Indian Institute of Technology Gandhinagar

Comprehensive Project Course
CE 401

Mid-Semester Project Report

Y-Shape Building with Normal Strength Concrete (M25)

Submitted to

Dr. Dhiman Basu
Professor, Civil Engineering

Submitted by: Group 3

Group members

Jyoti Prakash	**21110090**
Alok Kumar Vidyarthi	**21110019**
Anjana S A	**21110025**
Akshith Goud	**21110121**
Maloth Charan Tej	**21110116**
Pasupula Abhiteja	**21110153**
Swati Rawat	**20110209**

Under the Guidance of

Prof. Dhiman Basu

Prof. Gaurav Srivastava

Prof. Manish Kumar

Prof. Udit Bhatia

Prof. Sameer Patel

Prof. Pranab Mohapatra

Prof. S.R. Gandhi

Prof. Amit Prashant

Prof. Sushobhan Sen

Prof. Sudhir Kumar Arora

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ABSTRACT

The structural design of a Y-shaped four-storey reinforced concrete building utilizing M25 grade normal-strength concrete. The structure features a unique architectural layout with three outer blocks designed with flat slab systems for community facilities and a sports club. The central core, symmetrical along three axes, connects the outer blocks and serves as vertical circulation. The project involves multifaceted engineering processes, beginning with a geospatial survey using advanced GIS tools to model the site's Digital Elevation Model (DEM) and calculate cut-and-fill requirements for ground levelling. Hydrological studies, including flood risk mapping of the nearby Sabarmati River, were conducted to determine a safe plinth level, incorporating a 30% safety margin above historical flood data.

Architectural design and building information modeling (BIM) were executed using Autodesk Revit, creating detailed 3D models. Structural analysis and member design were performed with SAP2000, addressing load combinations, seismic impacts, and wind effects in compliance with IS codes. Key components such as beams, columns, slabs, and foundations were analyzed for safety and serviceability under varied load scenarios. Special attention was given to non-rectangular slab systems, designed using yield line theory to optimize performance.

The geotechnical investigation ensured a safe bearing capacity exceeding the applied loads, validating the foundation design with minimal settlement. Fire safety, water supply systems, and electrical networks were also planned, aligning with national standards to meet functional and safety requirements. The final design prioritizes structural integrity, cost-efficiency, and environmental resilience. This project underscores the integration of theoretical knowledge and practical engineering, providing a holistic approach to real-world construction challenges.

INTRODUCTION

We had the opportunity to gain in-depth knowledge of several topics of building design by working on a large-scale civil engineering project. It aided in our comprehension of the significance of each stage. It also provides an opportunity to gain firsthand knowledge about real-world construction projects. We started our trip with this difficult endeavour, guided by our instructors and teaching assistants.

This project includes many crucial steps and tasks, such as determining the amount of soil that must be moved in order to level the ground, estimating the location of the building's base while considering flood, planning the building's appearance, structural design, and analysis of various components of building such as beams, columns, slabs, shear walls, flat slabs, footing, staircases. Throughout this semester in this project, we successfully executed several key tasks, including site surveys and hydrological analyses, structural analysis and design of various structural elements. We conducted load combinations and structural analyses in SAP and used Revit for structural design.

GEOSPATIAL SURVEY

We did the land survey of the Temporary Sports Field (TSF) on campus. We have used the total station instrument for doing the survey and a total of **43 points** were taken from the plot during the survey for the elevation dataset. Further, to process the elevation data, we have used the QGIS software which has helped us in making Digital Elevation Model (DEM), contours, profile plot and calculating cut and fill volume of the site.

Volume = -3672.92 m^3
Pixel count = 41426658
Area = 4142.09 m^2

As the elevation difference is very minimal between the points (as the site area was almost plain without ups and downs), we have taken a 0.01-**meter interval** between the two contours. We assumed **the base level as 73 meters**, then the required **fill volume obtained was 3672.92 m2** from the survey area of **4142.09** m^2.

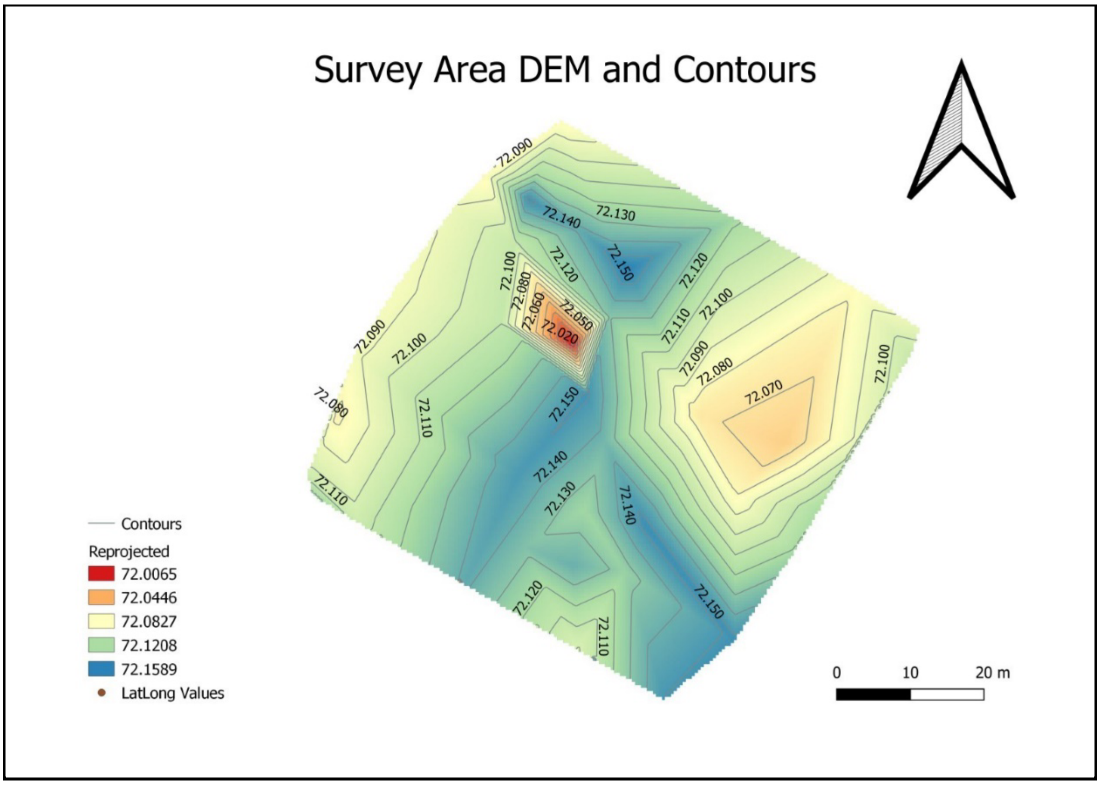


Figure 1: DEM profile showing contour lines of the survey region

Profile Plot

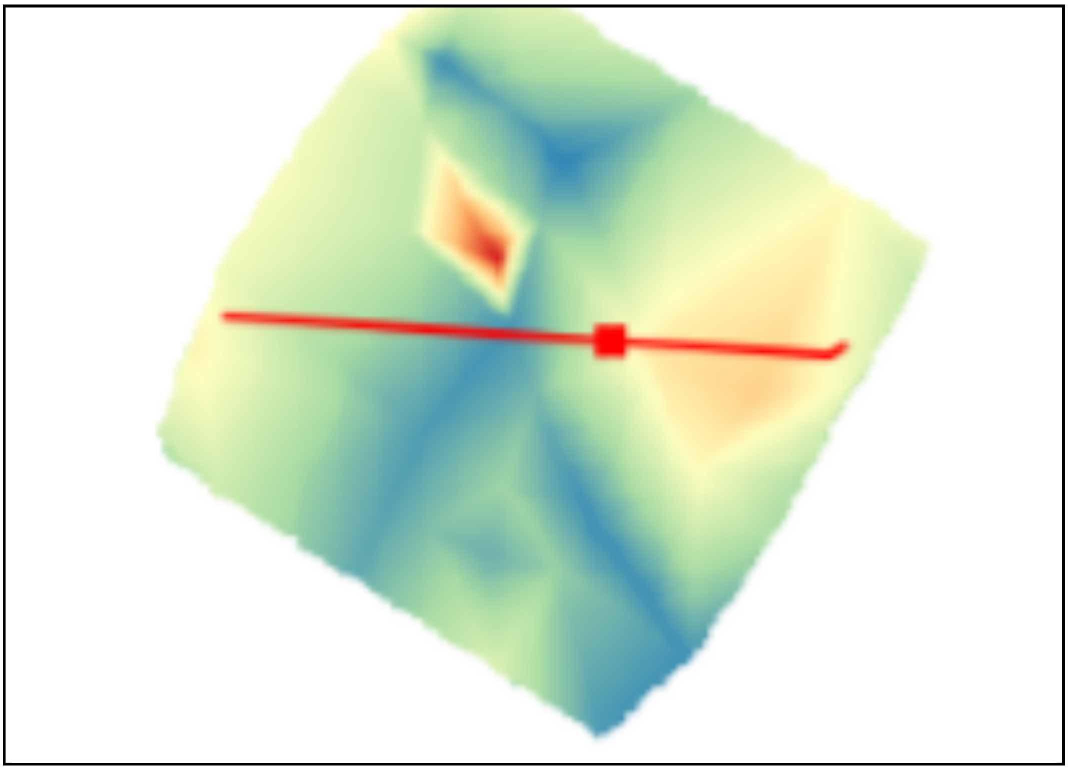


Figure 2: Profile plot which captures the depth differences using red line

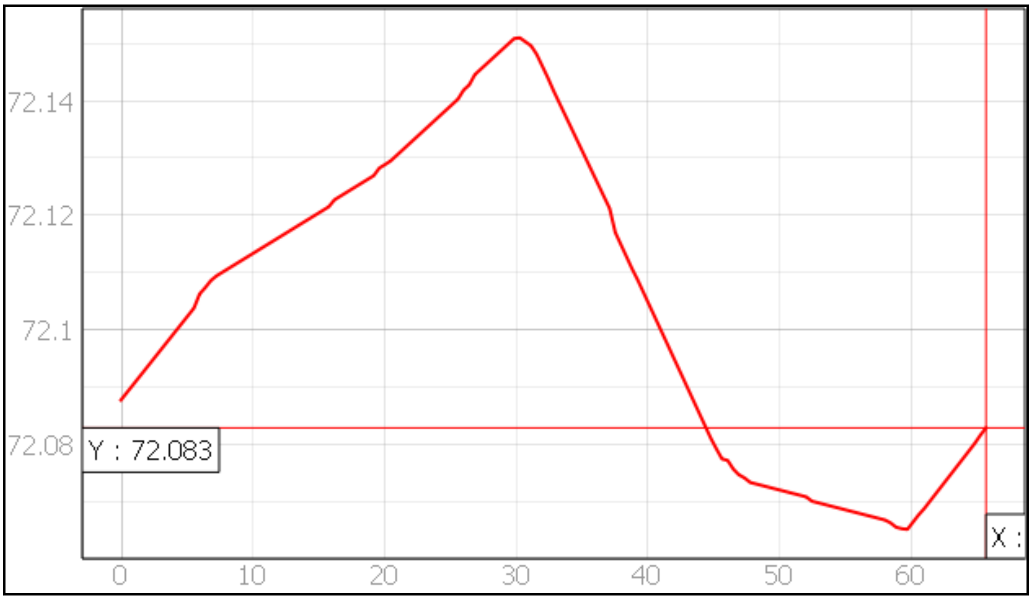


Figure 3: The graph depicts the depth variation of the red line defined in figure 2

Here, the Y axis shows the elevation in meters, and the X axis shows the distance in meters. The site is almost flat, and there is very **minimal change in the elevation** from one point to another.

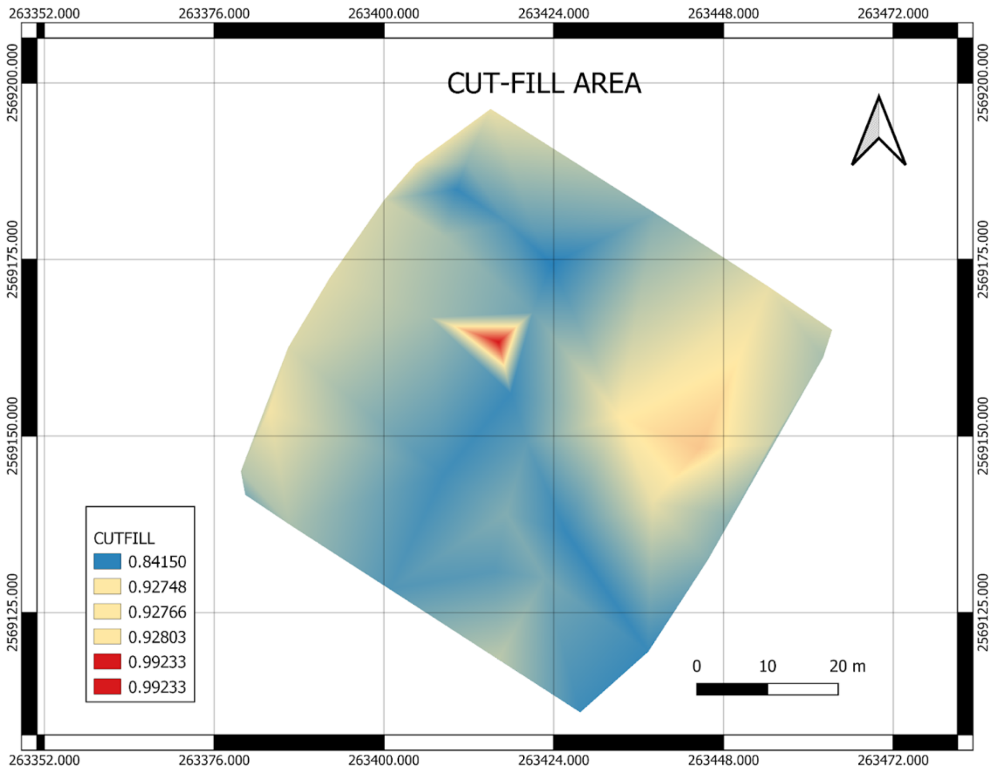


Figure 4: The cut fill volume of the site area

HYDROLOGICAL STUDIES

HEC-RAS Analysis

As our college is located on the banks of the Sabarmati River, it is crucial for us to analyze the risk of flooding due to the overflow of the river. The highest flow rate observed for the Sabarmati River was 10365 m3/s at the Gandhinagar Station in the year 1993. So, we did a steady flow analysis of the river at the highest flow rate of 10365 m3/s and found that our site of the building is safe from floods.

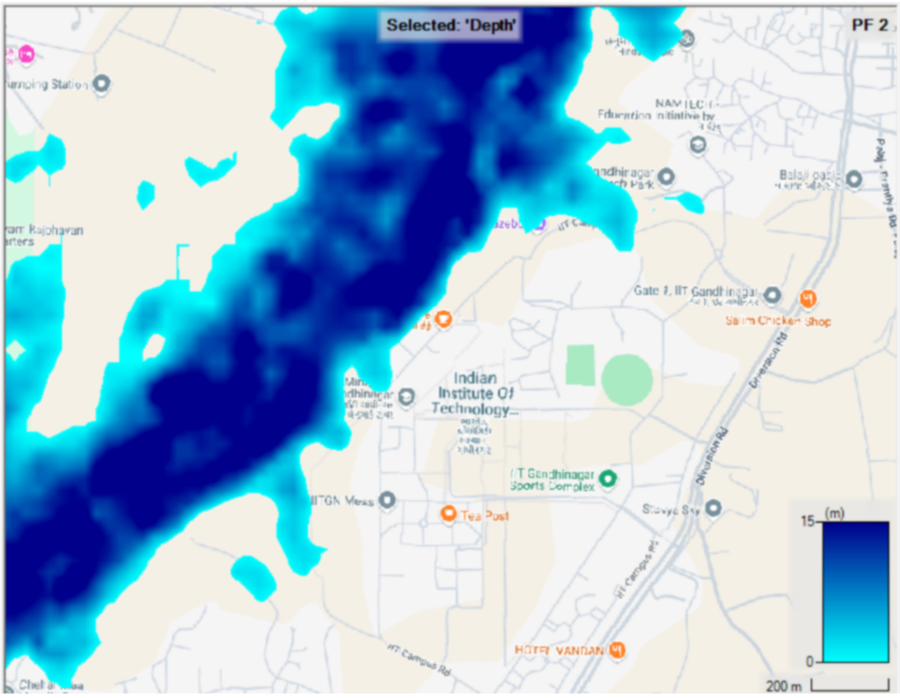


Figure 5: Flood mapping of Sabarmati river at steady flow rate of 10365 **m3**/s

The plinth level of the building is 0.45 meters as per NBC 2016, as our building lies in the plain area. Our site is not flood-prone, but still, for safety reasons, we are considering an extra margin of 30%, so our final plinth level will be 0.585 meters from the ground.

As the ground level of our site now from the mean sea level is 73 meters after cutting and filling, our final level of the building will be 73.585 meters from the mean sea level.

Precipitation Data Analysis

The monthly precipitation data for Gandhinagar from 1970 to 2020 was obtained from the Climate Engine website.

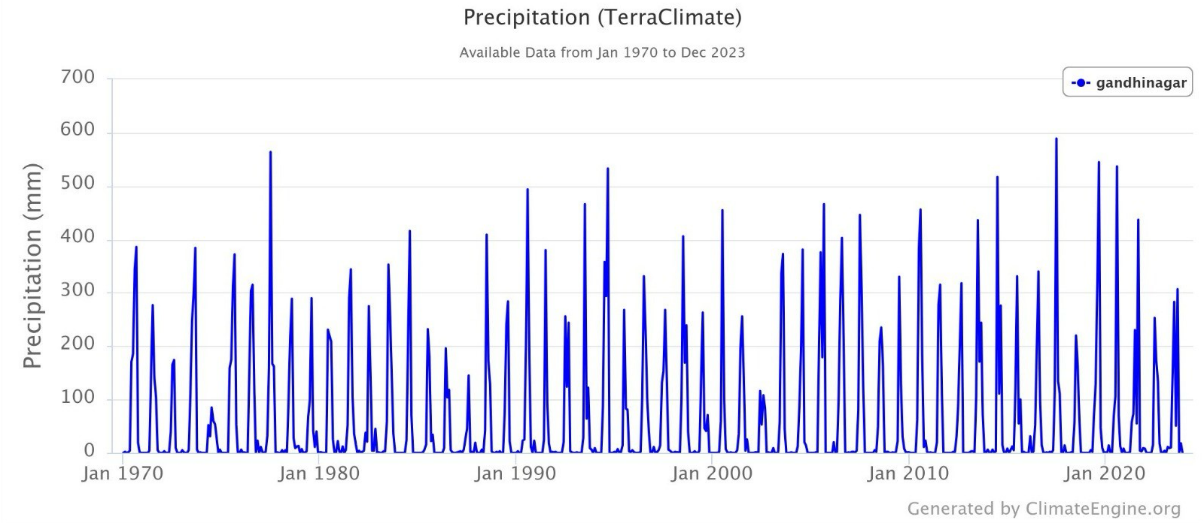


Figure 6: 50 year Monthly precipitation Vs time graph

Analysis showed that the probability of experiencing continuous rainfall for more than three consecutive days is quite low, especially given that our site is located in an area not prone to frequent flooding or heavy rainfall. After reviewing 50 years of data, we discovered that the maximum total rainfall recorded over a cumulative three-day span was 368.39 mm.

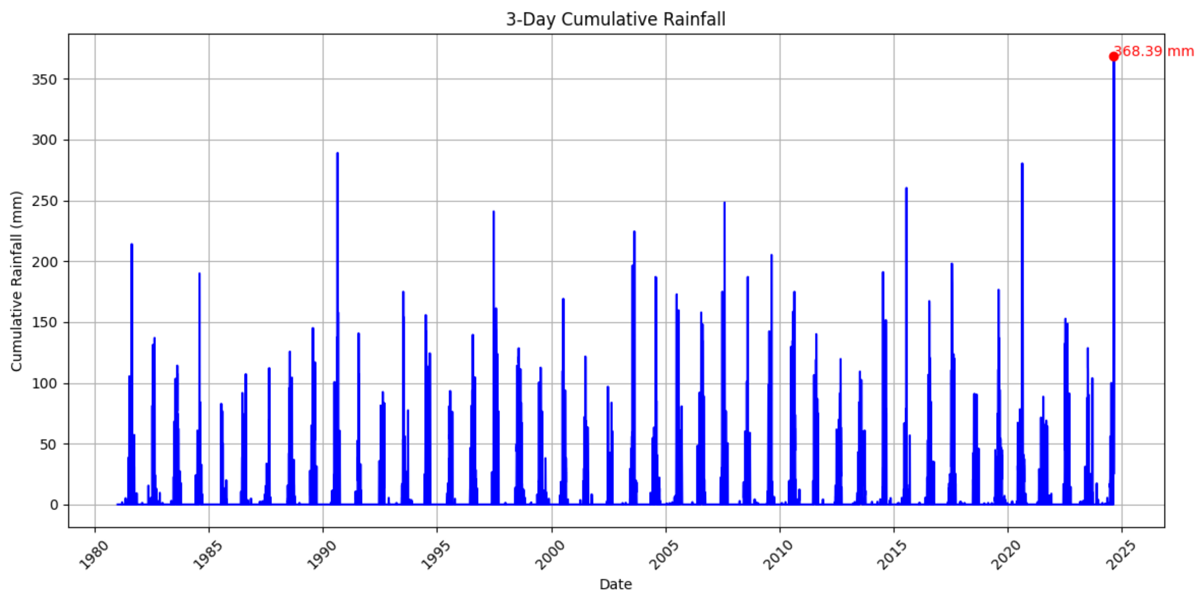


Figure 7: 3-day cumulative rainfall plot

ARCHITECTURAL LAYOUT

3D - MODEL OF THE PROJECT USING REVIT

Revit is a software tool in Building Information Modelling (BIM) from Autodesk, offering a full set of tools for architects, engineers, and even construction professionals in mastering building projects. It allows intelligent 3D models to be created so that changes in one view automatically update every other view. Revit supports multidisciplinary collaboration, bringing together architects, structural, and MEP engineers working in a single environment for the overall benefit of providing parametric modelling, construction documentation, and cloud-based collaboration tools for streamlining the design process to ensure accurate coordination and early project delivery.

