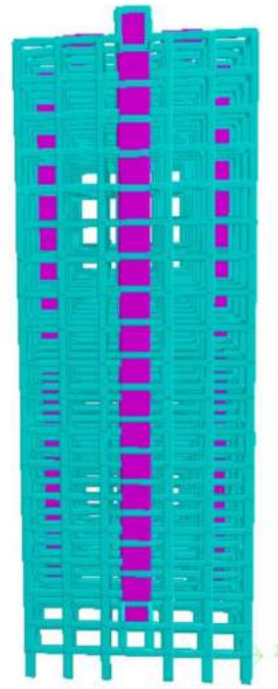
Thickness- 150mm



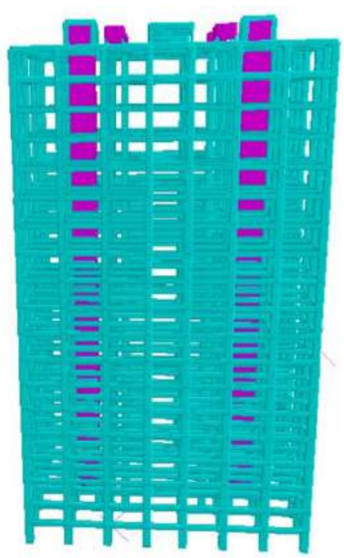
Shear Walls

Thickness- 200mm

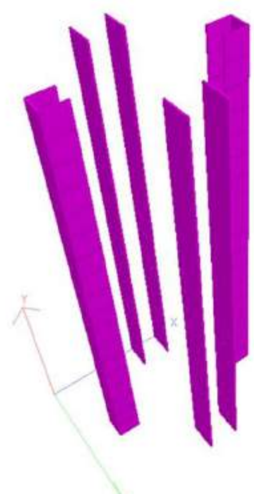




Front View

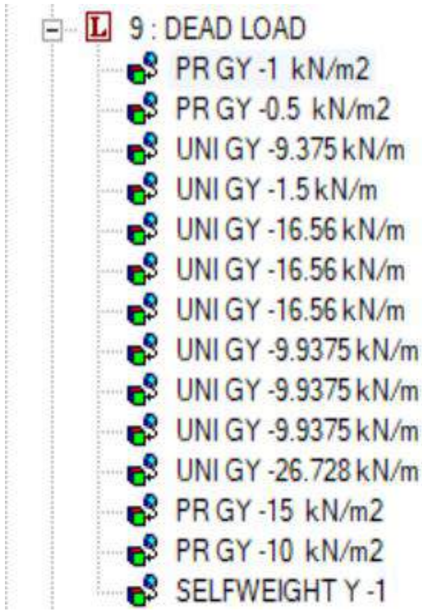


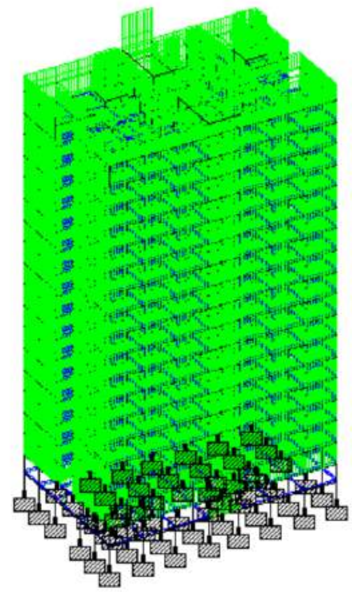
Side View



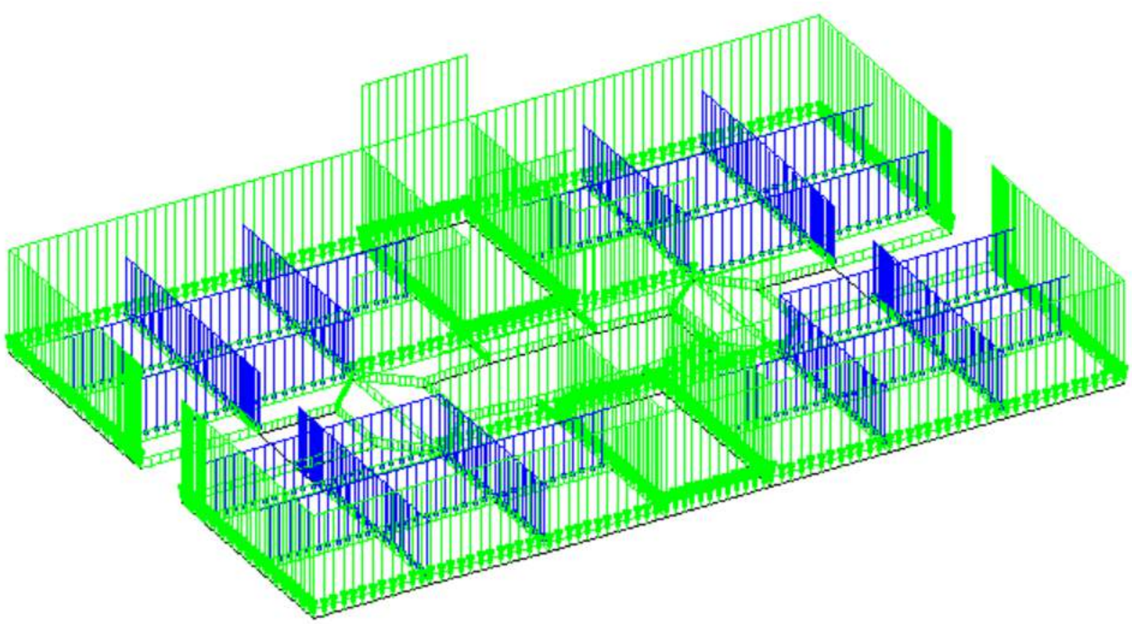
Loads And Load Combinations

Dead Load and Dead Load Profile





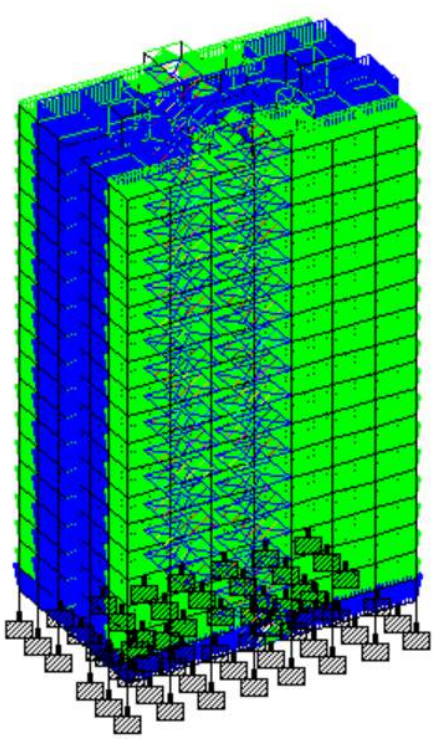
Dead Load on a typical Floor

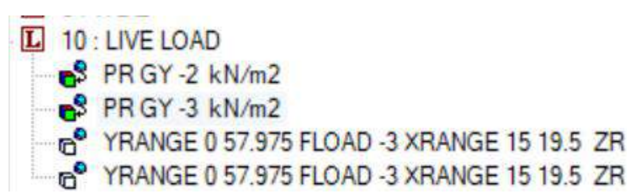


Live Load

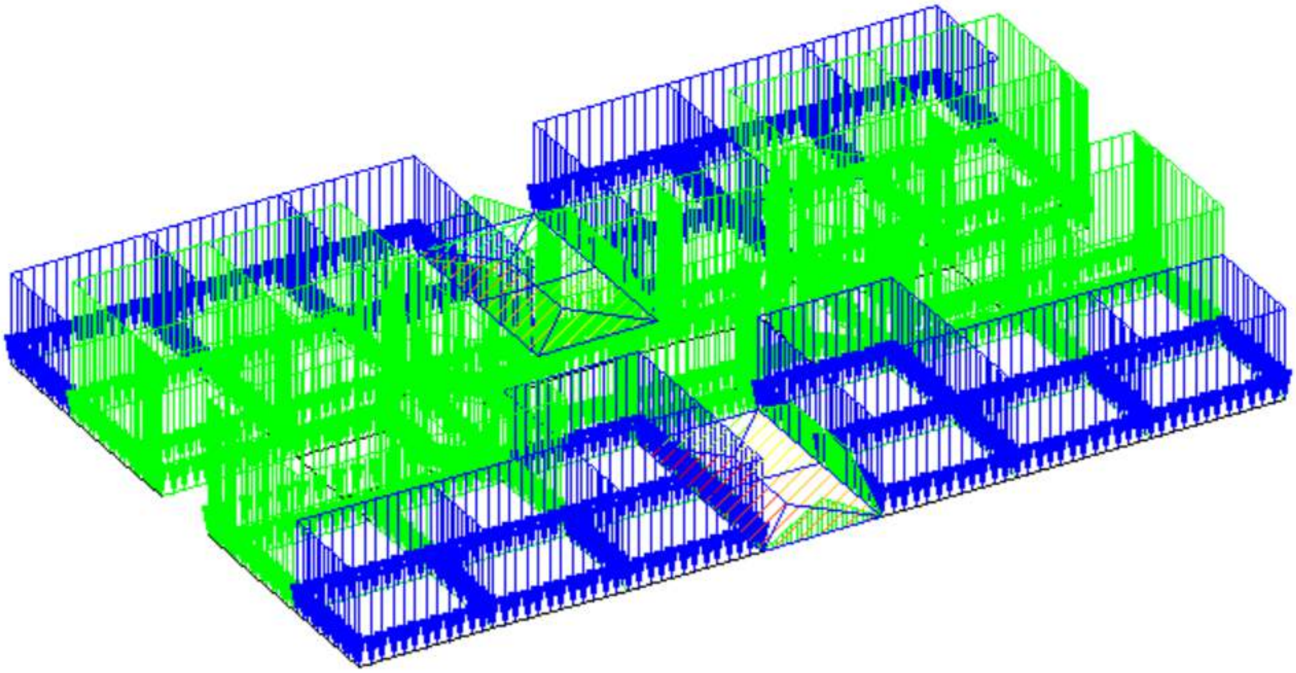
LIVE LOAD	VALUE
Bedroom	2 KN/m^2
Toilet and Bathroom	2 KN/m^2
Kitchen	2 KN/m^2
Dinning Cum Living Room	3 KN/m^2
Staircase	3 KN/m^2
Common Spaces	3 KN/m^2
Balcony	3 KN/m^2

Live Load Profile





Dead Load on a typical Floor



Wind Load And Design Forces

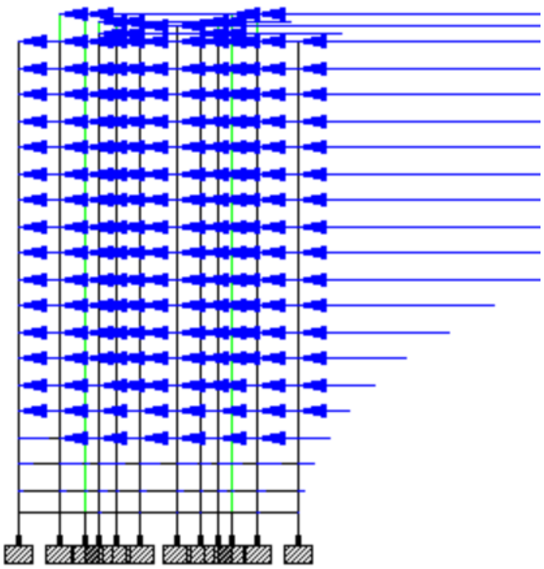
Wind Data	*Value*	*Reference*
Wind Zone	Zone IV	Refer IS:875, pt 3, Sec 5.2
Basic Wind Speed	50 m/s	Refer IS:875, pt 3, Sec 5.2
Terrain Category	Category 4	Refer IS:875, pt 3, Sec 5.3.2.1

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
Design Wind Speed Vz=Vb*k1*k2*k3	50*k2 m/s	Refer IS:875, pt 3, Sec 5.3
Pz=0.6(Vz)2	1500 *$(k2)^2$ N/m2	Refer IS:875, pt 3, Sec 5.4

