CE39004 - DESIGN SESSIONAL PROJECT REPORT

Department of Civil Engineering
IIT Kharagpur
Residential tower located at Guwahati

Group Members:

Anjana Jawahar	21CE10010
Ayush Yadav	21CE10013
Challa Naga Sujan	21CE10015
Daksh	21CE10016
Dangeti Narendra Gandhi	21CE10017
Aditya Singh Anand	21CE30003

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Design Basis Report

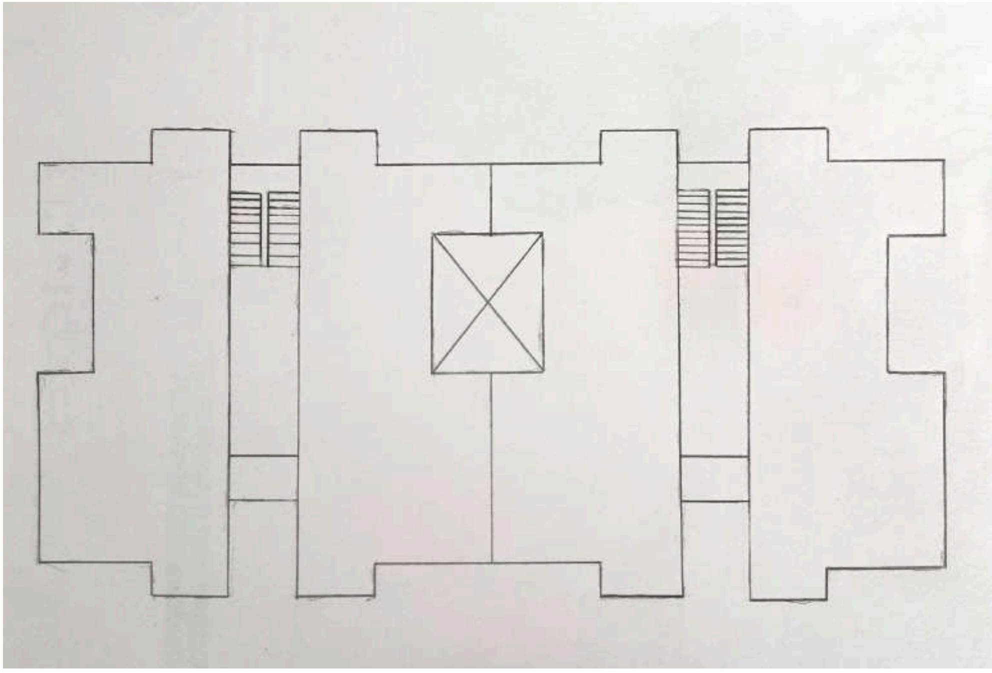
Problem Statement

- Building Location : Guwahati
- No. of floors: B+G+9
- No. of apartments on a floor: 4
- Area of each apartment: 143 m^2
- Height of the basement and ground floor: 2.7m
- Height of each floor: 3.4m
- Wall thickness:
- exterior: 0.25m
- interior: 0.15m
- Bearing Capacity: 20 ton/m^2

Materials considered

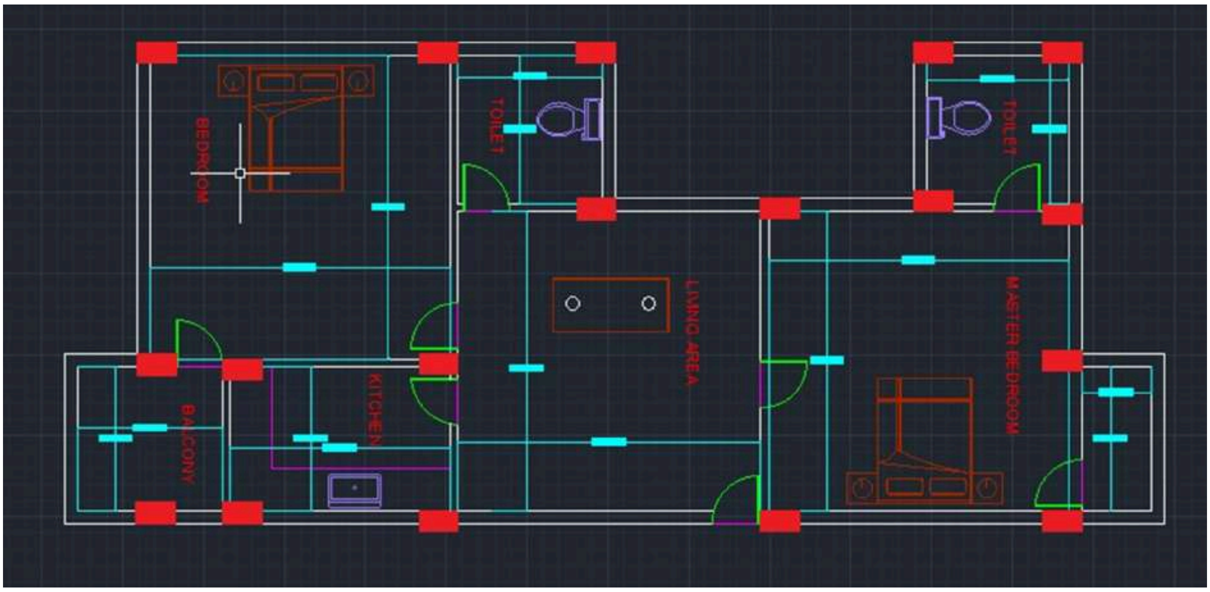
- M30 Grade concrete for RCC Structural elements
- Fe500 Steel for RCC Structural elements

Geometry

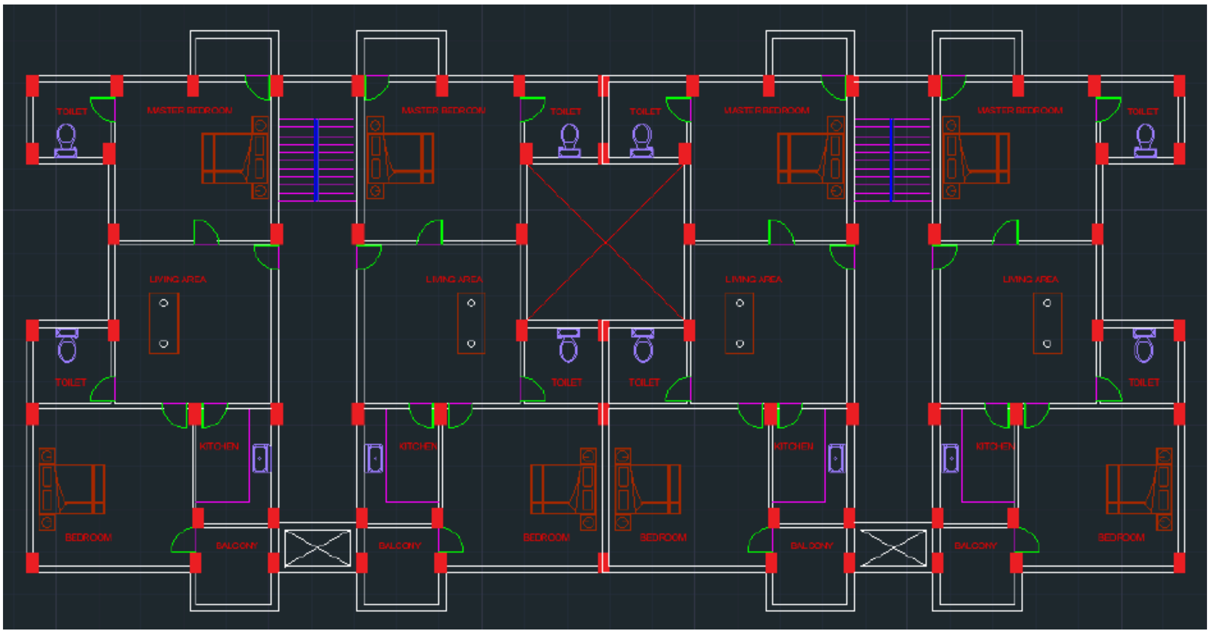


Apartments on each floor with 2 Hallways with stairs and lifts in each one.

Apartment Plan



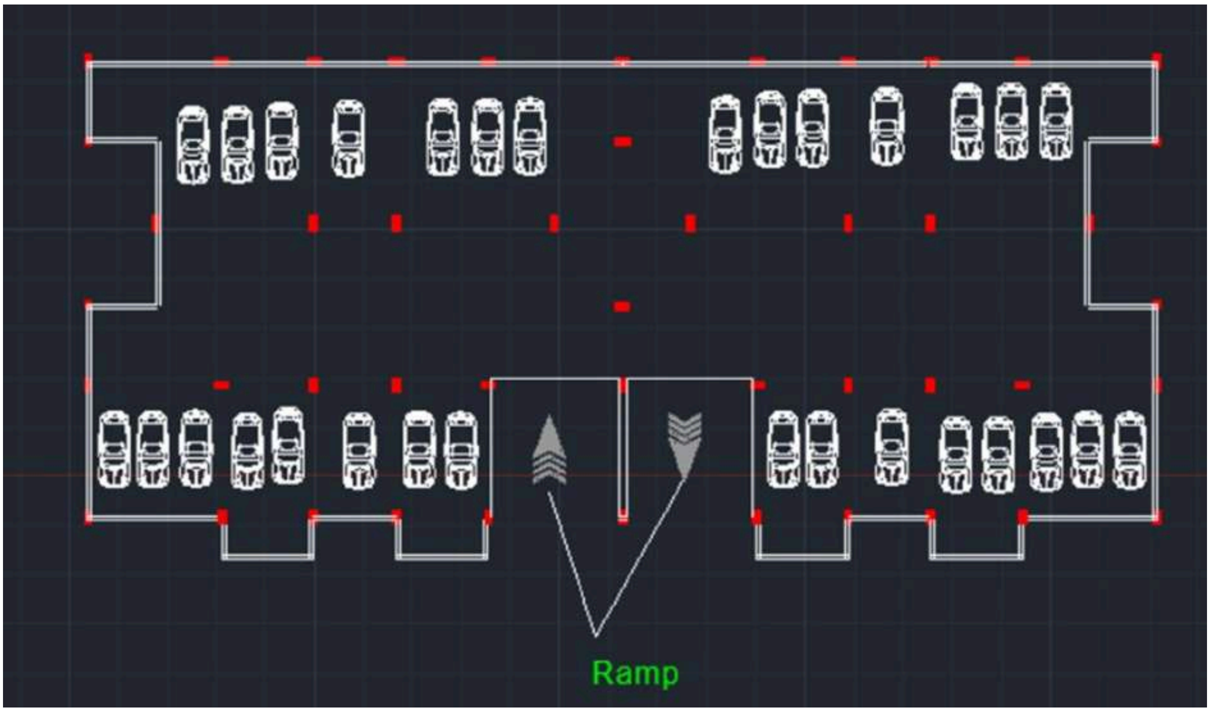
Floor Plan



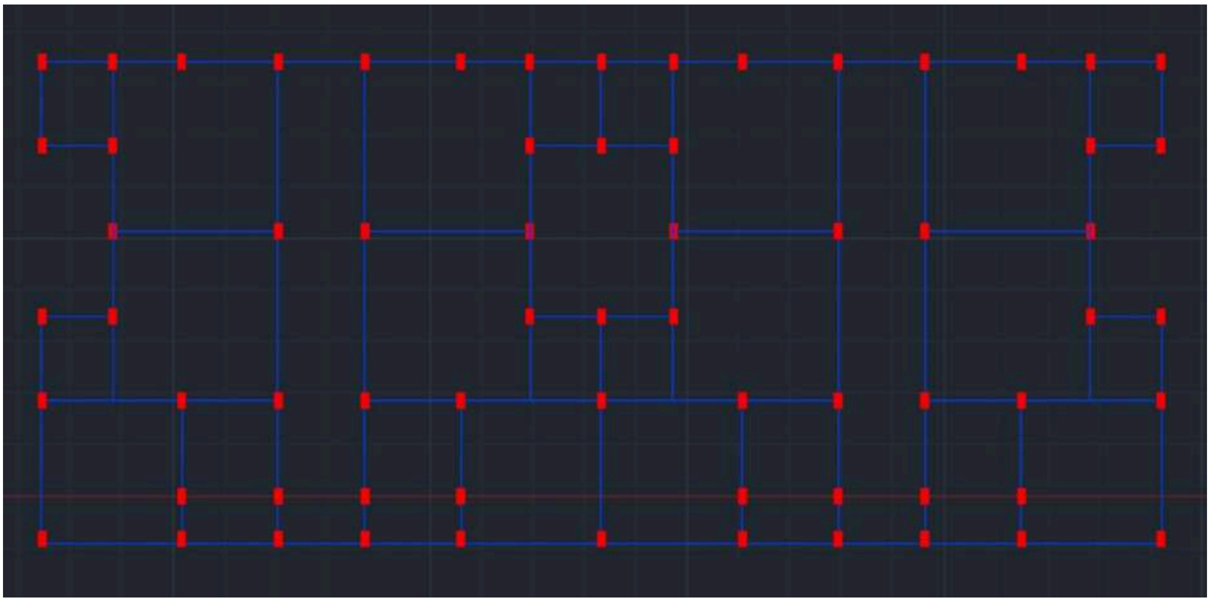
Advantages of the Plan

- Each apartment features two balconies, strategically positioned to face the east and west directions respectively. This thoughtful layout ensures ample natural light exposure for all apartments, both in the morning and evening.
- Between the 2nd and 3rd apartments lies an open space, facilitating easy access for plumbing repairs.