Project Setup :

- Started a new project using the "Architectural Template" in Revit.
- Created levels and established horizontal and vertical grids of given dimensions to arrange the structural and architectural elements such as columns and walls.
- Placed structural columns (0.45mX0.45m) on each grid intersection, ensuring they extended from the foundation up to the roof level.
- Added beams(0.3mX0.4m) aligned with the grid to support floor slabs.
- Created floor slabs for each level using the floor tool, by sketching the boundaries according to the grid and wall layout.
- For building and slap support added shear walls on four sides of the buildings.
- Added walls, doors and windows on all floors using pre-defined Revit families, adjusting their sizes and positions.
- Added balconies and railing support to the building. Designed the staircase for the middle building for connecting levels(floors).
- Painted the building for a shiny look which also results in the cost estimation of painting.

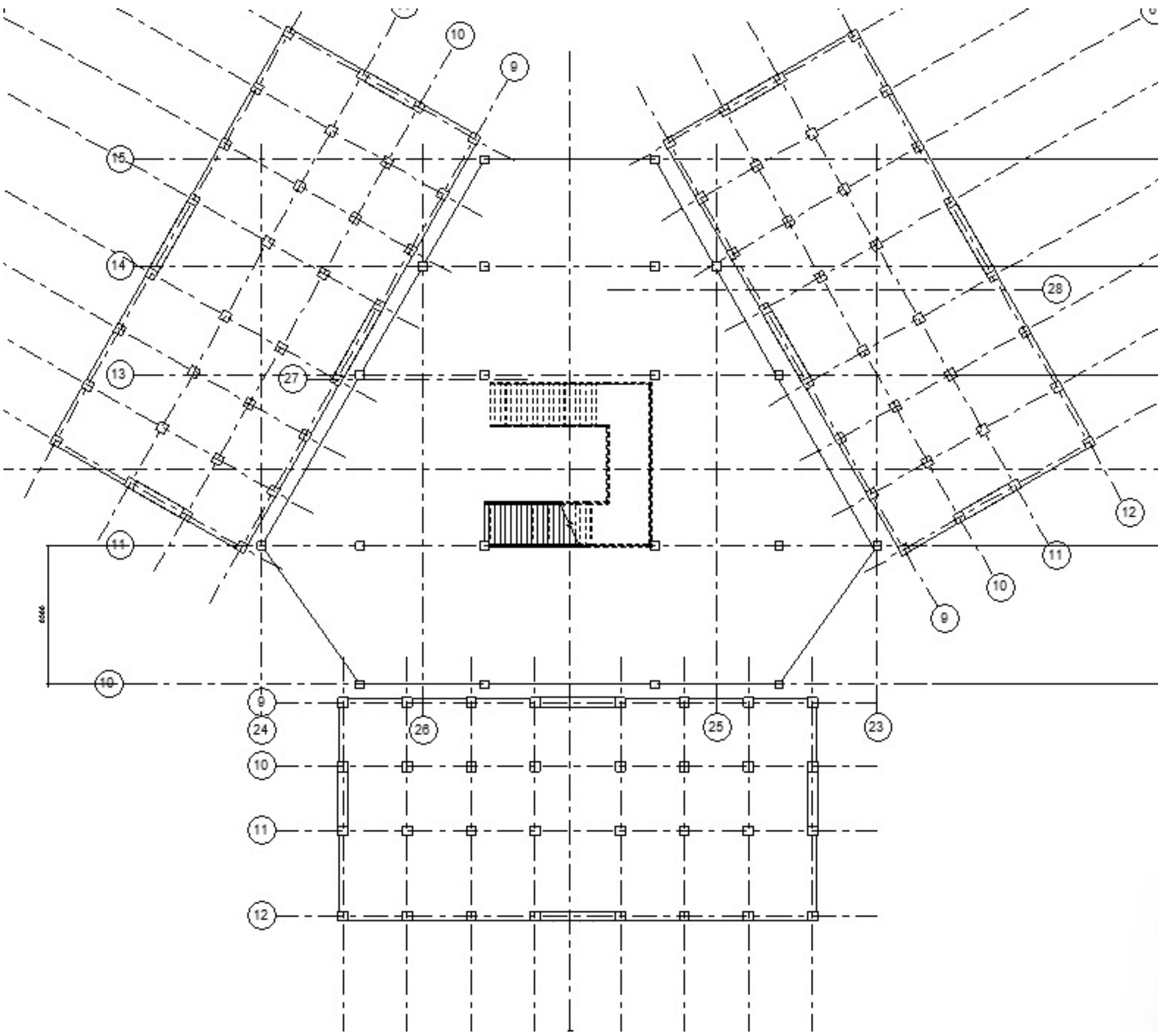


Figure 8: Structural layout of the building at foundation level

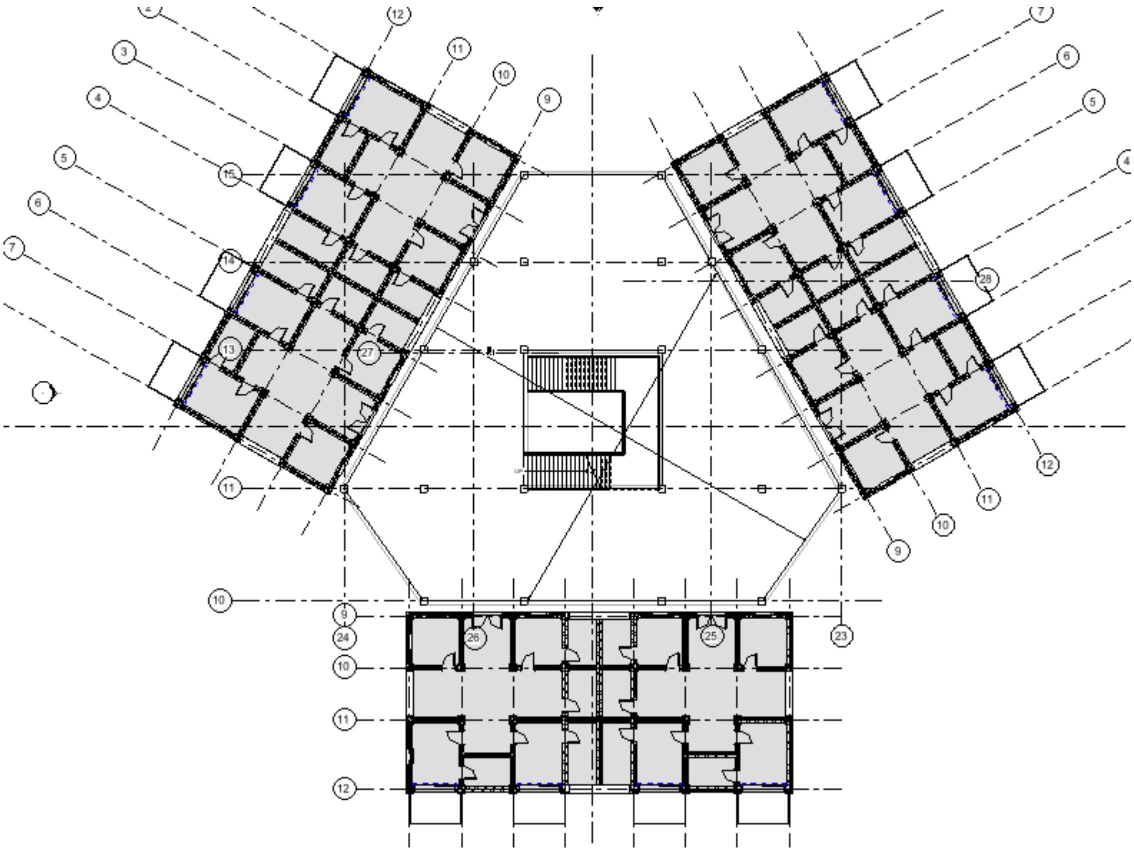


Figure 9: Structural layout of the building at 1st floor

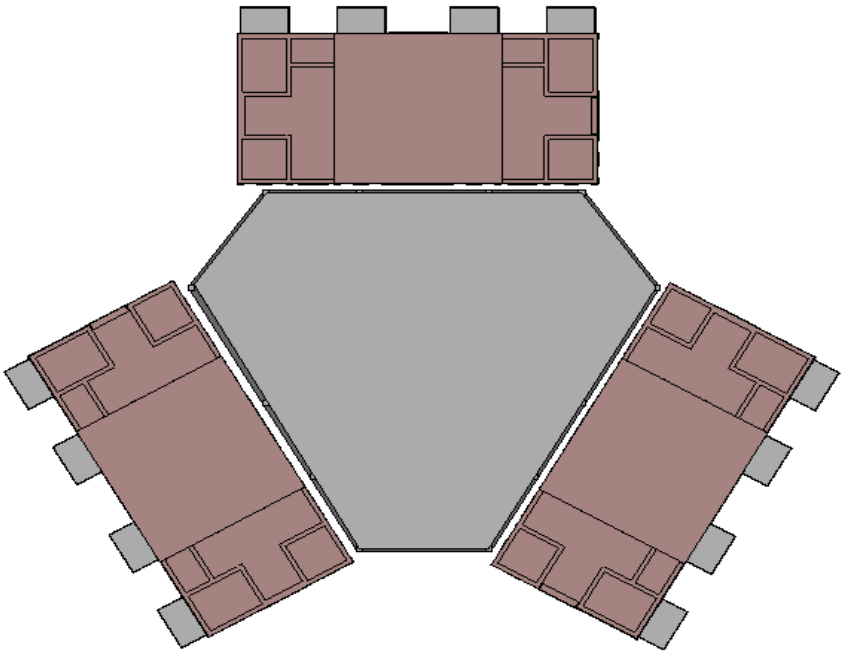


Figure 10: Top view of the building

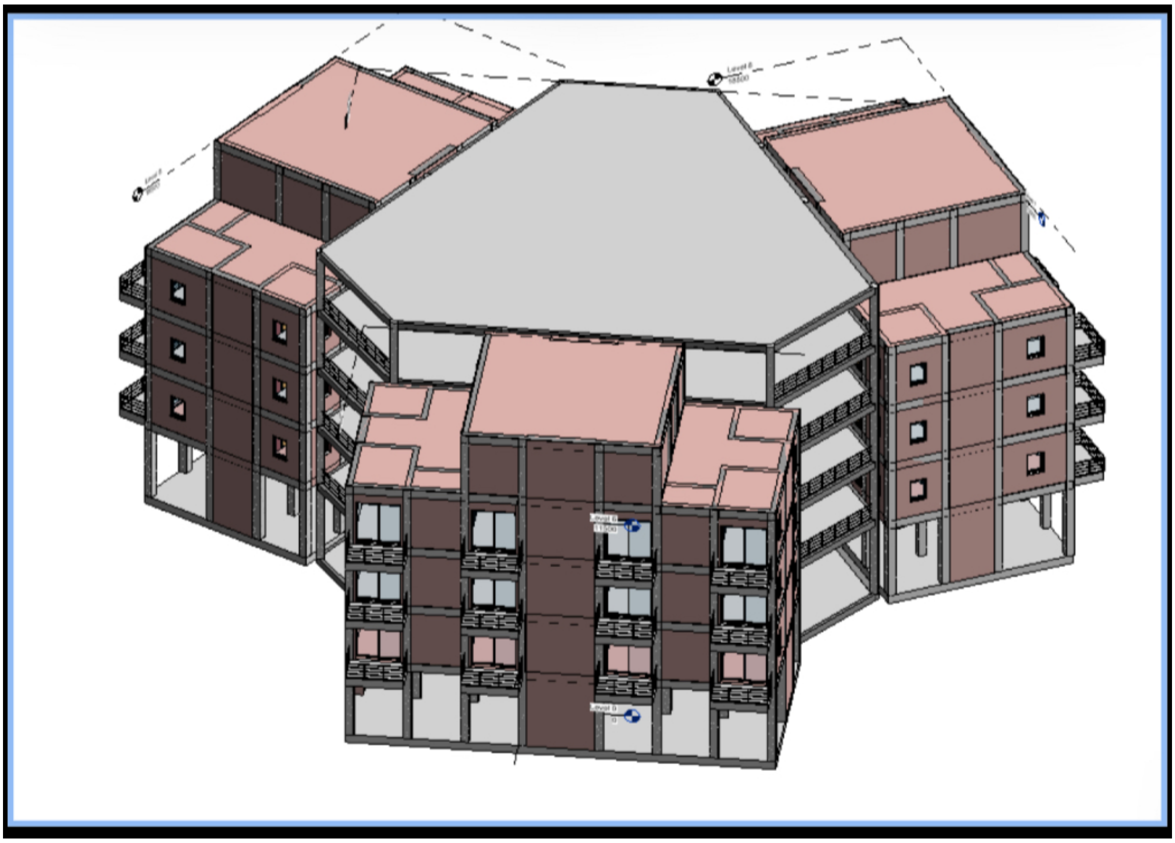


Figure 11: 3D view of the building from top

MATERIAL PROPERTIES

Concrete: -

- The Strength of concrete is M25 (Normal Strength Concrete)

  f_{ck} = 25 MPa

- Young Modulus of Elasticity E_c = 5000 * $\sqrt{f_{ck}}$ …. *(IS 456:2000 Clause 6.2.3.1)*

  E_c = 5000 * √25 = 25000 MPa

- Flexural Strength of Concrete f_{ct} = 0.7 * $\sqrt{f_{ck}}$ …. *(IS 456:2000 Clause 6.2.3.1)*

  f_{ct} = 0.7 * √25 = 3.5 MPa

Steel:-

We are using Fe500 steel for providing reinforcement,
Therefore, f_y = 500 MPa

Young Modulus of Elasticity E_s = 200000 MPa

STRUCTURAL ANALYSIS

The structural analysis of the Y shaped building is done by considering the building into two parts; the Middle building and three equivalent Side buildings. So analysis of side and middle buildings are done in SAP2000 software for G+3 (Ground plus 3 floors) plan.

The objective of using SAP2000 is to analyze and design buildings efficiently, ensuring structural safety under various loads like seismic, wind, live and dead loads. It helps optimize design while complying with relevant building codes and standards.
Steps to Model the Building in SAP2000:

Open SAP2000:
Start a new model by choosing a blank template or selecting a predefined grid system for easy positioning of structural elements.

Define Units:
Set units (e.g., kN, m) from the bottom right corner of the interface to ensure consistency throughout the project.

Define Material Properties:
Go to Define > Material Properties, and add materials (e.g., concrete, steel). Choose properties like compressive strength for concrete and yield strength for steel based on the design requirements.

Define Sections:
For beams and columns:

- Go to Define > Section Properties > Frame Sections, and define cross-sections for the columns (0.45m x 0.45m) and beams (0.3m x 0.4m).

For slabs:

- Go to Define > Section Properties > Area Sections, and define the slab thickness (0.15m thick RCC slab).

Create the Grid System:
Right click on the window or go to Edit > Edit Grid, and modify grid spacing to match the building's column layout. Define the X, Y and Z) grid spacing for the plan of the building. Below figure shows the grid spacing for the side building of the plan.

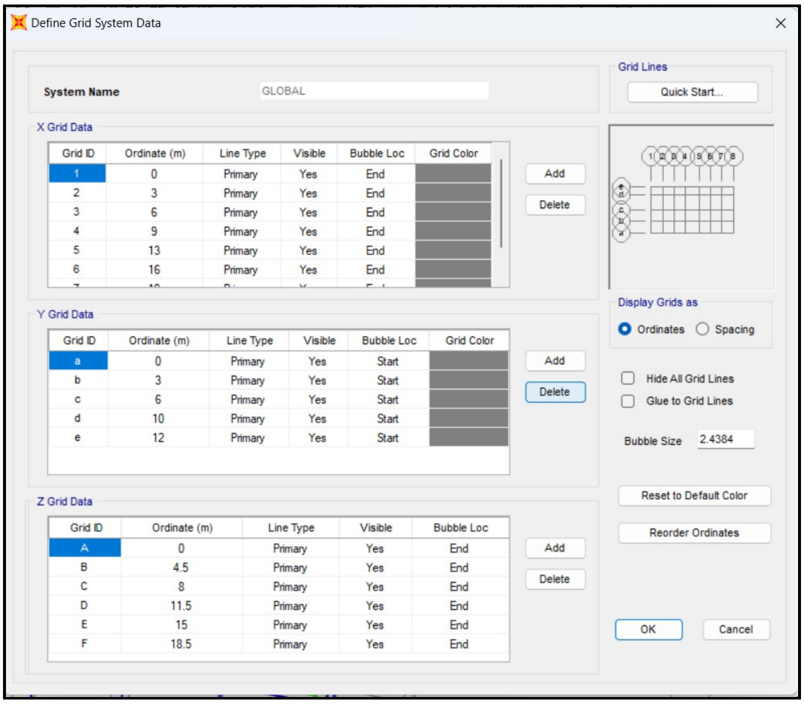


Figure 12: Grid system data of side building of the plan

Draw the Frame Elements (Columns and Beams):
Use the Quick Draw Frame tool (icon looks like a column) to draw columns at grid intersections from the base to the top of the structure following the grid points.

Similarly, use the Quick Draw Frame tool to place beams between columns in each storey. Ensure beams are placed in all required directions (X and Y axes).

Draw Slabs (Area Elements):
Go to Draw > Draw Area and select the slab section which was previously defined. Draw the slab by clicking on the four points that define the slab's perimeter (connecting the beam ends in each storey).

Assign Supports:
At the base of the columns, go to Assign > Joint > Restraints, and assign fixed supports at the foundation level to anchor the structure.

Assign Loads:
Define load cases for **dead load** (self-weight), **live load**, **earthquake** and **wind load**:

- Go to Define > Load Patterns, and input the relevant load types like **Dead**, **Live**, **Earthquake** and **Wind**. For the dead load, the multiplier is 1 since we are defining the element's dimensions, SAP will calculate its self weight accordingly.

Assign loads to different elements:

- Dead Load: Automatically applied to self-weight as the multiplier is 1.

- Live Load: Go to Assign > Shell Loads > Uniform, and apply live loads (e.g., 3 kN/m²) to slabs.
- Wind Load: Define wind loads using Define > Load Patterns, specifying wind intensity based on the building's location, and apply to the building's exterior using wind load combination.

- Earthquake Load: Define a seismic load pattern under Define > Load Patterns and input the relevant seismic parameters. Then assign the load to joints using Assign > Joint Loads > Forces.

Add Diaphragm:
Diaphragms need to be installed on the slabs at each floor level to ensure they are rigid. Go to Assign > Shell > Diaphragms and select the desired floor slabs, then assign them as rigid diaphragms.

Provide Load Combinations:
Go to Define > Load Combinations, create a new combination, and specify the desired load cases and their factors based on the IS 1893:2016 part-1 design code.

Run Analysis:

- Go to Analyze > Run Analysis, and check for any modeling errors or issues in the structure.
- Review the deformation, forces, and moments in the structure using the display tools.

Results:
Use the Display menu to view forces, moments, displacements, and other results for the selected load cases and combinations.