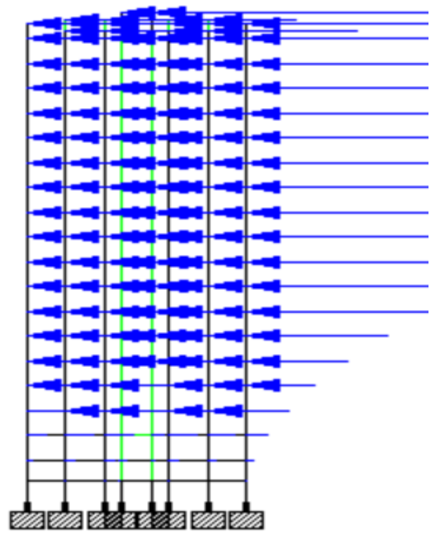
VARIATION OF Pz WITH HEIGHT

Height(m)	*k2*	*Vz (m/s)*	*Pz (KN/m2)*
10	0.80	40	0.96
20	0.80	40	0.96
30	0.97	48.5	1.411
40	1.035	51.75	1.606
50	1.11	55	1.815
60	1.12	56	1.881
65	1.13	56.5	1.915

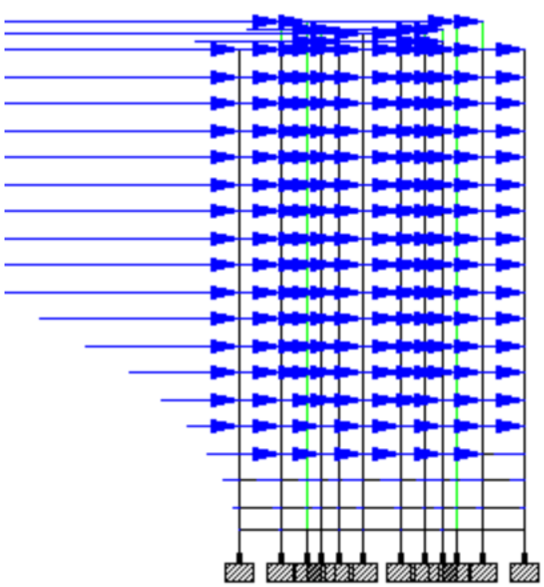
WL X



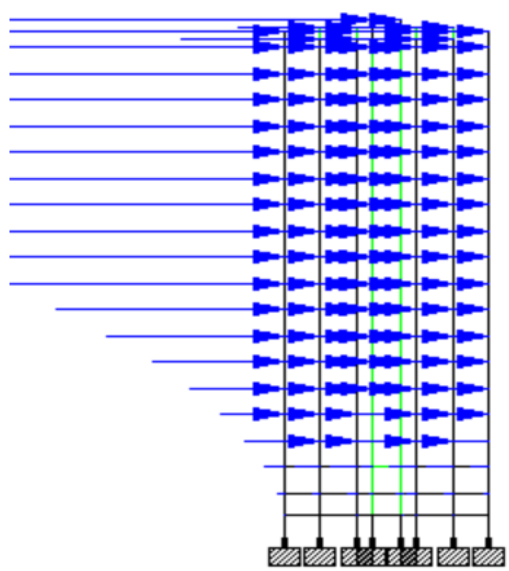
WL Z



WL –X

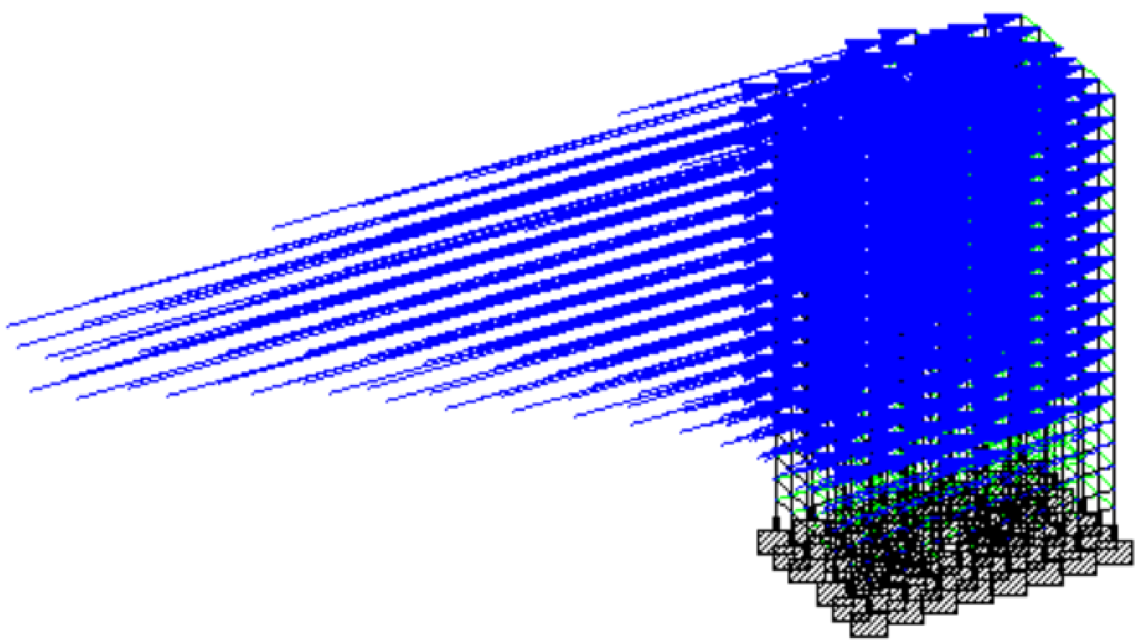


WL -Z



Seismic Load

Seismic Parameter	Value	Reference
Seismic Zone	V	Refer IS 1893: table 2
Zone Factor	0.36	Refer IS 1893: table 2
Importance Factor	1	Refer IS 1893: table 2
Response Reduction Factor	5	Refer IS 1893: table 7
Soil Type	Medium	Refer IS 1893: table 7
Structure Type	RC Frame Building	Refer IS 1893: table 7



EQ X

Load Combinations

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LOAD COMB 11 GENERATED INDIAN CODE GENRAL_STRUCTURES 1
9 1.5 10 1.5
LOAD COMB 12 GENERATED INDIAN CODE GENRAL_STRUCTURES 2
9 1.2 10 1.2 5 1.2
LOAD COMB 13 GENERATED INDIAN CODE GENRAL_STRUCTURES 3
9 1.2 10 1.2 6 1.2
LOAD COMB 14 GENERATED INDIAN CODE GENRAL_STRUCTURES 4
9 1.2 10 1.2 7 1.2
LOAD COMB 15 GENERATED INDIAN CODE GENRAL_STRUCTURES 5
9 1.2 10 1.2 8 1.2
LOAD COMB 16 GENERATED INDIAN CODE GENRAL_STRUCTURES 6
9 1.2 10 1.2 5 -1.2
LOAD COMB 17 GENERATED INDIAN CODE GENRAL_STRUCTURES 7
9 1.2 10 1.2 6 -1.2
LOAD COMB 18 GENERATED INDIAN CODE GENRAL_STRUCTURES 8
9 1.2 10 1.2 7 -1.2
LOAD COMB 19 GENERATED INDIAN CODE GENRAL_STRUCTURES 9
9 1.2 10 1.2 8 -1.2
LOAD COMB 20 GENERATED INDIAN CODE GENRAL_STRUCTURES 10
9 1.2 10 1.2 1 1.2
LOAD COMB 21 GENERATED INDIAN CODE GENRAL_STRUCTURES 11
9 1.2 10 1.2 2 1.2
LOAD COMB 22 GENERATED INDIAN CODE GENRAL_STRUCTURES 12
9 1.2 10 1.2 3 1.2
LOAD COMB 23 GENERATED INDIAN CODE GENRAL_STRUCTURES 13
9 1.2 10 1.2 4 1.2
LOAD COMB 24 GENERATED INDIAN CODE GENRAL_STRUCTURES 14
9 1.2 10 1.2 1 -1.2
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LOAD COMB 25 GENERATED INDIAN CODE GENRAL_STRUCTURES 15
9 1.2 10 1.2 2 -1.2
LOAD COMB 26 GENERATED INDIAN CODE GENRAL_STRUCTURES 16
9 1.2 10 1.2 3 -1.2
LOAD COMB 27 GENERATED INDIAN CODE GENRAL_STRUCTURES 17
9 1.2 10 1.2 4 -1.2
LOAD COMB 28 GENERATED INDIAN CODE GENRAL_STRUCTURES 18
9 1.5 5 1.5
LOAD COMB 29 GENERATED INDIAN CODE GENRAL_STRUCTURES 19
9 1.5 6 1.5
LOAD COMB 30 GENERATED INDIAN CODE GENRAL_STRUCTURES 20
9 1.5 7 1.5
LOAD COMB 31 GENERATED INDIAN CODE GENRAL_STRUCTURES 21
9 1.5 8 1.5
LOAD COMB 32 GENERATED INDIAN CODE GENRAL_STRUCTURES 22
9 1.5 5 -1.5
LOAD COMB 33 GENERATED INDIAN CODE GENRAL_STRUCTURES 23
9 1.5 6 -1.5
LOAD COMB 34 GENERATED INDIAN CODE GENRAL_STRUCTURES 24
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LOAD COMB 37 GENERATED INDIAN CODE GENRAL_STRUCTURES 27
9 1.5 2 1.5
LOAD COMB 38 GENERATED INDIAN CODE GENRAL_STRUCTURES 28
9 1.5 3 1.5
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LOAD COMB 39 GENERATED INDIAN CODE GENRAL_STRUCTURES 29
9 1.5 4 1.5
LOAD COMB 40 GENERATED INDIAN CODE GENRAL_STRUCTURES 30
9 1.5 1 -1.5
LOAD COMB 41 GENERATED INDIAN CODE GENRAL_STRUCTURES 31
9 1.5 2 -1.5
LOAD COMB 42 GENERATED INDIAN CODE GENRAL_STRUCTURES 32
9 1.5 3 -1.5
LOAD COMB 43 GENERATED INDIAN CODE GENRAL_STRUCTURES 33
9 1.5 4 -1.5
LOAD COMB 44 GENERATED INDIAN CODE GENRAL_STRUCTURES 34
9 0.9 1 1.5
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9 0.9 2 1.5
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9 0.9 1 -1.5
LOAD COMB 49 GENERATED INDIAN CODE GENRAL_STRUCTURES 39
9 0.9 2 -1.5
LOAD COMB 50 GENERATED INDIAN CODE GENRAL_STRUCTURES 40
9 0.9 3 -1.5
LOAD COMB 51 GENERATED INDIAN CODE GENRAL_STRUCTURES 41
9 0.9 4 -1.5
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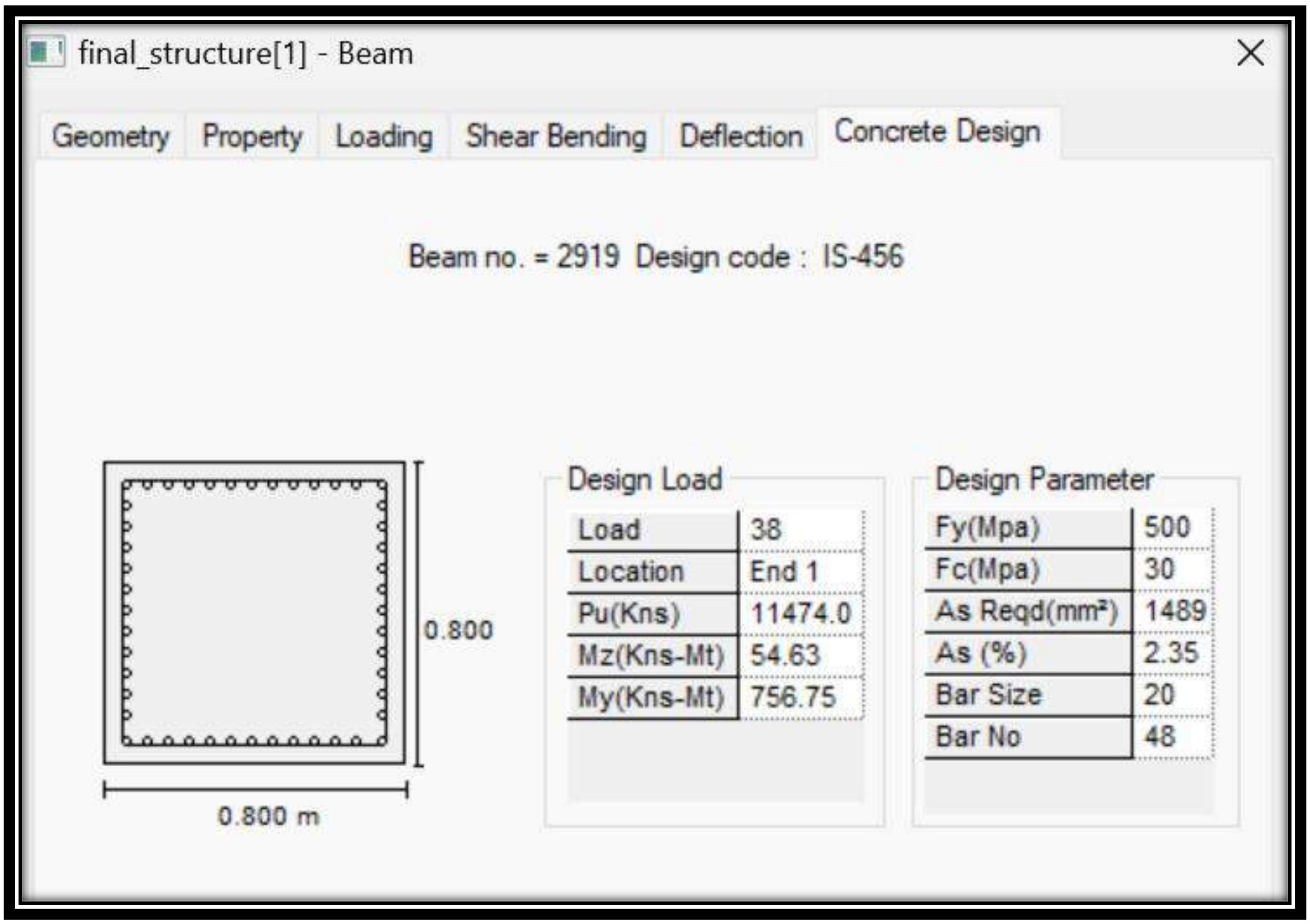
Column Design From Staad