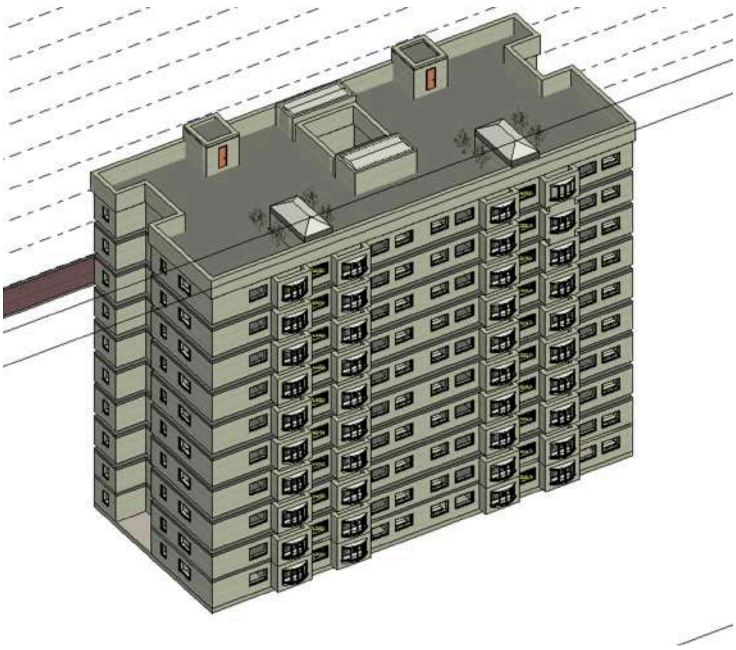
Basement Plan



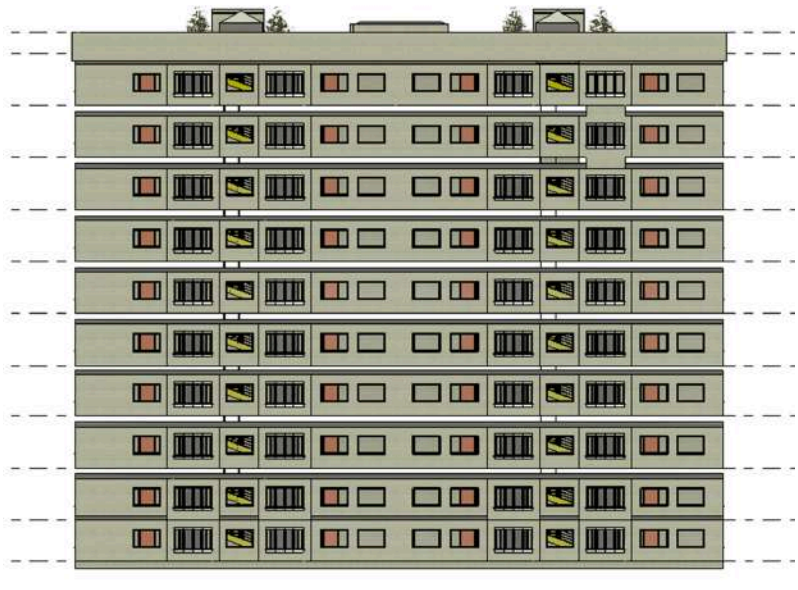
Beam-Column Layout



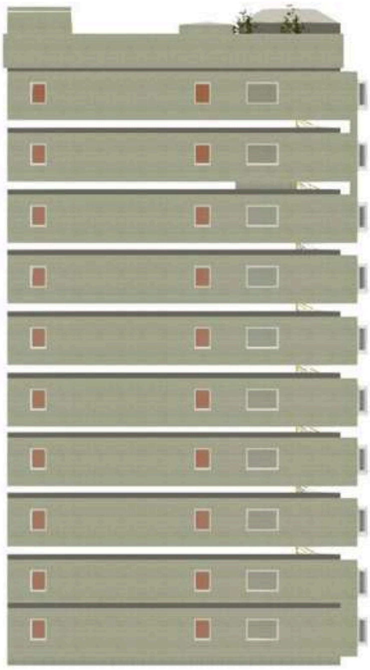
Isometric View



Front View

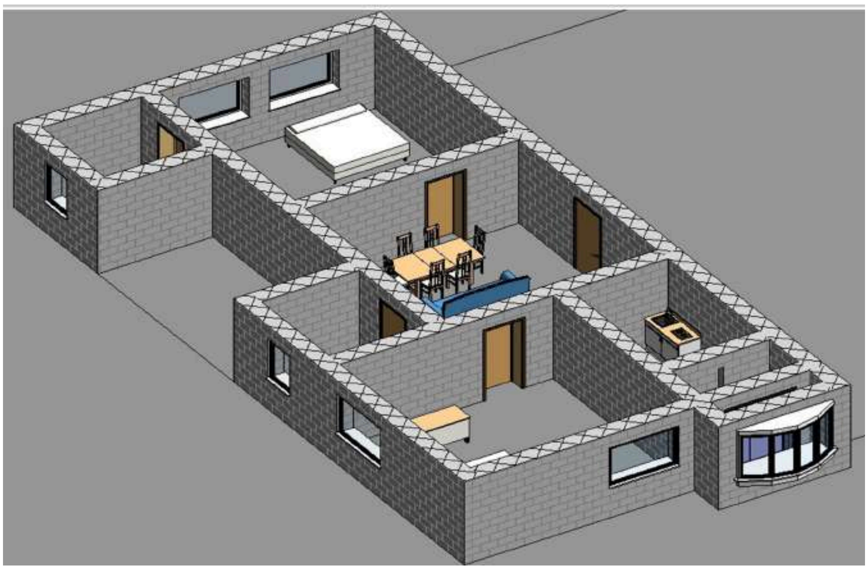


Side View

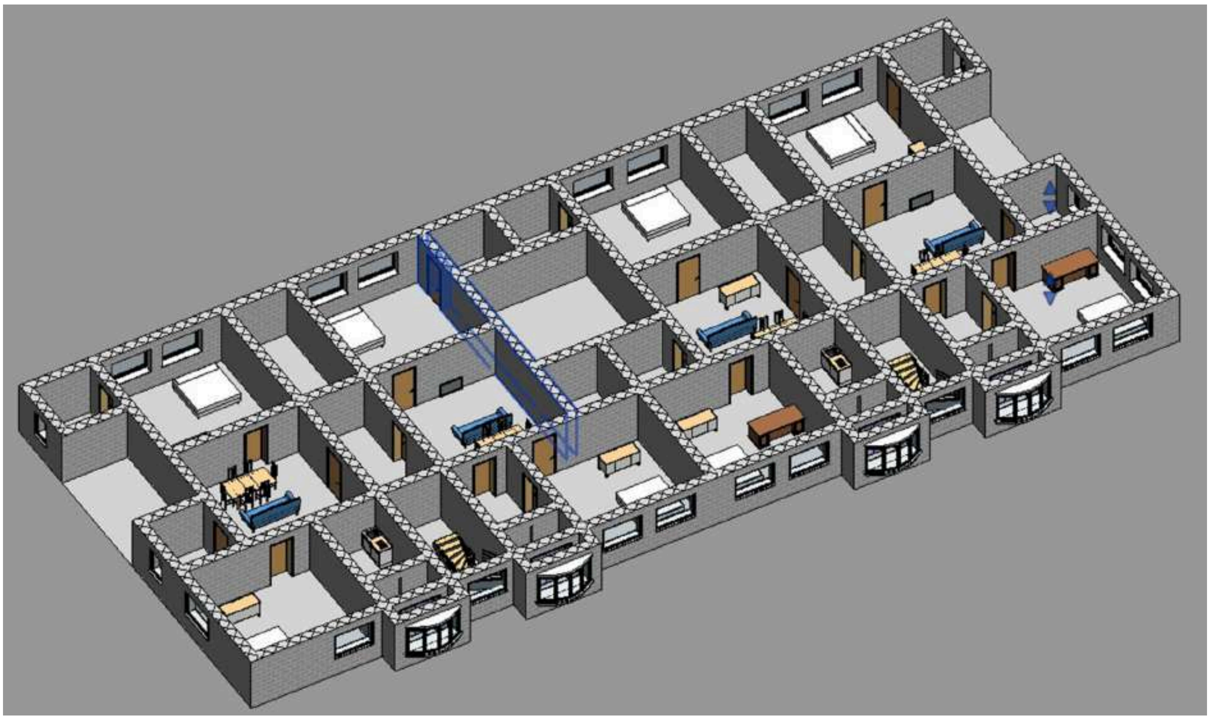


Back View

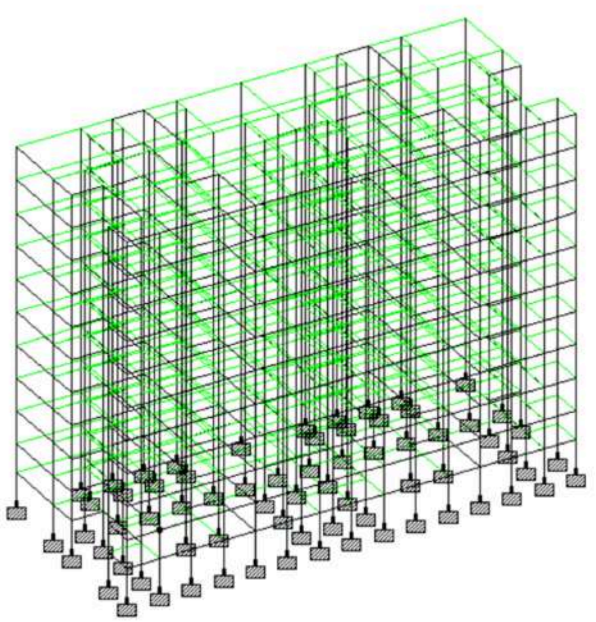
3D View of an apartment



3D View of a floor



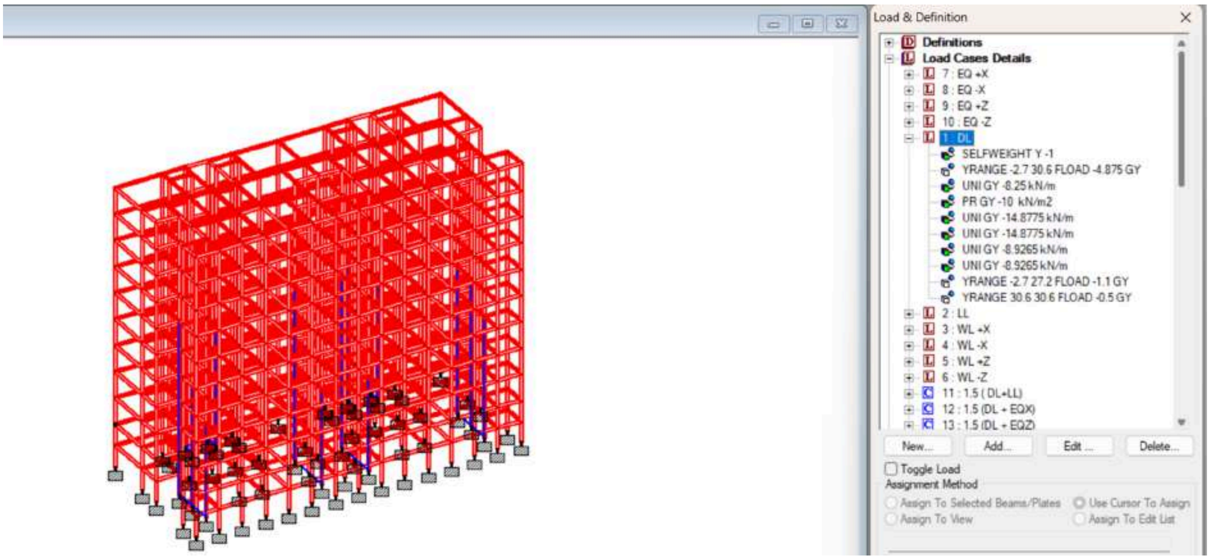
STAAD PRO MODEL



Dead Load (Parameters used from IS 875 Part 1)

Dead load	Value
Self Weight	-1
Slab	4.875 kN/m^2
Exterior Wall	14.8775 kN/m
Interior Wall	8.965 kN/m
Parapet Wall	8.25 kN/m
Floor finish	1.1 kN/m^2
Roof finish	0.5 kN/m^2
Water tank load	10 kN/m^2