SAP2000 Model for the side building: -

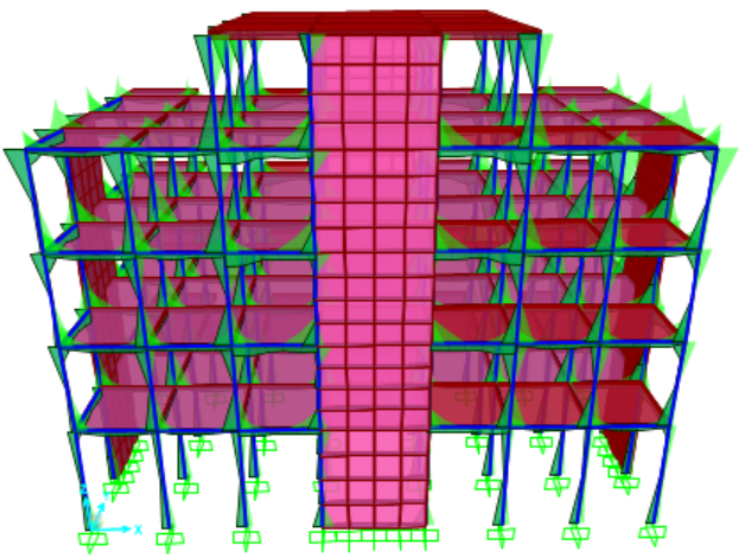


Figure 13: Bending Moment Diagram of Side Building using SAP2000 Analysis

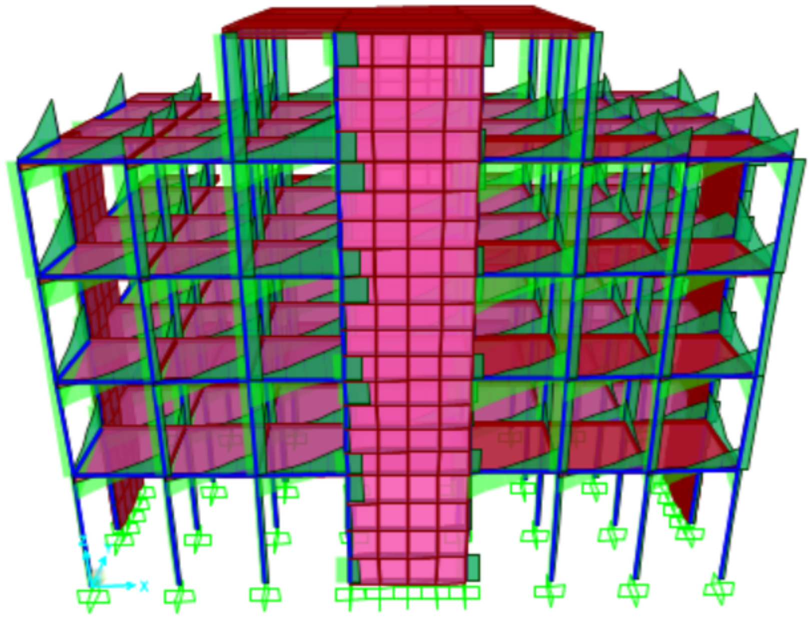


Figure 14: Shear Force Diagram of Side Building using SAP2000 Analysis

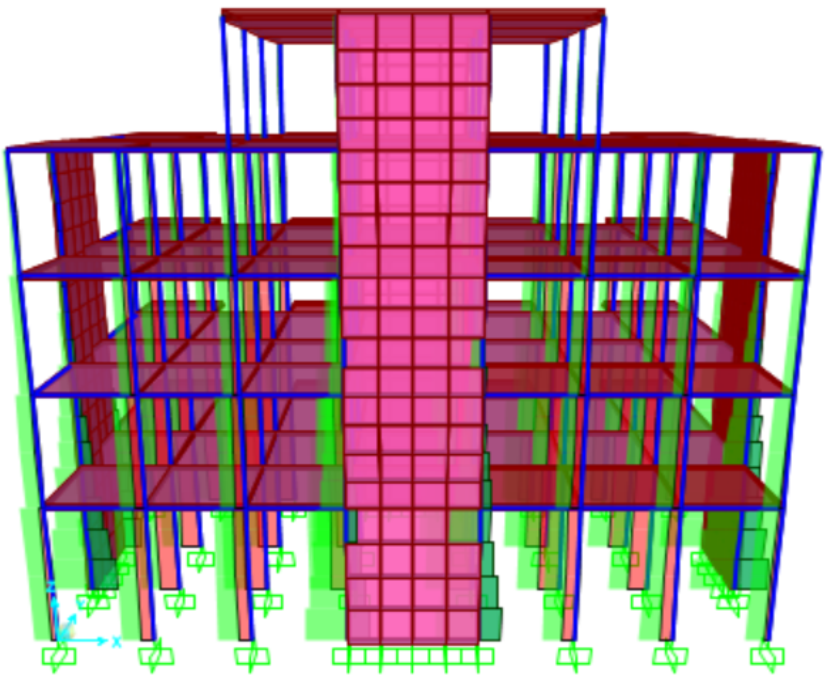


Figure 15: Axial Force Diagram of Side Building using SAP2000 Analysis

SAP2000 Model for the middle building: -

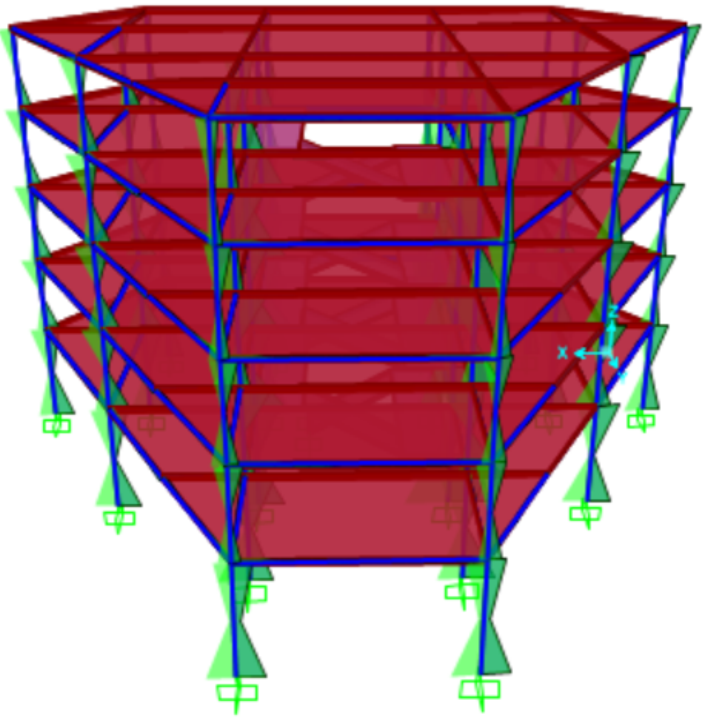


Figure 16: Bending Moment Diagram of Middle Building using SAP2000 analysis

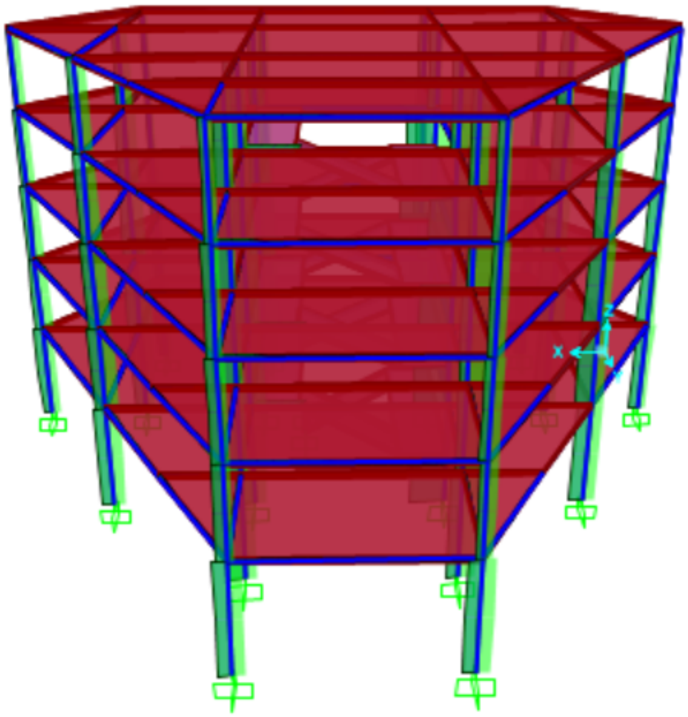


Figure 17: Shear Force Diagram of Middle Building using SAP2000 Analysis

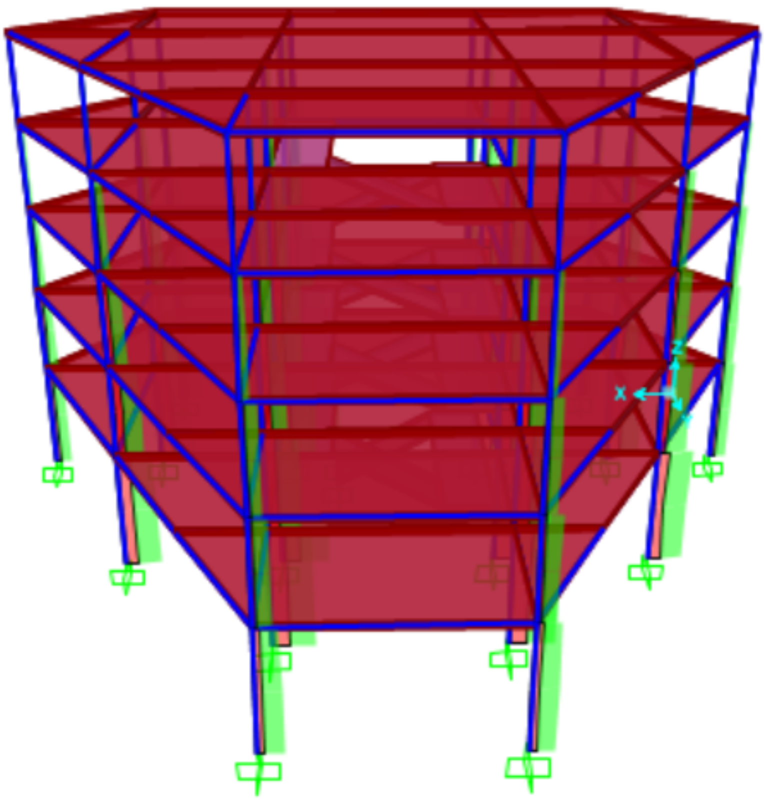


Figure 18: Axial Force Diagram of Middle Building using SAP2000 Analysis

LOAD CALCULATIONS:

Dead Load

It refers to the permanent, static forces acting on a structure due to its own weight and any fixed components. This includes the weight of structural elements like beams, columns, walls, slabs, shear wall, and any non-removable fixtures. Dead loads remain constant throughout the building's life and are crucial for determining the foundation and overall stability of the structure. These calculations are performed in accordance with IS 875: 1987 Part 1.
The below table provides the densities of various construction materials.

Density of RCC (in $\frac{KN}{m^3}$)	25
Density of masonry. Bricks (in $\frac{KN}{m^3}$)	19
Density of Plaster (in $\frac{KN}{m^3}$)	21

Table 1: Densities of building materials

Building 1
(Side Building)

BEAM						
Width(m)	**Depth(m)**	**Beam Length[c/c] (m)**	**Effective Beam length(m)**	**Number of Beams**	**Weight of One Beam (kN)**	**Total Weight (kN)**
0.3	0.4	3	2.55	38	7.65	290.7
0.3	0.4	4	3.55	10	10.65	106.5
Total dead load of beams of one floor						397.2

Table 2: Table showing Dead Load calculations of beams in side building

COLUMN					
Column Width (m)	**Column Depth (m)**	**Column Height(m)**	**Number of Columns**	**Weight of One Column (KN)**	**Total Weight (kN)**
0.45	0.45	3.5	32	17.71875	567
		4.5	32	22.78125	729

Table 6.3: Table showing Dead Load calculations of columns in side building

SLAB					
Length(m)	**Breadth(m)**	**Thickness (m)**	**Number of slabs**	**Weight of slab(kN)**	**Total weight of slabs(kN)**
3	3	0.15	12	33.75	405
4	3	0.15	8	45	360
4	4	0.15	1	60	60
3	2	0.15	4	22.5	90
Total self-weight of slab due to one floor					915

Table 6.4: Table showing Dead Load calculations of slab in side building

Floor weight = Self weight + Floor Finish
● Floor Finish = Area of slab * Floor Finish factor
(Floor Finish factor = 1.25 KN/m^2) .... (IS 875:1987 Part 1)
● Floor Finish = (22*10+3*2) * 1.25 = 282.5 KN
● Floor weight = 915 + 282.5 =1197.5 KN
● Roof weight = Self weight + Floor Finish + Terrace waterproofing
(Terrace Waterproofing factor = 1.5 KN/m^2) .... (IS 875:1987 Part 1)
● Terrace waterproofing = Area of slab * Terrace Waterproofing factor
Terrace waterproofing = (22*10+3*2) * 1.5 = 339 KN
● Roof weight = 915 + 282.5 + 339 = 1536.5 KN

Wall Load							
Length of wall(m)	**Effective Length(m)**	**Breadth(m)**	**Height(m)**	**Volume(m^3)**	**Density of Bricks(kN/m^3)**	**Weight of wall(kN)**	**Wt per unit Length(kN/m**
3	2.55	0.23	3.5	2.05275	19	39.00225	15.295
4	3.55	0.23	3.5	2.85775	19	54.29725	15.295

Table 5: Wall load calculation

Building 2 (Middle Building)

BEAM						
Width(m)	**Depth(m)**	**Beam Length[c/c] (m)**	**Effective Beam length(m)**	**Numbe r of Beams**	**Weight of One Beam (kN)**	**Total Weight (kN)**
0.3	0.4	5.81	5.36	6	16.08	96.48
		8	7.55	9	22.65	203.85
		2.89	2.44	2	7.32	14.64
		7.97	7.52	2	22.56	45.12
		6.5	6.05	4	18.15	72.6
		5.07	4.62	2	13.86	27.72
		5	4.55	2	13.65	27.3
		4.61	4.16	2	12.48	24.96
		5.77	5.32	2	15.96	31.92
		9.23	8.78	2	26.34	52.68

		5.85	5.4	2	16.2	32.4
Total dead load of beams of one floor						629.67

Table 6: Dead Load calculations of beam in middle building

COLUMN					
Column Width (m)	**Column Depth (m)**	**Column Height(m)**	**Number of Columns**	**Weight of One Column (KN)**	**Total Weight (kN)**
0.45	0.45	3.5	20	17.71875	354.375
		4.5	20	22.78125	455.625

Table 7: Dead Load calculations of column in middle building

SLAB			
Area of slab of one floor(m^2)		**Thickness of slab(m)**	**Weight of slab of one floor(KN)**
332.1997		0.15	1245.748875
Floor Finish =	**Area of slab * Floor finish factor**	**Floor finish Factor =**	1.5 KN/m^2
Floor Finish =	499 kN		

Live Load

Live load refers to the temporary, changing forces on a structure due to occupants, furniture, or movable objects. Unlike dead load, it varies with usage and external factors like snow or wind. Proper consideration of live loads ensures the structure can handle these changing forces safely.
The Live load given below is according to IS 875:1987 part-2

Side Building:

Type	Load(KN/m^2)	Number	Area(m^2)	Total area	Load(KN)
Living Room	3	2	33	66	198
Bedroom1	3	2	9	18	54
Bedroom2	3	4	12	48	144
Kitchen	3	2	9	18	54
Bathroom1	3	6	6	36	108
Bathroom2	3	2	12	24	72
Open Dining	3	2	9	18	54
Balcony	3	4	6	24	72
Total live load on one floor					**756**

Table 8: Live load calculations of side building

Type	Load(KN/m^2)	Area(m^2)	Load(KN)
SPORTS CENTRE	3	100	300
COMMUNITY HALL	3	100	300

Table 9: Live load calculations of top floor of side building

Middle Building:

Location	Area(m^2)	Load Intensity(KN/m^2)	Load(KN)	Remarks from IS 875 Part 2 1987
Common Area of Blocks A,B,C	268.1997	3	804.5991	Pg 7, i), c), (9)
Staircase	48	3	144	Pg 7, i), c), (9)
Roof	332.1997	1.5	498.29955	Pg 14, i), (a)

Table 10: Live load calculations of middle building

Wind Load

Wind load refers to the forces exerted on a structure by the pressure of wind acting against its surface. When wind hits a building, it creates both positive pressure on the windward side and suction (negative pressure) on the leeward side and roof. The magnitude of wind load depends on factors such as wind speed, building height, shape, and location.
The wind load given below is according to IS 875:1987 part-3.

The assumption regarding the Wind load calculation is given below:

Wind Load Calculations_Side building

Terrain Category	2
Topography	Plane area with upwind (θ) slope less than 3 degree
Desing life of structure (years)	50
Basic Wind Speed (m/s), Vb	39
Probability Factor or Risk coefficient, k1	1
Topography Factor (k3)	1
Importance Factor (k4)	1
Total Height of the building (m)	18.5
Length of building in X-direction (m)	22
Length of building in Y-direction (m)	10

Value for Terrain Category 2

Height	**k2**
10	1
15	1.05
20	1.07

$The\ design\ wind\ speed\ (Vz) = Vb * K1 * K2 * K3 * K4$ ....(IS CODE 875 Part-3 cl.6.3)
$The\ design\ wind\ pressure\ (Pz) = 0.6 * Vz2$ ....(IS CODE 875 part-3 cl. 7.2)

$The\ design\ wind\ pressure\ (Pd) = Kd * Ka * Kc * Pz.$ ....(IS CODE 875 Part-3 cl. 7.2)

Design wind pressure coefficient

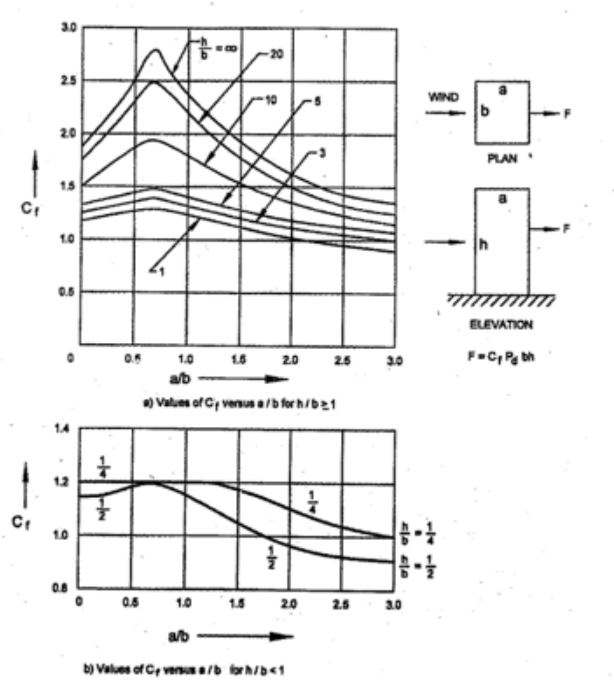
Ka	1
Wind Directionality Factor Kd	0.9
Combination Factor Kc	0.9

The value of Pd, however, shall not be taken as less than 0.7*Pz or MAX(Pd, 0.7*Pz)

$F = Cf * Ae * Pd$ ....*(IS 875:1987 Part-3)*

Where:

- Cf = Force Coefficient of Building
- Ae = Area Averaging Factor



Force X-Direction					
		Forces at Joints from left to write			
		1	2	3	4
Forces at Joints from down to up	1	1.849798334	3.699596669	4.316196114	1.849798334
	2	3.699596669	7.399193338	8.632392228	3.699596669
	3	3.811416978	7.622833957	8.893306283	3.811416978
	4	4.078805328	8.157610655	9.517212431	4.078805328
	5	2.094149295	4.188298591	4.886348356	2.094149295

Force X-Direction					
		Forces at Joints from left to write			
		1	2	3	4
Forces at Joints from down to up	1	1.8498	3.699596669	4.3162	1.8498
	2	3.6996	7.399193338	8.63239	3.6996
	3	3.81142	7.622833957	8.89331	3.81142
	4	4.07881	8.157610655	9.51721	4.07881
	5	2.09415	4.188298591	4.88635	2.09415

For Y-Direction									
	Area Joints from left to write								
		1	2	3	4	5	6	7	8
Area Joints from down to up	1	1.849798334	3.699596669	3.699596669	4.31619611	4.316196114	3.699596669	3.69959667	1.849798334
	2	3.699596669	7.399193338	7.399193338	8.63239223	8.632392228	7.399193338	7.39919334	3.699596669
	3	3.811416978	7.622833957	7.622833957	8.89330628	8.893306283	7.622833957	7.62283396	3.811416978
	4	2.039402664	4.078805328	6.118207991	9.51721243	9.517212431	6.118207991	4.07880533	2.039402664
	5	0	0	2.094149295	4.88634836	4.886348356	2.094149295	0	0

Seismic Load

Seismic load refers to the lateral forces exerted on structures due to ground motion during an earthquake. These dynamic forces are caused by the structure's inertia resisting sudden ground movements. We are designing the building for earthquake load according to the IS1893: 2016 which is given below: -

To calculate the earthquake load, the seismic weight of each floor is calculated as

Seismic weight = Dead Load + Live Load

For seismic weight calculation,

● Live load on roof = 0 .... *(IS 1893:2016 Part 1 Clause 7.3.2)*

Side building:

Base shear, V_b = 1051.296 KN

W(KN)	h (m)	Imposed loaf	Wi	Wi * h^2	Qi
1130	18.5	75	1205	412411.3	170.8223
4120	15	140	4260	958500	397.0143
5217	11.5	140	5357	708463.3	293.4481
5217	8	140	5357	342848	142.0089
5583	4.5	140	5723	115890.8	48.00238

Table 11: Seismic data of side building

Middle building:

- Area of slab = 332.1997 m^2
- Total live load acting on the slab = 332.1997 * 0.75

= 250 KN (Rounded off)

Total weight of the roof = Dead load + Live load

(Live load = 0 KN/m^2 for roof as per Clause 7.3.2 IS 1893:2016 Part 1)

Therefore,

Total weight of the roof = Weight of (roof slab + beams + columns + wall load + shear wall)

= 2010 + 629.67 + 354.37 + KN

= 2994 KN → Seismic weight of roof

Percentage of Imposed load(same as live load) to be considered in calculation of seismic weight is as follows:

As our live load is up to and including 3 KN/m^2, so the percentage of Imposed Load is 25%. Therefore, live load on floor = 3KN/m^2 * 25% of live load

= 0.75 KN/m^2

Floor	**Wi(KN)**	**Hi**	**WiHi^2**	**WiHi^2/(∑WiHi^2)**	**Qi**	**Vb**
5	3298	18.5	1128740.5	0.424624677	**293.72**	691.72
4	3456.5	15	777712.5	0.292570275	**202.38**	691.72
3	3456.5	11.5	457122.125	0.171966306	**118.95**	691.72
2	3456.5	8	221216	0.083219989	**57.565**	691.72
1	3625.5	4.5	73416.375	0.027618752	**19.104**	691.72

Table 12: Seismic data of middle building

LOAD COMBINATIONS:

(*As per IS 1893: Part 1: 2016)*

1. 1.5(DL+LL)
2. 1.2[DL+LL+EQ_x]
3. 1.2[DL+LL-EQ_x]
4. 1.2[DL+LL+W_x]
5. 1.2[DL+LL-W_x]
6. 1.2[DL+LL+EQ_y]
7. 1.2[DL+LL-EQ_y]
8. 1.2[DL+LL+W_y]
9. 1.2[DL+LL-W_y]
10. 1.5[DL+EQ_x]
11. 1.5[DL-EQ_x]
12. 1.5[DL+W_x]
13. 1.5[DL-W_x]
14. 1.5[DL+EQ_y]
15. 1.5[DL-EQ_y]
16. 1.5[DL+W_y]
17. 1.5[DL-W_y]
18. 0.9DL+1.5 EQ_x
19. 0.9DL-1.5 EQ_x
20. 0.9DL+1.5 W_x
21. 0.9DL-1.5 W_x
22. 0.9DL+1.5 EQ_y
23. 0.9DL-1.5 EQ_y
24. 0.9DL+1.5 W_y
25. 0.9DL -1.5 W_y

We have used these load combinations in the SAP2000 model of both side and middle buildings.