For Column No 2919



As per Staad Pro,

For Column 2919 with dimensions 800mm x 800mm

- Pu= 11474 KN
- Mz= 54.63 KN
- My= 756.75 Kn
- As(%)= 2.35%

Column Manual Calculation

Column Number: 2919

$P_u = 11474\ kN$ $\quad$ $L = 2.7\ m$

$M_z = 54.63\ kNm$ $\quad$ $B = 800\ mm$

$M_y = 756.75\ kNm$ $\quad$ $D = 800\ mm$

$$e_{min.\ z-z} = max\left(\left(\frac{L}{500} + \frac{D}{30}\right), 30\ mm\right) = 32.06\ mm$$
$$e_{min.\ y-y} = max\left(\left(\frac{L}{500} + \frac{B}{30}\right), 30\ mm\right) = 32.06\ mm$$

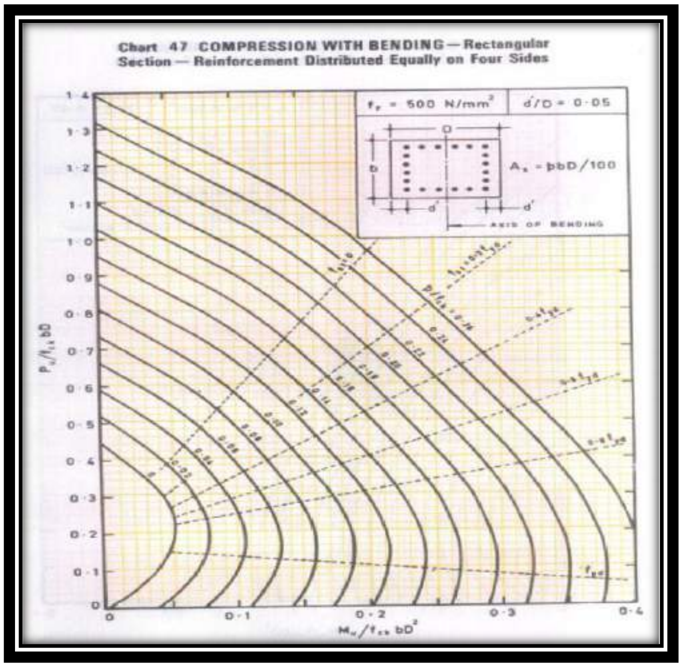
$$M_{u\ min.\ z-z} = 11474\ kN \times \frac{32.06}{1000}\ m = 367.856\ kNm$$
$$M_{u\ min.\ y-y} = 11474\ kN \times \frac{32.06}{1000}\ m = 367.856\ kNm$$

Here, $M_y > M_{u\ min.\ y-y}$ & $M_z < M_{u\ min.\ z-z}$

Hence, the column is subjected to axial load and uniaxial bending moment.

Taking $\phi = 36\ mm$ and clear cover $20\ mm$

$$d' = \frac{\phi}{2} + clear\ cover$$
$$\implies d' = \frac{36}{2} + 20 = 38\ mm$$
$$\frac{d'}{D} = \frac{38}{800} \simeq 0.05$$



$$\frac{P_u}{f_{ck}BD} = \frac{11474 \times 10^3}{30 \times 800 \times 800} = 0.6$$

$$\frac{M_u}{f_{ck}BD^2} = \frac{756.75 \times 10^6}{30 \times 800 \times 800^2} = 0.05$$

From graph,

$$\frac{\%\,P}{f_{ck}} = 0.06 \Longrightarrow \%\,P = 1.8$$

$$A_s = \frac{0.06 \times 30 \times 800 \times 800}{100} \Longrightarrow A_s = 11520\ mm^2$$

$$n \times \frac{\pi}{4} \times 36^2 = 11520 \Longrightarrow n \simeq 12$$

Size of Tie bars

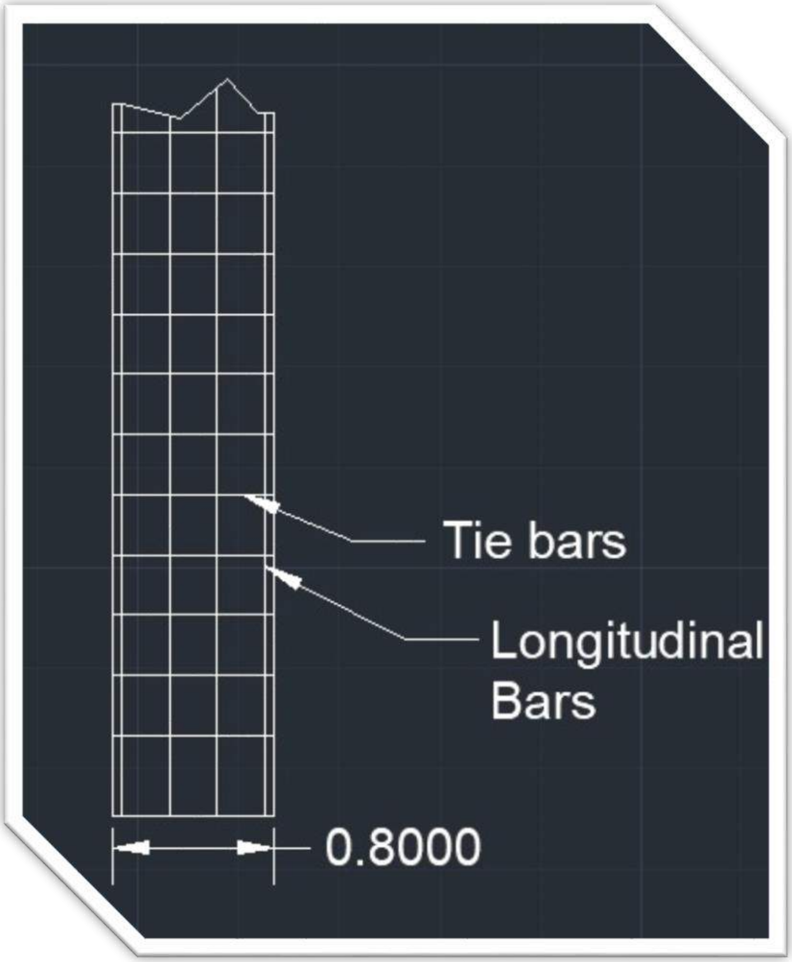
$$\phi \not< max\left(\frac{\phi_{main}}{4}, 6\ mm\right)$$