Dead load definition added to the model in Staad Pro:

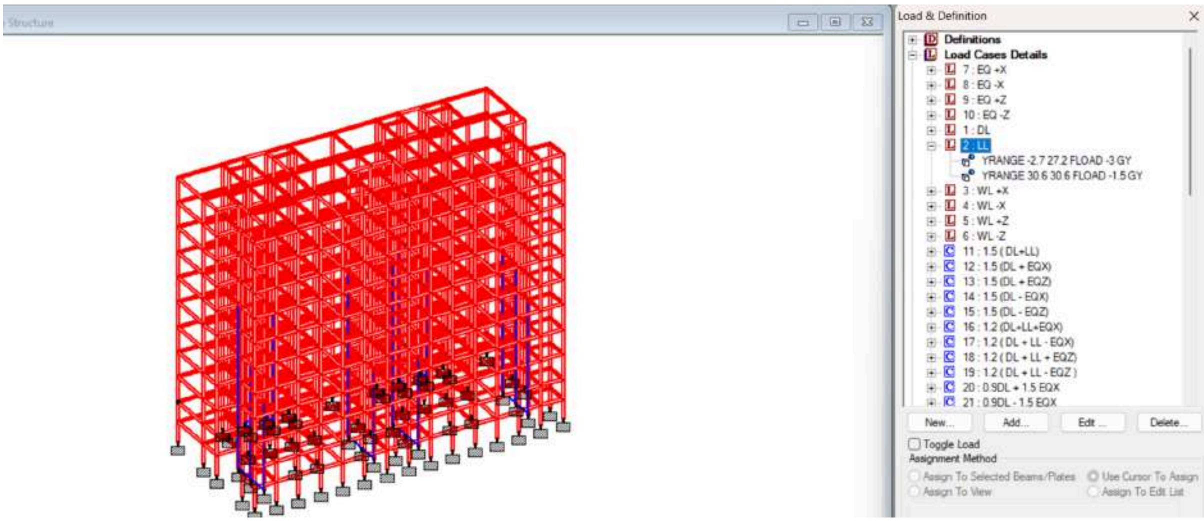


<u>Live Load</u>

Parameters used from IS 875 Part 2

Live Load	Value
Bedroom	2 kN/m2
Toilet & Bathroom	2 kN/m2
Kitchen	2 kN/m2
Dinning cum Living Room	3 kN/m2
Staircase	3 kN/m2
Common Space	3 kN/m2
Balcony	3 kN/m2

Live load definition added to the model in Staad Pro:



Wind Pressure & Design Forces (Reference: IS:875 PART 3)

Parameters used from IS:875 PT.3

Wind Data	Value	Reference
Basic Wind Speed	50m/s	Fig 1, IS 875, Part 3
Wind Zone	5	Fig 1, IS 875, Part 3
Terrain Category	4	IS 875, Part 3, SEction 4.3.2.1

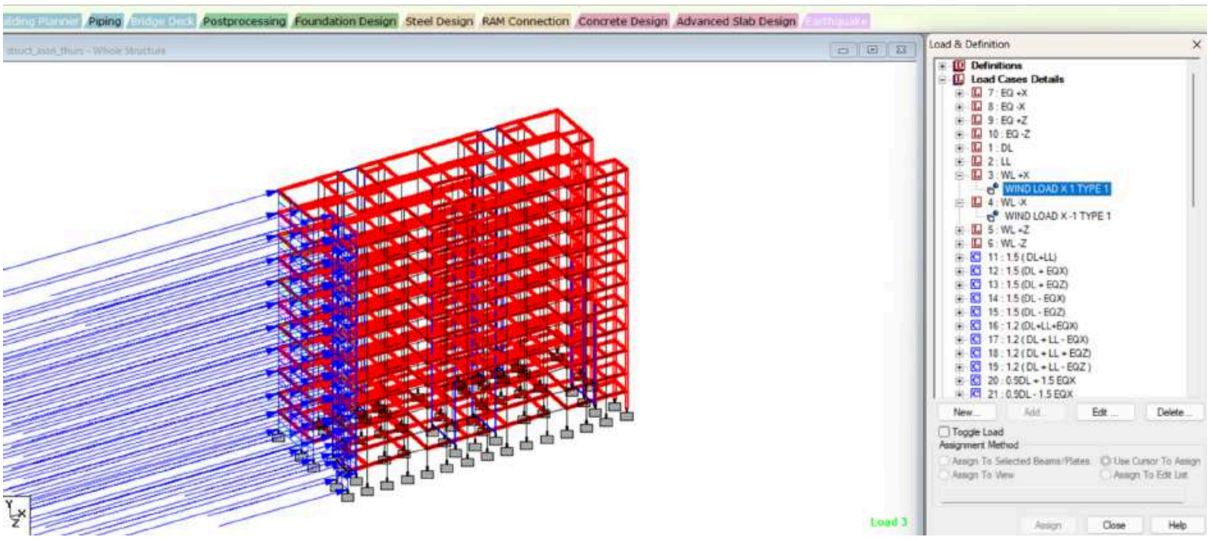
Design Factors	Value	Reference
Risk Coefficient Factor, K1	1	Table 1, IS 875, Part 3
Terrain and Height Factor, K2	Varies with height	Refer IS 875, Part 3
Topography Factor, K3	1	Refer IS 875, Part 3, Clause 6.3.3.1

Design Wind Parameters	Value	Reference
Design Wind Speed Vz= Vb*K1*K2*K3	50*K2	Clause 6.3, IS 875 Part 3
Pz= .6(Vz)^0.5	1500 Kz^2	Clause 7.2, IS 875, Part3

Height(m)	K2	Vz(m/s)	Pz(KN/m^2)
10	0.8	40	0.96
15	0.8	40	0.96
20	0.8	40	0.96
30	0.97	48.5	1.41135
50	1.1	55	1.815

Wind Load

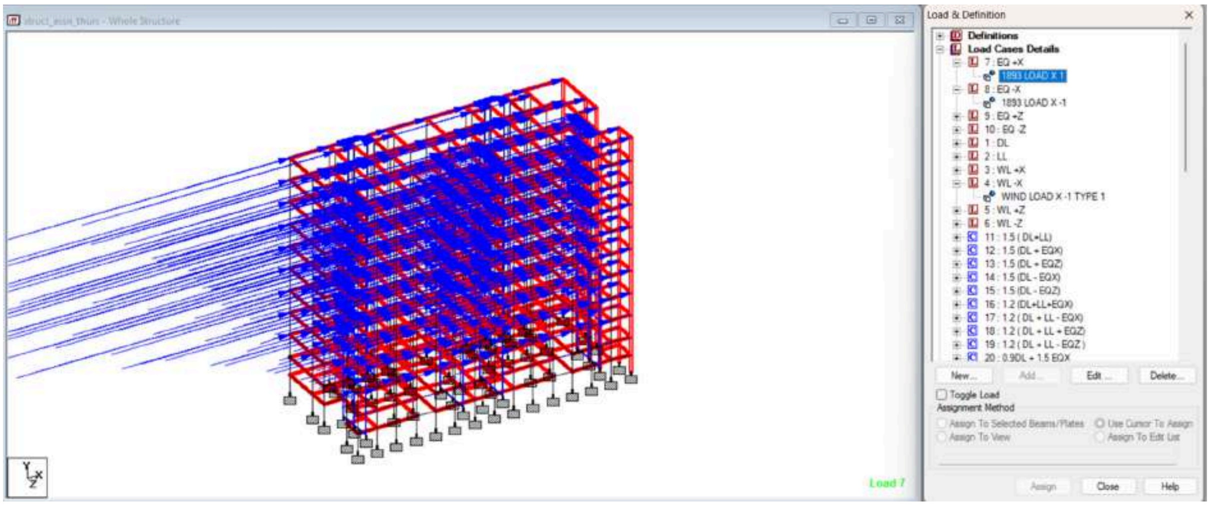
Wind Load applied along X Direction



Earthquake Load (Reference: IS:1893)

Seismic Parameter	Value
Zone	V
Zone Factor	0.36
Importance Factor	1
Response reduction Factor	5
Soil Type	Medium
Structure Type	RC Frame Building

Seismic Load applied along X Direction



Seismic Load calculation

Calculation of Self Weight

1. Slabs:

For Roof,
Dead Load= 4.875+0.5= 5.375 kN/m2
Total Area= 572-36= 532 m2
Hence, Load =5.375*532=2859.5 kN

For Apartments,
Dead Load= 4.875+1.1=5.975 kN/m2
Total Area=536 m2
Hence, Load = 5.975*536= 3202.6 KN

Total Load = 6062.1 KN

2. Beams(0.5m x 0.25m):

Length of beam per floor = 366 m
Length of beam in ground = 366 m
Total Floors=11
Total Load = unit weight x Total Length x Floor x C/S Area
=25 x 11 x 366x 0.5 x 0.25 kN=12851.25 kN

3. Columns:

Total columns= 67 (per floor)
Height of basement and ground floor = 2.7m

Height of other floors = 3.4m
Total weight =[25.8 x0.75 x 0.4 + 10.2x0.6x0.3] x25x67
=16039 KN

4. Walls:

Interior walls 0.15 m thick & 100 m per floor
Load = 24*.15*2.9*10*100= 10,440 KN

Exterior walls 0.25 m thick & 257 m per floor
Load= 24*0.25*2.9*10*257= 26830KN

Total Self Weight of Building= 71642.1 KN

Zone V

Zone Factor = 0.36(Z)
Importance Factor= 0.1
Response Reduction Factor= 5
Fundamental Natural Period, T= 0.075*(h^0.75)
= 0.075*(36^0.75)
=1.10227 s
From Software, T = 1.10227 s
h=36m
Sa/g= 0.907 (From Software)
Ah= (Z*I*Sa/g) / (2*RF) = .36*.1*0.907/ 2*5 = .0326
Base Shear, VB = Ah * W = .00326* 71642 = 2335.53KN
From Software= 2171.73 KN

Beam and Column Dimensions

Beam

- Depth = L/10 or L/12
  Depth = 6000/12 = 500mm

- As per the IS Code,
  W/D > 0.4 - 0.45
  W/D> 0.4
  W>200
  Let's consider W=250mm

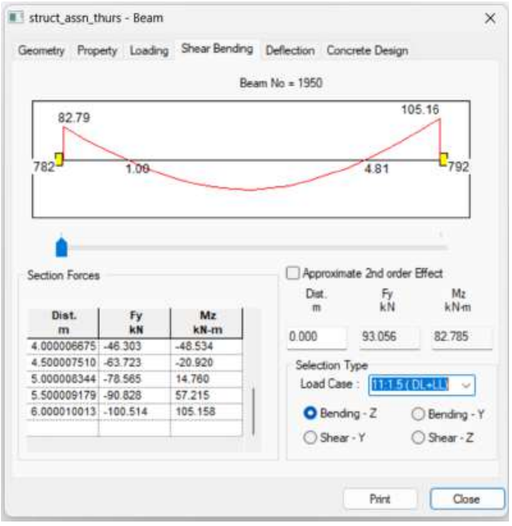
Column

- Group 1 : 750mmx400mm

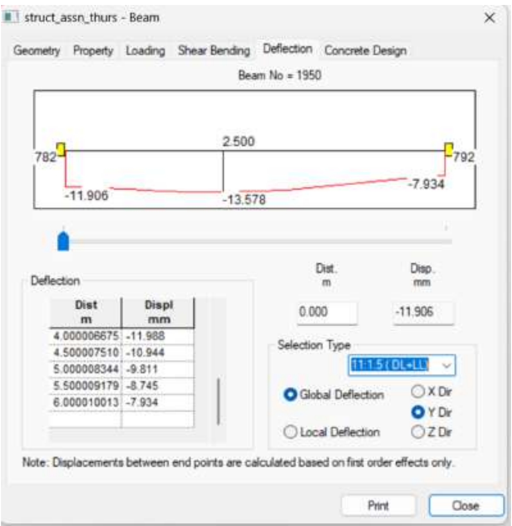
(upto 6floors)

- Group 2: 600mmx300mm
  (Above the 6th floor)

Design of Beams

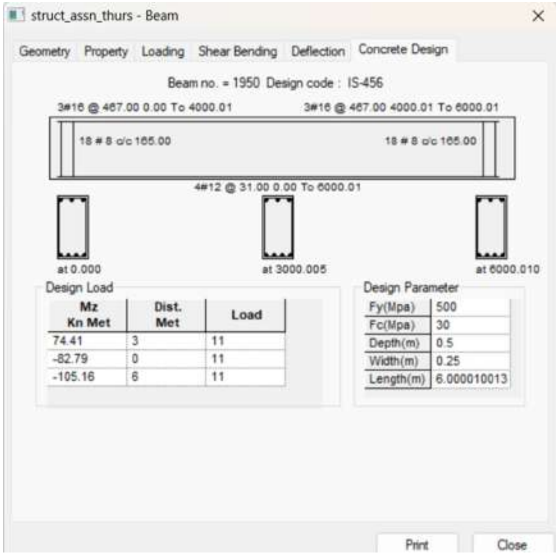


Bending moment Mz= 105.158 KN-m
Max shear: Vmax = 100.514 KN



Maximum permissible deflection in beam
= min (L/250, 20 mm)
= min (6000/250, 20 mm)
= 20 mm
Hence beam deflection is under permissible limit.