GEOTECHNICAL INVESTIGATION

BEARING CAPACITY

The 'bearing capacity' of soil refers to the ability of the ground to support the loads applied to a foundation without experiencing failure or excessive settlement. It is typically expressed in terms of 'ultimate bearing capacity', which is the maximum pressure the soil can withstand before failure, and 'allowable bearing capacity', which is the safe limit considering a factor of safety.

The ultimate bearing capacity is influenced by factors such as the soil's cohesion, unit weight, and the foundation's depth and width, and is calculated using formulas like Terzaghi's equation. The allowable bearing capacity is obtained by dividing the ultimate bearing capacity by a safety factor, which typically ranges from 2.5 to 3.0, depending on soil conditions and project requirements. The bearing capacity is essential in foundation design, ensuring that the structure can be safely supported by the underlying soil without risk of settlement or failure.

We used IS CODE 6403(1981) {code of practice for determination of breaking capacity of shallow foundation} to find out the bearing capacity of the soil.

>The dimension of the raft/mat foundation is

- Width of the foundation = 12 m
- Length of the foundation = 24 m
- Depth of the foundation = 3 m
- The stress coming on the foundation = 82.12 KN/m^2

> According to the dimension of the foundation

- The cohesion of the soil, C = 13.89 KN/m^2
- Angle of Internal friction Φ = 20.625° (Local Shear failure because Φ < 29°)
- Using LSF Φ', angle of internal friction = Tan-1 (2/3 * TAN (20.625)) = 14.09°

Shear Parameter						
Depth (m)	Cohesion, C(kg/cm^2)	Angle of internal Friction, Φ (degree)	Depth*C	Depth*Φ	γ (gm/cm^3)	γ KN/m^3
3	0.08	24.25	11.772	72.75	1.7	16.677
6	0.14	22.5	41.202	67.5	1.73	16.9713
9	0.16	16.5	47.088	49.5	1.81	17.7561
12	0.16	16.25	47.088	48.75	1.86	18.2466
15	0.13	27	19.1295	81	1.88	18.4428
18	0.16	19.75	47.088	59.25	1.89	18.5409
21	0.1	27.25	29.43	81.75	1.94	19.0314
24	0.2	17.25	58.86	51.75	1.93	18.9333
27	0.17	16.25	50.031	48.75	1.92	18.8352
30	0.15	24.5	44.145	73.5	1.96	19.2276
33	0.16	24	47.088	72	1.98	19.4238

TABLE 1 BEARING CAPACITY FACTORS
(*Clause* 5.1.1)

BEARING CAPACITY FACTORS

ϕ (Degrees)	N_c	N_q	N_γ
0	5.14	1.00	0.00
5	6.49	1.57	0.45
10	8.35	2.47	1.22
15	10.98	3.94	2.65
20	14.83	6.40	5.39
25	20.72	10.66	10.88
30	30.14	18.40	22.40
35	46.12	33.30	48.03
40	75.31	64.20	109.41
45	138.88	134.88	271.76
50	266.89	319.07	762.89

NOTE — For obtaining values of N'_c, N'_q and N'_γ, calculate $\phi' = \tan^{-1}$ (0 67 tan ϕ). Read N_c, N_q, and N_γ, from the Table corresponding to the value of ϕ' instead of ϕ which are values of N'_c, N'_q, N'_γ respectively.

Table 13: Bearing capacity factors

>Bearing capacity Factor:-

Nc = 10.47

Nq = 3.65

Nγ = 2.36

> Depth Factor:-

$Dc = 1.064$

$Dq = 1.032$

$D\gamma = 1.032$

>Shape Factor:-

$Sc = 1.3$

$Sq = 1.1$

$S\gamma = 0.8$

> Inclination Factor:-

$Ic = 1$

$Iq = 1$

$I\gamma = 1$

> Bearing Capacity = 407.49 KN/m^2

>Assume FOS = 3

>Safe Bearing Capacity = 407.49/ 3 = 135.83 KN/m^2, greater than 82.12 KN/m^2,

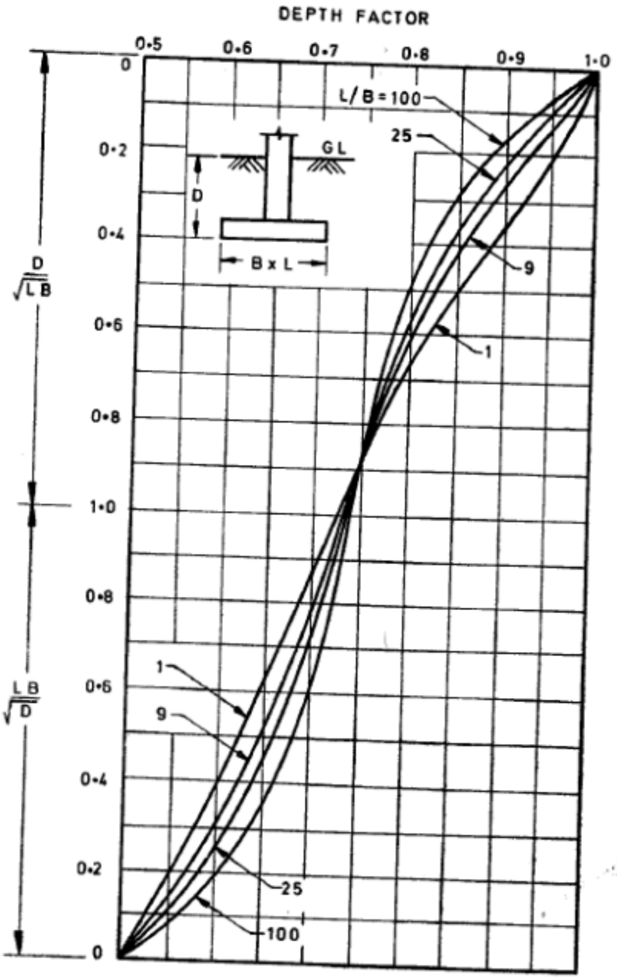
So safe in Bearing.

SETTLEMENT

'Settlement' refers to the downward movement of a structure due to the compression of the underlying soil under the applied loads. It occurs when the soil beneath the foundation undergoes deformation, usually as a result of increased stress from the building or structure. Settlement can be 'elastic' (immediate and reversible) or 'plastic'(permanent), and it can be caused by various factors such as the weight of the structure, soil type, moisture content, and depth of the foundation. The amount of settlement depends on the characteristics of the soil, including its compressibility and strength, as well as the magnitude and distribution of the loads applied. Excessive settlement can lead to structural damage, misalignment of components, or failure of the foundation, which is why estimating and controlling settlement is critical in foundation design. Engineers use methods such as soil testing and settlement analysis to predict and mitigate potential settlement, ensuring the stability and safety of the structure.

> For settlement Calculation we used the IS CODE 8009-1 (1976)

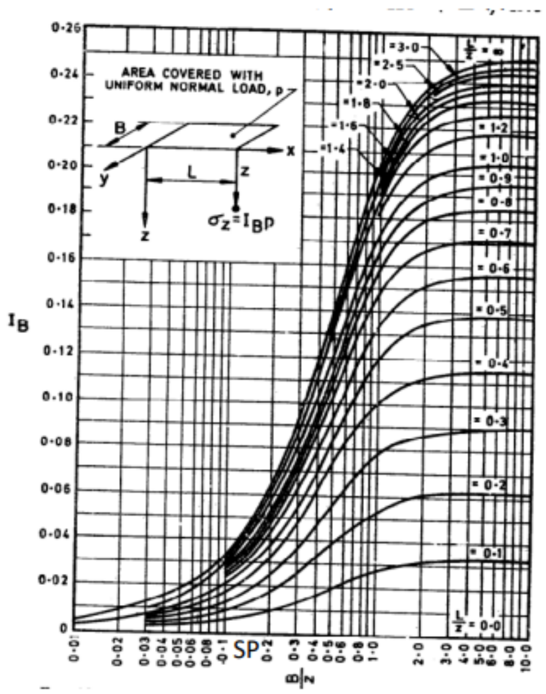
Stress on the foundation KN/m^2	82.1084
Width of the foundation B (m)	12
Length of the foundation L (m)	24
Depth of the footing (m)	3
Depth factor, λ	0.9
Rigidity Factor	0.8 For Raft Foundation
Pore Pressure Parameter, λ	0.7 Assuming the soil is NC



IG. 12 Fox's Correction Curves for Settlements of Flexible Rectangular Footings of $L \times B$ at Depth D

Settlement for NC soil

$$S_c = \frac{C_c H_c}{1+e_o} \log\left(\frac{\sigma'_o + \Delta\sigma'_{av}}{\sigma'_o}\right)$$



> The Settlement is **176 mm**

BEAM DESIGN

(As per IS 13920 : 2016)

Design of beams based on the results obtained from SAP2000 analysis is as follows:

Beam design of Side Building

Check for axial stress:

Axial force acting on beam = 0 KN < $0.08f_{ck}$ = 0.08×25 = 4 KN … *(IS 13920:2016 Cl 6.1)*

Therefore, the member can be designed as a beam.

Check for dimensions of beam:

Width to depth ratio of beam = $\frac{300}{400}$ = 0.75 > 0.3 ………….. *(IS 13920:2016 Cl 6.1.1)*

Width of beam = 300 mm > 200 mm .... *(IS 13920: 2016 Cl 6.1.2)*

Depth of beam = 400 mm < $\frac{clear\ span}{4} = \frac{3000}{4}$ = 750 mm .... *(IS 13920: 2016 Cl 6.1.3)*

Width of beam(b_w) = 300 mm

Width of supporting member(c_2) = 450 mm

a) c_2 = 450 mm
b) $0.75\ times\ breadth\ of\ supporting\ member(c_1)$ = 0.75 × 450 = 337.5 mm

Width of the beam *bw* shall not exceed width of supporting member c_2 plus distance on either side of supporting member equal to the smaller of (a) and (b).

……. *(IS 13920: 2016 Cl 6.1.4)*

Therefore, $b_w <$ 400 + 337.5 = 737.5 mm (OK)

Longitudinal Reinforcement:

Minimum longitudinal steel ratio ρ_{min} required on any face at any section is:

$\rho_{min} = 0.24\frac{\sqrt{f_{ck}}}{f_y}$ ……..... *(IS 13920: 2016 Cl 6.2.1)*

$\rho_{min} = 0.24\times\frac{\sqrt{25}}{500}$ = 0.0024 = 0.24%

Considering clear cover(c/c) of 20 mm as per IS 456:2000 Table 16 (Assuming the condition is mild)

Initially considering 16mm φ for main bars and 8mm φ for stirrups.

Therefore, effective depth(d),

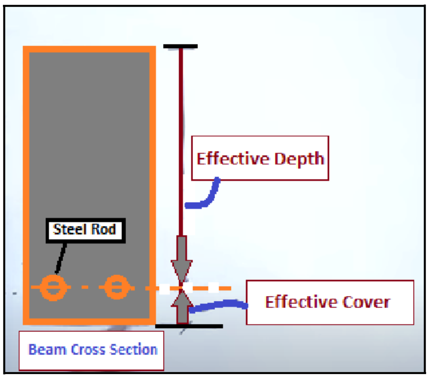


Figure 19: D,d calculation

d = D – c/c - $\frac{\phi_{main}}{2}$

Where

D is the overall depth of beam

c/c is the clear cover as per IS 456:2000 Table 16

ϕ_{main} is the diameter of the main bars

Therefore,

d = 400 – 20 - $\frac{16}{2}$ = 372 mm

$A_{st,min}$ calculation:

$$A_{st,min} = \frac{\rho_{min} \times b \times d}{100} = \frac{0.24 \times 300 \times 372}{100} = 267.84\ mm^2$$

Maximum Longitudinal Reinforcement in beams ρ_{max} provided on any face at any section is 0.025 i.e. 2.5%.

$$A_{st,max} = \frac{\rho_{max} \times b \times d}{100} = \frac{2.5 \times 300 \times 372}{100} = 2790\ mm^2$$

Design of Beam for Flexure:

We are designing the beam for hogging moment, sagging moment and shear forces.
Maximum Hogging Moment = - 96.6 KNm

Maximum Sagging Moment = 58.2 KNm

Maximum Shear Force = 91.8 KN

a) Design of beam for Hogging Moment:

$$M_{u,lim} = 0.36 \times \frac{x_{u,max}}{d} \times \left(1 - 0.42\ \frac{x_{u,max}}{d}\right) \times f_{ck} \times \text{b} \times d^2$$

.... *(IS 456:2000 Annex G G.1.1 (a))*

For Fe500, $\frac{x_{u,max}}{d}$ = 0.46 .... *(IS 456:2000 Note Table Pg: 70*

Therefore,

$M_{u,lim}$ = 143.189 KNm

As, $M_{u,lim} > M_u$

Therefore, design the beam as a singly reinforced beam section.

$A_{st,req} = \frac{f_{ck}}{2f_y}\left(1 - \sqrt{1 - \frac{4.598 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$

$A_{st,req} = \frac{25}{2 \times 500}\left(1 - \sqrt{1 - \frac{4.598 \times 96.6 \times 10^6}{25 \times 300 \times 372^2}}\right) \times 300 \times 372$

$A_{st,req}$ = 679.9 mm^2 > $A_{st,min}$ [312.48 mm^2] (OK)

Provide **4** no's of **16mm φ** bars as hogging steel.

b) Design of beam for Sagging Moment:

As, $M_u < M_{u,lim}$

Therefore, design the beam as a singly reinforced beam section.

$A_{st,req} = \frac{f_{ck}}{2f_y}\left(1 - \sqrt{1 - \frac{4.598 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$

$A_{st,req} = \frac{25}{2 \times 500}\left(1 - \sqrt{1 - \frac{4.598 \times 58.2 \times 10^6}{25 \times 300 \times 372^2}}\right) \times 300 \times 372$

$A_{st,req}$ = 386.5 mm^2 > $A_{st,min}$ [312.48 mm^2] (OK)

Provide **3** no's of **16mm φ** bars as sagging steel. (>50% of top reinforcement at the column face is provided as the bottom reinforcement)

Design of Beam for Shear:

For of beam L = 3 m

For Sway to Right:

$M_U^{As} = 0.87 \times f_y \times A_{St} \times (d - d')$ = 95.5 KN
$M_U^{Bh} = 0.87 \times f_y \times A_{st} \times (d - d')$ = 127.73 KN
Sway to Left
$M_U^{Ah} = 0.87 \times f_y \times A_{st} \times (d - d')$ = 127.73 KN
$M_U^{Bs} = 0.87 \times f_y \times A_{st} \times (d - d')$ = 95.95 KN
As per Cl-6.3.3 of IS 13920, The shear force capacity of beam shall be greater of:

1) Factored Shear force as per linear structural analysis = 91.8KN

2) Factored Gravity shear force + equilibrium shear forces when plastic hinges are formed at both the edges. From above calculations,

From SAP,

$$V_{ua}^{\ D+L} = 91.8KN$$

i) For Sway to right,

$$V_{ua} = V_{ua}^{\ D+L} + 1.4\frac{M_U^{As} + M_U^{Bh}}{L_{Ab}} = 158.28\,KN$$

$$V_{uB} = V_{ua}^{\ D+L} - 1.4\frac{M_U^{As} + M_U^{Bh}}{L_{Ab}} = -1.7KN$$

For sway to left

$$V_{ua} = V_{ua}^{\ D+L}\ 1.4\frac{M_U^{As} + M_U^{Bh}}{L_{Ab}} = -1.7KN$$

$$V_{uB} = V_{ua}^{\ D+L} + 1.4\frac{M_U^{As} + M_U^{Bh}}{L_{Ab}} = 158.28KN$$

So we took the V = 158.28 KN

Spacing required for providing stirrups considering 10mm 2 legged stirrups as per is code 456 page 73

$$s_v = \frac{(0.87 X f_y\ X\ A_{sv}\ X\ d)}{V_{us}} = 80.26\text{mm}$$

The first link shall be at a distance not exceeding 50 mm from the joint face

…. *(IS 13920: 2016 Cl 6.3.5.1)*

Closely spaced links shall be provided over a length equal to 2d on either side of a section where flexural yielding may occur under earthquake effects.. Over the remaining length of the beam, vertical links shall be provided at a spacing not exceeding d/2

…. *(IS 13920: 2016 Cl 6.3.5.1)*

For this case 2d = 744 mm. So provide 1000mm confining zone
Checking for stirrups spacing in confining zone as per IS 13920 clause 6.3.5
The spacing of links over a length of 2d at either of a beam shall not exceed

…. *(IS 13920: 2016 Cl 6.3.5.)*

a. d/4 = 372/4 = 93
b. 8 18 = 144mm
c. 100mm

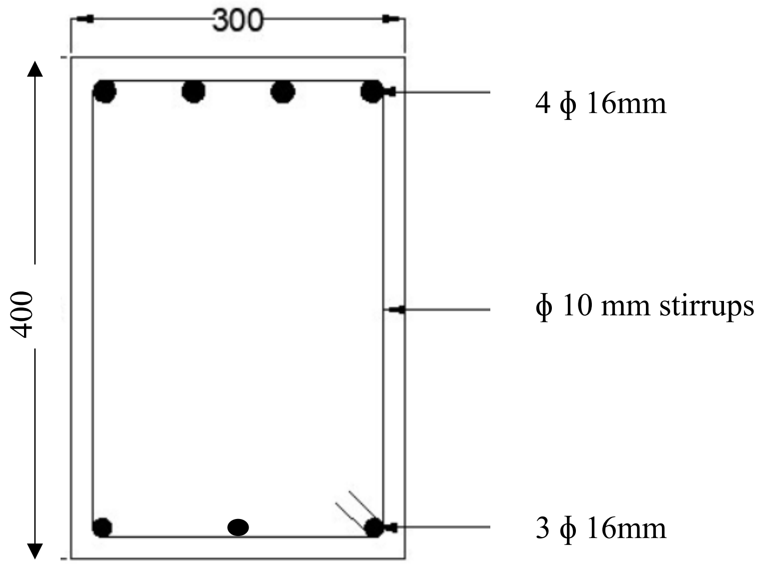
Therefore provide 2-legged 10mm stirrups at 100mm c/c up to 1000mm. Place the 1st stirrup at a distance of 50mm from the face of the support.
At an exterior joint, the top and bottom bars of beams shall be provided with anchorage length beyond the inner face of the column, equal to the development length of the bar in tension plus 10 times the bar diameter minus the allowance for 90° bends

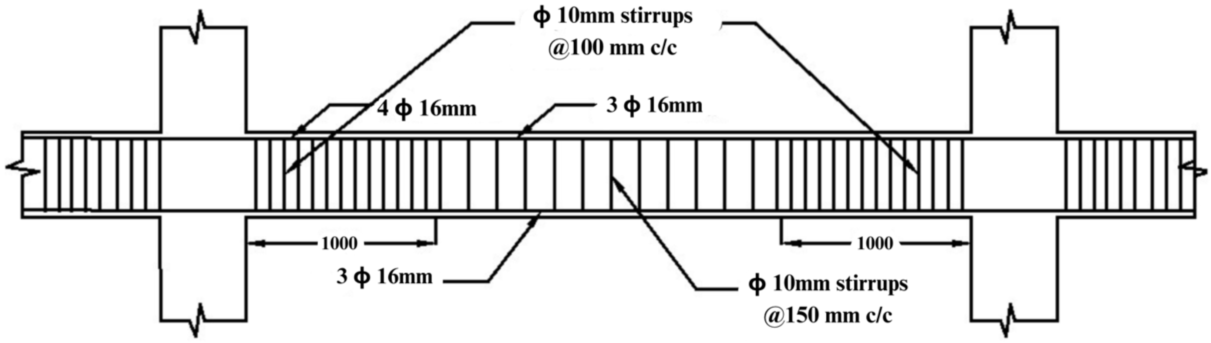
…. *(IS 13920: 2016 Cl 6.2.5.)*

Therefore anchorage length = 50d + 10d - 8d
= (50+10 - 8)x16

Anchorage length for 16mm rod mm

Detailing





Beam design of Middle Building:

Check for axial stress:

Axial force acting on beam = 0 KN < $0.08f_{ck}$ = 0.08×25 = 4 KN … *(IS 13920:2016 Cl 6.1)*

Therefore, the member can be designed as a beam.