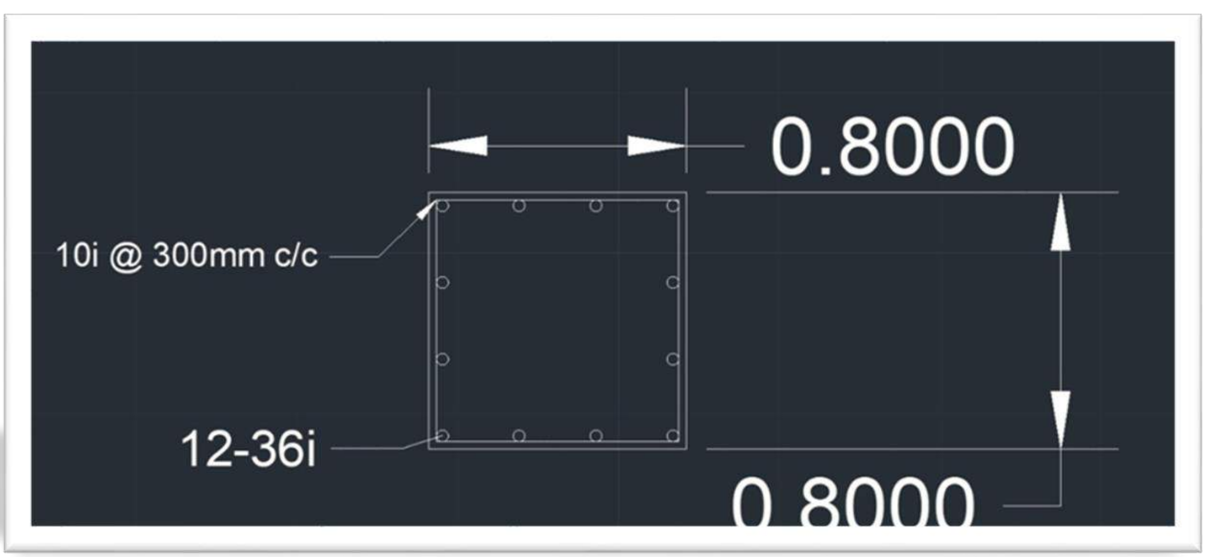
$$\phi \not< max(9\ mm, 6\ mm)$$

Take $\phi = 10\ mm$

$$Spacing \not> max(LLD, 16\ \phi_{main}, 300\ mm) = 300\ mm$$

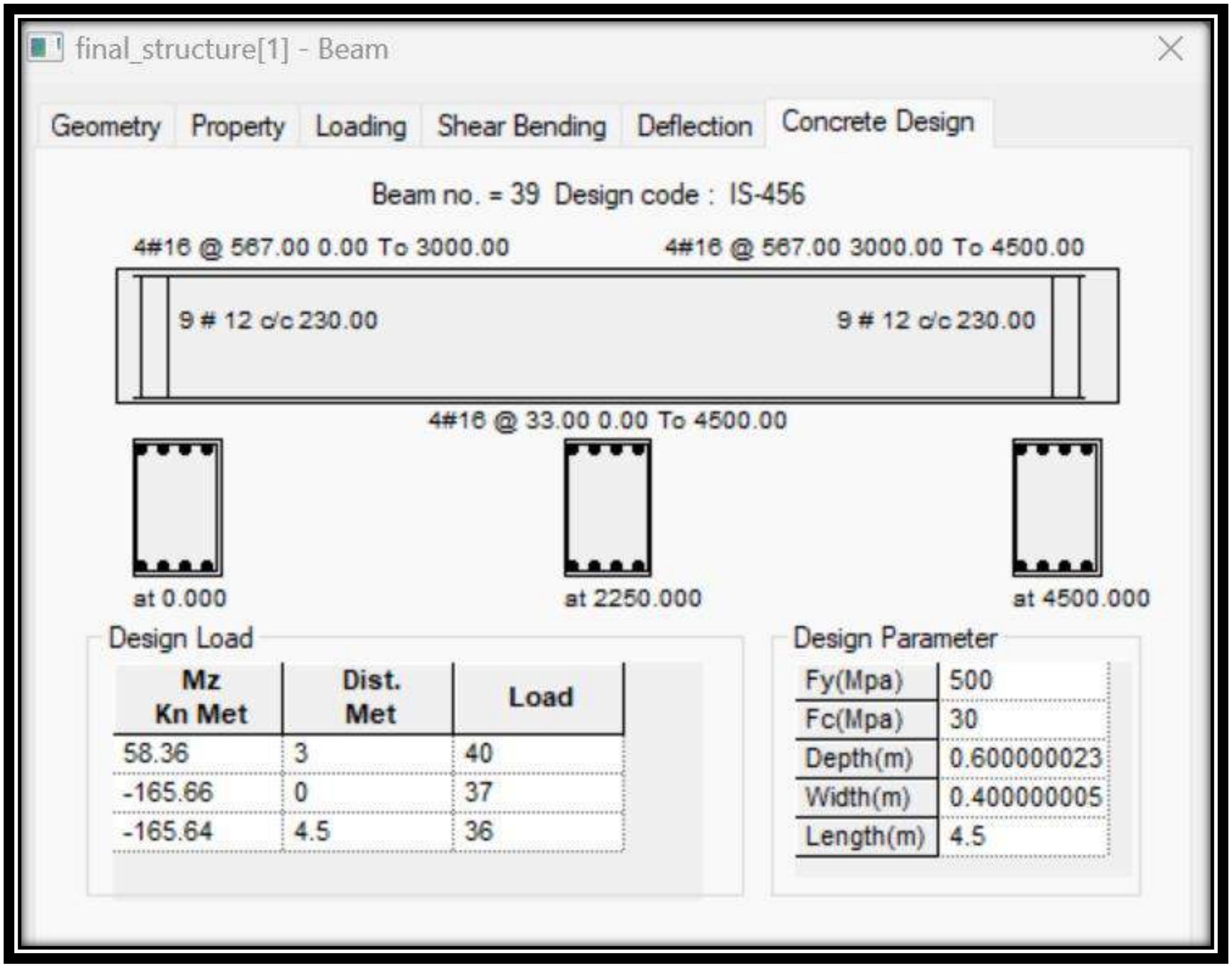
Column Detailing in Auto Cad





Beam Design From Staad

For Beam No 39



As per Staad Pro,

For Beam 39 with dimensions 600mm x 400mm

- Fu= 147.534 KN
- Mu= 165.659 KN

Beam Manual Calculation

Beam Number: 39

$L = 4.5\ m$ $\quad F = 147.534\ kN$ $\quad d = D - 50 = 5$

$B = 400\ mm$ $\quad M = 165.659\ kNm$

$D = 600\ mm$

$$M_{u_{lim}} = Q\, f_{ck}\, Bd^2$$
$$= 0.133 \times 30 \times 400 \times 500^2$$
$$= 482.8\ kNm$$
$$M_u < M_{u_{lim}}$$

$$P_t = 50\frac{f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6M_u}{f_{ck}Bd^2}}\right] = 50\times\frac{30}{500}\left[1-\sqrt{1-\frac{4.6\times147.534\times10^6}{30\times400\times550^2}}\right] = 0.29\ \%$$

Provide - 16 mm bars

$$\frac{0.3}{100}\times400\times550 = n\times\frac{\pi}{4}\times16^2$$
$$\Longrightarrow n \simeq 4$$

$\therefore$ Provide 4 - 16 mm bars

$$\tau_v = \frac{V_u}{Bd} = 147.534\times10^3 = 0.67$$
$$\tau_{c_{max}} = 0.63\sqrt{30} = 3.45$$
$$f_{ck} = 30\ N/mm^3,\ P_t = 0.3\ \%$$
$$So,\ \tau_c = 0.246$$
$$\tau_v > \tau_c$$
$$\Longrightarrow V_u = V_{cc} + V_{us} = 147.534\times10^3 - 0.246\times300\times550$$
$$\Longrightarrow V_{us} = 106.944\ kN$$

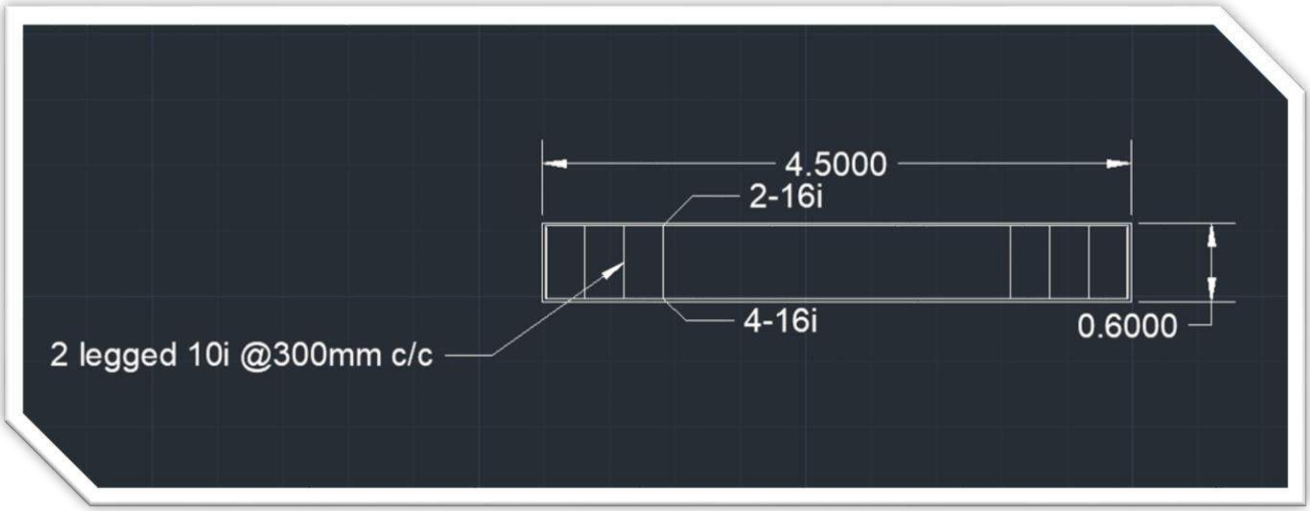
Assume 2-legged 10mm diameter bars

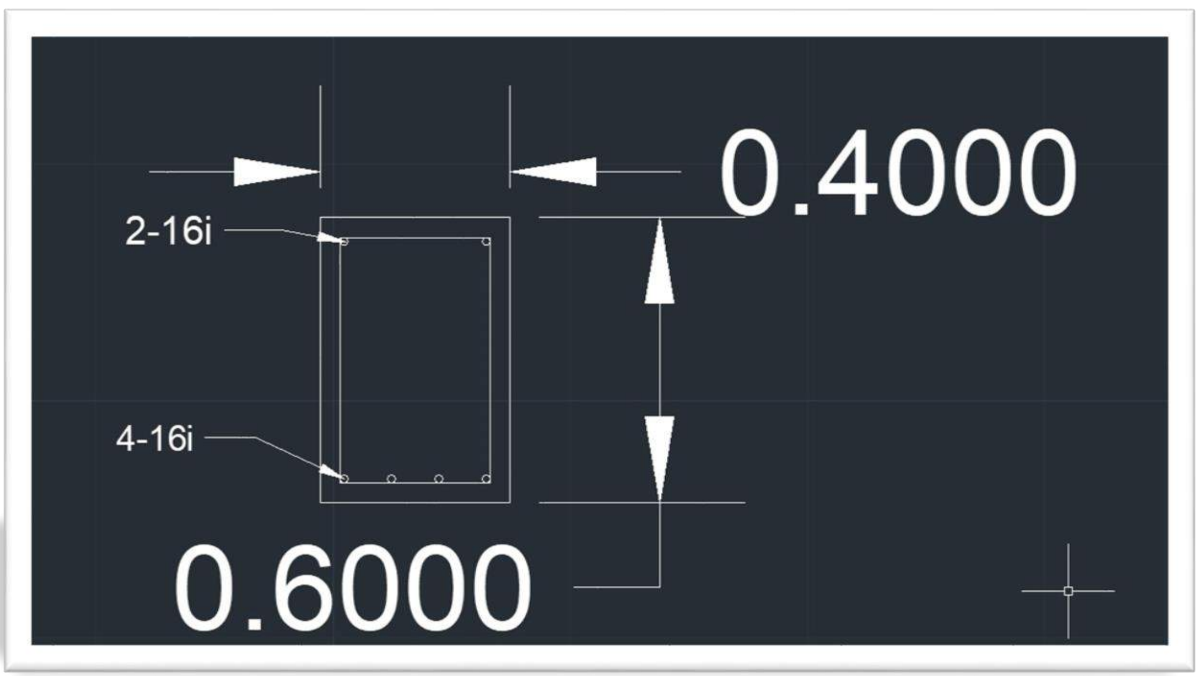
$$A_{sv} = 2\times\frac{\pi}{4}\times10^2 = 157.08mm^2 \text{ (Assumed)}$$
$$S_v = \frac{0.87f_yA_{sv}d}{V_{us}} = \frac{0.87\times500\times157.08\times550}{106.944\times10^3} = 351.411\ mm$$

Maximum $S_v = min(0.75\ d, 300\ mm) = 300\ mm$

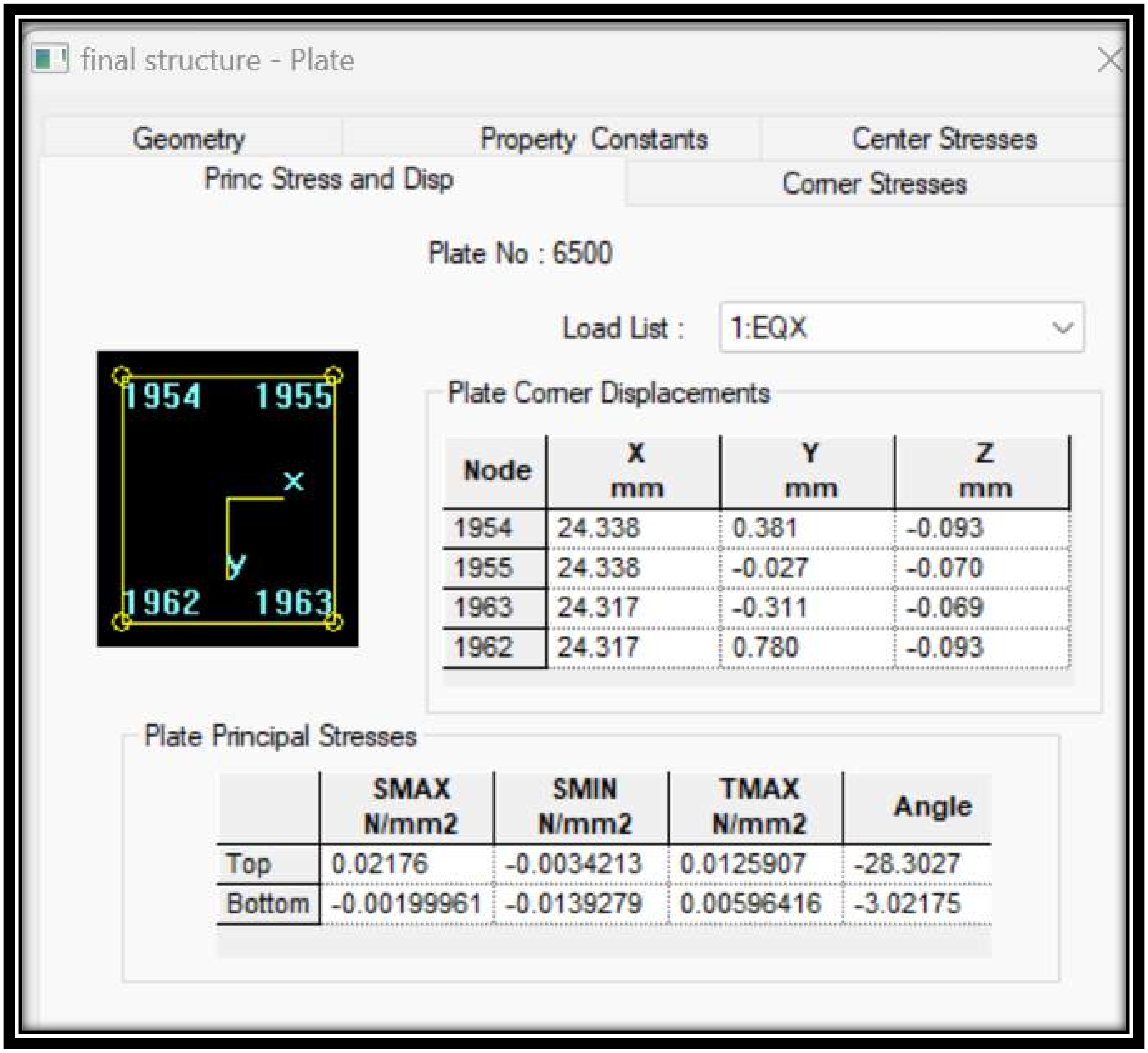
So, provide 300 mm spacing.