Beam Concrete Design



Manual Calculations and Results:

For Flexure,

Assume clear cover = 30mm
Steel bar diameter = 12mm
Effective cover = d’ = 30+6= 36mm
So, effective depth = d= D-d’= 500-36= 464mm
x/d= 1.22-[(1.22)^2 -(6.61Mu/fck bd^2)]^0.5
= 1.22- [(1.22)^2 -(6.61*105.158*10^6/ 30*250*464^2]^0.5
=.191444
(x/d) < (x/d)max = 0.46
Since it is an under reinforced section, beam is safe

Now, Pt = 41.38*(fck/fy)*(x/d)
=41.38*(30/500)*0.1914
=.475%
(Pt)min = (.85/100) = 0.17%
(Pt)max = 4%
(Pt)min < (Pt)provided < (Pt)max,
Hence beam is safe
So, Ast = Pt*b*d= (0.475/100)*250*464
= 551 mm^2

Ast (From Software) = 597.10 mm^2

For Shear,
Vmax = Vu= 100.514 KN

Tv = V/bd= 100.514/250*464 = 0.722N/mm^2
(Tc)max = 3.1 N/mm^2 (As per Table 20)
Tc= 0.578 N/mm^2
(By interpolation from Table 19, IS 456:2000)
Tu > Tc => Shear reinforcement should be provided

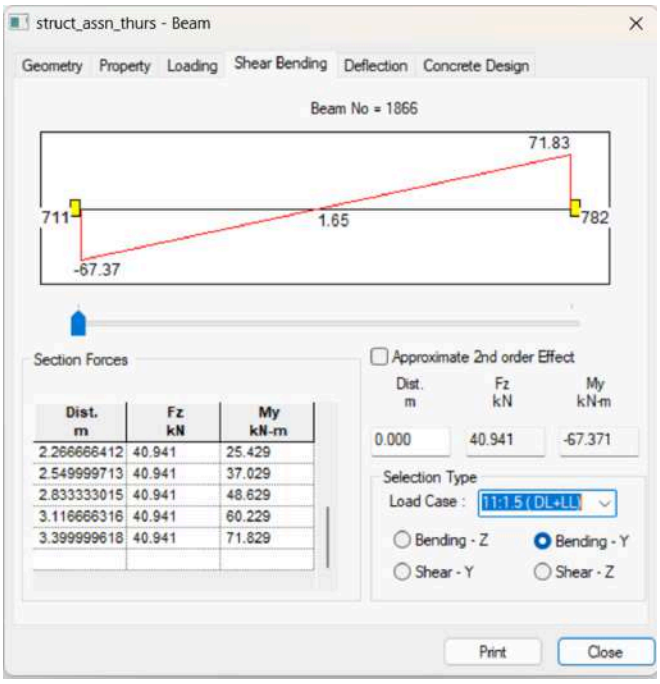
For V-Stirrups,
Vus= Vu-Vc = (Tv-Tc)*b*d
= (.722-.578)*250*464 = 16.7 KN

Taking 2- 8mm dia V-Stirrups. So, Asv = 100.53mm^2
Sv = (0.87fy*Asv*d / Vus) = 121mm

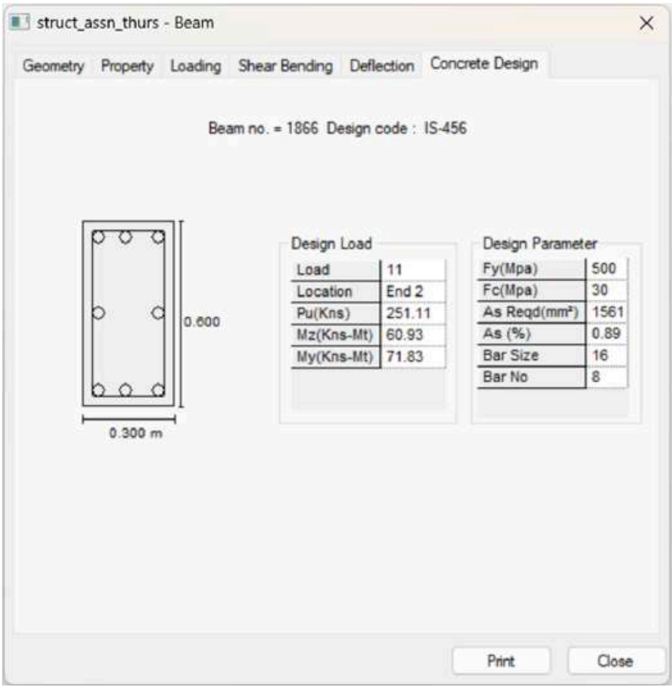
Provide 2legged- 8mm dia stirrups @ 120mm c/c

Design of Columns

Bending Moment Diagram



Concrete Design



A sc from Software =1561 mm^2
A sc from calculation = 1602 mm^2

MANUAL CALCULATIONS AND RESULTS:

Considering Beam No. 1866
My = 71.83 KNm
Mz = 60.93 KNm
Pu = 251.11 KN
L = 3.4m , b=300mm , D=600mm
Check for short column:
L eff = 0.65*L = 0.65*3.4 = 2.21 (Since both ends are restarined)
So, slenderness ratio, λ = L eff / b = 2.21/.3 = 7.367 < 12
Hence this is short column
Area of concrete = 180000mm^2
Assuming d’ = 30mm
d’ / D = 30/600 = 1/20 = 0.05

SP 16 Chart 47,
Pu / fck*b*D = (251.11*10^3)/(30*300*600)
= 0.05
P/fck = 0.89/30 = .03
Mu/fck*b*D^2 = 0.08

Mu y1 = 0.08*30*300*600^2
= 259.2 KNm
Now, d‘ / B = 30/300 = 0.1

Chart 48 SP 16:

$Pu/ fck*b*D = 251.11*10^3 / 30*300*600$

$= 0.05$

$P/fck = 0.03$

$Mu\ z1 = .080*30*600*300^2$

$= 129.6\ KN$

Checking for

$(Mu\ y / Mu\ y1)^{\alpha n} + (Mu\ z / Mu\ z1)^{\alpha n} \le 1$

$\alpha n \Rightarrow P\ u2 = .45*fck*Ag + (0.75*fy-0.45*fck)*Asc$

$Asc = .89*300*600/100 = 1602mm^2$

$P\ u2= 0.45*30*600*300 +(.75*500-.45*30)*1602$

$= 3009.123KN$

Alternatively referring to chart 63 of SP16,

$P\ u2/Ag =16.8$

$P\ u2= 16.8*600*300=3024\ KN$

$Pu/Pu2 = 251.11/3009.123= 0.083$

$Pu /Pu2 < 1 \Rightarrow \alpha n=1$

$\Rightarrow Mu\ y/Mu\ y1 + Mu\ z/ Mu\ z1 \le 1$

$71.83/259.2 + 60.93/129.6 \le 1$

$0.747 \le 1$

Hence OK

Load Combinations (IS:456)

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

Validation of Reaction Forces

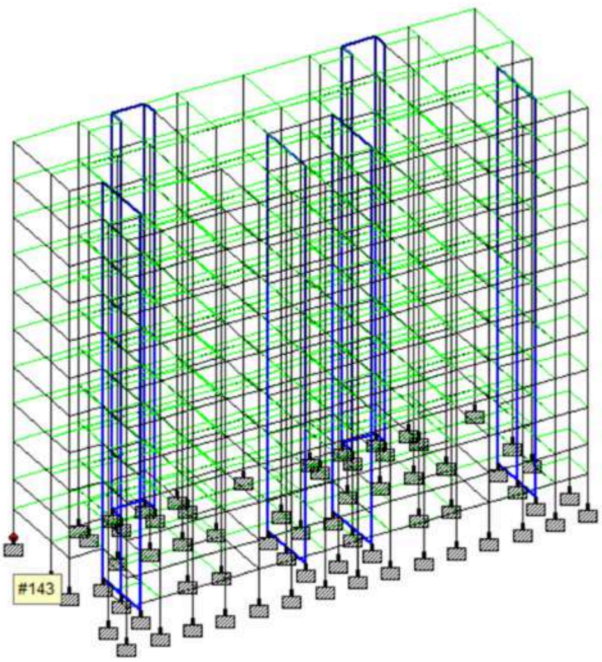
For Node Number 143 :
For Dead load -

Column weight above node = Height x Area xunit weight
= [(25.8*.75*.4) + (3*3.4*.6*.3)] *25 = 239.4 KN

2. Beam weight above node = Area x Length/2 x No. Of beams x No. Of Floors x unit wt.
= 0.5*.25*6/2 *2 *11 *25kN =206.25 KN
3. Wall weight above node = Height x Area x unit Weight
=[(3.4 *9 + 2.7*2]*[0.25 x6 x2/2] x24 kN =1296 KN
4. Slab weight above node = 1/4 x Area of Beam x Thickness x unit wt. x No of Floor
= 1/4 x(6x6) x0.195 x25 x11 kN =482.625 KN

Net reaction(Manual) = 2224.275 kN
Net reaction (software) = 2371.95 KN

For Node Number: 143
For Live load
We see that only the slab will have a live load
Live load of slab above node = 1/4 xArea of Room x Load xNo of Floor
= 1/4 x(6 x6) x(3x10 + 1.5x1) =283.5 KN
Net reaction(manual) =283.5 kN
Net reaction (software) = 285.553 KN



struct_assn_thurs - Support Reactions:

All | Summary | Envelope

		Horizontal	Vertical	Horizontal	Moment		
Node	L/C	Fx kN	Fy kN	Fz kN	Mx kN-m	My kN-m	Mz kN-m
143	1 DL	19.511	2371.995	18.745	17.396	-0.103	-16.896
	2 LL	3.900	285.553	3.858	3.689	-0.021	-3.331
	3 WL +X	-12.100	-34.002	-0.991	-1.444	0.068	21.493
	4 WL -X	7.119	33.445	0.602	0.710	0.062	-19.658
	5 WL +Z	-1.772	-59.131	-19.254	-29.401	-0.054	1.450
	6 WL -Z	2.588	58.619	14.121	27.210	-0.115	-3.307
	7 EQ +X	-21.930	-148.425	-1.804	-1.997	-0.168	62.471
	8 EQ -X	21.930	148.425	1.804	1.997	0.168	-62.471
	9 EQ +Z	-2.568	-108.953	-16.021	-31.472	0.086	2.877
	10 EQ -Z	2.568	108.953	16.021	31.472	-0.086	-2.877
	11 1.5 (DL+L	35.117	3986.322	33.904	31.626	-0.187	-30.340
	12 1.5 (DL +	-3.628	3335.356	25.411	23.098	-0.407	68.362
	13 1.5 (DL +	25.414	3394.564	4.086	-21.115	-0.027	-21.029
	14 1.5 (DL -	62.162	3780.630	30.823	29.089	0.097	-119.050
	15 1.5 (DL -	33.119	3721.422	52.149	73.301	-0.283	-29.659
	16 1.2 (DL+L	1.777	3010.948	24.959	22.905	-0.351	50.693
	17 1.2 (DL +	54.409	3367.167	29.288	27.697	0.052	-99.237

Slab Design Manual

M30, Fe500
fck=30 N/mm^2, fy= 500 N/mm^2
γc = 25KN/mm^2
Span/Effective depth = 35
=> 6000/d = 35
d= 172 mm

Let us consider 12 dia bars,
Therefore thickness of slab, h = d+Φ/2+cover
=172+12/2+15=193mm
193mm is not appreciable
Let us consider h=195mm
so updated d=174mm

Step 2 (Load Calculation):
Dead Loads:
Self weight of Slab (DL) = (1*1*.195)*25
= 4.875 KN/m^2
Live Load (LL) = 3 KN/m^2
TL = 7.85 KN/m^2
Factored Design Load (W)= 1.5*TL
= 11.8125 KN/m^2

Step 3 (IS Code 456:2000, pg 90-91, Table 27)
Ly/Lx = 1
αx = 0.062
αy = 0.062
Mx= αx*W*Lx^2= .062*11.8125*6^2=26.3655 KNm
My= αy*W*Ly^2=.062*11.8125*6^2=26.3655 KNm

Step 4 (Checking depth requirement from moment)
Mu =0.138 fck*b*d^2
26.3655*10^6=.138*30*100*d^2
d= (26.3655*10^6/.138*30*1000)^0.5
= 80mm < 174mm(provided)
Hence OK

Step 5 (Checking for Shear)
Max design shear, V = 0.5W*Lx
=0.5*11.8125*6=35.4375 KN
Approx Avg. shear stress = V/bd
=35.4375*10^3/174*1000
=0.20 N/mm^2

Consider a very less % of reinforcement, referring to Table 19(Pg 73) of IS:456:2000,
The design shear strength,
Tc= 0.29N/mm^2 > 0.2N/mm^2
Hence OK

Step 6: Calculation of Area
(SP 16, Table 3, Pg 49)
Mu/bd^2 = 26.3655*10^6/1000*174^2 = 0.87
Pt= 0.207 (Table 3, SP16, Pg 49)
As = Pt*bd/100 = .207*1000*174/100= 360mm^2/m width

Our both spans are same
For main reinforcements, using 12 dia, a s = pi*12^2/4 =113mm^2
No of rods within 1000mm = 360/113 = 4 no.