Check for dimensions of beam:

Width to depth ratio of beam = $\frac{300}{400}$ = 0.75 > 0.3 ………….. *(IS 13920:2016 Cl 6.1.1)*

Width of beam = 300 mm > 200 mm .... *(IS 13920: 2016 Cl 6.1.2)*

Depth of beam = 400 mm < $\frac{clear\ span}{4} = \frac{3000}{4}$ = 750 mm .... *(IS 13920: 2016 Cl 6.1.3)*

Width of beam(b_w) = 300 mm

Width of supporting member(c_2) = 450 mm

c) c_2 = 450 mm
d) $0.75\ times\ breadth\ of\ supporting\ member(c_1)$ = 0.75 × 450 = 337.5 mm

Width of the beam bw shall not exceed width of supporting member c_2 plus distance on either side of supporting member equal to the smaller of (a) and (b).

…..... *(IS 13920: 2016 Cl 6.1.4)*

Therefore, $b_w <$ 400 + 337.5 = 737.5 mm (OK)

Longitudinal Reinforcement:

Minimum longitudinal steel ratio ρ_{min} required on any face at any section is:

$\rho_{min} = 0.24\frac{\sqrt{f_{ck}}}{f_y}$ ……..... *(IS 13920: 2016 Cl 6.2.1)*

$\rho_{min} = 0.24\times\frac{\sqrt{25}}{500} = 0.0024 = 0.24\%$

Considering clear cover(c/c) of 20 mm as per IS 456:2000 Table 16 (Assuming the condition is mild)

Initially considering 16mm φ for main bars and 8mm φ for stirrups.

Therefore, effective depth(d),

$d = D - c/c - \frac{\phi_{main}}{2}$

Where

D is the overall depth of beam

c/c is the clear cover as per IS 456:2000 Table 16

ϕ_{main} is the diameter of the main bars

Therefore,

$d = 400 - 20 - \frac{16}{2} = 372$ mm

$A_{st,min}$ calculation:

$A_{st,min} = \frac{\rho_{min} \times b \times d}{100} = \frac{0.24\times300\times372}{100} = 267.84\ mm^2$

Maximum Longitudinal Reinforcement in beams ρ_{max} provided on any face at any section is 0.025 i.e. 2.5%.

$A_{st,max} = \frac{\rho_{max} \times b \times d}{100} = \frac{2.5\times300\times372}{100} = 2790\ mm^2$

Design of Beam for Flexure:

We are designing the beam for hogging moment, sagging moment and shear forces.
Maximum Hogging Moment = - 123.7 KNm

Maximum Sagging Moment = 63.9 KNm

Maximum Shear Force = 84 KN

c) Design of beam for Hogging Moment:

$$M_{u,lim} = 0.36 \times \frac{x_{u,max}}{d} \times \left(1 - 0.42\ \frac{x_{u,max}}{d}\right) \times f_{ck} \times b \times d^2$$

.... *(IS 456:2000 Annex G G.1.1 (a))*

For Fe500, $\frac{x_{u,max}}{d} = 0.46$ .... *(IS 456:2000 Note Table Pg: 70*

Therefore,

$M_{u,lim}$ = 143.189 KNm

As, $M_{u,lim} > M_u$

Therefore, design the beam as a singly reinforced beam section.

$$A_{st,req} = \frac{f_{ck}}{2f_y}\left(1 - \sqrt{1 - \frac{4.598 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$$

$$A_{st,req} = \frac{25}{2 \times 500}\left(1 - \sqrt{1 - \frac{4.598 \times 123.7 \times 10^6}{25 \times 300 \times 372^2}}\right) \times 300 \times 372$$

$A_{st,req}$ = 914.28 mm^2 > $A_{st,min}$ [312.48 mm^2] (OK)

Provide **5** no's of **16mm φ** bars as hogging steel.

d) Design of beam for Sagging Moment:

As, $M_u < M_{u,lim}$

Therefore, design the beam as a singly reinforced beam section.

$$A_{st,req} = \frac{f_{ck}}{2f_y}\left(1 - \sqrt{1 - \frac{4.598 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$$

$$A_{st,req} = \frac{25}{2 \times 500}\left(1 - \sqrt{1 - \frac{4.598 \times 63.9 \times 10^6}{25 \times 300 \times 372^2}}\right) \times 300 \times 372$$

$A_{st,req}$ = 427.7 mm^2 > $A_{st,min}$ [312.48 mm^2] (OK)

Provide **3** no's of **16mm φ** bars as sagging steel. (>50% of top reinforcement at the column face is provided as the bottom reinforcement)

Design of Beam for Shear:

For of beam L = 3 m

For Sway to Right:

$M_U^{As} = 0.87 \times f_y \times A_{St} \times (d - d') = 95.5$ KN
$M_U^{Bh} = 0.87 \times f_y \times A_{st} \times (d - d') = 159.18$ KN
Sway to Left
$M_U^{Ah} = 0.87 \times f_y \times A_{st} \times (d - d') = 159.18$ KN
$M_U^{Bs} = 0.87 \times f_y \times A_{st} \times (d - d') = 95.95$ KN
As per Cl-6.3.3 of IS 13920, The shear force capacity of beam shall be greater of:

1) Factored Shear force as per linear structural analysis = 91.8KN

2) Factored Gravity shear force + equilibrium shear forces when plastic hinges are formed at both the edges. From above calculations,

From SAP,

$$V_{ua}^{D+L} = 91.8KN$$

i) For Sway to right,

$$V_{ua} = V_{ua}^{D+L} + 1.4\frac{M_U^{As} + M_U^{Bh}}{L_{Ab}} = 128.64\ KN$$

$$V_{uB} = V_{ua}^{D+L} - 1.4\frac{M_U^{As} + M_U^{Bh}}{L_{Ab}} = 39KN$$

For sway to left

$$V_{ua} = V_{ua}^{D+L}\ 1.4\frac{M_U^{As} + M_U^{Bh}}{L_{Ab}} = 39\ KN$$

$$V_{uB} = V_{ua}^{D+L} + 1.4\frac{M_U^{As} + M_U^{Bh}}{L_{Ab}} = 128.85KN$$

So we took the V = 128.64 KN

Spacing required for providing stirrups considering 10mm 2 legged stirrups as per is code 456 page 73

$$s_v = \frac{(0.87 X f_y X A_{sv} X d)}{V_{us}} = 98.95\text{mm}$$

The first link shall be at a distance not exceeding 50 mm from the joint face

…. *(IS 13920: 2016 Cl 6.3.5.1)*

Closely spaced links shall be provided over a length equal to 2d on either side of a section where flexural yielding may occur under earthquake ef2cts.. Over the remaining length of the beam, vertical links shall be provided at a spacing not exceeding d/2

…. *(IS 13920: 2016 Cl 6.3.5.1)*

For this case 2d = 744 mm. So provide 1000mm confining zone
Checking for stirrups spacing in confining zone as per IS 13920 clause 6.3.5
The spacing of links over a length of 2d at either of a beam shall not exceed

…. *(IS 13920: 2016 Cl 6.3.5.)*

d. d/4 = 372/4 = 93
e. 8 18 = 144mm
f. 100mm

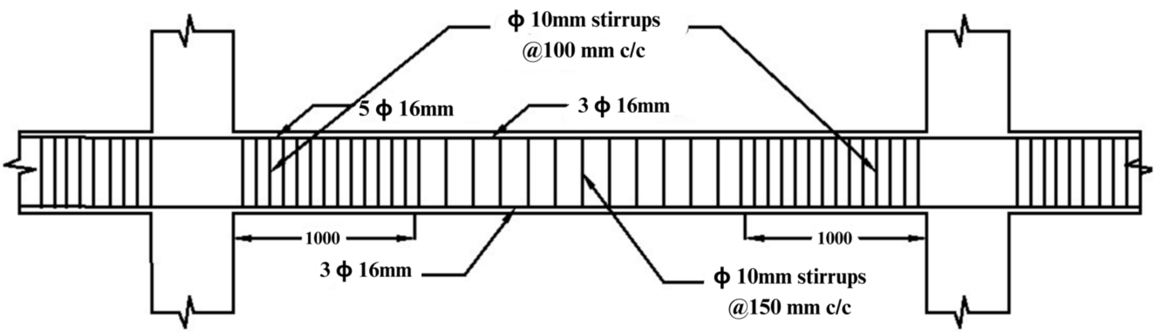
Therefore provide 2-legged 10mm stirrups at 100mm c/c up to 1000mm. Place the 1st stirrup at a distance of 50mm from the face of the support.
At an exterior joint, the top and bottom bars of beams shall be provided with anchorage length beyond the inner face of the column, equal to the development length of the bar in tension plus 10 times the bar diameter minus the allowance for 90° bends

…. *(IS 13920: 2016 Cl 6.2.5.)*

Therefore anchorage length = 50d + 10d - 8d
= (50+10 - 8)x16

Anchorage length for 16mm rod = 832 mm

Detailing



COLUMN DESIGN

Design forces for a column: P_u = 1552.5KNm

M_{u2}= 78 KNm

M_{u3}= 63 KNm

V_{u3} = 87.1 KNm

Checks according to IS code 13920: 2016

Axial stress

A structural member may be constructed as a column, if the factored axial stress is greater than $0.08f_{ck}$ and less than $0.4f_{ck}$ *IS 13920:2016 Clause 7.1*

$0.08f_{ck}$ = 0.08 x 25 = 2MPa

$0.4f_{ck}$ = 0.4 x 25 = 10MPa

The axial stress is $\frac{P_u}{A}$ = 7.6

These members can be designed as a column.

Column member size.

The minimum column dimension should not be less than *(Clause 7.1.1, IS 13920: 2016)*

- 20 d_b = 360mm bar in the beam.
- 300 mm

The Column dimensions provided Breadth = 450 and Depth = 450

The Aspect ratio = $\frac{B}{D} = \frac{450}{450}$ = 1

The Aspect ratio is greater than 0.45 The design is (OK) *(Clause 7.1.2, IS 13920: 2016)*

The slenderness ratio according to IS 456:2000 is $\frac{L_{ex}}{D}$

$$\frac{L_{ex}}{D} = \frac{4500}{450} = 10$$

The slenderness ratio is less than 12 so the column is designed as a short column.

(Clause 25.1.2 of IS 456: 2000)

Limiting longitudinal reinforcement

minimum and maximum reinforcement should be 0.8% and 4% of the cross-sectional area *(Clause 26.5.3.1 of IS 456: 2000)*

$A_{st,min}$ = 0.8% of cross-sectional area = 1280 mm^2

$A_{st,max}$ = 4% of cross-sectional area = 6400 mm^2

Minimum eccentricity

All columns shall be designed for minimum eccentricity, equal to the unsupported length of column/ 500 plus lateral dimensions/30, subject to a minimum of 20 mm. Where bi-axial bending is considered, it is sufficient to ensure that eccentricity exceeds the minimum about one axis at a time.

(Clause 25.4 of IS 456: 2000)

e_{min} = max $\left(\frac{L_x}{500} + \frac{D}{30} = 24 \ , or\ 20mm\right)$

Design

initially, an approximate design of the column is performed. Subsequently, the interaction check is performed for all the load combinations.

Factored axial load on the column P_u = 1552.5 KNm

The first design is performed for uniaxial eccentricity.

$$Mu = 1.15\sqrt{M_{u2}^2 + M_{u3}^2} = 1.15\sqrt{78^2 + 63^2} = 115.304 \text{ KNm}$$

Checks for strong column weak beam *(Clause 7.2.1 IS 13920:2016)*

$$\sum Mc > 1.4 \sum Mb$$

$$2Mc \geq 1.4(127+95.5)$$

$$Mc \geq 155.75 \text{ KNm}$$

The moment capacity required for a strong column weak beam is more so Mu = (155.75) kNm is considered for design moments they factored axial load *Pu*.

Effective cover is d' = 40 +10 + $\frac{20}{2}$= 60 mm

$$\frac{d'}{d} = 0.1$$

According to SP16 chat 48 Parameters *Pu* and *Mu* can be normalized

$\frac{P_u}{f_{ck}bD}$ = 0.306 , $\frac{M_u}{f_{ck}bD^2}$ = 0.06

The required longitudinal reinforcement ratio *p*

$$\frac{p}{f_{ck}} = 0.04$$

$$p = 1\%$$

The area of longitudinal steel in the concrete cross-section is u

$$Asc,\ required = \frac{1}{100} \times 450 \times 450 = 2025$$

Provide 12 bars of 16 mm diameter (equal on all sides).

Ast,provided = 2413 mm2

Check the safety of the column under biaxial loading

M^1_{u2} *and* M^1_{u3} *are the uniaxial moment capacities of the column cross-section about the two orthogonal axes passing through the geometric center.*

The $M^1_{u2} = M^1_{u3}$ *, because of symmetry.* $M^1_{u2} = M^1_{u3}$ *is 227 kNm from chat 48 of SP16*

According to is ***456:2000 clause 39.6*** *the interaction checks have been performed*

$$Puz = 0.45fckAg + (0.75fy - 0.45fck)Asc = 2331.2285\ kN$$

$$For\ \frac{P_u}{P_{UZ}} = 0.6 \qquad , \alpha_n = 1.66$$

Interaction check under biaxial moment is

$$(\frac{M_{u2}}{M^1_{u2}})^{\alpha_n} + (\frac{M_{u3}}{M^1_{u3}})^{\alpha_n} = \left(\frac{78}{227}\right)^{1.6} + \left(\frac{63}{227}\right)^{1.6} = 0.309$$

Therefore the section is safe under the applied load. Similar checks were performed for other load combinations and other columns.

Design for shear

The shear capacity of the column

The design shear strength of concrete τc is determined by assuming 25% of the longitudinal steel in tension.

the percentage of steel reinforcement is

$$p_t = 1.86 \times \frac{25}{100} = 0.465\%$$

Design shear strength of concrete τc for p_t *= 0.465% is 0.47* (Table 19 : IS 456:2000)

τ_c *should be multiplied by the factor* δ (clause 40.2.2, IS 456:2000)

$$\delta = 1 + \frac{3P_u}{A_g f_{ck}} = 1.93$$

$$r_{c\alpha} = 0.465 \times 1.93 = 0.897\ MPa$$

Shear capacity of concrete is

$$Vc \;= \frac{0.897\times450\times450}{1000} = 181\ KNm$$

The design shear force in column V_u *it the maximum value of* *(clause 7.5 ,IS 13920:2000)*
a. Factored shear force as per linear structural analysis
b. Shear force due to plastic hinge formation at beam ends

The maximum factored shear force obtained through the structural analysis performed using SAP2000 is 87 kN.

Shear force due to plastic hinge formation at the ends of the beam is

For sway to right:

$$V_u \;= \frac{1.4(M_u^{AS} + M_u^{BH})}{L} \;= \frac{1.4(127.73+95.5)}{4.5} = 69.44\ KNm$$

For sway to left:

$$V_u \;= \frac{1.4(M_u^{AS} + M_u^{BH})}{L} \;= \frac{1.4(123.73+95.5)}{4.5} = 69.44\ KNm$$

The design shear force V_u *= 87.1*

The design shear stress $\tau_v \;= \frac{V_u}{bd} = \frac{87.1}{450\times450} \;=\; 0.0.43$ (clause 40.1 , IS 456:2000)

Since $\tau_v \; < \tau_c$ *nominal shear reinforcement is provided in the column. The spacing of nominal shear reinforcement* s_v *is determined by assuming 2 legged 10 mm diameter stirrups and 2 cross-links* (clause 26.5.2.6 , IS 456:2000)

$$\boldsymbol{s_v} \;\leq\; \frac{\boldsymbol{0.87 f_y A_{sv}}}{\boldsymbol{0.4b}} = 312$$

The value of s_v *should be less than* (clause 26.5.3.2 , IS 456:2000)

- *Least lateral dimension of column = 450*
- *16 × diameter of smallest main reinforcement bars = 320*
- *300mm*

Provide 2 legged 10 mm diameter stirrups with a spacing of 150 mm center-to-center

Special confining reinforcement

The special confining reinforcement should be provided up to a certain length on either side of the joint.

the length of the confinement zone should not be less than either of the following

(clause 8.1(a), IS 13920:2000)

- *The larger lateral dimension of the member at the section where yielding occurs i.e., 450mm*
- *1/6 of the clear span of the member = 585 mm*
- *450 mm*

Therefore the special confining reinforcement is provided over a length of 585mm.

The area of the shear reinforcement bar A_{sh} provided over this length is given by the formula

$$A_{sh} = max\left(0.18 s_v h \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right) \quad , \quad 0.05 S_v h \frac{f_{ck}}{f_y}\right)$$

Parameters A_g and A_k are the gross area of the column cross-section and area of confined concrete core in the rectangular link measured to its outer dimensions. These parameters are calculated as follows.

$$A_g = 450 \times 450 = 202500\, mm^2$$

$$A_k = (450 - 2 \times 40)^2 = 136900\, mm^2$$

A_{sh} *is used to calculate the spacing of shear reinforcement* s_v *considering 10 mm stirrups.*

$$A_{sh} = max\left(0.18 s_v h \frac{f_{ck}}{f_y}\left(\frac{A_g}{A_k} - 1\right) \quad , \quad 0.05 S_v h \frac{f_{ck}}{f_y}\right)$$

$$A_{sh} = 64$$

The spacing of shear reinforcement s_v is S122mm. This spacing should not be more than the following

(clause 8.1(b), IS 13920:2000)

- $\frac{\mathbf{1}}{\mathbf{4}}\boldsymbol{th}$ *of minimum member dimension of the column = 112.5 mm*
- *6 times the diameter of the smallest longitudinal bars = 120 mm*
- *100 mm*

On either side of the joint, place two 10-mm-legged and two cross-tie shear-hoops with a diameter of 10mm with 70 mm c/c over a 600 mm length. displays the specifics of a typical ground story column's reinforcement.

Detailing

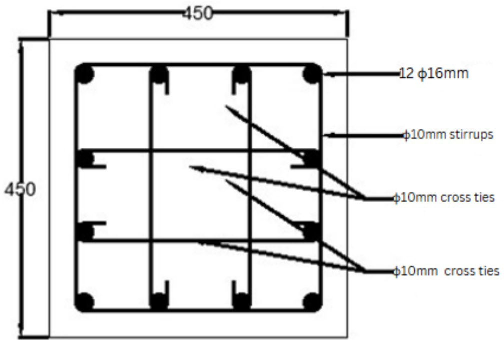


Figure20 : Top view of column cross section

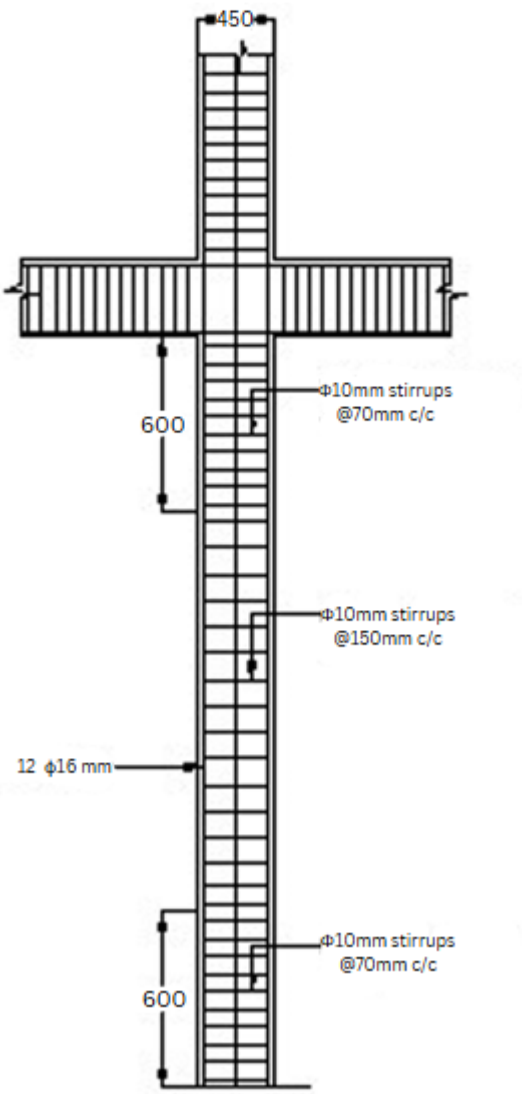


Figure 21: Side view of column

Joint Design

Joint 1: Interior joint

Beam-column joint dimensions

The dimensions of the beam-column joint are determined according to Clause 9.1.2 of IS 13920: 2016.

The depth of joint D_J is 450 mm.