Beam Detailing in Auto Cad





Slab Design From Staad



Node	X mm	Y mm	Z mm
1954	24.338	0.381	-0.093
1955	24.338	-0.027	-0.070
1963	24.317	-0.311	-0.069
1962	24.317	0.780	-0.093

	SMAX N/mm2	SMIN N/mm2	TMAX N/mm2	Angle
Top	0.02176	-0.0034213	0.0125907	-28.3027
Bottom	-0.00199961	-0.0139279	0.00596416	-3.02175

Loads on this Slab are

- ❖ Live Load = 3 KN/m2
- ❖ Floor Finishing = 1 KN/m2

Slab Manual Calculation

Raw dimensions of slab = $5\ m\ \times 4\ m$
Factored Load, $w = 1.5 \times (1+3) = 6\ kN$

$\frac{span}{depth} = 40$ (as per IS 456-2000)
Depth of slab $D = 150\ mm$
Assuming a 10 mm diameter bar and 15 mm clear cover
Effective depth $d = 150 - 15 - \frac{10}{2} = 130\ mm$
Support thickness $= 250\ mm$
$\frac{L_y}{L_x} = \frac{5}{4} = 1.25$
Hence, it is a two way slab.

Effective span for,
$l_x = min(clear\ span + depth,\ clear\ span + c/c\ support) = 4130\ mm$
$l_y = min(clear\ span + depth,\ clear\ span + c/c\ support) = 5130\ mm$

Moment calculation
From table 26 of 15456-2000
$\alpha_x = 0.088 \quad \alpha_y = 0.036$
$M_x = \alpha_x \cdot w \cdot l_x^2 = 0.088 \times 6 \times 4.13^2 = 9.006\ kNm$
$M_y = \alpha_y \cdot w \cdot l_y^2 = 0.036 \times 6 \times 4.13^2 = 3.684\ kNm$

Limiting Moment $= 0.138 f_{ck} b d^2 = 0.138 \times 30 \times 1000 \times d^2$
$\implies d = 46.64\ mm$ (required) $< 150\ mm$ (provided)

Along L_x,

$$P_t = 50\frac{f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6M_u}{f_{ck}Bd^2}}\right] = 50\times\frac{30}{500}\left[1-\sqrt{1-\frac{4.6\times 9.006\times 10^6}{30\times 1000\times 150^2}}\right] = 0.093\ \% \simeq 0.09\ \%$$

$$A_{st} = \frac{P_t}{100}\cdot bd = \frac{0.09}{100}\times 1000\times 150 = 139.5\ mm^2$$

For $10\ mm$ bars $A_{st} = 78.53\ mm^2$

Spacing $= \dfrac{78.53}{139.5}\times 1000 = 562$

Provide $500\ mm$ spacing.

Along L_y,

$$P_t = 50\frac{f_{ck}}{f_y}\left[1-\sqrt{1-\frac{4.6M_u}{f_{ck}Bd^2}}\right] = 50\times\frac{30}{500}\left[1-\sqrt{1-\frac{4.6\times 3.684\times 10^6}{30\times 1000\times 150^2}}\right] = 0.038\ \% \simeq 0.04\ \%$$

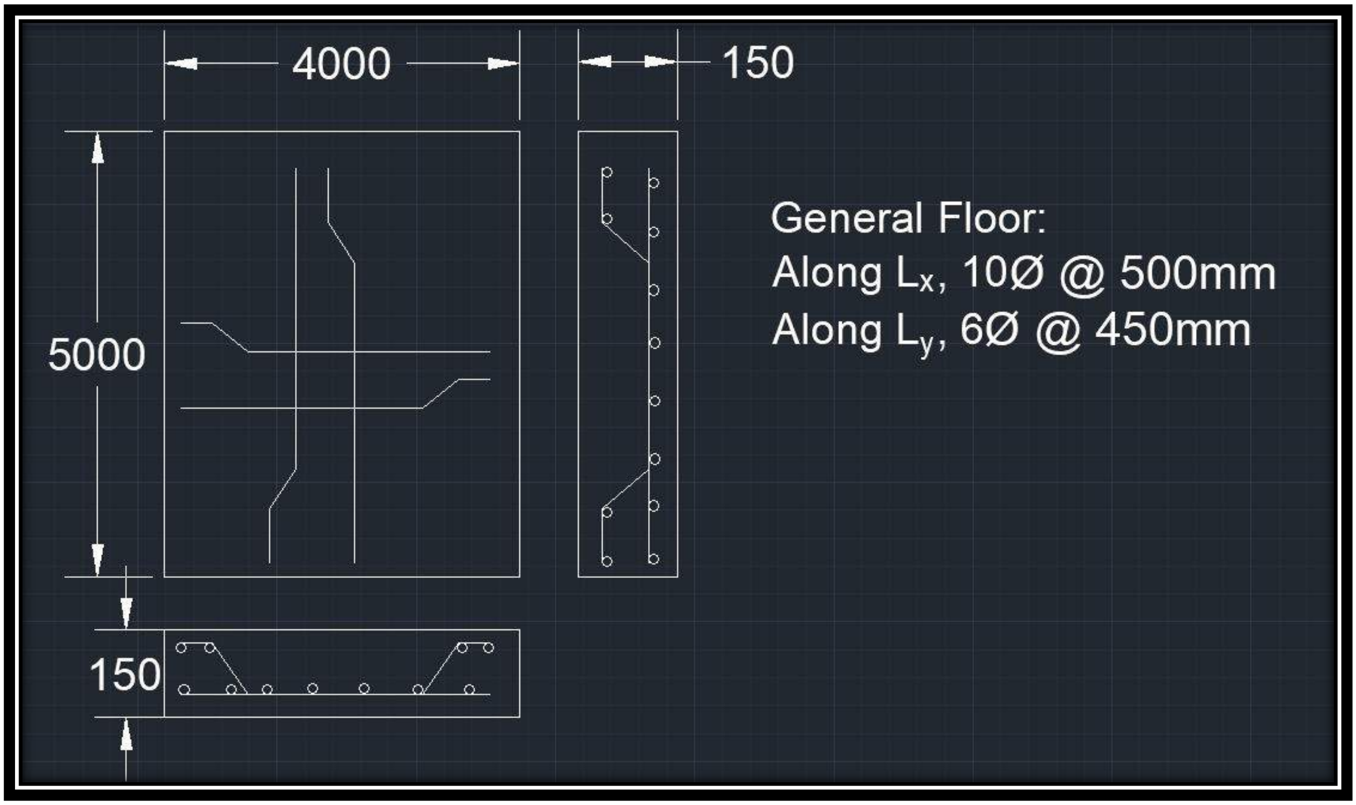
$$A_{st} = \frac{P_t}{100}\cdot bd = \frac{0.04}{100}\times 1000\times 150 = 60\ mm^2$$

For $6\ mm$ bars $A_{st} = 28.27\ mm^2$

Spacing $= \dfrac{28.27}{60}\times 1000 = 471$

Provide $450\ mm$ spacing.

Slab Detailing in Auto Cad



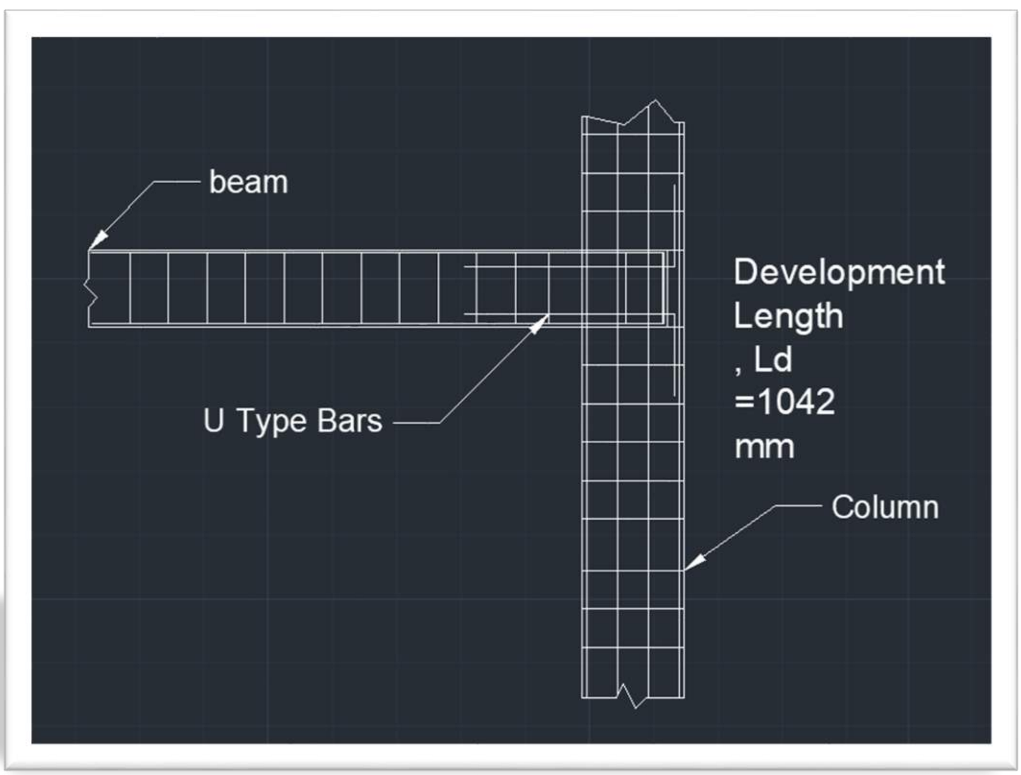
Development Length Manual Calculation

$$V_u = F_y = 147.534\, kN$$
$$\tau_{bd} = \frac{V_u}{n\pi\phi \cdot LA} \quad \{LA = d \cdot j\} \quad j = 0.8$$
$$\implies \tau_{bd} = \frac{147.534 \times 10^3}{4 \times \pi \times 16 \times 550 \times 0.8}$$
$$\implies \tau_{bd} = 1.67\, MPa$$

$$\tau_{bd}\,(permissible) = 1.6 \times 1.5 = 2.4\, MPa$$
$$\therefore \tau_{bd} < \tau_{bd}\,(permissible)$$

$$L_d = \frac{0.87 f_y \phi}{4\tau_{bd}} = \frac{0.87 \times 500 \times 16}{4 \times 1.67}$$
$$\implies L_d = Development\ length \simeq 1042\, mm$$

Beam Column Junction in Auto Cad



Pile Foundation Design in Staad

PILE CAP DESIGN

Design For PileCap P1185

PILE ARRANGEMENT

Column Dimensions

Column Shape :	Rectangular
Column Length - X (Pl) :	0.800 m
Column Width - Z (Pw) :	0.800 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 $\mathbf{P_{CL}}$ =	4.600 m
Pile Cap Width $\mathbf{P_{CW}}$ =	4.198 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 $\mathbf{e}$ =	0.050 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)
101	16.902	-6532.309	253.648	223.982	0.000	-15.563
102	13.522	-5225.847	202.919	179.185	0.000	-12.451
201	25.353	-9798.464	380.473	335.973	0.000	-23.345
202	16.226	-6271.017	243.503	215.022	0.000	-14.941
203	20.282	-7838.771	304.378	268.778	0.000	-18.676
204	15.212	-5879.079	228.284	201.584	0.000	-14.007

Pile Cap size (in investigated direction)	**H** =	4.600 m
Pile Cap size (in investigated perpendicular direction)	**B** =	4.198 m

