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Seismic Analysis and Design of Residental Building According to Nbc Code

Prof. Rajiv Manandhar[1], Malvika Adhikari[2], Neelam Yadav[3], Navraaz Budhathoki[4], Rakshya Laxmi Sharma[5]

[1]Asst. Professor Civil Engineering Nepal Engineering College
[2]Engineer Civil Engineering Nepal Engineering College

Abstract
This project explores the seismic design of a six-story reinforced concrete residential building located in Kathmandu one of Nepal's most earthquake-prone regions. Using ETABS and the Response Spectrum Method, we analyzed the building under NBC code.
We focused on key design parameters like base shear, reinforcement detailing. It is focused on the process and outcomes of analysis of building according to Nepal Building Code. Structural drawings were prepared using NBC provisions to meet Nepal's regulatory standards.
This study highlights how codes formulation can impact both safety and cost, and offers practical insights for engineers working in seismic Zone V areas like Kathmandu Valley.

Main Objectives:
- To become familiar with seismic-resistant design of structures.
- To gain familiarity with the Nepal Building Code (NBC).
- To fulfill the requirements of the BE Civil Engineering syllabus.

Specific Objectives:
- Modeling of the building for structural analysis.
- Sectional design and structural detailing of the members.

Brief Description of the Proposed Project
- Type of Project: Structural analysis and design of a residential building
- Building Type: Residential Building
- Location: Kathmandu
- Structural Type: Special moment-resisting RCC framed structure
- Plinth Area: 1477.35 sq.ft
- Number of Storeys: 6, including basement
- Lower Basement Height: 3.2 m
- Type of Slab: Two-way slab
- Type of Staircase: Open well staircase
- Type of Foundation: Mat foundation
- Method of Analysis: Dynamic analysis
- Design Concept: Limit state design

- Concrete Grade: M25
- Reinforcement: Fe500
- Dead Load: Calculated as per IS 875 Part I: 1987
- Live Load: Calculated as per IS 875 Part II: 1987
- Seismic Load: Calculated as per NBC 105:2020
- Preliminary Design: IS 456:2000
- Soil Type: Medium soil
- Bearing Capacity of Soil: 163 KN/m² (assumed)

Units

SI (metric) units are used in this report. Unless specified in figures and drawings, all dimensions should be taken as millimeters.

Interpretation

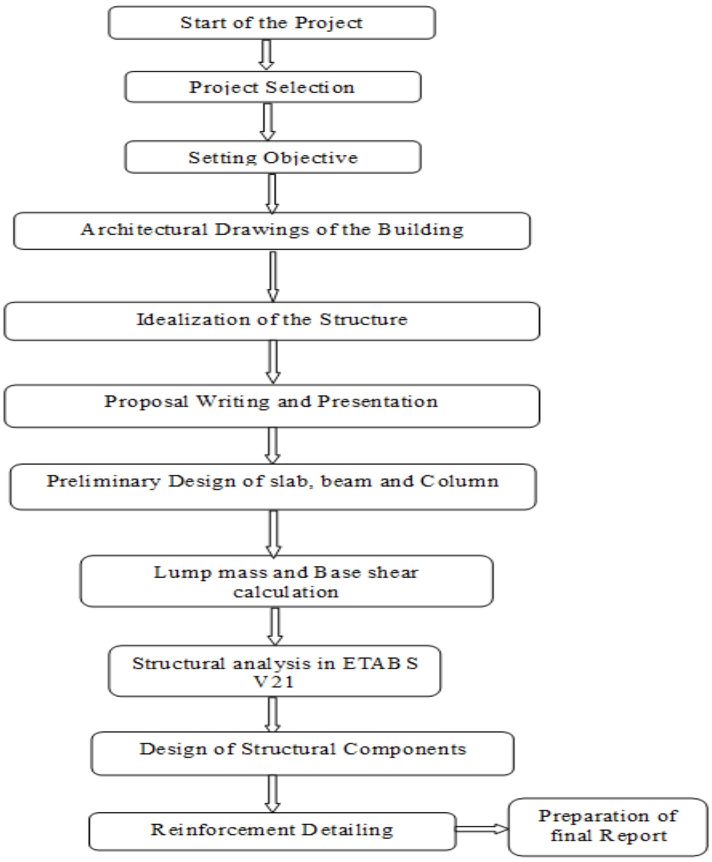
References to clauses of Indian Standards are denoted as IS 456:2000 for structural design. Some clauses are sourced from IS 1893:2002, SP-16, and other relevant reference books.