Let us adopt a spacing = 280mm c/c
Check spacing $\ngtr$ 3d or 300mm [whichever is less]
280 $\ngtr$ 300 Hence OK

Actual As provided =>
(As) provided =4*113=452 mm^2 within 1000mm width
For distribution reinforcements using 12 dia as well
(Same calculations as above)

Design of Stair Case

R = 150mm
T = 300mm
G = 250mm
N = 50mm

Thickness of waist slab = 150mm
Rise =150mm, Tread=300mm
Self weight of waist slab = .15*25*(R^2+T^2)/T
= 4.19 KN/m^2
Self wt. of steps = 25*(1/2)*.15 = 1.875 KN/m^2
Floor finish = .05*20 = 1 KN/m^2
LL = 3 KN/m^2
fck = 30N/mm^2
fy = 500N/ mm^2

Considering 12 steps,
Considering 1m = 1000mm strip
One way slab, Span/eff. depth =25
12*250/d = 25

$d = 120mm$

Let us assume that we will use 12 dia bars

w = d+cover+ϕ/2 =120+20+6= 146 mm

Not feasible

Take w= 150mm

d=124mm

FT= Tiles finish= 15mm

Self wt. Tiles= 23.5KN/m^3

B= (G^2+R^2)^0.5 = 292mm

Dead load = [{WB+(1/2)RT}*25 + {FT*23.5}(1/4)

=[{.15*.292 + (1/2)*150*.3}25 +

{.015*.3*23.5}]*(1/.25)

=7.053KN/m^2

Live Load =3 KN/m^2

Factored design load= 1.5*7.055= 15.08 KN/m^2

Step 3: Checking (Mu,d req)

Design moment = M= WL^2/10=15.08*(3)^2/10 = 13.572 KNm

But Mu= .138 fck*b*d^2

if d=129mm remaining (Mu)avail = 68.89 KNm > 13.572 KNm

Hence OK

Step 4: Calculation of reinforcement

Mu/bd^2 = 13.572*10^6/1000*124^2 = 0.88

Mu = .36 fck bx(d-.42x)

Solving above eqn

x/d = 1.2 -((1.2)^2 -(6.61Mu/fck*b*d^2))^0.5

=1.2-((1.2)^2 -(6.61*.88)30)^0.5 = 0.084

x=0.084d = 10.416mm

Lever arm =z=d-.42x =119.625mm

Mu = .82 fy*As*x^2

13.522*10^6 =.87*500*As*119.625

As=400 mm^2/m width

Width of staircase considered 1.5m

Total As(req) =1.5*400=600mm^2

We considered 12 dia bars

so Total no of bars= 600/(pi/4)*12^2 = 6 bars

Distribution reinforcement =.12*1000*1.5/100

=180 mm^2/m width

Assuming 8 dia to be used

as=(pi/4)*8^2 = 50mm^2

Total steel required = 180*3750=675 mm^2 lover 3750mm length

Total no of rods= 675/50 = 14 number

Spacing b/w bars = 3750/14 = 270mm c/c

Design of Shear wall

Shear walls provide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces the lateral sway of the building and thereby reduces damage to the structure and its contents.

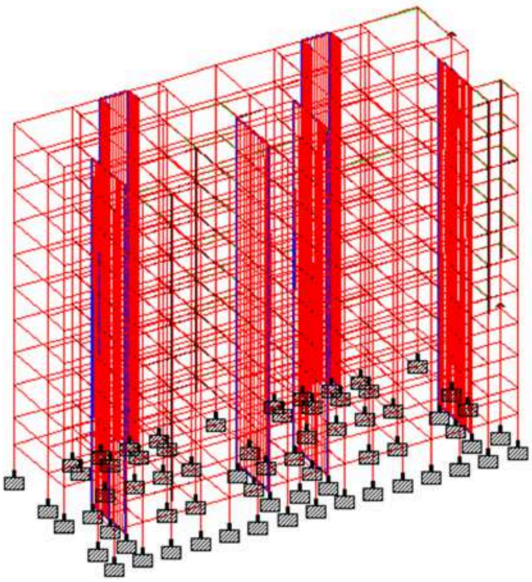
Deflection before adding the shear wall

	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	839	26 1.5 DL + 1	24.848	-9.101	-9.580	28.143	0.000	-0.000	-0.001
Min X	792	27 1.5 DL -1.	-24.803	-9.100	-9.533	28.087	0.000	0.000	0.001
Max Y	748	10 EQ -Z	0.000	0.798	-26.835	26.847	-0.000	0.000	-0.000
Min Y	739	11 1.5 (DL+L	0.029	-22.780	-2.690	22.939	-0.003	0.000	-0.000
Max Z	782	22 0.9 DL + 1	-0.007	-5.690	41.346	41.736	0.001	-0.000	-0.000
Min Z	852	31 1.5 DL - 1.	-4.618	-4.241	-46.998	47.414	-0.001	0.000	0.000
Max rX	640	30 1.5 DL + 1	-4.241	-9.435	34.337	35.861	0.002	0.000	-0.000
Min rX	739	11 1.5 (DL+L	0.029	-22.780	-2.690	22.939	-0.003	0.000	-0.000
Max rY	851	14 1.5 (DL -	-23.853	-5.185	-6.649	25.299	-0.000	0.000	0.001
Min rY	785	24 1.5DL + 1.	23.999	-5.026	1.932	24.596	0.000	-0.000	-0.001
Max rZ	845	11 1.5 (DL+L	-0.124	-12.291	-3.152	12.689	-0.000	-0.000	0.001
Min rZ	789	11 1.5 (DL+L	0.202	-12.293	-3.103	12.680	-0.000	0.000	-0.001
Max Rs	810	29 1.5 DL - 1.	4.871	-19.647	-45.935	50.197	-0.002	-0.000	-0.001

Maximum Drift Calculation

Max deflection (δ) = 50.197 mm
Height of building (h) = 36 mm
Max drift = δ/h
= 50.197/36000
= 0.139%
As per Cl. 11.1 (IS 1893:2002), Max drift ≤ 0.4%
No shear wall is required in the building against deflection.
Still, shear walls are provided adjacent to lifts and a few other places.

Shear wall Placement



Deflection after adding the shear wall

			Horizontal	Vertical	Horizontal	Resultant	Rotational		
	Node	L/C	X mm	Y mm	Z mm	mm	rX rad	rY rad	rZ rad
Max X	851	26 1.5 DL + 1	24.440	-5.683	-6.157	25.836	0.000	-0.000	-0.001
Min X	785	27 1.5 DL -1.	-23.843	-5.505	-3.756	24.757	0.000	0.000	0.001
Max Y	790	10 EQ -Z	-0.034	1.339	-15.267	15.325	-0.000	0.000	0.000
Min Y	739	11 1.5 (DL+L	-0.088	-21.063	-0.470	21.068	-0.005	0.000	-0.000
Max Z	786	28 1.5 DL + 1	5.029	-5.854	24.314	25.509	0.000	0.000	-0.000
Min Z	850	29 1.5 DL - 1.	4.764	-6.530	-26.109	27.331	-0.002	0.000	0.000
Max rX	831	11 1.5 (DL+L	-0.137	-15.509	-0.805	15.530	0.002	0.000	-0.000
Min rX	810	11 1.5 (DL+L	-0.140	-20.630	-0.454	20.635	-0.005	0.000	-0.000
Max rY	785	29 1.5 DL - 1.	4.549	-4.363	-20.557	21.501	0.000	0.000	0.000
Min rY	801	14 1.5 (DL -	-22.677	-2.166	-0.544	22.786	0.001	-0.000	0.000
Max rZ	825	11 1.5 (DL+L	-0.417	-8.358	-0.707	8.398	0.000	0.000	0.002
Min rZ	806	11 1.5 (DL+L	-0.372	-8.315	-0.150	8.324	0.000	0.000	-0.002
Max Rs	837	29 1.5 DL - 1.	4.878	-17.726	-25.256	31.239	-0.000	0.000	0.001

Max deflection after adding – 31.239 mm

Foundation Design

Using IS 2911-2010 Part 1

We have for node 143,
Load= 3986.222KN
pile capacity =750 KN
Assume group efficiency of 90%
Number of piles, n= 3986.222/(750*0.9)=5.90

Provide 6 piles
Now centre to centre distance between pile=3D
take D= 500mm
c/c distance=1.5m
Now edge distance=0.5m

Choosing 2 rows x 3 Columns arrangement:
Thus length of pile cap = (1.5*2)+(0.5*2)= 4m
Width of pile cap= (1.5*1) + (0.5*2)= 2.5m

PILE ARRANGEMENT

Column Dimensions

Column Shape : Rectangular
Column Length - X (Pl) : 0.750 m
Column Width - Z (Pw) : 0.400 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 $\mathbf{P_{CL}}$ = 4.000 m
Pile Cap Width $\mathbf{P_{CW}}$ = 2.500 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 $\mathbf{e}$ = 0.500 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 $\mathbf{f'_c}$ = 25000.004 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

Reaction: Fx=0.15 Fy=242.49 Fz=-20.57 Mx=-36.11 My=0.01 Mz=-0.27
Reaction: Fx=2.14 Fy=-112.02 Fz=-16.64 Mx=-32.77 My=-0.02 Mz=-1.57
Reaction: Fx=1.23 Fy=-160.29 Fz=-23.62 Mx=-38.96 My=0.07 Mz=-1.12
Reaction: Fx=0.80 Fy=57.50 Fz=-17.61 Mx=-33.67 My=0.01 Mz=-1.40
Reaction: Fx=1.42 Fy=-107.73 Fz=-18.04 Mx=-33.78 My=-0.01 Mz=-1.58
Reaction: Fx=-1.87 Fy=236.15 Fz=-18.07 Mx=-33.80 My=-0.03 Mz=1.48

Load Comb 101 : 1.000 x DL+1.000 x DL+1.000 x DL+1.000 x DL+1.000 x DL+1.000 x DL+1.000 x DL+1.000 x D
Load Comb 102 : 0.800 x DL+0.800 x DL+0.800 x DL+0.800 x DL+0.800 x DL+0.800 x DL+0.800 x DL+0.800 x D
Load Comb 201 : 1.500 x DL+1.500 x DL+1.500 x DL+1.500 x DL+1.500 x DL+1.500 x DL+1.500 x DL+1.500 x D
Load Comb 202 : 0.960 x DL+0.960 x DL+0.960 x DL+0.960 x DL+0.960 x DL+0.960 x DL+0.960 x DL+0.960 x D
Load Comb 203 : 1.200 x DL+1.200 x DL+1.200 x DL+1.200 x DL+1.200 x DL+1.200 x DL+1.200 x DL+1.200 x D
Load Comb 204 : 0.900 x DL+0.900 x DL+0.900 x DL+0.900 x DL+0.900 x DL+0.900 x DL+0.900 x DL+0.900 x D

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	-19.246	-2656.479	-17.081	-18.159	0.000	20.250
102	-15.397	-2125.183	-13.665	-14.527	0.000	16.200
201	-28.869	-3984.718	-25.621	-27.238	0.000	30.374
202	-18.476	-2550.220	-16.398	-17.433	0.000	19.440
203	-23.095	-3187.774	-20.497	-21.791	0.000	24.299
204	-17.322	-2390.831	-15.373	-16.343	0.000	18.225

PILE CAP DESIGN CALCULATION

Pile Reactions

Total pile number **N** = 6

	Arrangement		Reaction		
Pile No.	X (m)	Y (m)	Axial (kN)	Lateral (kN)	Uplift (kN)
1	-1.500	-0.750	-692.987	8.135	0.000
2	-1.500	0.750	-674.411	8.135	0.000
3	0.000	-0.750	-686.175	8.135	0.000
4	0.000	0.750	-667.598	8.135	0.000
5	1.500	-0.750	-679.362	8.135	0.000
6	1.500	0.750	-660.786	8.135	0.000

Reinforcement Calculation

Maximum bar size allowed along length # 32

Maximum bar size allowed along width # 20

Bending Moment At Critical Section = -1538.295 kNm (Along Length)

Bending Moment At Critical Section = -1132.168 kNm (Along Width)

Pile Cap Thickness **t** = 1.219 m

Selected bar size along length # 12

Selected bar size along width # 12

Selected bar spacing along length = 66.33 mm

Selected bar spacing along width = 77.76 mm

Pile Cap Thickness Check

Calculated Thickness (t) = 1.219 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	-779.597	0.000
2	-758.698	0.000
3	0.000	0.000
4	0.000	0.000
5	0.000	-764.269
6	0.000	-743.370