The width of the joint

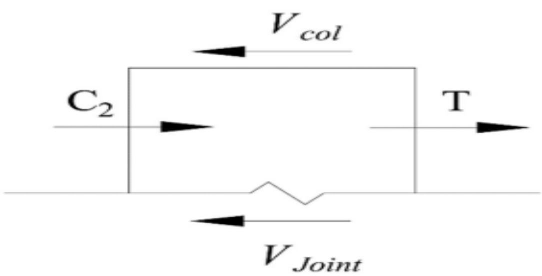
$$b_j = min\{b_c, b + 0.5D_c\} = min\left\{450, 300 + \frac{450}{2}\right\} = 450$$

Shear force in joint

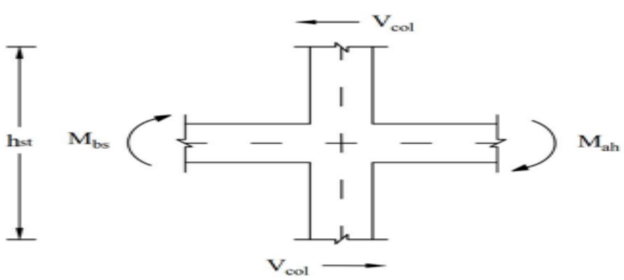
The free body diagram of an interior beam-column joint is shown v_{col} is the shear force in the column, T is the tension force in the top reinforcement bars v_{joint} is the joint shear, and C_2 is the compressive force in the top reinforcement bars. These parameters are calculated next

Calculation of column shear

The free-body diagram of the column joint for swaying towards the right. The shear calculated considering sway towards right is same as that calculated for sway towards left.



Free body diagram of an interior beam-column joint



Free body diagram for a beam-column junction

The shear force in the column v_{col} is

$$v_{col} = 1.4\frac{(m_{AS}+M_{bh})}{h_{st}} = 76$$

Force developed in beam reinforcement

Tension force developed in top reinforcement bar T_1 is

$$T_1 = C_1 = 1.25A_{st}\,f_y = \text{ 502KN}$$

Compressive force developed in bottom reinforcement bar is

$$c_2 = 1.25A_{st}f_y = 377\text{KN}$$

Shear force in the joint v_{joint} can be determined by applying equilibrium at the joint

$$V_{joint} = T_1 + C_2 - V_{col} = 803\text{KN}$$

Shear strength of a beam-column joint

The joint shear strength τ_{jc} is

$$\tau_{jc} = 1.5\sqrt{f_{ck}} = 7.5\text{Mpa}$$ *(clause 9.1.1, IS 13920:2016)*

Joint shear resistance V_{jc} in the terms of force is

$$V_{jc} = A_{cj}\tau_{jc} = 1518.7\text{KN}$$

Since $v_{jc} > v_{joint}$ the beam column joint is safe is shear

Joint 2: Exterior Joint(connected to 3 beams)

The dimensions of the beam-column joint are determined according to Clause 9.1.2 of IS 13920:2016.

Depth of joint D_j is 450mm

The width of the joint b_j is

$$b_j = min\{b_c, b + 0.5D_c\} = min\left\{450, 300 + \frac{450}{2}\right\} = 450$$

Calculation of column shear

The shear force in the column v_{col} is

$$v_{col} = 1.2\frac{(m_{AS}+M_{bh})}{h_{st}} = 76KN$$

Force developed in beam reinforcement

Tension force developed in top reinforcement bar T_1 is

$$T_1 = C_1 = 1.25A_{ST}\,F_y = 502\;KN$$

Compressive force developed in bottom reinforcement bar is

$$c_2 = 1.25 A_{st} f_y = 377 \text{ KNK}$$

Shear force in the joint v_{joint} can be determined by applying equilibrium at the joint

$$V_{joint} = T_1 + C_2 - V_{col} = 803 \text{ KN}$$

Shear strength of a beam-column joint

The joint shear strength τ_{jc} is

$$\tau_{jc} = 1.2\sqrt{f_{ck}} = 6Mpa \qquad \text{(clause 9.1.1, IS 13920:2016)}$$

Joint shear resistance V_{jc} in the terms of force is

$$V_{jc} = A_{cj}\tau_{jc} = 1215KN$$

Since $v_{jc} > v_{joint}$ the beam column joint is safe is shear

Joint 3: (Exterior Corner Joint)

The dimensions of the beam-column joint are determined according to Clause 9.1.2 of IS 13920:2016.

Depth of joint D_j is 450mm

The width of the joint b_j is

$$b_j = min\{b_c, b + 0.5D_c\} = min\left\{450, 300 + \frac{450}{2}\right\} = 450$$

Calculation of column shear

The shear force in the column v_{col} is

$$v_{col} = 1.2\frac{(m_{AS} + M_{bh})}{h_{st}} = 76 \text{ KN}$$

Force developed in beam reinforcement

Tension force developed in top reinforcement bar T_1 is

$$T_1 = C_1 = 1.25 A_{ST} F_y = 502 \text{ KN}$$

Compressive force developed in bottom reinforcement bar is

$$c_2 = 1.25 A_{st} f_y = 377KN$$

Shear force in the joint v_{joint} can be determined by applying equilibrium at the joint

$$V_{joint} = T_1 - V_{col} = 426KN$$

$$V_{joint} = C_2 - V_{col} = 301\text{KN}$$

Shear strength of a beam-column joint

The joint shear strength τ_{jc} is

$$\tau_{jc} = 1 * \sqrt{f_{ck}} = 5\ Mpa \qquad \text{(clause 9.1.1, IS 13920:2016)}$$

Joint shear resistance V_{jc} in the terms of force is

$$V_{jc} = A_{cj}\tau_{jc} = 1012\text{KN}$$

Since $v_{jc} > v_{joint}$ the beam column joint is safe is shear

SLAB DESIGN

Side Building

Thickness of the slab = 150 mm (Considered value)
For all the slabs $\frac{l_y}{l_x} \leqq$ 2 [Therefore, two-way slab]
Considering simply supported restrained slab (Where the comers of a slab are prevented from lifting) *(IS 456:2000 Annx D-1.0)*

According to Clause 24.1 of IS 456: 2000, shorter span to overall depth ratio for continuous slab is 40.
As we are using Fe 415 bars, additionally 0.8 is multiplied:

$$\frac{Span}{Overall\ Depth} = \frac{4000}{D} = 0.8 \text{ x } 40$$

Therefore, Minimum overall depth required is

$$D_{min} = 125 \text{ mm}$$

Assuming, overall depth (D) = 150 mm (which is greater than D_{min})

Load calculation:
Total Dead load of slab = self-weight + Floor finishing
= 25*0.15 + 1.25
= 5 kN/m^2
Total Live load on slab = 3 kN/m^2 ……….…..... *(IS 875:1987 Part 2)*
Total factored load is given by
w = 1.5(5+3) = 12 kN/m^2
Total 8 different type of slab are there in building as shown in figure
From Table 26 of IS 456:2000, α_x and α_y coefficient are used to compute bending moment in slab from following equation as per Annex D-1.1 of IS 456:2000.

My = α_x wlx^2
My = $\alpha_y$wlx^2

Slab Name	I		II		III		IV		V		VI		VII		VIII		
	Edge	Middle	Edge	Middle	Edge	Middle	Edge	Middle	Edge	Middle	Edge	Middle	Edge	Middle	Edge	Middle	
alpha	0.047	0.035	0.037	0.028	0.032	0.024	0.057	0.044	0.051	0.039	0.047	0.036	0.037	0.028	0.084	0.064	
lx	3	3	3	3	3	3	3	3	3	3	3	3	4	4	2	2	**Max Mu**
Mx	5.076	3.78	3.996	3.024	3.456	2.592	6.156	4.752	5.508	4.212	5.076	3.888	7.104	5.376	4.032	3.072	**7.104**

Table showing calculations of Mx for different slabs after yield line division

Maximum bending moment among 8 different slabs is

$$M_u = 7.104 \text{ KNm}$$

Considering clear cover(cc) for moderate conditions as 40 mm, as per Table 16 IS 456:2000 the minimum clear cover for moderate conditions is 30 mm.

Therefore, clear cover(cc) = 40 mm [Considered]

Assuming the diameter of reinforcing bars as ϕ = 8 mm.
Therefore, effective depth(d)

$d = D - cc - \frac{\phi}{2}$

$d = 150 - 40 - \frac{8}{2}$

d = 106 mm

Now calculating Limiting Moment of Resistance $M_{u,lim}$,

$M_{u,lim} = 0.36 f_{ck} \left(\frac{x_{u,max}}{d}\right)\left[1 - 0.42\ \frac{x_{u,max}}{d}\right] \mathrm{b}d^2$

For Fe415, $\frac{x_{u,max}}{d} = 0.48$ *[Note point Pg: 70, IS 456:2000]*

$M_{u,lim} = 0.36 \times 25 \times 0.48(1 - 0.42 \times 0.48) \times 1000 \times 106^2$ [b = 1000 mm, as we are considering 1 m strip]

$M_{u,lim}$ = 38.75 KNm > M_u *(Under reinforced section, good for design)*

Calculating A_{st} ,

$M_u = 0.87 f_y A_{st} d \left(1 - 0.42 \frac{x_{u,max}}{d}\right)$

A_{st} = 235 mm^2

The minimum reinforcement required for Fe415 steel is 0.12% of the total cross sectional area. …… *[Cl 26.5.2.1 IS 456:2000]*

$A_{st,min}$ = 0.0012×b×D

$A_{st,min}$ = 0.0012×1000×150 = 180 mm^2

Providing spacing between rods,

Spacing = $\frac{1000 * Area\ of\ assumed\ bar}{A_{st}} = \frac{1000 \times \frac{\Pi}{4} \times (8^2)}{235}$ = 214 mm

Maximum spacing between rods as per Cl 26.3.3 IS 456:2000 is

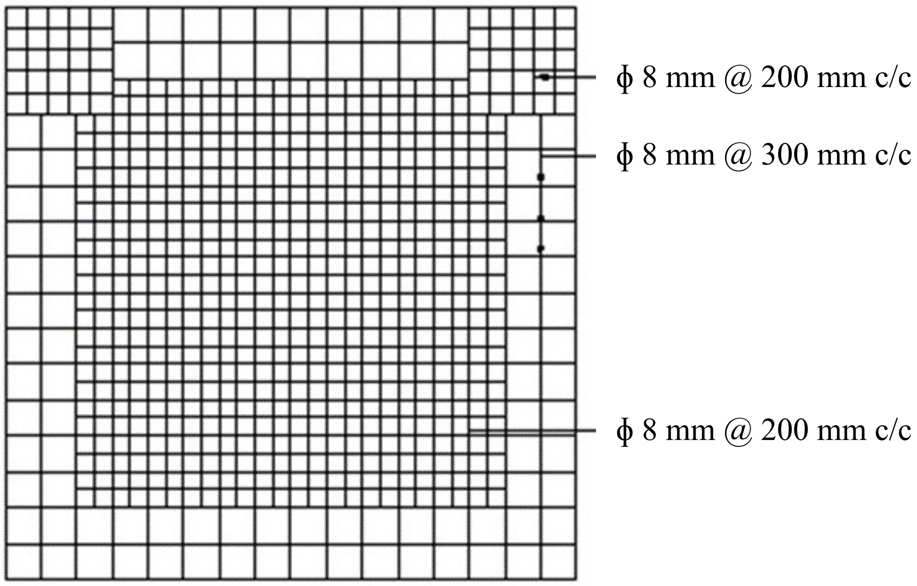
Maximum spacing = min$\begin{cases} 3d = 3 \times\ 106 = 318\ mm \\ 300\ mm \end{cases}$

Maximum spacing = 300 mm

As the dimension of the slab is 4×4 m^2, providing 8ϕ mm diameter bar with 200 mm c/c spacing at centre of middle span ($\frac{3l}{4}$ = 3m) and at each edge ($\frac{l}{8}$ = 0.5m) 8 mm diameter bar with 300 mm c/c is provided is provided in both direction.

Torsion reinforcement shall be provided at any corner where the slab is simply supported on both edges meeting at that comer. Torsion reinforcement is provided for slab meeting at corner for length of [$\frac{shorter\ span}{8}$] i.e. 0.8 m, 8 mm diameter bar with 200 mm c/c spacing at top on both direction.

Detailing



FLAT SLAB DESIGN

A) Direct design method: -

a) $\frac{L}{B} < 2 = \frac{4}{3} = 1.3 < 2$

b) $LL < \frac{4}{4} DL$

$4.5 < \frac{3}{4}\ 6.25$

4.5 < 4.6875

- As per clause 31.1.1.1

Proportioning

a) Along longer Span ($L_1 = 4.0m$, $L_2 = 3.0m$)
Width of column strip = 0.25 L_2 (>0.25 $L_1 = 0.25 \times 4.0 = 1$)
= 0.25×3.0
= 0.75 m
Column strip width= $2 \times 0.75 = 1.5$
Width of Middle strip = 3-1.5 = 1.5 m

b) Along Shorter Span
Width of column strip= 0.75 m
Width of Middle Strip = 4.0 – 1.5
= 2.5 m

Depth of Flat Slab: -

Minimum thickness = 250 mm

$\frac{Span}{Depth} = 26$ (from Clause 23.21)

$\frac{4000}{Depth} = 26$

$Depth = \frac{4000}{26} = 153.84$

Assuming reinforcement= 2.6

Modification factor =1.0

$\frac{Span}{Depth} = 26 \times 1\ (30\ to\ 35)$

$d = \frac{Longer\ Span}{Ratio} = \frac{4000}{30}$ = 133.3 mm

Assuming clear of 25 mm, 10 mm bar diameter

Overall depth = d + Clear Cover + $\frac{Bar\ diameter}{2}$

= 133 + 25+ $\frac{10}{2}$

= 163 mm

A depth of Overall depth = 175 mm

Effective Cover = 35 mm

Drop dimension (clause 31.2.2)

Minimum length along longer spans = $\frac{4.0}{3} = 1.3$

Minimum length along short spans = $\frac{1}{3} \times 3.0$= 1 m

Consider width of drop = 1.9 mx 1.9 m = 3.61m

Drop thickness = $\frac{D}{4} = \frac{163}{4} = 40.75$mm < (min=100 mm)
Drop thickness= 100 mm
Colum head dimensions; - (clause 31.2.2) $(\frac{1}{5}\ to\ \frac{1}{4}\ average\ span = 0.7)$
Load Calculation: -

Live load = 4.5 KN/m^2
Dead load = $\left(\frac{0.27+0.17}{2}\right) \times 25 = 5.5\ KN/m^2$
We have square supports- 450mm x 450mm columns

Ln – along long span= 4-0.45=3.55m

Ln-along shorter span=3-0.45=2.55m

Total load= 4.5+5.5 = 10 KN/m^2
Factored load = 10 x 1.5 = 15 KN/m^2
BM along long span: -
$M_0 = \frac{Wln}{8} = \frac{(15*3*3.55)*3.55}{8}$ = 70.8 kN-m
BM along short span: -
$M_0 = \frac{Wln}{8} = \frac{(15*4*2.38)*2.38}{8}$ =36.5 kN-m

Moment distribution in long span: - **(from clause 31.4.3.2, 31.4.5)**

Moment Type	Column strip	Middle Ship
-Ve moment (Top) Support	0.65 x 0.75 x 70.8 = 34.515	0.65 x 0.25 x 70.8 = 11.505
+Ve moment (Bottom) Centre	0.35 x 0.6 x 70.8 = 14.868	0.35 x 0.4 x 70.8 = 9.912

Moment distribution in Short span: - **(from clause 31.4.3.2, 31.4.5)**

Moment Type	Column strip	Middle Ship
-Ve moment (Top) Support	0.65 x 0.75 x 36.57 = 18.31	0.65 x 0.25 x 37.57 = 6.105
+Ve moment (Bottom) Centre	0.35 x 0.6 x 37.57 = 7.8	0.35 x 0.4 x 37.57 = 5.259

Check for depth provided
Negative moment is more than positive moment therefore consider only negative moments for depth check
Along longer Span: -

Column strip: -

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

$d_{min} = \sqrt{\frac{34.515\times10^6}{0.13\times25\times1500}} = 84.1\ mm$

$d_{provided} = D - d^{-}$ = $273 - 35 = 238\ mm$ $(d_{provid} > d_{min})$

Middle Strip: -

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

$d_{min} = \sqrt{\frac{11.505\times10^6}{0.13\times25\times1500}} = 48.578\ mm$

$d_{provided} = D - d^{-}$ = $273 - 35 = 238\ mm$ $(d_{provid} > d_{min})$

Along Shorter Span: -

Column strip: -

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

$d_{min} = \sqrt{\frac{18.31\times10^6}{0.13\times25\times2500}} = 61.28\ mm$

$d_{provided} = D - d^{-}$ = $273 - 35 = 238\ mm$ $(d_{provid} > d_{min})$

Middle Strip: -

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

$d_{min} = \sqrt{\frac{5.259\times10^6}{0.13\times25\times1500}} = 32.84\ mm$

$d_{provided} = D - d^{-}$ = $273 - 35 = 238\ mm$ $(d_{provid} > d_{min})$

Depth provided is sufficient from BM consideration

Along short span: -

Shear Strength Check [Punching Shear] :

$T_v = \frac{V_u}{b\ d}$ (31.6.2.61)

Critical Surface is at distance $\frac{d}{2}$

b_0= (2x1.5)+(1.5x1.5) = 5.25

W = 0.675x0.675x 15 KN/m^2 = 6.8 KN (0.45+$\frac{d}{2} + \frac{d}{2}$ =0.675 (d=0.225))

Total load on Column = $3 \times 4 \times 15 = 180$

$V_u = 180 - 6.8$ = 173.2 KN

$\tau_v = \frac{V_u}{b_0\ d} = \frac{173.2\times10^3}{5.25*0.275} = 0.119 N/mm$^2 =

$\tau_v \ngtr$ K3 τ_v $K_s = 0.5 + \beta_c$= 0.5+1= 1.5

Reinforcement calculation: -

Along long span

Negative BM

(i) Column strip [Mu= 34.515*10^6 , b=1500 , d=275

$M_u = 0.85 f_y A_{st}.d[1 - \frac{A_{st}f_y}{bdf_{ck}}]$

$$A_{st} = 0.5 \times \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$$

A_{st} = 566.6 mm^2

$$A_{st\,min} = 0.12\%\ Cross\ section\ of\ strip$$
$$= \frac{0.12}{100} \times 1500 \times 566.6 = 1019.88\ mm^2$$

Using 12 mm diameter $bars$,

Spacing $= \frac{\frac{\pi}{4} \times 12^2}{1019.88} \times 1500 = 166.33$ mm c/c

Along long span

+ve BM

(i) Column strip [Mu= 14.868*10^6 , b=1500 , d=275

$$M_u = 0.85 f_y A_{st}.d[1 - \frac{A_{st} f_y}{b d f_{ck}}]$$

$$A_{st} = 0.5 \times \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$$

A_{st} = 237.9 mm^2

$$A_{st\,min} = 0.12\%\ Cross\ section\ of\ strip$$
$$= \frac{0.12}{100} \times 1500 \times 113.5 = 428.22\ mm^2$$

Using 12 mm diameter $bars$,

Spacing $= \frac{\frac{\pi}{4} \times 12^2}{428.22} \times 1500 = 396.16$ mm

Along long span

Negative BM

middle strip [Mu= 11.505*10^6 , b=2500 , d=175

$$M_u = 0.85 f_y A_{st}.d[1 - \frac{A_{st} f_y}{b d f_{ck}}]$$

$$A_{st} = 0.5 \times \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$$

A_{st} = 183.4 mm^2

$$A_{st\,min} = 0.12\%\ Cross\ section\ of\ strip$$
$$= \frac{0.12}{100} \times 2500 \times 183.4 = 550.2 mm^2$$

Using 12 mm diameter $bars$,

Spacing $= \frac{\frac{\pi}{4} \times 12^2}{550.2} \times 1500 = 308.33$ mm

Along long span

+VE BM

(i) middle strip [Mu= 9.912*10^6 , b=2500 , d=175

$M_u = 0.85 f_y A_{st}.d[1 - \frac{A_{st} f_y}{b d f_{ck}}]$

$A_{st} = 0.5 \times \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$

A_{st}= 157.86 mm^2

$A_{st\,min} = 0.12\%\ Cross\ section\ of\ column\ strip$

$= \frac{0.12}{100} \times 2500 \times 157.86 = 473.58\ mm^2$

Using 12 mm diameter $bars,$

Spacing $= \frac{\frac{\pi}{4} \times 12^2}{473.58} \times 2500 = 597.033$ mm

Along short span

Negative BM

(ii) Column strip [Mu= 18.31*10^6 , b=1500 , d=275

$M_u = 0.85 f_y A_{st}.d[1 - \frac{A_{st} f_y}{b d f_{ck}}]$

$A_{st} = 0.5 \times \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$

A_{st} = 295.45 mm^2

$A_{st\,min} = 0.12\%\ Cross\ section\ of\ column\ strip$

$= \frac{0.12}{100} \times 1500 \times 295.45 = 531.81\ mm^2$

Using 12 mm diameter $bars,$

Spacing $= \frac{\frac{\pi}{4} \times 12^2}{531.81} \times 1500 = 318.99$ mm

Along short span

+ve BM

(ii) Column strip [Mu= 7.8*10^6 , b=1500 , d=275

$M_u = 0.85 f_y A_{st}.d[1 - \frac{A_{st} f_y}{b d f_{ck}}]$

$A_{st} = 0.5 \times \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$

A_{st} = 124.4 mm^2

$A_{st\,min} = 0.12\%\ Cross\ section\ of\ column\ strip$

$= \frac{0.12}{100} \times 1500 \times 124.4 = 223.92\ mm^2$

Using 12 mm diameter $bars,$

Spacing $= \frac{\frac{\pi}{4} \times 12^2}{223.92} \times 1500 = 757.61$ mm

Along short span

Negative BM

middle strip [Mu= 6.105*10^6 , b=1500 , d=175

$M_u = 0.85 f_y A_{st}.d[1 - \frac{A_{st} f_y}{b d f_{ck}}]$

$A_{st} = 0.5 \times \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$

A_{st} =97.26 mm^2

$A_{st\,min} = 0.12\%\ Cross\ section\ of\ column\ strip$

$= \frac{0.12}{100} \times 1500 \times 97.26 = 175.06\, mm^2$

Using 12 mm diameter bars

Spacing $= \frac{\frac{\pi}{4} \times 12^2}{175.06} \times 1500 = 969.07$ mm

Along Short span

+VE BM

(ii) middle strip [Mu= 5.25*10^6 , b=1500 , d=175]