Detailing

Detailing is performed using IS 13920:1993, Handbook on Concrete Reinforcement and Detailing (SP 34:1987), and other reference books cited in the project.

1. METHODOLOGY

To carry out any project work, various methods or steps are followed from the initial stage to the final stage. The following are the methods adopted during the analysis and design of a nine-story commercial and office building.



1. Seismic Coefficient Method (Static)
2. Response Spectrum Method (Dynamic)

Load Cases

Load cases represent independent loading conditions for which the structure is explicitly analyzed. Earthquake forces occur randomly in all directions. For buildings with lateral load-resisting elements oriented in two principal directions, analysis in the X and Y directions separately is sufficient. The adopted load cases are:

1. Dead Load (DL)
2. Live Load (LL)
3. Earthquake Load in X Direction (EQX)
4. Earthquake Load in Y Direction (EQY)

Load Combinations for Design (IS Codes):

1. 1.5(DL + LL)
2. 1.2(DL + LL ± EQX)
3. 1.2(DL + LL ± EQY)
4. 1.5(DL ± EQX)
5. 1.5(DL ± EQY)
6. 0.9DL ± 1.5EQX
7. 0.9DL ± 1.5EQY

Load Combinations for Design (NBC):

1. 1.2DL + 1.5LL
2. DL + λLL + EQX
3. DL + λLL - EQX
4. DL + λLL + EQY
5. DL + λLL - EQY

Modeling and Analysis of Building

For seismic analysis, the structural analysis program ETABS was used. It includes a special feature for modeling horizontal rigid floor diaphragm systems. A floor diaphragm is modeled as a rigid horizontal plane parallel to the global X-Y plane, ensuring no relative displacement of points on the diaphragm in the **X-Y plane. The following steps were taken:**

- Material properties were defined (Concrete: M25; Reinforcement: Fe500).
- Load cases and their combinations with load factors were specified.
- The structure was analyzed for various load combinations, and results were obtained in the form of shear forces (SF), bending moments (BM), and axial forces (AF).

2. PRELIMINARY DESIGN

2.1 Design data

Concrete Grade	: M25	
Steel Grade	: Fe500	
Live load on slab	: 3 KN/m^2	(IS 875-1987 Table I)
Load due to floor finish	: 1.5KN/m^2	

Live load on roof	: 1.5 KN/m^2	(IS 875-1987 Table II)
Brick masonry	: 19.2KN/m^3	(IS 875-1987 Table I)
RCC Specific Weight	: 25KN/m^3	

Bearing Capacity of Soil : 163 KN/ m^2 (assumed)

2.2 Preliminary design of slab

Refrence slab = 4.11*5.38m
% of steel (0.1% to 0.4%) = 0.2%
fy= yield strength of the steel=500N/mm^2
Lx = Effective length of the member with respect to the x-x axis = 4.1148m
Ly = Effective length of a member with respect to the y-y axis = 5.3848m
LY/LX = 1.30864198 < 2
So, it is Two way slab
From IS456:2000
A = Coefficient = 26
B =Shape factor, buckling modifier, load distribution- factor = 1
δ=Small deflection or strain
For modification factor 'λ' for tension reinforcement
fs= 0.58*fy*Area of x-section of steel required/Area of x-section of steel required
fs=0.58*500*1
fs=290 n/mm
from graph of modification factor is 1.6
Effective Depth d=Lx/αβδλ
d=4.1158/1*26*1.6*1
d=98.8mm
Provide 10mm dia bar and 15mm clear cover
Overall Depth D= d+10/2+15=98.8+10/2+15=118.8mm
Adopt D =125mm

2.3 Preliminary design of beam

For numeric Grid:
Span length,
L= 5380mm (Longest span)
Let us consider effective depth of beam be 1/12 to 1/15th of span

Depth(D)= L/13= 5380/13 = 418.84mm
Assume dia. of bars in beam = 20mm
Assume, eff. Cover 20mm
Overall depth (D)=418.84+20+20/2 =450mm
Adopt depth (D) =450mm
Width of beam = (1/2 to 2/3) of depth
= 450*2/3=300mm
Adopt, b=300mm
Hence, size of beam = 300mm * 450mm
ii) For Alphabetical Grid
Span length, L = 4110mm (Longest span)
Let us consider effective depth of beam be 1/10 to 1/15th of span
Depth(D)=L/13= 4110/13 = 316.15mm
Assume dia. of bars in beam= 20mm
Clear cover= 25mm
Overall depth (D)=316.15+25+20/2 = 351.16mm
Adopt depth (D)=355mm
Width of beam
B= (1/2to2/3)of the depth = 355*2/3=240mm
Adopt, b= 240mm