PILE CAP DESIGN CALCULATION

Pile Reactions

Total pile number **N** = 14

_	Arrangement		Reaction		
Pile No.	**X (m)**	**Y (m)**	**Axial (kN)**	**Lateral (kN)**	**Uplift (kN)**
1	-2.250	-0.750	-696.654	27.237	0.000
2	-2.250	0.750	-719.395	27.237	0.000
3	-1.500	-2.049	-677.697	27.237	0.000
4	-1.500	2.049	-739.825	27.237	0.000
5	-0.750	-0.750	-698.128	27.237	0.000
6	-0.750	0.750	-720.868	27.237	0.000
7	0.000	-2.049	-679.171	27.237	0.000
8	0.000	2.049	-741.299	27.237	0.000
9	0.750	-0.750	-699.602	27.237	0.000
10	0.750	0.750	-722.342	27.237	0.000
11	1.500	-2.049	-680.645	27.237	0.000
12	1.500	2.049	-742.773	27.237	0.000
13	2.250	-0.750	-701.075	27.237	0.000
14	2.250	0.750	-723.816	27.237	0.000

Reinforcement Calculation

Maximum bar size allowed along length # 40

Maximum bar size allowed along width # 40

Bending Moment At Critical Section = -4699.403 kNm (Along Length)

Bending Moment At Critical Section = -4677.367 kNm (Along Width)

Pile Cap Thickness **t** = 1.607 m

Selected bar size along length # 16

Selected bar size along width # 16

Selected bar spacing along length = 88.74 mm

Selected bar spacing along width = 97.48 mm

Pile Cap Thickness Check

Calculated Thickness (t) = 1.607 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	-1288.786	0.000
2	-1330.856	0.000
3	-745.453	0.000
4	-813.793	0.000
5	-244.340	0.000
6	-252.299	0.000
7	0.000	0.000
8	0.000	0.000
9	0.000	-244.856
10	0.000	-252.815
11	0.000	-748.696
12	0.000	-817.035
13	0.000	-1296.966
14	0.000	-1339.035

Effective Depth(d_{ef}) = $h_{cap} - \left(p_{id} + cc + 0.5 \times d_b\right)$ = 1.476 m

Depth of neutral axis for balanced section(x_u) $\dfrac{700 \times d_{eff}}{1100 + 0.87 \times f_y}$ = 0.707 m

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

Ultimate moment of resistance(M_{ulim}) = $0.36 \times f_c \times b \times X_u \times \left(d_{eff} - 0.416 \times X_u\right)$ = 37890.391 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	-243.824	0.000
2	0.000	-251.783
3	-1117.502	0.000
4	0.000	-1219.949
5	-244.340	0.000
6	0.000	-252.299
7	-1119.933	0.000
8	0.000	-1222.379
9	-244.856	0.000
10	0.000	-252.815
11	-1122.363	0.000
12	0.000	-1224.810
13	-245.372	0.000
14	0.000	-253.331

Governing moment (M_u) = -4677.367 kNm

We assume singly reinforced and under reinforcement section

Effective Depth(d_{ef}) = $h_{cap} - \left(p_{id} + cc + 0.5 \times d_b\right)$ = 1.476 m

Depth of neutral axis for balanced section(x_u) $\dfrac{700 \times d_{eff}}{1100 + 0.87 \times f_y}$ = 0.707 m

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

Ultimate moment of resistance(M_{ulim}) = $0.36 \times f_c \times b \times X_u \times \left(d_{eff} - 0.416 \times X_u\right)$ = 41518.770 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	-422.130	0.000
2	-435.775	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
12	0.000	0.000
13	0.000	-424.783
14	0.000	-438.428
TOTAL	-857.905	-863.211

Design Shear Force for One-Way Action V_u = -863.211 kN

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

Design Shear Stress (T_v) = $\frac{V_u}{B \times d}$ = -139.312 kN/m^2

Allowable Shear Stress (T_c) = $\frac{0.85 \times \sqrt{0.8 \times f_c}}{6 \times \beta} \times \left(\sqrt{1 + 5 \times \beta} - 1\right)$ = 275.645 kN/m^2

Where Beta = $\max\left(\frac{0.8 \times f_c}{6.89 \times .pt}, 1\right)$ = 26.661

and percentage of steel required (p_t) = $\frac{A_{st}}{B \times d} \times 100$ = 0.131

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	-347.500	0.000
4	0.000	-379.036
5	0.000	0.000
6	0.000	0.000
7	-348.248	0.000
8	0.000	-379.784
9	0.000	0.000
10	0.000	0.000
11	-348.996	0.000
12	0.000	-380.532
13	0.000	0.000
14	0.000	0.000
TOTAL	-1044.743	-1139.351

Design Shear force (V_u) = -1139.351 kN

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

Design Shear Stress (T_v) = $\frac{V_u}{B \times d}$ = -183.878 kN/m^2

Allowable Shear Stress (T_c) = $\frac{0.85 \times \sqrt{0.8 \times f_c}}{6 \times \beta} \times \left(\sqrt{1 + 5 \times \beta} - 1\right)$ = 275.645 kN/m^2

Where Beta = $\max\left(\frac{0.8 \times f_c}{6.89 \times .pt}, 1\right)$ = 26.661

and percentage of steel required (p_t) = $\frac{A_{st}}{B \times d} \times 100$ = 0.131

Here $T_v <= T_c$ Hence safe

Check for Two Way Shear (Along Length)

Pile No.	Two-way Shear at column face (kN)
1	-422.130
2	-435.775
3	-410.756
4	-448.033
5	0.000
6	0.000
7	-411.641
8	-448.917
9	0.000
10	0.000
11	-412.525
12	-449.801
13	-424.783
14	-438.428
TOTAL	-4302.789

Design Two-Way Shear force = -4302.789 kN

As Per IS 456 2000 Clause 31.6.2.1

Two Way Shear Stress(T_v) = $\frac{V_t}{b_0 \times d_{eff}}$ = -320.207 kN/m^2

Where,perimeter of critical section(b_0) = $2 \times (b + h + 2 \times d)$ = 9.104 m

As Per IS 456 2000 Clause 31.6.3.1

Allowable shear stress = $K_s \times \tau_c$ = 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}) = $0.5 \times (B - b) - C_s$ = 1.850 m

$l_{db} > l_d$ hence, safe

Along Width

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) = $\dfrac{0.87 \times d_b \times f_y}{4 \times \tau_{bd}}$ = 1.471 m

Allowable Length(l_{db}) = $0.5 \times (H - h) - Cs$ = 1.649 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}) = $0.12\% \times B \times h_{cap}$ = 7898.957 mm2

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

Area of steel required (A_{sq}) = $0.5 \times \left(\frac{f_c}{f_y}\right) \times \left(1 - \sqrt{1 - \frac{4.5977 \times M_u}{f_c \times b \times d \times d}}\right) \times b \times d$ = 9349.410 mm2

Area of steel provided (A_{st}) = = 9349.410 mm2

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

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

Selected spacing (S) = 88.74 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}) $0.12\% \times B \times h_{cap}$ = 8655.360 mm2

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

Area of steel required (A_{sq})= $0.5 \times \left(\frac{f_c}{f_y}\right) \times \left(1 - \sqrt{1 - \frac{4.5977 \times M_u}{f_c \times b \times d \times d}}\right) \times b \times d$ = 9286.344 mm2

Area of steel provided (A_{st}) = = 9286.344 mm2

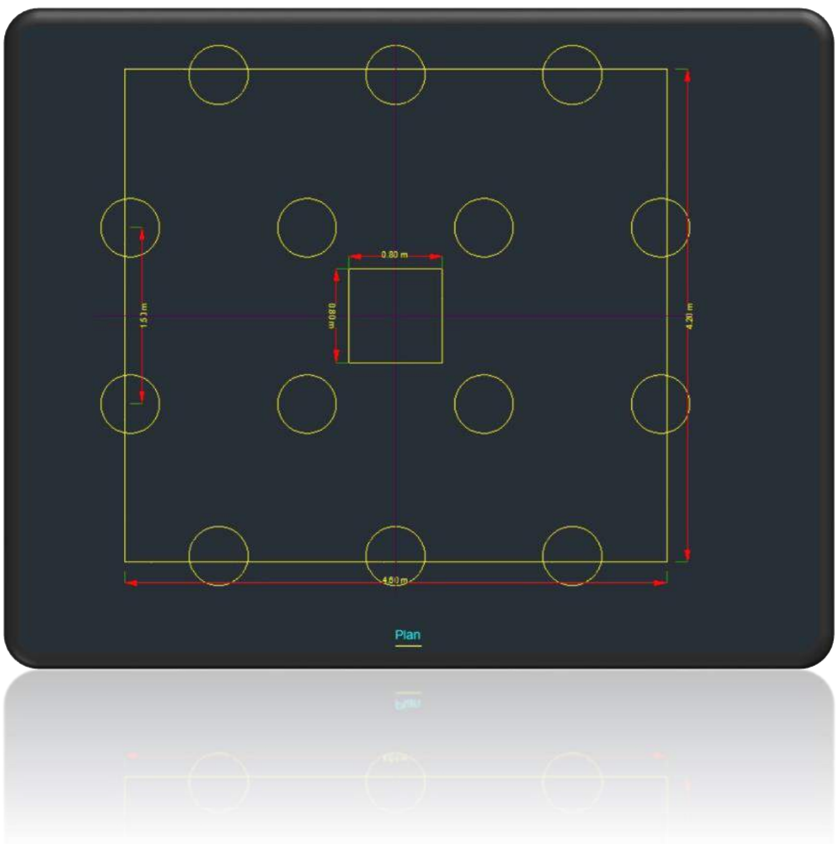
$A_{stmin} <= A_{st}$ Steel area is accepted

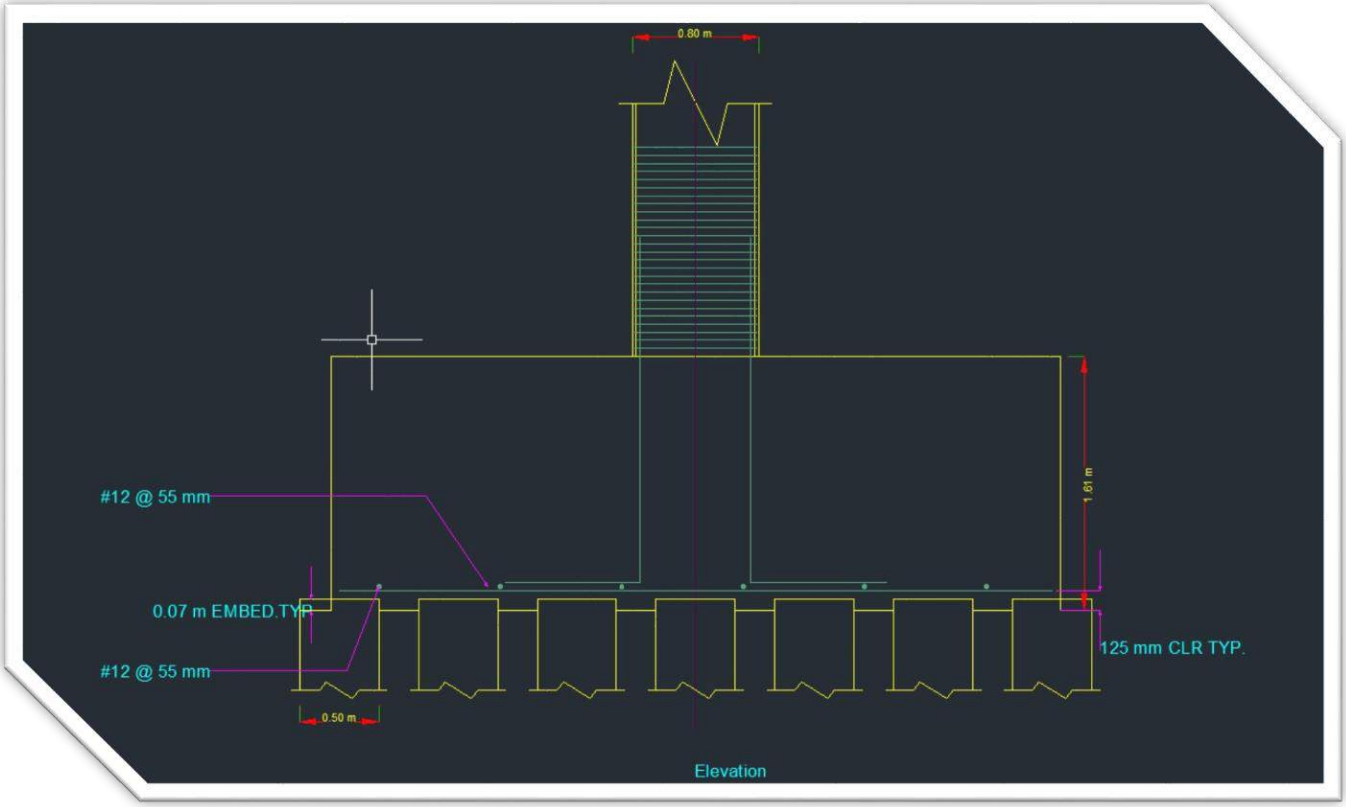
Minimum spacing allowed (S_{min}) = 40 + d_b = 56.00 mm