$M_u = 0.85 f_y A_{st}.d[1 - \frac{A_{st} f_y}{b d f_{ck}}]$ $(f_{ck} = 25, f_y = 415)$

$A_{st} = 0.5 \times \frac{f_{ck}}{f_y}\left(1 - \sqrt{1 - \frac{4.6 \times M_u}{f_{ck} \times b \times d^2}}\right) \times b \times d$

= 75.6 mm^2

$A_{st\,min} = 0.12\%\ Cross\ section\ of\ column\ strip$

$= \frac{0.12}{100} \times 1500 \times 75.6 = 136.08\, mm^2$

Using 12 mm diameter *bars*,

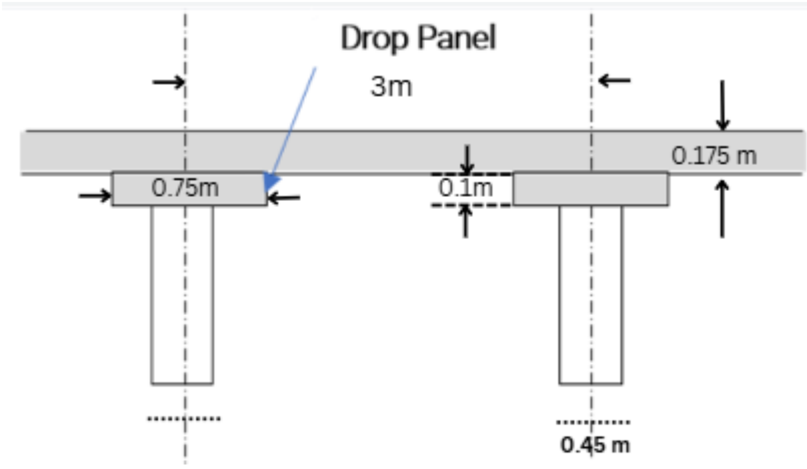
Spacing $= \frac{\frac{\pi}{4} \times 12^2}{136.08} \times 1500 = 1246.66$ mm

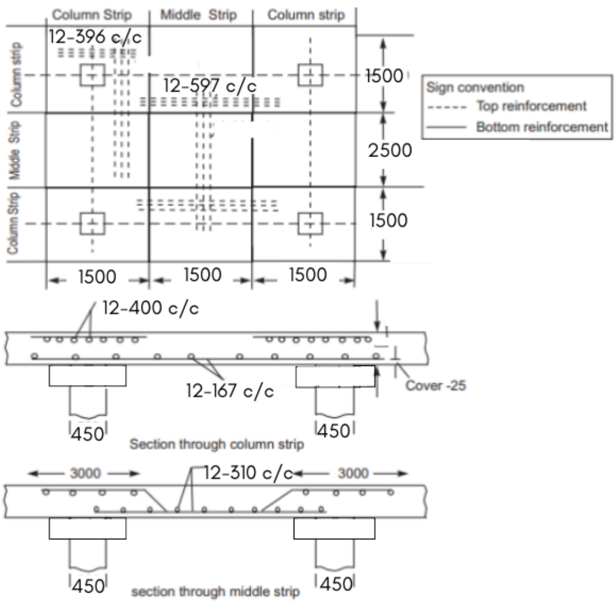
Short Span

	-Ve Reinforcement calculation	+Ve Reinforcement calculation
Column span	318.99 mm c/c	757.61 mm c/c
Middle span	969.07 mm c/c	1246.66 mm c/c

Longer Span

	-Ve Reinforcement calculation	+Ve Reinforcement calculation
Column span	166.33 mm c/c	396.16 mm c/c
Middle span	308.33 mm c/c	597.033 mm c/c





	-Ve Reinforcement calculation	+Ve Reinforcement calculation
Column span	639.20 mm c/c	1494.6 mm c/c
Middle span	1857.7mm c/c	2377.9 mm c/c

SHEAR WALL DESIGN

Thickness of shear Wall = 250 mm
Length of shear wall = 4000 mm
Height of shear Wall = 18.5 m
Grade of concrete = 25 $\frac{N}{mm^2}$
Grade of steel = 415 $\frac{N}{mm^2}$
Axial Load = 2727.4 kN
Shear Force = 711.9 kN
Moment = 6105.09 kN-m

<u>Ratio</u>

$\frac{\text{Length of wall}}{\text{Thickness}} = \frac{4000}{250} = 16 > 4$ (OK) ...Cl 10.1.3

<u>Min Thickness of shear wall</u>

250mm >150 mm .. (OK) …Cl 10.1.2

<u>Eff depth of shear wall(dw)</u>

d_w = 0.8×Lw = 0.8× 4000 = 3200 mm

<u>Section Classification</u>

$\frac{h_w}{L_W} = \frac{18000}{4000} = 4.5 > 2$ (Slender Wall) .. Cl 10.1.4

<u>Max spacing of vertical and horizontal reinforcement is min of</u>

a) $\frac{4000}{5}$ = 800 mm
b) 3 × 250 = 750 mm
c) 450 mm
So, max spacing 450 mm …Cl 10.1.9

<u>Max diameter of bar to be provided</u>

$\frac{1}{10}$ of the thickness of slab = $\frac{250}{10}$ = 25 mm .... Cl 10.1.8

Requirement of boundary element

Extreme fibre Stress = $\frac{P}{t_w \times L_w} + \frac{6M}{t_w \times L_w^2} = \frac{2727.4 \times 1000}{250 \times 4000} + \frac{6 \times 6105.09 \times 10^6}{250 \times 4000^2} = 11.8 \ \frac{N}{mm^2}$

Without boundary element max stress = 0.2fck = 0.2×25 = 5 <11.8 $\frac{N}{mm^2}$ ….Cl 10.4.1
So, we need to provide boundary elements,
Provide boundary element of 500 mm X 500 mm
L_w = 4000 – (500 × 2) = 3000 mm
Minimum required reinforcements …Table-1, Cl-10.2

$p_{v,be} = \frac{0.8 \times 500 \times 500}{100} = 2000\ mm^2$

$p_{v,web} = \frac{0.25 \times 250 \times 3000}{100} = 1875\ mm^2$

Moment of resistance of web portion

$\frac{x_u^*}{L_w} = \frac{0.0035}{0.0035+\frac{0.87fy}{E_s}} = \frac{0.0035}{0.0035+\frac{0.87\times 415}{2\times 10^5}} = 0.66$

$\lambda = \frac{P_u}{f_{ck} \times t_w \times L_w} = \frac{2727.4}{25\times 250\times 4000} = 0.1$

p = vertical reinforcement ratio = $\frac{A_{st}}{t_w \times L_w} = \frac{1875}{250\times 3000} = 0.0025$

$\Phi = \frac{0.87 \times f_y \times p}{f_{ck}} = \frac{0.87 \times 415 \times 0.0025}{25} = 0.036$

$\frac{x_u}{L_w} = \frac{\Phi + \lambda}{2\,\Phi + 0.36} = \frac{0.136}{0.432} = 0.31$

Now,

$\frac{x_u}{L_w} < \frac{x_u^*}{L_w}$,

$\beta = \frac{\frac{0.87 f_y}{E_s}}{0.0035} = \frac{0.87 \times 415}{200 \times 1000 \times 0.0035} = 0.51$

$\frac{M_u}{f_{ck} \times t_w \times L_w^2} = \Phi\ [(1 + \frac{\lambda}{\Phi})\ (\frac{1}{2} - 0.416 \times \frac{x_u}{L_w}) - (\frac{x_u}{L_w})^2\ (0.168 + \frac{\beta^2}{3})]$

$\frac{M_u}{f_{ck} \times t_w \times L_w^2} = 0.036[(1+\frac{0.1}{0.036})(\frac{1}{2} - 0.416 \times \frac{0.416}{0.31}) - (0.31^2)\ (0.168+(\frac{0.621^2}{3})]$

$\frac{M_u}{f_{ck} \times t_w \times L_w^2} = 0.036[(3.77)(0.37)-(0.09)(0.296)] = 0.036[1.39 - 0.026] = 0.05$

$M_u = 0.05 \times 25 \times 1000 \times 0.25 \times 3 \times 3 = 2812.5$ kN-m

Resisting moment from boundary elements,

$M_{be} = 6105.09 - 2812.5 = 3292.59$ kN-m

$P_{eq} = \frac{M_{be}}{\text{lever arm}} = \frac{3292.59}{4-0.5} = 940.74$ kN

Fraction of Boundary Element Area = $\frac{500 \times 500}{4000 \times 500} = 0.125$

Load on boundary element due to fraction = 0.125 × 2727.4 = 340.9

Compressive Load = 340.9 + 940.74 = 1281 kN

Tensile Load = 340.9 - 940.74 = -599.8 kN

Vertical shall be more than 0.8% in boundary elements …Cl 10.4.3

$A_{st} = \frac{500 \times 500 \times 0.8}{100} = 2000\ mm^2$

As boundary element can be considered as short column ….Cl 10.4.2

$P_u = 0.4 f_{ck} \times A_c + 0.67 f_y \times A_{st}$ …IS 456 2000 Cl 39.3

$P_u = 0.4 \times 25 \times [(500 \times 500) - 2000] + 0.67 \times 415 \times 2000 = 2480000-556100 = 1923.9$ kN >1281 kN (OK)

Providing 12, 16mm diameter bars = 200.96 ×12 = 2411.52 mm^2

Web Design

Longitudinal Reinforcement:

Max shear stress,

$(T_C)_{max} = 3.1\ \frac{N}{mm^2}$ for M25 …IS 456 - Table 20

$(p_{v,net})_{min} = 0.0025 + 0.01375\frac{t_w}{L_w} = 0.0025 + 0.01375(\frac{250}{4000}) = 0.00335 = 0.335\%$

$T_v = \frac{V_u}{t_w \times d_w} = \frac{711.9 \times 1000}{250 \times 3200} = 0.88$

$T_c = 0.404\ \frac{N}{mm^2}$ ………………IS 456 – 2000

$T_c < T_v < (T_v)_{max}$

Need to design for shear,

$V_{us} = V_u - T_c \times t_w \times d_w$

$V_{us} = 711.9 - \frac{0.404 \times 250 \times 3200}{1000} = 711.9 - 323.2 = 388.7$ kN

$(p_h)_{min} = 0.0025 + 0.5(\frac{h_w}{L_w} - 2)(p_{v,web} - 0.025)$

Putting p_v, web value = 0.0025,

$(p_h)_{min} = 0.0025$

Area of horizontal reinforcement$_{min}$ = 0.0025* 250*1000 = 625 mm^2

STAIRCASE DESIGN

Height between floor = 3.5 m
Riser = 150 mm
Tread = 300 mm
Width of flight = 2 m
Landing Width = 2 m
Slab Thickness = 0.15 m
c/c distance between supports = 8.3 m
Clear Cover = 20 mm
Diameter of bars = 12 mm
Effective Depth = 150 – 20 – 12/2 = 124 mm

For going,

Self-weight of Slab = 25× 0.15 = 3.75 kN/m^2

Self-weight of Steps (for one flight) = $\frac{25\times0.15}{2}$= 1.875 kN/$m2$

Floor finishing Load = 1.5 kN/$m2$

Live Load = 3 kN/m2

Total = 10.125 kN/m2

Factored Load = 1.5× 10.56 =15.187 kN/$m2$

For Landing,
Self-weight of Slab = 25×0.15 = 3.75 kN/m2

Finishing Load = 1.5 kN/m2

Live Load = 3 kN/m2

Total = 8.25 kN/m2

Factored Load = 12.375 kN/m2

Design Moment,

Width = 1000 mm (Design for 1m strip)
Reaction = $[(2 \times 2.15 \times 12.375) + (4 \times 15.187)]/2$ = [53.21 +60.74]/2 = 57 kN/m

Maximum moment at midspan =$[(57 \times 4.15)] - \left[(12.375 \times 2.15) \times \left(4.15 - \frac{2.15}{2}\right)\right] -$ $\left[\left(15.187 \times \left(\frac{(4.15-2.15)^2}{2}\right)\right)\right]$

= 236.55 – 81.6 - 30.3 = 124.6 kNm/m

R = $\frac{M_u}{bd^2} = \frac{124.6}{10^3 \times 0.136^2}$= 6.8 MPa

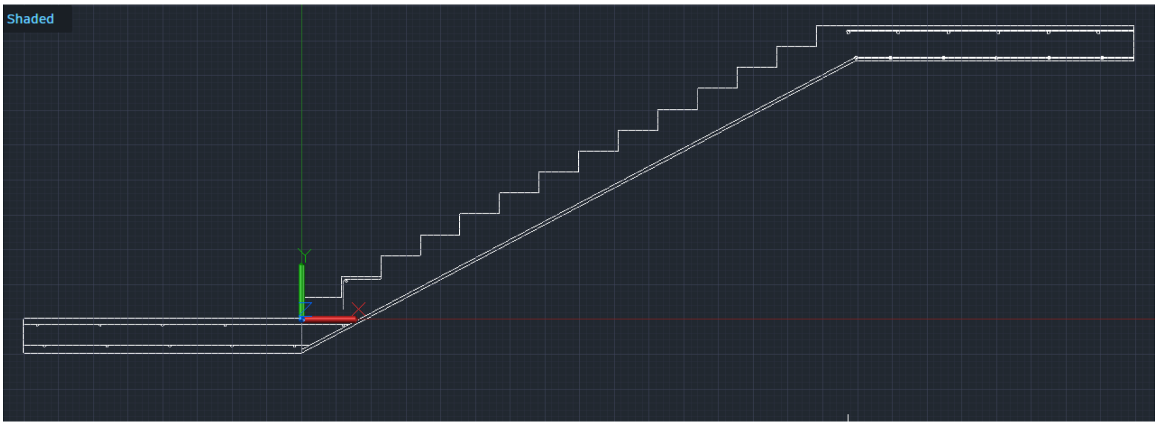
$$A_{st,req} = \frac{f_{ck}}{2f_y}\left(1 - \sqrt{1 - \frac{4.598 \times M_u}{f_{ck} \times b \times d^2}}\right) \times bd$$

$A_{st,req} =$ 240 mm^2/m

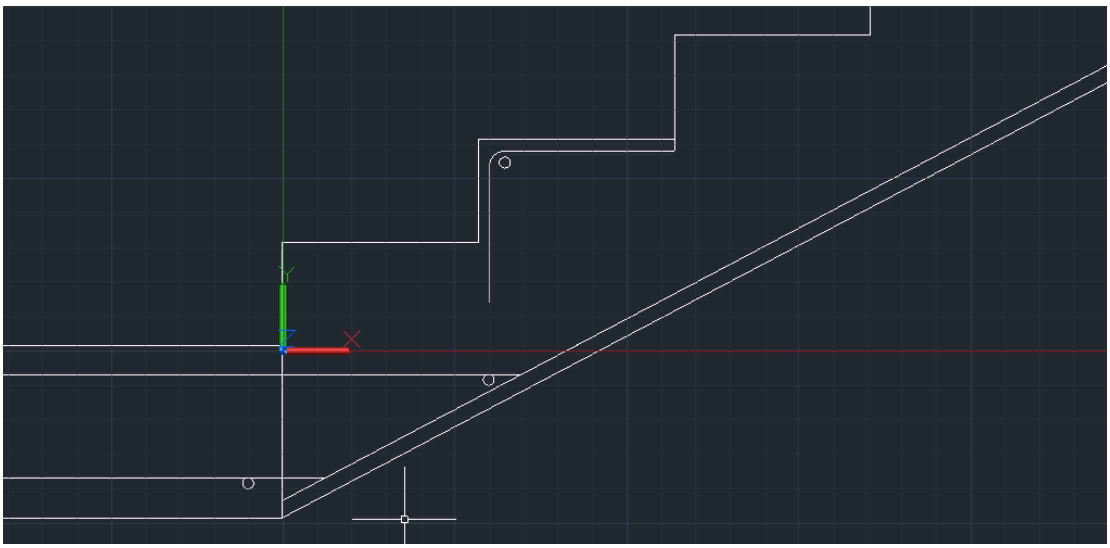
Spacing of bars 12 dia bars = 113.09*1000/240 = 471 mm c/c

Distributors
(Ast)req = 0.0012 b*r = 0.0012*1000*150 = 180 mm^2/m

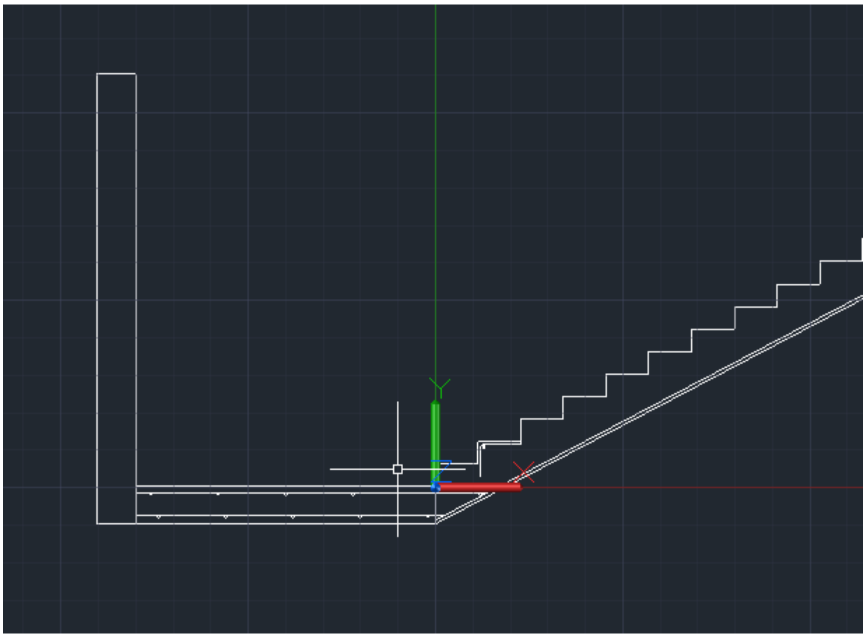
Spacing
10 dia bars = 78.5*1000/180 = 436.33 mm



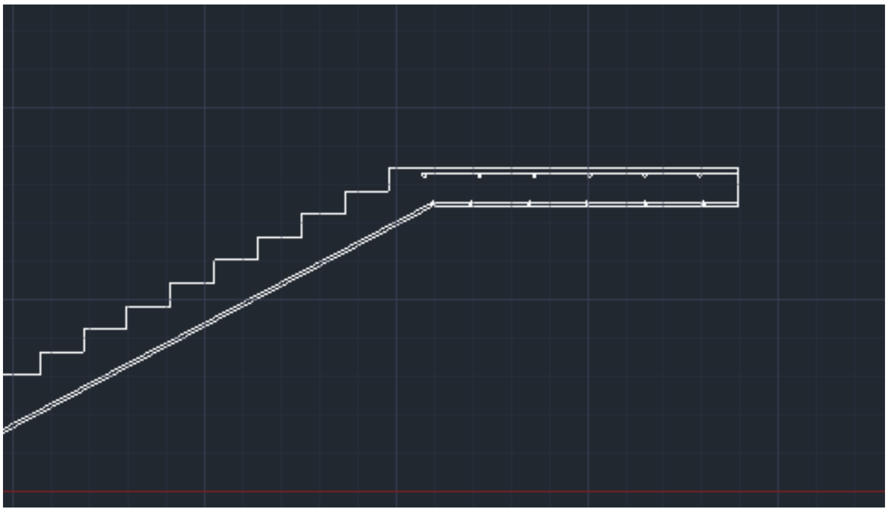
Figure



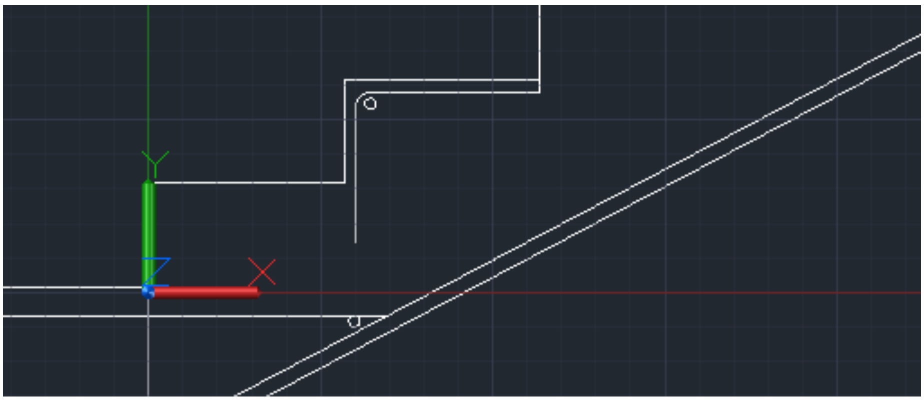
Figure



Figure



Figure



Figure

DESIGN OF FOUNDATION

For foundation, we used the conventional method and IS CODE 2950-1 (1981).