2.3 Preliminary design of column

load from 5th floor to just above 4th floor

S. N	**Description**	**No**	**Length**	**Breadth**	**Height**	**Quantity**	**Unit wt**	**Load**
1	Dead load							
a.	slab	1	3.378	3.912	0.125	1.652	24.000	39.645
b.	beam along alphabetic grid	1	3.912	0.230	0.230	0.207	24.000	4.967
c.	beam along numeric grid	1	2.692	0.300	0.325	0.262	24.000	6.299
d.	secondary beam							

e.	column	1	0.457	0.457	3.075	0.642	24.000	15.413
f.	floor finish	1	3.378	3.912		13.215	1.500	19.822
g.	false ceiling	1	3.378	3.912		13.215	1.000	13.215
h.								
i.	Parapet wall	1	4.038	0.102	1.000	0.412	19.200	7.909
2	Roof load (upper terrace)	1	3.378	3.912		13.215	1.500	11.893
						Load in KN =		**119.163**

load from 4th floor to just above 3rd floor								
S.N	**Description**	**No**	**Length**	**Breadth**	**Height**	**Quantity**	**Unit wt**	**Load**
1	Dead load							
a.	slab	1	5.384	3.912	0.125	2.633	24.000	63.187
	beam along alphabetic grid	1	3.912	0.230	0.230	0.207	24.000	4.967
	beam along numeric grid	1	5.384	0.300	0.325	0.525	24.000	12.599
	secondary beam							
	column	1	0.457	0.457	3.075	0.642	24.000	15.413
	floor finish	1	5.384	3.912		21.062	1.500	31.593
	false ceiling	1	5.384	3.912		21.062	1.000	21.062
	4"wall with opening	1	5.943	0.102	2.743	1.663	19.200	20.753
	4"wall with opening	1	4.038	0.102	3.073	1.266	19.200	15.799
2	LiveLoad		3.378	3.937		13.299	3.000	23.938
2.1	Roof load (Lower terrace)	1	2.464	3.937		9.699	1.500	8.729
						Load in KN=		**218.039**

load from 3rd floor to just above 2nd floor

S.N	**Description**	**No**	**Length**	**Breadth**	**Height**	**Quantity**	**Unit wt**	**Load**
1	**Dead load**							
a.	slab	1	5.384	3.912	0.125	2.633	24.000	63.187
	beam along alphabetic grid	1	3.480	0.230	0.230	0.184	24.000	4.418
	beam along numeric grid	1	4.927	0.300	0.325	0.480	24.000	11.530
	column	1	0.457	0.457	3.075	0.642	24.000	15.413
	floor finish	1	5.384	3.912		21.062	1.500	31.593
	false ceiling	1	5.384	3.912		21.062	1.000	21.062

	4"wall with opening	1	4.267	0.102	2.743	1.194	19.200	14.900
	4"wall without opening	1	4.369	0.102	2.743	1.222	19.200	23.468
	4"wall with opening	1	0.406	0.102	3.073	0.127	19.200	1.590
	4" wall without opening	1	2.032	0.102	3.073	0.637	19.200	12.229
2	**Live load**	1	5.384	3.912		21.062	3.000	44.231
						Load in KN =		**243.620**

load from 2nd floor to just above 1st floor								
S.N	**Description**	**No**	**Length**	**Breadth**	**Height**	**Quantity**	**Unit wt**	**Load**
1	Dead load							
a.	slab	1	5.384	3.912	0.125	2.633	24.000	63.187
	beam along alphabetic grid	1	3.480	0.230	0.230	0.184	24.000	4.418
	beam along numeric grid	1	4.927	0.300	0.325	0.480	24.000	11.530
	column	1	0.457	0.457	3.075	0.642	24.000	15.413
	floor finish	1	5.384	3.912		21.062	1.500	31.593
	false ceiling	1	5.384	3.912		21.062	1.000	21.062
						0.000		0.000
	4"wall with opening	1	4.267	0.102	2.743	1.194	19.200	14.900
	4"wall without opening	1	4.369	0.102	2.743	1.222	19.200	23.468
	4"wall with opening	1	0.406	0.102	3.073	0.127	19.200	1.590
	4" wall without opening	1	2.032	0.102	3.073	0.637	19.200	12.229
2	Live load	1	5.384	3.192		17.186	3.000	41.246
						Load in KN=		**240.635**