Effective Depth(d_{ef}) = $h_{cap} - (p_{id} + cc + 0.5 \times d_b)$ = 1.088 m

Depth of neutral axis for balanced section(x_u) $\frac{700 \times d_{eff}}{1100 + 0.87 \times f_y}$ = 0.521 m

As Per IS 456 2000 ANNEX G G-1.1 C

Ultimate moment of resistance(M_{ulim}) = $0.36 \times f_c \times b \times X_u \times (d_{eff} - 0.416 \times X_u)$ = 10217.168 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	-241.535	0.000
2	-235.137	0.000
3	0.000	0.000
4	0.000	0.000
5	0.000	-236.842
6	0.000	-230.444
TOTAL	-476.672	-467.287

Design Shear Force for One-Way Action V_u = -476.672 kN

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

Design Shear Stress (T_v) = $\frac{V_u}{B \times d}$ = -175.247 kN/m^2

Allowable Shear Stress (T_c) = $\frac{0.85 \times \sqrt{0.8 \times f_c}}{6 \times \beta} \times \left(\sqrt{1 + 5 \times \beta} - 1\right)$ = 276.952 kN/m^2

Where Beta = $\max\left(\frac{0.8 \times f_c}{6.89 \times pt}, 1\right)$ = 21.590

and percentage of steel required (p_t) = $\frac{A_{st}}{B \times d} \times 100$ = 0.134

Here $T_v <= T_c$ Hence safe

Calculation of Maximum Bar Size

Along Length

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) = $\frac{0.87 \times d_b \times f_y}{4 \times \tau_{bd}}$ = 1.289 m

Allowable Length(l_{db}) = $0.5 \times (B - b) - C_s$ = 1.575 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}) = $0.12\% \times B \times h_{cap}$ = 3627.000 mm2

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

Area of steel required (A_{sq}) = $0.5 \times \left(\frac{f_c}{f_y}\right) \times \left(1 - \sqrt{1 - \frac{4.5977 \times M_u}{f_c \times b \times d \times d}}\right) \times b \times d$ = 4093.596 mm2

Area of steel provided (A_{st}) = = 4093.596 mm2

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

Minimum spacing allowed (S_{min}) = 40 + d_b = 52 mm

Selected spacing (S) = 66.33 mm

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

Similarly, all the calculations are done along width too for the pile cap design

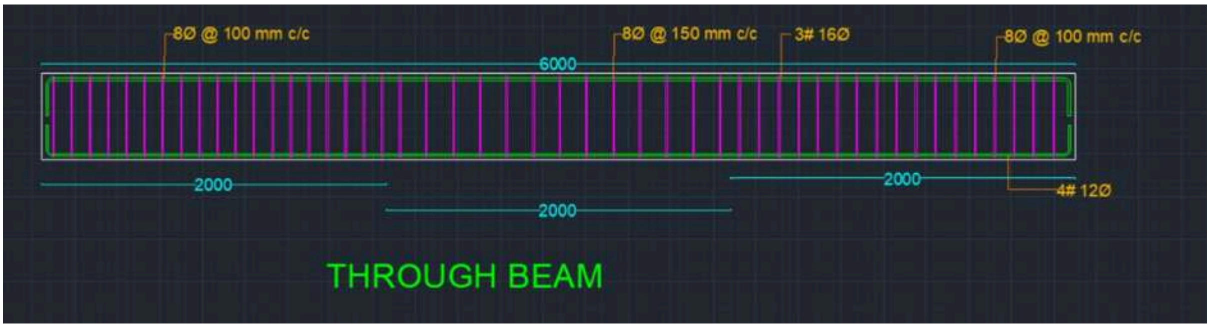
Pile Design

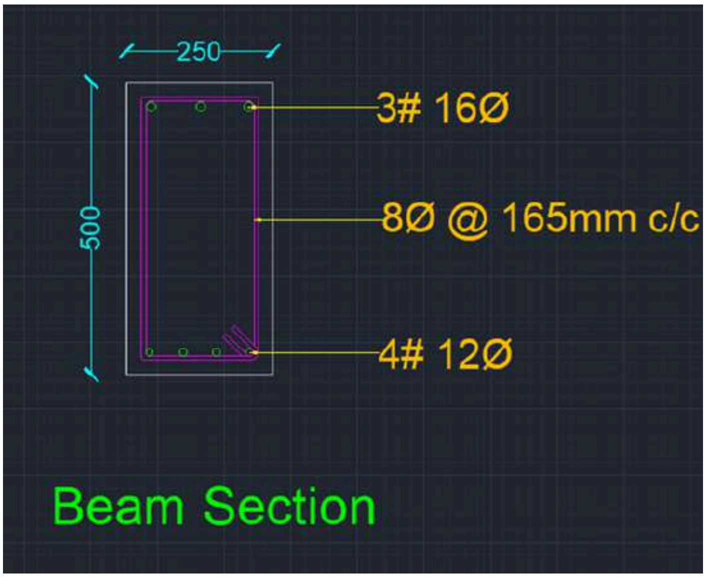
D=0.5m
Q design = 750 KN
Taking Factor of Safety=2.5 as per Cl.B-5 (IS 2911 (Part 1/Sec 1) : 2010)
Q ultimate = 2.5*750= 1875 KN
C = qu/2 = 20 ton/m^2/2 = 10 ton/m^2 = 99.64 KN/m^2
q tip = c*Nc= 10*9=90 ton/m^2= 896.76 KN/m^2
Q tip= q tip*(pi/4)*D^2
=896.76*(3.14/4)*0.5^2= 175.98 KN
Q friction= Q ult - Q tip
= 1875-175.98= 1699.02 KN
Q friction = α*C*(pi)*D*L
1324.02 = .45*99.64*3.14*.5*L ([α obtained from Fig.2
IS 2911 (Part 1/Sec 1) : 2010]
=> L= 24.135 m