Selected spacing (S) = 97.48 mm

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

Pile Detailing in Auto Cad





0.80 m
1.61 m
#12 @ 55 mm
0.07 m EMBED.TYP
#12 @ 55 mm
125 mm CLR TYP.
0.50 m
Elevation

Plumbing Details

Water Supply design for FTA:

Water demand = 135 LPCD

For total apartment:

There are 4 houses in each floor**, Total population**= 68*6=408 (Considering the average family size as 6)

Total water demand = 408 * 135 = 55,080 liters/day

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

Average rate of supply = 55,080 liters /day, i.e,

Assuming 8 hours pumping into the overhead storage tank, **Rate of pumping** = 55,080/8=6,885 liters/hr

Assuming that the pattern of pumping is from 6 to 10 am and 2 to 6 pm

Let a be the hourly demand (a =55,080/24 =2295 lit/hr)

According to SP 35:

Hours	Hourly demand	Supply
0-4	0.2a	459
4-5	0.4a	918
5-6	0.8a	1836
6-10	2.25a	5163.75
10-12	a	2295
12-13	0.6a	1377
13-14	2.25a	5163.75

14-17	0.7a	1606.5
17-18	2.25a	5163.75
18-20	0.9a	2065.5
20-22	0.7a	1606.5
22-23	0.4a	918
23-24	0.2a	459

Required water is supplied by pumping in 8 hours.

∴ **Rate of pumping** = 55080/8 ≈ 6900 lit/hr (considering losses in pipes)

Capacity of overhead tank:

Based on a and the pattern of pumping, i.e, for 4 hours from 6 to 10 am and for 4 hours from 2 to 6 pm, the capacity of the ground level storage tank is calculated as in the table below

FIG. 7 MODIFIED MASS-DIAGRAM OR RIPPLE METHOD FOR DETERMINATION OF THE YIELD OF IMPOUNDING RESERVOIRS

TABLE 15 CAPACITY OF STORAGE RESERVOIR

(*Clause* 5.4.2.3)

Time		Hourly Demand	Cumulative Demand	Average Rate of Pumping Per Hour $= \frac{24 \times a}{16}$	Cumulative Pumping	Cumulative Deficit or Surplus {Surplus + ve, Deficit − ve}	Storage in Reservoir at the End of Period
From h	To h						
0	4	0.2 *a*	0.8 *a*	1.5 *a*	6,0 *a*	+5.2 *a*	7.6 *a*
4	5	0.4 *a*	1.2 *a*	1.5 *a*	7.5 *a*	+6.3 *a*	8.7 *a*
5	6	0.8 *a*	2.0 *a*	1.5 *a*	9.0 *a*	+7.0 *a*	9.4 *a*
6	10	2.25 *a*	11.0 *a*	—	9.0 *a*	−2.0 *a*	0.4 *a*
10	12	*a*	13.0 *a*	1.5 *a*	12.0 *a*	−1.0 *a*	1.4 *a*
12	13	0.6 *a*	13.6 *a*	1.5 *a*	13.5 *a*	−0.1 *a*	2.3 *a*
13	14	2.25 *a*	15.85 *a*	1.5 *a*	15.0 *a*	−0.85 *a*	1.55 *a*
14	17	0.7 *a*	17.95 *a*	1.5 *a*	19.5 *a*	+1.55 *a*	3.95 *a*
17	18	2.25 *a*	20.20 *a*	1.5 *a*	21.0 *a*	+0.8 *a*	3.2 *a*
18	20	0.9 *a*	22.0 *a*	—	21.0 *a*	−1.0 *a*	1.4 *a*
20	22	0.7 *a*	23.4 *a*	—	21.0 *a*	−2.4 *a*	0
22	23	0.4 *a*	23.8 *a*	1.5 *a*	22.5 *a*	+1.3 *a*	1.1 *a*
23	24	0.2 *a*	24.0 *a*	1.5 *a*	24.0 *a*	0	2.4 *a*

Capacity of storage reservoir = maximum deficit + maximum surplus = 2.4 *a* + 7.0 *a* = 9.4 *a*

Reservoir will be empty at 2200 hours and will be full at 0600 hours when the pumping stops.

Maximum storage = 9.4 *a*, that is, 9.4 hours of average supply = 9.4 × .09 = 0.846 million liters = 39 percent of daily demand.

48 HANDBOOK ON WATER SUPPLY AND DRAINAGE

Capacity of overhead tank:

Based on a and the pattern of pumping, i.e, for 4 hours from 6 to 10 am and for 4 hours from 2 to 6 pm, the capacity of the ground level storage tank is calculated as in the table below

Time	Hourly demand	Cumulative demand	Average rate of pumping (per hr)	Cumulative pumping	Cumulative deficit or surplus (surplus +ve, deficit -ve)	Storage
0-4	459	1836	3442.5	13770	11934	17442
4-5	918	2754	3442.5	17212.5	14458.5	19966.5
5-6	1836	4590	3442.5	20655	16065	21573
6-10	5163.75	25245	NONE	20655	-4590	918
10-12	2295	29835	3442.5	27540	-2295	3213
12-13	1377	31212	3442.5	30982.5	-229.5	5278.5
13-14	5163.75	36375.75	3442.5	34425	-1950.75	3557.25
14-17	1606.5	41195.25	3442.5	44752.5	3557.25	9065.25
17-18	5163.75	46359	3442.5	48195	1836	7344
18-20	2065.5	50490	NONE	48195	-2295	3213
20-22	1606.5	53703	NONE	48195	-5508	0
22-23	918	54621	3442.5	51637.5	2983.5	2524.5
23-24	459	55080	3442.5	55080	0	5508

Storage of Overhead tank

Storage = Max deficit + Max surplus

= 2.4a + 7.0a

= 9.4a lit = 21,573 Lit

The storage capacity as per the norms given in Table 16 of 'SP 35: Water Supply and Drainage' based on the population in the residential building

= No. of population X 70 lit

= 6*68*70

= 28560 lit

Storage needed for flushing water closets as given in Table 17 of SP 35: Water Supply and Drainage

= 68*270 + 68*180

= 30,600 liters

Minimum storage as given in clause 5.4.2.3 of SP35: Water Supply and Drainage

=½ * (Day's supply)

= ½ * 55,080= 27,540 lit

Storage of overhead tank

= Max (21573, 28560, 30600, 27540)

= **30600** lit

Capacity=30600 liters

Using concrete tank of dimension of 4m*2m*1m

No. of tanks required=30.6/8 = 3.825

No. of tanks provided =4

There are two set water pipe lines, one for pumping water from ground to overhead tank. In this pipe line one direct pipe connection i.e without going to the over-head tank directly to the apartment is provided. Another pipeline is to supply water from overhead tanks to each apartment's utilities.

Pipe Type	Diameter
Main	33mm
Subsidiary	22mm
Distribution	15mm

Sewerage System

Design of Sewerage pipe system:

A Two-pipe system will be provided in the residential blocks for disposal of sewage where soil pipes are connected to the building drain directly and the waste pipes are connected to the building-drain using a trapped gully.

A Main ventilation pipe (MVP) is provided for the main waste pipes (MWP) and the main soil pipe (MSP).

Waste Appliances (Internal diameters) (Table 54 of SP 35)

Waste Appliance	Internal Diameter
Wash Basin	30mm
Bathroom	40mm
Water Closet	50mm
Urinal	40mm

The load on soil pipes is 13 units and the load on waste pipes is 50 units in each floor. So, from table 52 of SP 35 the following pipe diameters can be obtained

Horizontal branches for each floor (Diameters) (Table 52 of SP 35)

FLOOR	BLOCKS	
	MWP	MSP
All Floors	100 mm	75mm

MWP and MSP columns indicate the diameter of the horizontal branches joining MWP and MSP respectively.

MWP and MSP (Table 53 of SP 35)

Diameter provided for all the MWP and MSP is 100 mm.

MVP (Main ventilation pipe) (Clause 6.7.5.3 of SP 35)

The diameter provided for the MVP is 50 mm.

Main sewer pipe Slope of the main sewer pipe is 1/1000. n (Manning's coefficient) = 0.012.

Specification for sewer pipes:

The sewer should be designed for the wastewater flow of 135 Lpcd.

80% of total consumption may be expected to reach the sewer.

The flow in the sewer or the ratio of maximum flows to average flows, depends upon the contributory population.

- ❑ If the population is less than 20,000 then peak factor should be taken as 3.
- ❑ A self-cleansing velocity of 1.0 m/s provided for minimum flow to avoid deposition in the line. Maximum velocity in the sewer is 3 m/s.

Diameter of main pipe is 150 mm and that of secondary pipe is 100 mm

A sewer runs at 80% full at ultimate peak flow. Pipes are laid at a depth of 1 m.

where vehicular loading is present, a concrete encasement is provided for the sewer.

Calculation of diameter for sewer:

Total consumption = 135 Lpcd. Design population = 408 Peak factor = 3

Peak consumption

= 3*(408*135)

= 165240 L/d Or 1.9125L/s.

$\therefore$ As per CPHEEO Manual, only 80% of total consumption is expected to reach the sewer.

Therefore, sewer flow

= 0.8*1.9125

= 1.53 L/s.

Peak discharge, Q=91.8 lpm

Using Manning's formula,

$$\boldsymbol{Q = \frac{1}{n} \cdot A \cdot R^{\frac{2}{3}} \cdot S^{\frac{1}{3}}}$$

where:

Q is the discharge,

n is the Manning's roughness coefficient=0.012,

A is the cross-sectional area of flow= $\pi d^2/4$,

R is the hydraulic radius =d/(4 π), and

S is the slope of the energy line (head loss per length of pipe) = 1/1000.

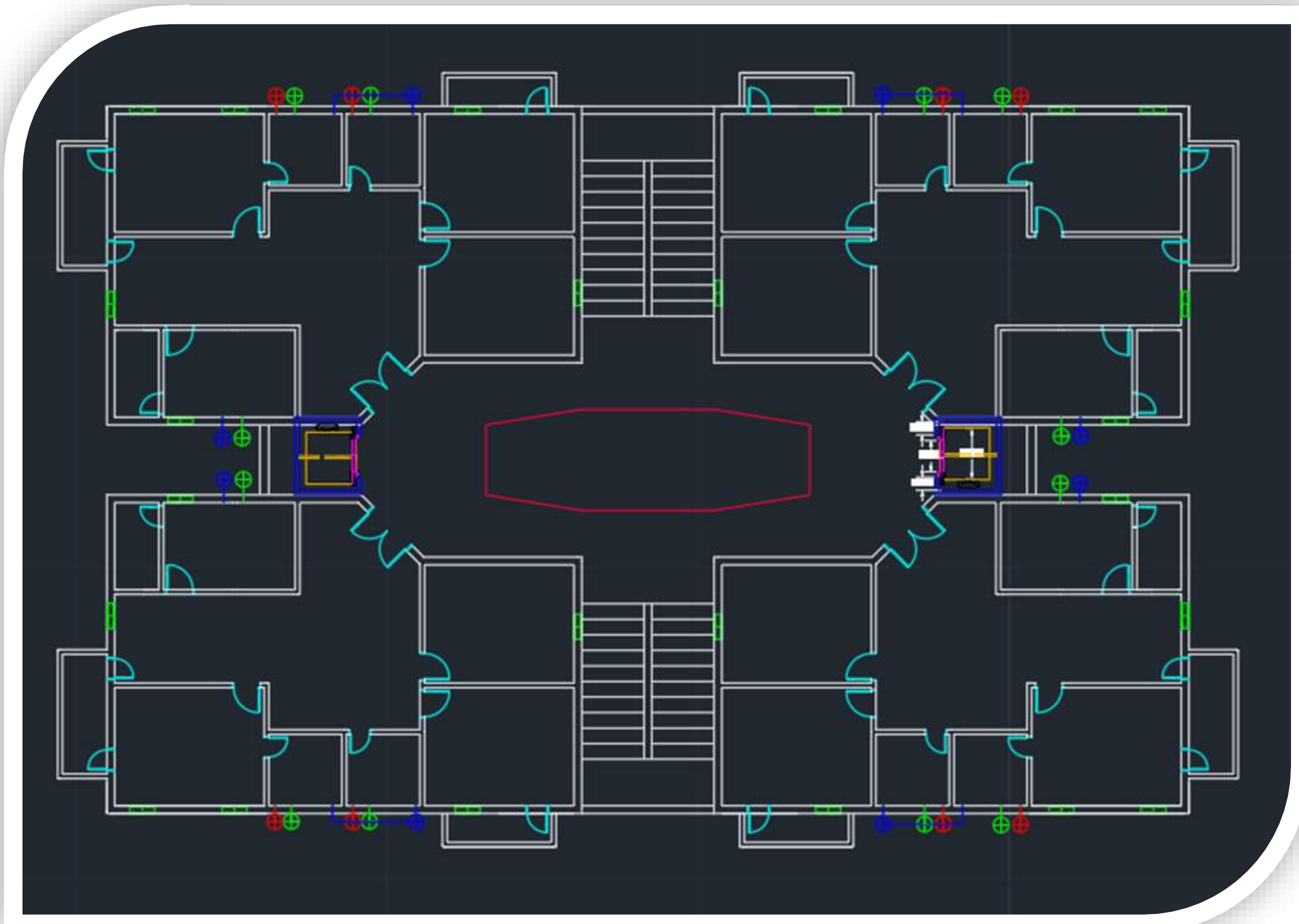
91.8/ (1000x60) = 1/0.012 x πd^2 /4 x (d/(4 π))$^{2/3}$ x (1/1000)$^{1/2}$

$\therefore$ d =126.065 ≈ 126 mm

So, The **diameter** provided for the main sewer pipe is **150 mm**.