RAFT DIMENTSION	
Length of Raft in x-direction, B (m)	12
length of Raft in y-direction,L (m)	24
Depth of the foundation (m), Df	3
Area of the foundation (m^2)	288
Iyy	3456
Ixx	13824

		Column load in x, (KN)				
		1	2	3	4	sum
	A	540.955	679.724	1034.96	1159.328	3414.967
	B	694.708	561.188	569.201	1240.515	3065.612
	C	645.685	653.934	614.42	1114.534	3028.573
Column Load in y, (KN)	D	558.568	625.999	583.399	519.051	2287.017
	E	437.094	632.361	583.399	644.982	2297.836
	F	643.823	653.934	614.14	1114.534	3026.431
	G	707.629	569.88	580.903	1250.843	3109.255
	H	540.955	682.286	1034.96	1159.328	3417.529
	Total	4769.417	5059.306	5615.382	8203.115	
	Total load coming on footing	23647.22				

eccentricity in y-direction ey	0.01641859
eccentricity in x-direction ex	0.535591922

$$q = \frac{Q}{A'} \pm \frac{Qe_y}{I_x} y \pm \frac{Qe_x}{I_y} x$$

Where

q is soil pressure at different points

Q = total load coming from all columns = 23647.22 KN

A = total area of the foundation = 12*24 = 288 m^2

ey = eccentricity in y direction = 0.0164

ex = eccentricity in x direction = 0.536

Ixx = moment of inertia in x direction = BD^3/12 = 12*24^3/12 = 13824 m^4

Iyy = moment of inertia in y direction = DB^3/12 = 24*12^3/12 = 3456 m^4

Soil pressure at different points , q (KN/m^2)																
	1				2				3				4			
	x(m)	y(m)	q (kN/m^2)	Safety	x(m)	y(m)	q (kN/m^2)	Safety	x(m)	y(m)	q (kN/m^2)	Safety	x(m)	y(m)	q (kN/m^2)	Safety
A	-4	11	67.75847758	SAFE	-2	11	75.0879104	SAFE	1	11	86.0820598	SAFE	5	11	100.740925	SAFE
B	-4	8	67.67422106	SAFE	-2	8	75.0036539	SAFE	1	8	85.9978032	SAFE	5	8	100.656669	SAFE
C	-4	5	67.58996455	SAFE	-2	5	74.9193974	SAFE	1	5	85.9135467	SAFE	5	5	100.572412	SAFE
D	-4	2	67.50570804	SAFE	-2	2	74.8351409	SAFE	1	2	85.8292902	SAFE	5	2	100.488156	SAFE
E	-4	-2	67.39336603	SAFE	-2	-2	74.7227989	SAFE	1	-2	85.7169482	SAFE	5	-2	100.375814	SAFE
F	-4	-5	67.30910952	SAFE	-2	-5	74.6385424	SAFE	1	-5	85.6326917	SAFE	5	-5	100.291557	SAFE
G	-4	-8	67.22485301	SAFE	-2	-8	74.5542859	SAFE	1	-8	85.5484352	SAFE	5	-8	100.207301	SAFE
H	-4	-11	67.1405965	SAFE	-2	-11	74.4700294	SAFE	1	-11	85.4641787	SAFE	5	-11	100.123044	SAFE

Soil Pressure per strip in X-direction					
Average contact prussure (Kn/m^2)	Width of the strips, B1 (B)	Length of the strips, B (m)	Average laod on the strip, Qav (KN)	Modified average soil pressure, qav (KN/m^2)	Modification factor for the column load
82.41734332	2.5	12	2943.74365	98.12478832	0.862012327
82.33308681	3	12	3014.801563	83.74448785	0.983425679
82.2488303	3	12	2994.765445	83.18792904	0.988837134
82.16457378	3.5	12	2868.964549	68.30867975	1.254457028
82.05223177	3.5	12	2872.014867	68.38130636	1.249878088
81.96797526	3	12	2988.639055	83.01775152	0.987512702
81.88371875	3	12	3028.534438	84.1259566	0.974038616
81.79946224	2.5	12	2935.756434	97.85854779	0.859028975

Average Contact Pressure on the strip qav = Total pressure on the strip /4 =

$$(A1 + A2 + A3 + A4)/4 = 82.417 \text{ KN/m^2}$$

Average Load on the strip Qav = Total load on the strip + qav*B*B1

Modified average soil pressure = Qav / (B*B1)

Column Load modification factor , f = Qav/(Total load on the strip)

Modified column load = fQi , (ith column load)

Modified Column load (KN)			
1	2	3	4
466.30988	585.93047	892.14828	999.35503
683.19369	551.88669	559.76688	1219.9543
638.4773	646.63422	607.56131	1102.0926
700.69955	785.28885	731.84898	651.12717
546.31421	790.37416	729.17763	806.14887
635.78339	645.76813	606.47105	1100.6165
689.25797	555.08513	565.82195	1218.3694
464.69602	586.10344	889.06063	995.89634

Positive moment of strips = 3763242.232 N-m
Negative Moment of strips = 1980453.608 N-m

In Raft foundation provide

Clear cover = 100mm
Diameter of the main bar = 25 mm
Spacing between the bars = 180 mm

DESIGN FOR FIRE

Building classification: Subdivision A-4 apartment houses (flats)
Fire zone 1
Construction type 3

Floor area ratio(FAR) = area on which load is constructed/usable building area

Survey area total= 4142 m2
Area of building=1080 m2

FAR =3.835
Height of building= 18.5 m

IS : 1643 - 1988

TABLE 1 COMPARATIVE FLOOR AREA RATIOS FOR OCCUPANCIES FACING ONE PUBLIC STREET OF AT LEAST 9 m WIDTH

(*Clause* 4.1)

Occupancy Classification	Floor Area Ratio's Type of Construction			
	Type 1	Type 2	Type 3	Type 4
Residential	UL	2·0	1·4	1·0
Educational	UL	2·0	1·4	1·0
Institutional	UL	1·5	1·0	0·8
Assembly	UL	1·0	0·7	0·5
Business	UL	2·9	2·3	1·6
Mercantile	8·0	1·8	1·4	1·0
Industrial	7·5	1·9	1·6	1·3
Storage (*see* Note 4)	6·0	1·5	1·3	1·0
Hazardous (*see* Note 4)	2·8	1·1	0·9	NP

EXIT REQUIREMENTS

Protecting the safety of occupants during a fire is essential. Pathways must be designed to allow individuals to evacuate quickly and reach a safe area with minimal risk of encountering smoke, toxic fumes, debris, or other hazards. Every building intended for human use should include multiple unobstructed exits, ensuring occupants can evacuate safely and efficiently during fires or other emergencies.

PLACING OF FIRE EXIT SIGNS

TABLE 1 OCCUPANT LOAD

(*Clause* 2.4)

SL No	GROUP OF OCCUPANCY (*see* IS : 1641-1988*)		OCCUPANT LOAD, FLOOR AREA IN m²/PERSON
1.	Residential	(A)	12·5
2.	Educational	(B)	4
3.	Institutional	(C)	15†
4.	Assembly:	(D)	
	a) With fixed or loose seats and dance floors		0·6†
	b) Without seating facilities including dining rooms		1·5‡
5.	Mercantile:	(F)	
	a) street floor and sales basement		3
	b) upper sale floors		
6.	Business and industrial	(E and G)	10
7.	Storage	(H)	30
8.	Hazardous	(J)	10

For our building, our Occupant Load and floor Area in m2 /person is 12.5.

EXIT DOORS AND CAPACITIES

TABLE 2 OCCUPANTS PER UNIT EXIT WIDTH

(*Clauses* 2.5.2 *and* 2.5.3)

SL No.	GROUP OF OCCUPANCY (*See* IS : 1641-1988*)	NUMBER OF OCCUPANTS		
		Stairways	Ramps	Doors
(1)	(2)	(3)	(4)	(5)
1.	Residential (A)	25	50	75
2.	Educational (B)	25	50	75
3.	Institutional (C)	25	50	75
4.	Assembly (D)	40	50	60
5.	Business (E)	50	60	75
6.	Mercantile (F)	50	60	75
7.	Industrial (G)	50	60	75
8.	Storage (H)	50	60	75
9.	Hazardous (J)	25	30	40

2.5.1 The unit of exit width, used to measure the capacity of any exit, should be 50 cm. A clear width of 25 cm should be counted as an additional half unit. Clear widths less than 25 cm should not be counted for exit width.

EXIT DOORS AND OTHER CAPACITIES

Exit requirement- IS 1644 ,Cl 2

Doorways ,width of road ,Internal staircase ,horizontal exits

Classes of fire extinguishers:
https://law.resource.org/pub/in/bis/S03/is.2171.1999.svg.html#:~:text=The%20extinguisher%20shall%20be%20marked,is%20filled%20in%20the%20extinguisher.

<u>Once final building design made</u>

Horizontal Exits (IS 1644, Cl. 2.13)

1. The width of a horizontal exit must be equal to that of the exit doorway, with a minimum width of 100 cm.
2. At least one fire door within a horizontal exit must be of the self-closing type.
3. If there is a difference in elevation between interconnected spaces requiring horizontal exits, ramps with a maximum slope of 1 in 10 should be provided, and the use of steps should be avoided.
4. Doors within horizontal exits must remain operable from both sides at all times, ensuring unrestricted accessibility.
5.

<u>Escape Lighting Luminaires</u>
Signage Requirements: Clearly marked signage is required for all exits, emergency exits, and escape routes. These signs must comply with Indian Standards for graphical clarity, ensuring they are easily recognizable and informative.
Mounting Height: Luminaires (light fixtures) should be installed at a minimum height of 2 meters above floor level, ideally at a lower height when possible, to provide effective guidance during an emergency.

Placement of Lighting: Strategic lighting placement is essential throughout the building, including near exit doors, final exits, fire alarm points, firefighting equipment, corridor intersections, and staircases, to maintain well-lit and safe escape routes.
Intersection and Stairwell Focus: Extra lighting should be directed at critical areas such as corridor intersections and staircase landings, ensuring these points are well-lit to aid safe navigation during evacuations.

Exit Doors and Capacity Requirements: Clause 4.4.1

- The capacity of exits is measured in units of 500 mm. If the clear width is 250 mm or more, it counts as an additional half unit; clear widths under 250 mm are excluded from the calculation.
    - *Note:* For Type 3 construction, all occupants on a given floor must evacuate within 1 minute.
- Occupants per unit exit width in residential buildings are specified as:
    - Stairways - 25 occupants
    - Ramps - 50 occupants
    - Doors - 75 occupants

Doorways: IS 1644, Cl 4.7

1. All exit doors must lead to an enclosed stairwell, a horizontal exit within a corridor, or a passageway that provides a continuous, secure evacuation route.
2. Exit doorways must be at least 100 cm wide and 200 cm high.

3. In buildings with a central corridor, room doors should swing inward to avoid obstructing corridor movement.

Road Width Requirements: IS 1643, Cl 6

1. The building must be adjacent to a main street that is at least 12 meters wide, which should connect to another street of equal width.
2. Roads leading to the building should not end in a dead-end.
3. Parking is not permitted in the open spaces surrounding the building.
4. The main entrance should be wide enough for firefighting vehicles to access, with a minimum width of 4.5 meters.

Internal Staircase Specifications: NBC, Cl 4.9

1. All internal staircases must be constructed from non-combustible materials.
2. Stair tread width should be at least 250 mm, excluding nosing, in residential buildings.
3. Riser height should not exceed 190 mm, with no more than 15 risers in a single flight.
4. For buildings 15 meters or taller, access to the main staircase should be through a fire- or smoke-resistant door with a minimum fire rating of 2 hours, though this can be reduced to 1 hour for residential buildings.
5. An exit sign, complete with a directional arrow, should be mounted at an appropriate height above floor level. The sign must be illuminated by electric light connected to the corridor circuits.

Firefighting Equipment for Apartment Houses:

- Fire Extinguishers
- Hose Reels
- Downcomers
- Fire Hydrants
- Fire Sprinkler Systems
- Manually Operated Electric Fire Alarm System

Class of Fire	Type of Fire	Type of Fire Extinguisher Used	Comments
Class A	Ordinary combustibles (wood, paper, cloth, plastics)	Water, Foam, ABC Dry Chemical	Water and foam extinguishers are ideal for cooling and soaking materials to prevent re-ignition. ABC dry chemical extinguishers are also effective.
Class B	Flammable liquids (gasoline, oil, paint)	CO_2, Dry Chemical, Foam	CO_2 extinguishers smother fires without leaving residue. Dry chemical extinguishers are versatile and effective but may leave residue.
Class C	Electrical fires (wiring, circuit breakers, appliances)	CO_2, Dry Chemical	CO_2 and dry chemical extinguishers are safe as they don’t conduct electricity. Avoid water or foam extinguishers for electrical fires.

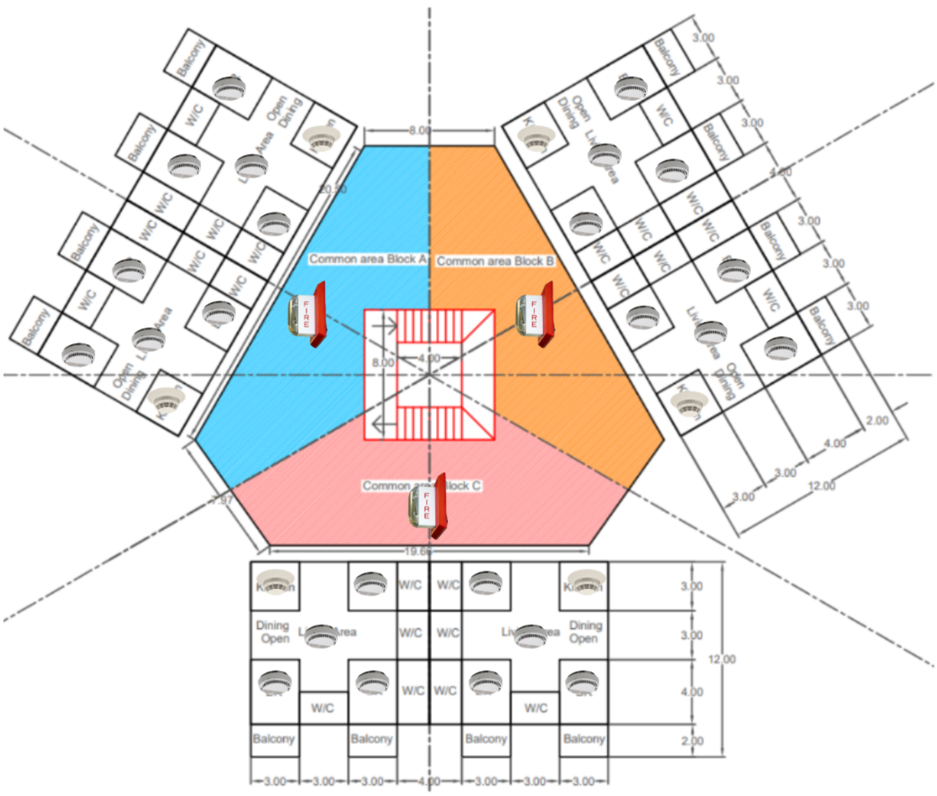
Class D	Combustible metals (magnesium, titanium, potassium)	Dry Powder	Specialized dry powder extinguishers are required, as other types may exacerbate metal fires.
Class K	Cooking oils and fats	Wet Chemical	Wet chemical extinguishers are designed to cool and smother high-temperature grease fires commonly found in commercial kitchens.

Fire& heat detection and alarm placement

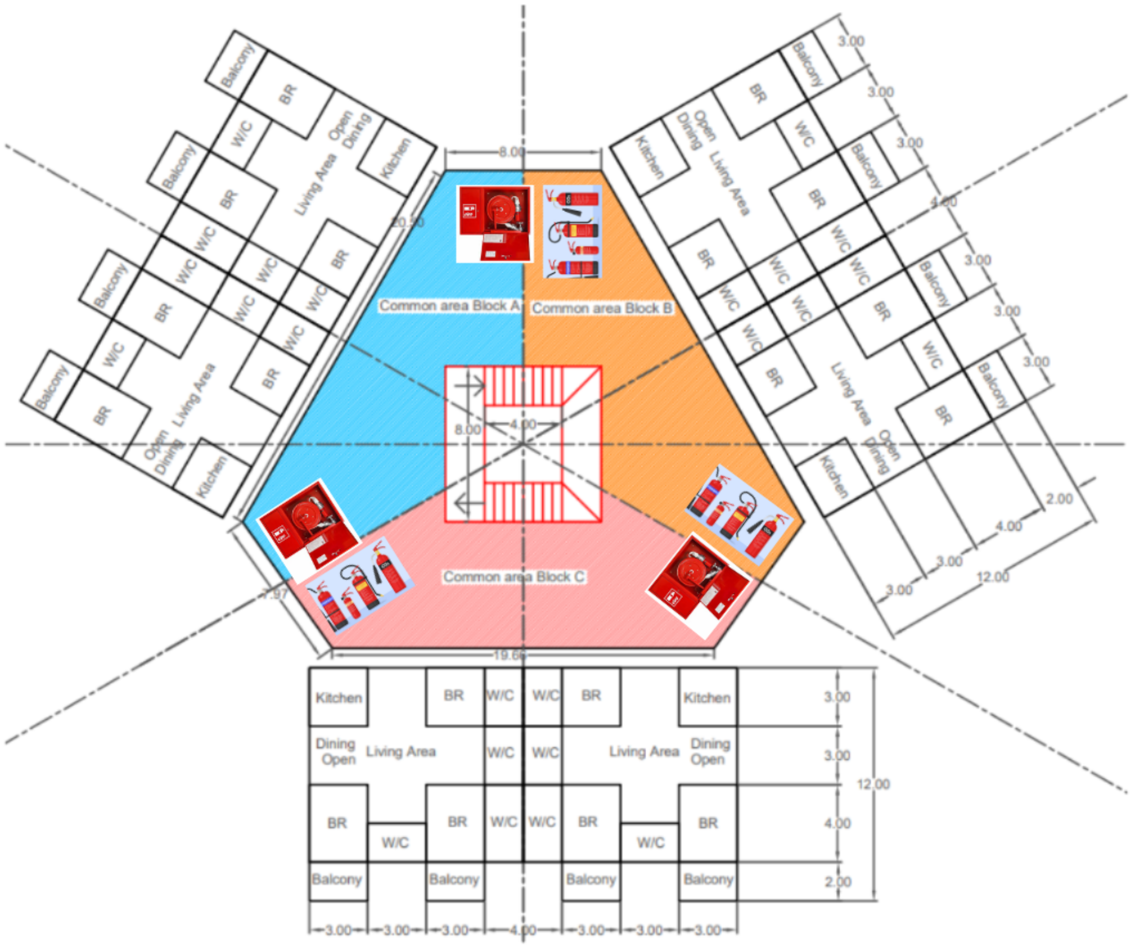
Heat detector

Fire detector

Fire alarm



Fire extinguisher and hose reel placement



WATER SUPPLY

Side Building		
Total Bedroom per floor	6	
Total people per room	2	
Total people per floor	12	
Total floor(including ground floor)	4	
Total People	48	
Water requirement of each person per day(litre)	135	

Total water requirement per day(litre)	6480	
Building Height(m)	18.5	
Water Tank Capacity(litre)	1000	
Diameter of tank(m)	1.1	
Height of tank(m)	1.05	
Volume(m^3)	0.9973425	
Total Tank Required	6.48	Around 7
Final Total Water required(litre)	7000	
Time for filling 1000 litre 7 tanks(assumption)(hour)	1	
Flow Rate(m^3/h)	7	
Efficiency of pump(assumption)	0.8	80%
Head of pump(building height + tank height)(metre)	19.55	
Density of water(Kg/m^3)	1000	
Gravity(m/s^2)	9.8	
Power of water pump required(watt)	465.6701389	
Power of water pump required(hp)	0.6242227063	

COST ESTIMATION

Labour type	unskilled/semiskilled/skilled/highly skilled	Minimum wages per hour(for an 8 hr day)
Bhisti	semiskilled	868
Blacksmith class 1	skilled	954
Blacksmith class 2	semiskilled	868
Carpenter class 1	skilled	954
Carpenter class 2	semiskilled	868
Chowkidar	unskilled	783
Beldar	unskilled	783
Mason class 1	skilled	954
Mason class 2	unskilled	783
Sewer man	unskilled	783
Stone chiseler	semi skilled	868
Rock cutter	semiskilled	868
Rock breaker	semiskilled	868
Driller	semiskilled	868
Technician	highly skilled	1035

Polisher class 1	skilled	954
Polisher class 2	semi skilled	868
Specialised technicians	highly skilled	1035
White washer	unskilled	783
Tile turner	unskilled	783
Mazdoor	unskilled	783
Concrete (hand mixer)	unskilled	783
Chowkidar	unskilled	783
Charpoy stringer	semiskilled	868
Mate	semiskilled	868

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APPENDIX

Survey Points	Survey_G3
Rainfall data	RainfallData_G3
Load Calculations	Load Calculations
Foundation Calculation	Foundation Calculation