Therefore length of Pile is taken as 25m

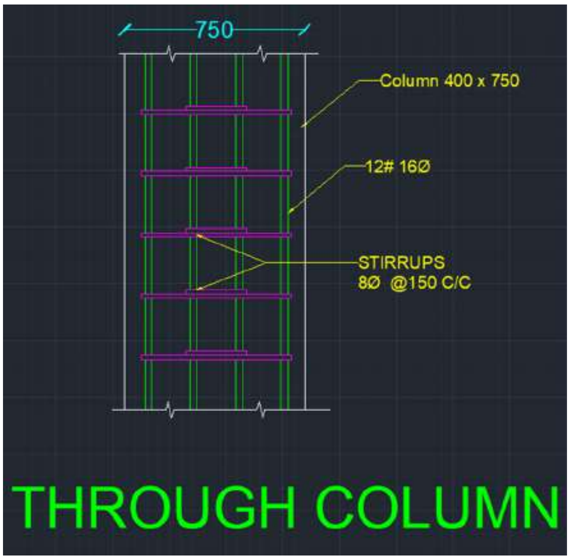
Reinforcement Detailings

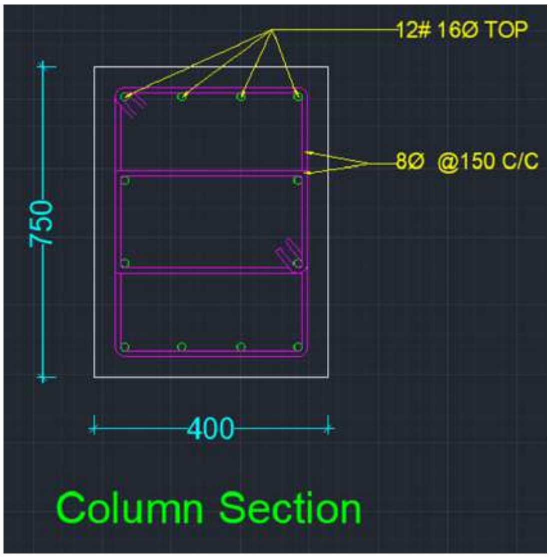
Beam Reinforcement Detailing



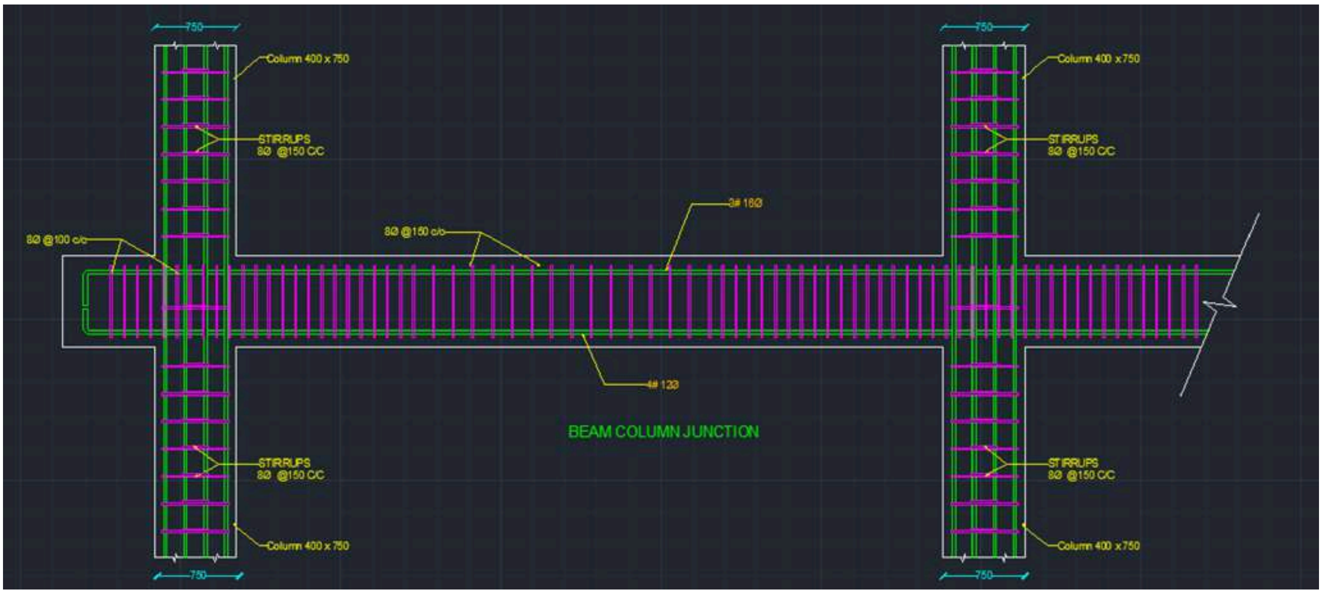


Column Reinforcement Detailing

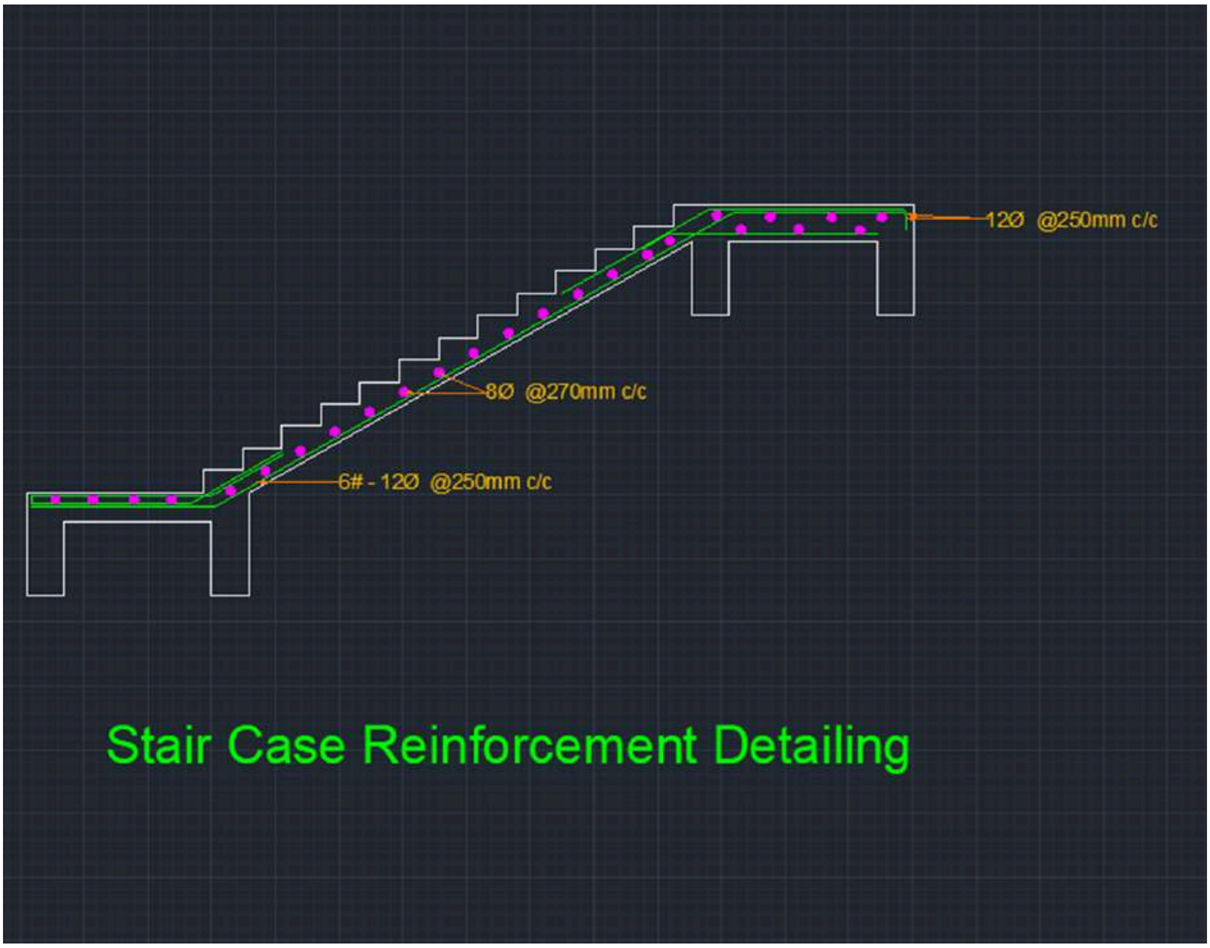




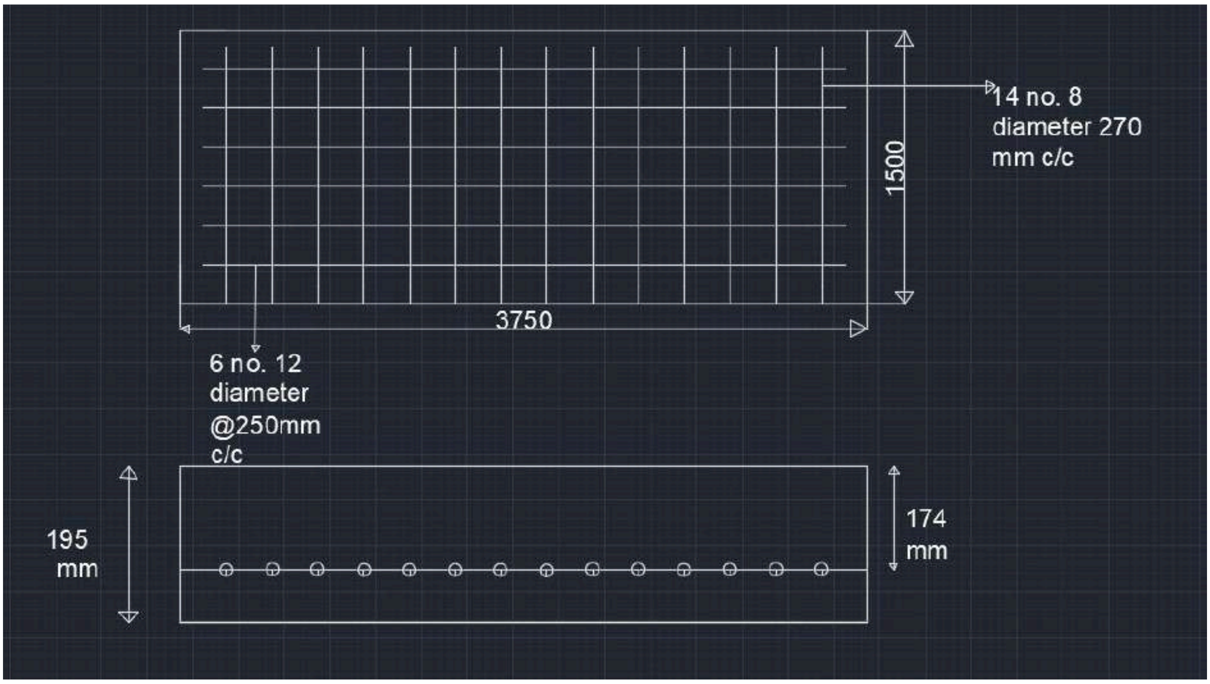
Beam-Column Junction Reinforcement detailing



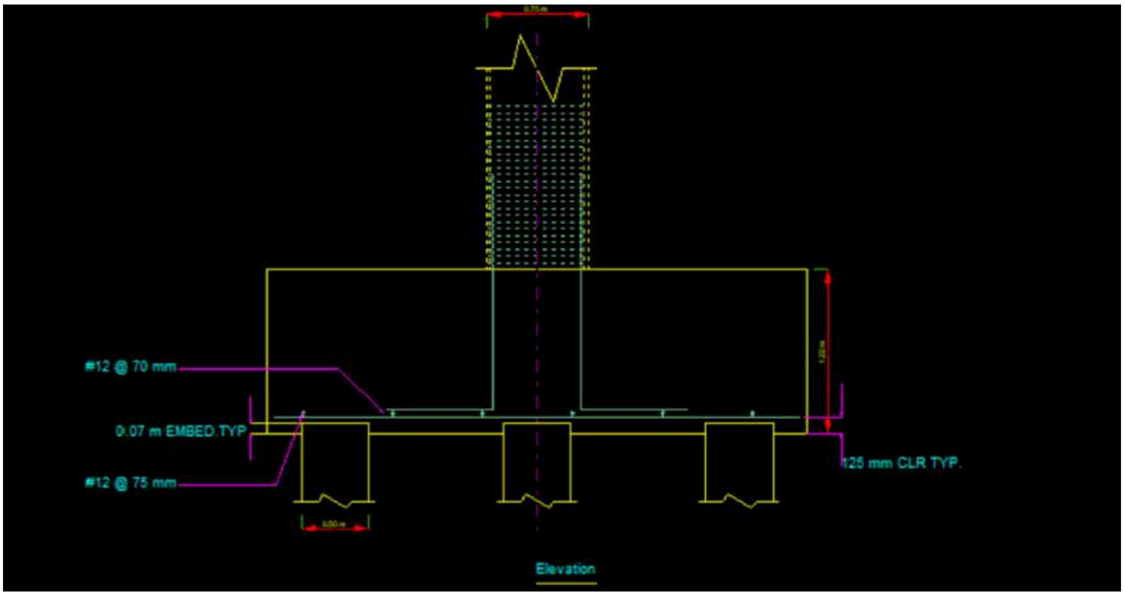
StairCase Reinforcement Detailing

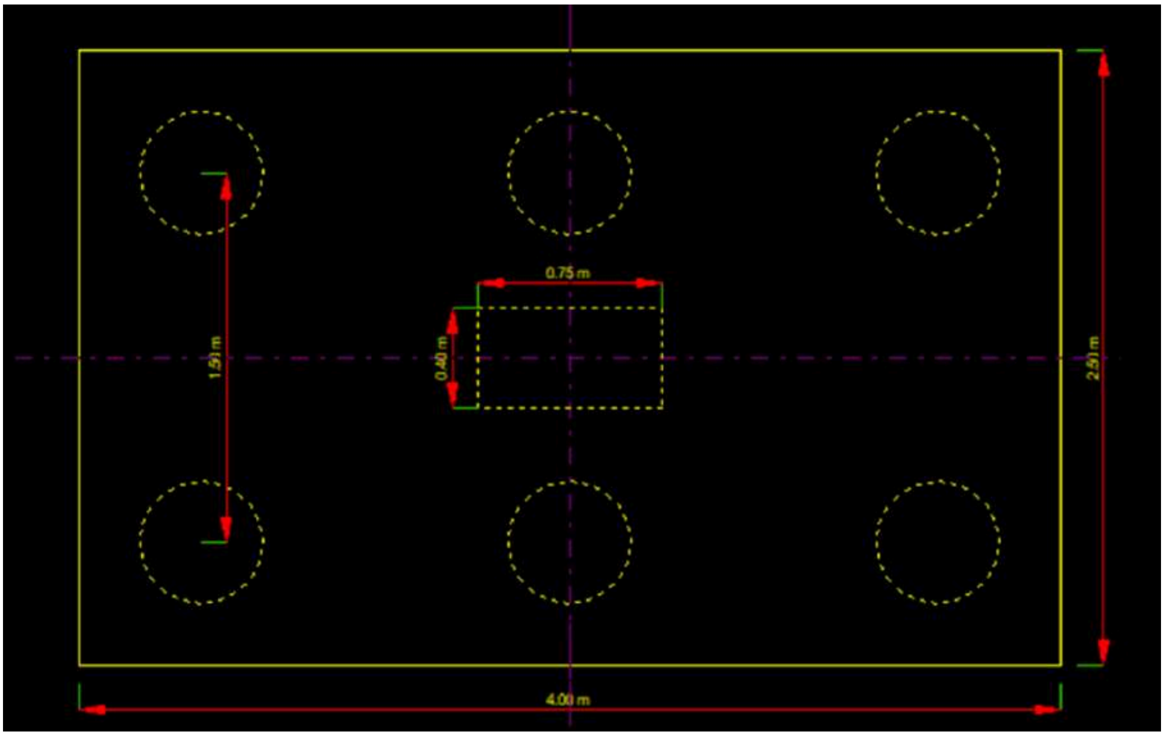


Slab reinforcement detailing



Pile Cap Detailing





WATER SUPPLY DESIGN

Water demand as per National Building Code 1983 = 135 LPCD
There are 4 Apartments on each floor, hence,
Total Population = 10*4*5 = 200 (assuming average family size to be 5)
Considering Water Demand as 140 LPCD
Total water demand = 140*200= 28,000 litres/day
Average rate of supply = 28,000 litres/day
Hourly Demand = 1166.67 litres/hr = 1200 litres/hr
(Considering water losses in pipes)
Assuming the pump works for 8 hours (4 h morning + 4 h evening) per day to fill overhead tank,
Rate of pumping = 28,000/8 = ≈ 3500 L per hour

OVERHEAD TANK CAPACITY

According to SP35 Table 15,
Capacity of storage reservoir = maximum deficit + maximum surplus
= 2.40 + 7.0a = 9.4a
= (9.4*2*1200) = 22,560 L

According to SP35 Table 16,
Storage of overhead tank according to number of people = 70 Litres per person x Number of inhabitants = 70 x 200 = 14,000 L

According to SP35 Table 17,
Minimum storage needed for flushing
= number of floors x (270 L for 1 toilet + (180 L per secondary toilet(s)))
= 10 floors x ((270 x 8) L + (180 x 0) L)
= 21,600 L

As per clause 5.4.2.3 of SP35,
Maximum Storage=0.5 x Daily Supply
=0.5 x 28000
=14000 L
Storage of Overload tank=Max(22560 , 14000, 21600, 14000)=22,560 L

Required Storage Capacity of overhead tanks = 22,560 L

OVERHEAD TANK DESIGN

Dimensions of Storage Tank on roof Diameter= 2 m, Depth = 1.8 m
Volume of a Single Storage Tank (VT) = (3.14 x 2x2 x 1.8 x 1000/4) = 5,652 L
Number of tanks required on Roof = Required Storage / VT
= 22,560 / 5652
= 3.99

Provide 4 tanks with each block having 1 tank.