Minimum pipe diameter recommended in CPHEEO manual is 150 mm except that in hilly areas, where extreme slopes are prevalent, 100 mm can be used

Plumbing Layout

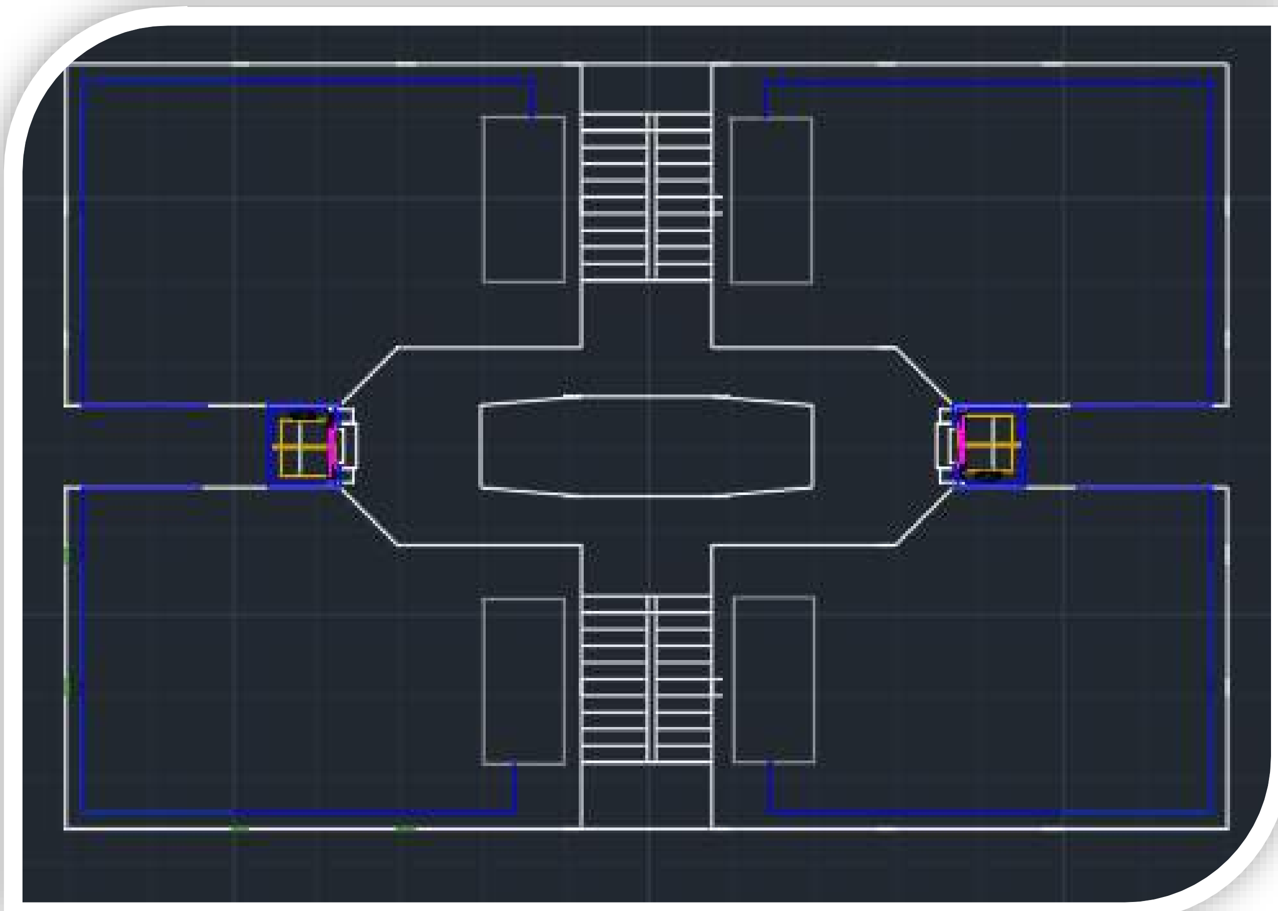


1.Blue coloured pipes are the inlet pipes.

2.Red and green colored pipes are the outlet pipes. They represent septic and sewage respectively.

3. Blue colored circle is the inlet from the tank and the green colored circle goes to the ground outlet.

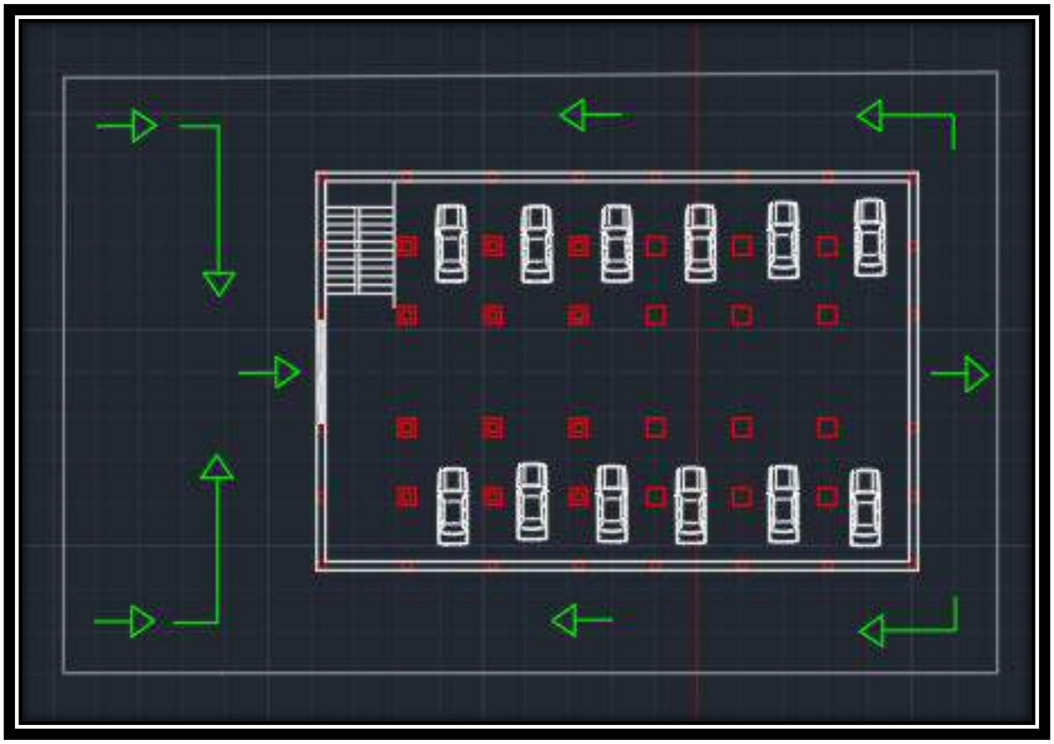
Plumbing Layout (roof)



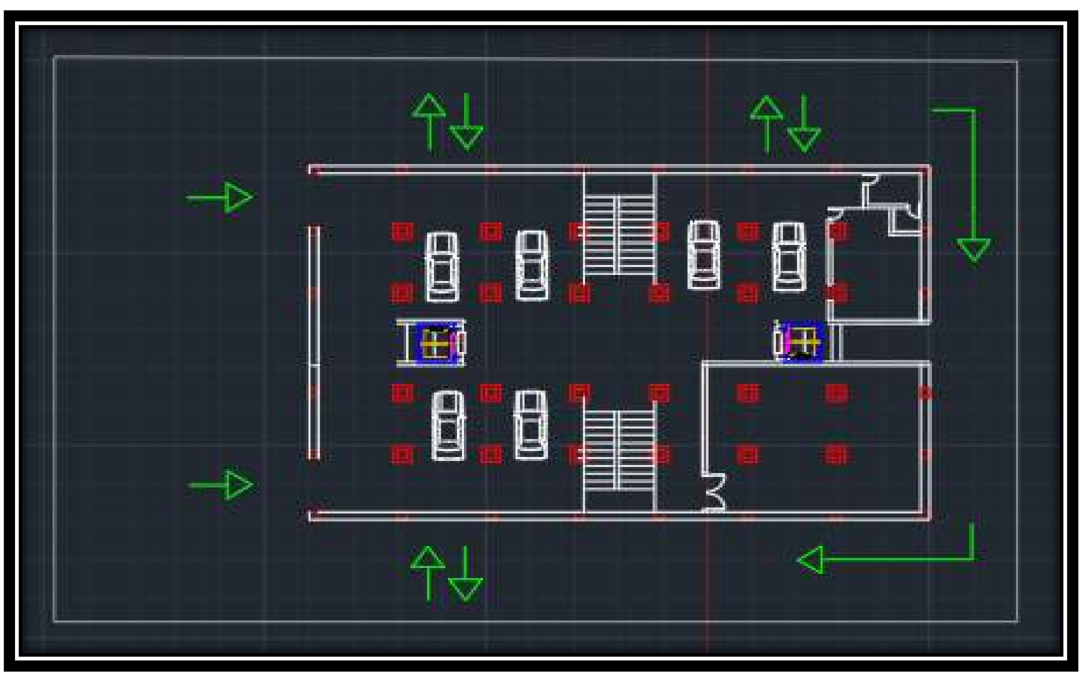
This shows the plumbing layout on the roof. The blue pipes are the inlet pipes from the water tank.

Road Network Details

Road Network For Basement Entry



Road Network For Ground Floor



COST ESTIMATION

-Steel Bar Cost:

12 mm => 81365.54 kg * Rs 48 = 3905550

16 mm => 172038.12 kg * Rs 54 = 9290060

20 mm => 10276.24 kg * Rs 59 = 606300

25 mm => 31723.03 kg * Rs 62 = 1966830

-Concrete Cost:

Rs 6500 * 3533.2 cubic meter = Rs 22965800/-

-Brick Cost:

Rs 9000000/-

IS CODES USED

- For RCC Design – IS : 456 – 2000
- For Steel Structures – IS : 800 – 2007
- For Dead Load calculations – IS : 875 (Part 1) – 1987
- For Live Load calculations – IS : 875 (Part 2) – 1987
- For Wind Load calculations – IS : 875 (Part 3) - 1987
- IS : 875 (Part 5) – 1987
- For Seismic Design – IS :1893 – 2002
- SP 16:1980 for concrete design
- As per IS 1172:1993, Clause 4.1 for water tank design

SOFTWARES USED

S no.	*Name*	*Used for*
1.	**AutoCAD**	To draw the plans
2.	**Revit**	For creating 3D rendered view of the building
3.	**Lumion**	To create the walkthrough
4.	**StaadPro**	For structural analysis
5.	**MS pptx**	For presentation
6.	**MS word**	For report