DESIGN OF WATER SUPPLY PIPES FROM TANKS(FTA)

REF- SP35

The sanitary fixtures in each apartment consist of the following:
a) 1 sink and 1 tap in the kitchen
b) 1 overhead flushing tank for water-closet and tap in each water closet room
c) 1 shower, a tap and 1 wash basin in each bathroom

SL.no	Description	No.of Fixtures per flat
1	Kitchen Sink	2
2	Kitchen tap	2
3	Ablution tank	2
4	Supply to overhead flushing tank	1
5	Shower	2
6	Tap	2
7	Wash Basin	2
	Total	13

DESIGN OF SEWERAGE PIPE SYSTEM

Two pipe systems are provided.
Soil pipes are connected to the Building drain while waste pipes are connected using a trapped gully.
MVP = Main Ventilation Pipe 50 mm
MWP = Main Waste Pipe 100 mm
MSP = Main Soil Pipe 100 mm

From Horizontal Branches for each floor Table 52 SP35 (Not including sewerage pipes)

MWP - 100 mm pipe diameter is connected for branching
MSP - 75 mm pipe diameter is connected for branching

Waste Appliances used Table52 SP35

Pipe type	Diameter
Main	35 mm
Susidiary	25mm
Distribution	15mm

Waste Appliances used Table 54 SP35

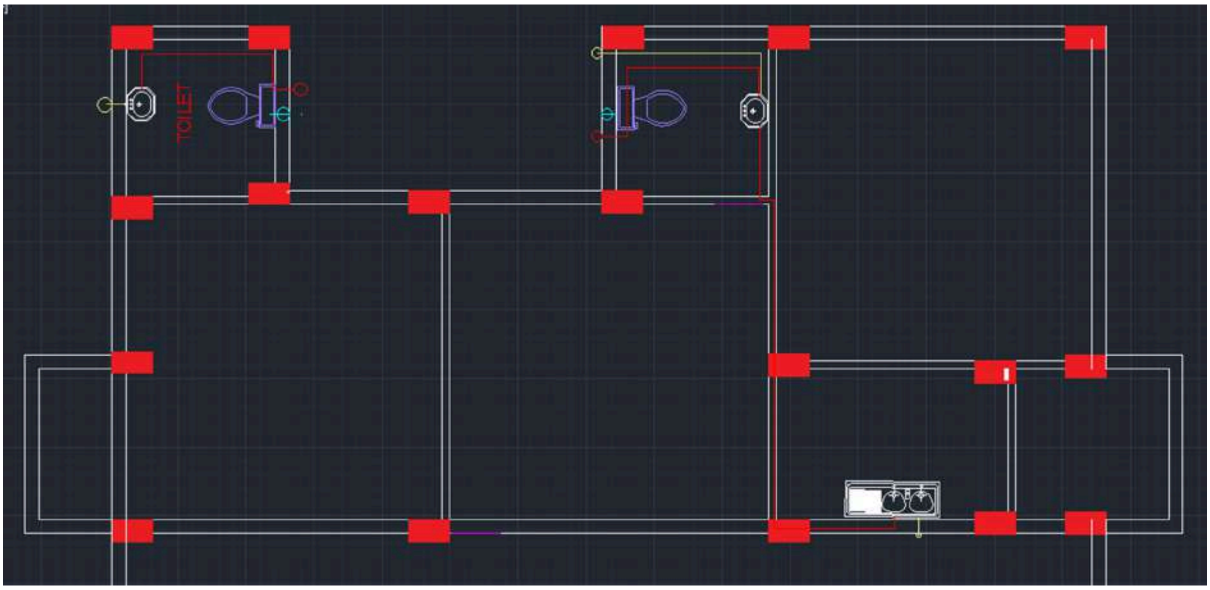
Waste Appliance	Internal Diameter (mm)
Wash Basin	30
Domestic Sink	40
Bathrooms(Showers)	40
Urinal Floor Trap	65

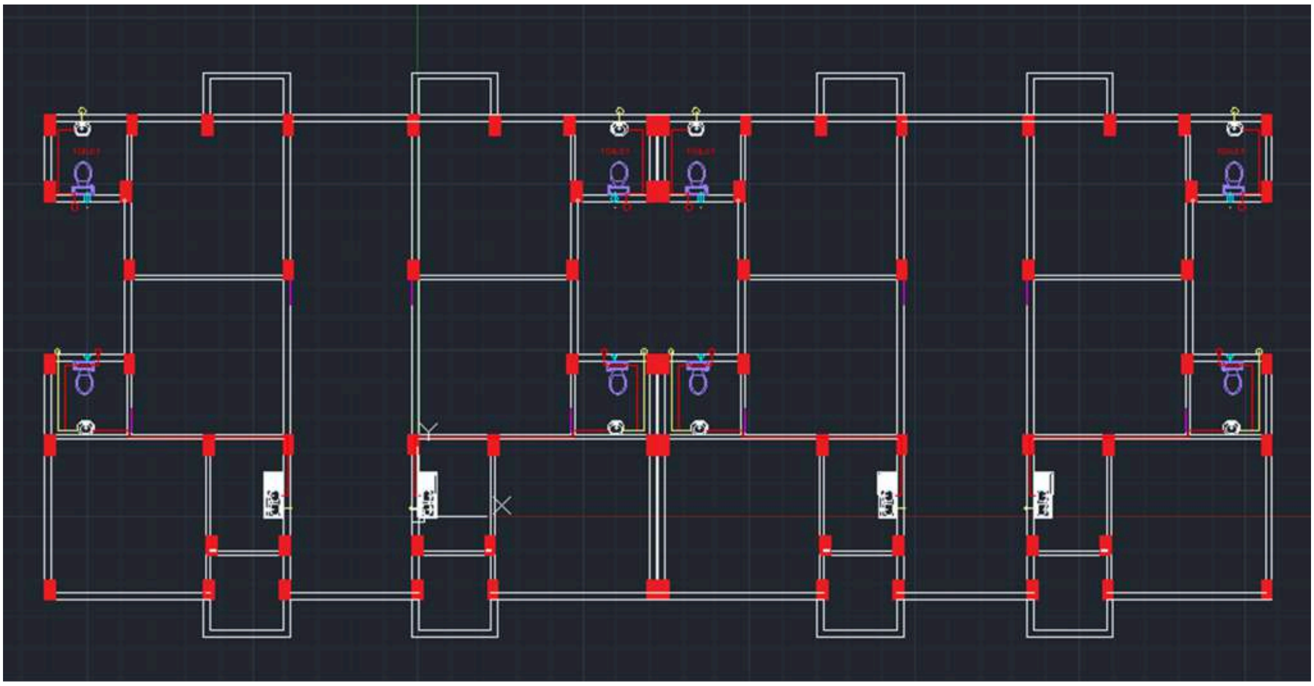
DESIGN OF SEWER PIPE

We have total water demand calculated in the previous section= 28,000 L/day
Now assume 90% of total consumption to reach sewer
Here, peak factor=3(since population <20,000)
>Total storage=28,000*0.9*3 L/day = 75,600 L/day
= 875 cm^3/s
Now using Mannings' formula $Q = AR^{2/3}\sqrt{s}/n$
Take slope of main sewer pipe= 1/1000
Mannings' coefficient=0.012
On Calculating we get, d=13.668 cm= 136mm

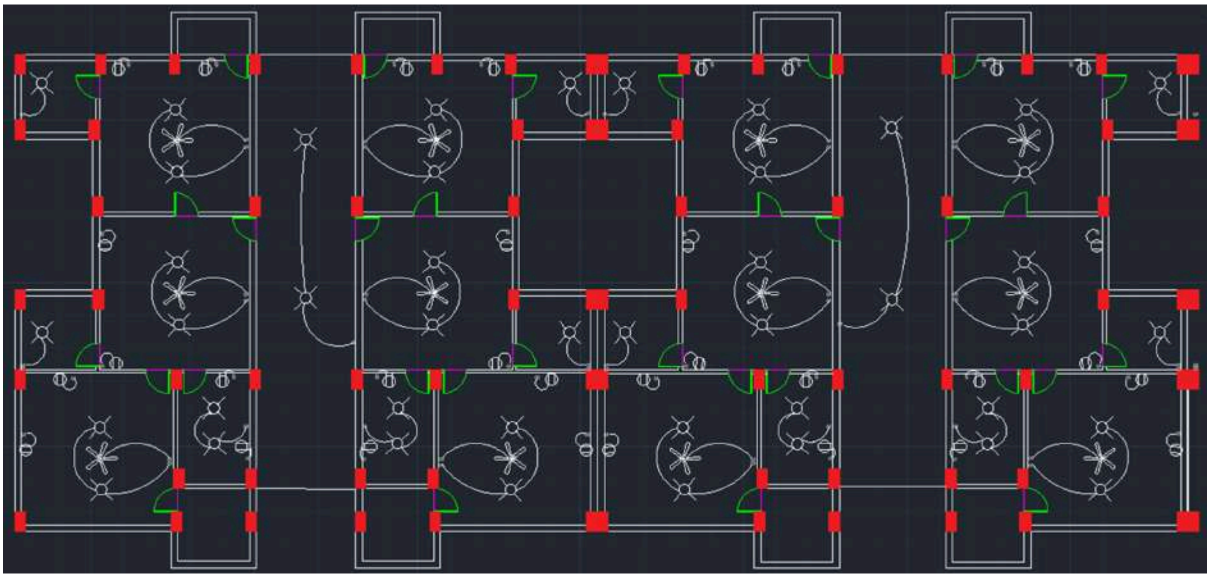
Provide 140 mm diameter pipe for the sewer pipe connecting the pipes from sewage from all the flats to the municipal sewage system.

Plumbing network



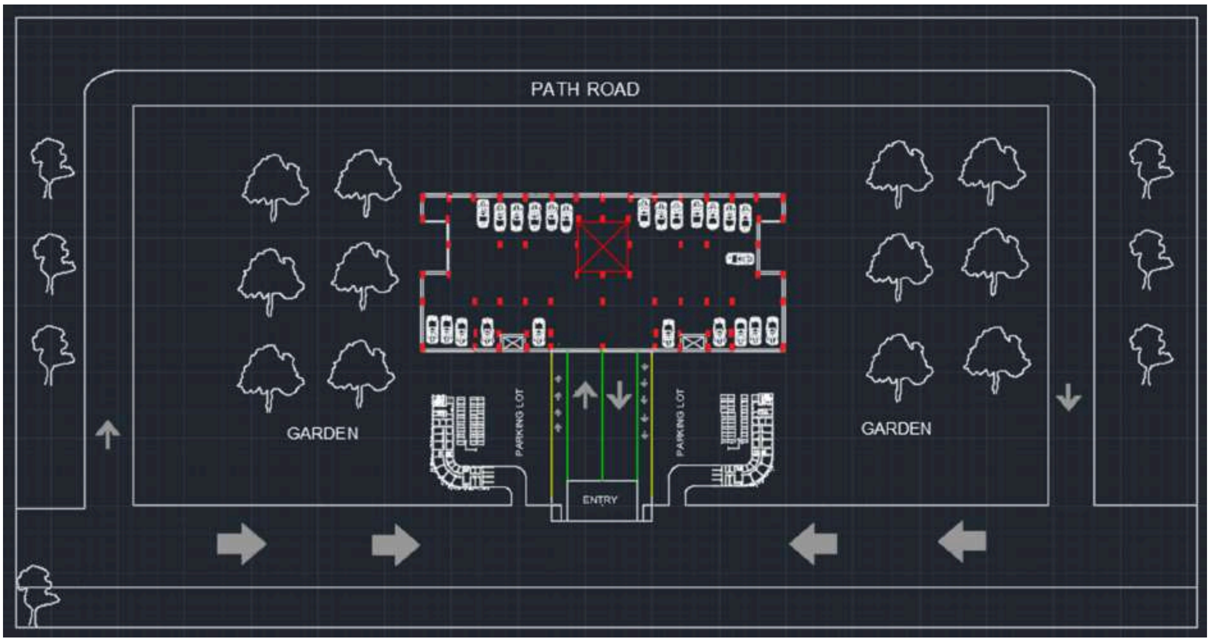


Electrical Wiring Network



GRAPHIC SYMBOL	DESCRIPTION
	LIGHT
	FAN
S	SWITCH
	SOCKET

Road Network



Cost Estimation

Column:
No. of columns each floor: 67

Volume of columns= 67*2*2.7*0.8*0.4+67*7*3.4*0.6*0.35+67*2*3.4*0.8*0.4
Total volume= 596.428 m^3

Concrete volume = 0.98*586.428=584.49 m^3
Steel Volume= 0.02*596.428= 11.928 m^3

Slabs:

Total slab area one floor= 500 m^3
Thickness=0.195 m
Volume of slab per floor= 0.195*500= 97.5 m^3

Total volume of slab= 11*97.5=1072.5 m^3

Volume of concrete= 1072.5*0.9915= 1063.383 m^3
Volume of steel= 1072.5*0.0085= 9.116 m^3

Beams:

Volume of beams per floor= 394*0.5*0.25=49.25 m^3

Volume of beam in whole building 11*49.25=541.75 m^3

Volume of concrete=0.985*541.75=533.62 m^3
Volume of steel=0.015*541.75=8.1265 m^3

Bricks:

Volume of 1 brick= 250*100*100= 0.0025 m^3
Volume of exterior wall on one floor= 229*3.4*0.25= 194.65 m^3
Volume of interior walls on one floor= 165*3.4*0.2= 112.2 m^3

For ground and basement
Volume of walls= 229*2.7*0.25*2+58*2.7*0.2=340.3 m^3
Total walls in structure=2761+340=3101.0 m^3

No. of bricks used= 3200/0.0025=1280000 bricks

Taking buffer, Total used bricks= 1400000

Final calculation:
Total volume of concrete in structure= 2181.493 m^3
Cost of concrete=2181.4*4000=8725972 Rs
Total volume of steel in structure= 29.1705 m^3
Cost of steel= 7850*65.5*29.1705=14998484.75 Rs

Cost of bricks=1400000*5= 7000000 Rs

Materials	**Quantity**		**Standard Price(Rs.)**		**Price (Rs.)**
Concrete	2181.4	m^3	4000	per m^3	8725972
Steel	136.5	tons	85000	per tonn	11602500
Bricks	1741390	pieces	8	per piece	13931120
Tiles	76100.85	sqft	50	per sqft	3805042.5
Plastering	96068	sqft	90	per sqft	8646120
Painting Exterior	99587.7	sqft	35	per sqft	3485569.5
Painting Interior	38750.08	sqft	120	per sqft	4650009.6
Putty	12255	Kg	50	per kg	612750
Primer	1200	liters	200	per liter	240000
				Total Cost	55699083.6
				Cost per sq.m	10,820