load from 1st floor to just above ground floor								
S.N	**Description**	**No**	**Length**	**Breadth**	**Height**	**Quantity**	**Unit wt**	**Load**
1	Dead load							
a.	slab	1	5.384	3.912	0.125	2.633	24.000	63.187
	beam along alphabetic grid	1	3.480	0.230	0.230	0.184	24.000	4.418
	beam along numeric grid	1	4.927	0.300	0.325	0.480	24.000	11.530
	column	1	0.457	0.457	3.075	0.642	24.000	15.413
	floor finish	1	5.384	3.912		21.062	1.500	31.593
	false ceiling	1	5.384	3.912		21.062	1.000	21.062
	4"wall with opening	1	4.267	0.102	2.743	1.194	19.200	14.900

	4"wall without opening	1	4.369	0.102	2.743	1.222	19.200	23.468
	4"wall with opening	1	0.406	0.102	3.073	0.127	19.200	1.590
	4" wall without opening	1	2.032	0.102	3.073	0.637	19.200	12.229
2	Live load	1	5.384	3.912		21.062	3.000	63.187
						Load in KN=		**262.576**

load from ground floor to just above basement

S.N	Description	No	Length	Breadth	Height	Quantity	Unit wt	Load
1	Dead load							
a.	slab	1	5.384	3.912	0.125	2.633	24.000	63.187
	beam along alphabetic grid	1	3.480	0.230	0.230	0.184	24.000	4.418
	beam along numeric grid	1	4.927	0.300	0.325	0.480	24.000	11.530
	column	1	0.457	0.457	3.075	0.642	24.000	15.413
	floor finish	1	5.384	3.912		21.062	1.500	31.593
	false ceiling	1	5.384	3.912		21.062	1.000	21.062
	4"wall with opening	1	5.156	0.102	2.743	1.443	19.200	18.004
	4"wall with opening	1	4.063	0.102	3.073	1.274	19.200	15.894
	4" wall without opening	1	4.013	0.102	3.073	1.258	19.200	24.153
2	Live load	1	6.604	7.061		46.631	3.000	139.893
						Load in KN=		**345.146**

Total load= 1429.18 KN
Add 30% for earthquake load= 1857.93 kN
Factored load = 1.5*1857.93 = 2786.900947 KN
For axially loaded short column (From IS 456:2000 clause no. 39.3)
Pu= 0.4fck*Ac + 0.67fy*Asc
Where,
fck = characteristic strength for M25
= 25 Mpa
Ac = Area of concrete
fy = characteristic strength of compression reinforcement fe500= 500 Mpa
= 500 Mpa
Asc = Area of longitudinal reinforcement for column

Let, Asc = 2% of Ag
= 0.02*Ag
Size of column = 400mm*400mm
If Asc= 1.5%
Size of column = 414mm*414mm equivalent to
= 450mm*450mm

3. Load Calculation

3.1 Dead Load Calculation

This load on a structure is a function of the site, maximum Earthquake intensity or strong ground motion and the local soil, the stiffness, and its orientation in relation to the incident seismic waves. It is the combination of overall dead load and approximate amount of live load acting on the building. For the calculation of the seismic weight of a floor the floor takes half the load above and half of the load below the floor for wall and column dead load this is called lumping of floor mass. The seismic weight W of the whole building is the sum of the seismic weight of the floor.

Calculation of Dead load		
Column size = 450 * 450 mm		
Slab = 125 mm		
Beam (Alphabetic grid) = 230 * 355 mm		
Beam (Numeric grid) = 300 * 450 mm		
Storey height = 3.2 m		
No of floor = 6		
Size of outer wall = 230 mm		
Size of inner wall = 102 mm		
For basement		
Length =	47.521	m
Load due to shear wall = 47.521 * 0.23 * 25 *3.2004	874.496	KN
1. Dead load of ground floor		
Slab		
Total area of slab =	109.270	sq.m
Slab area*unit weight*thickness = 109.27*25*0.125	**341.469**	KN
Floor Finish =	1.500	KN/sq.m
Slab area * floor finish = 109.27*1.5	**163.905**	KN

Beam		
Along alphabetic grid		
Length =	36.981	m
width of column =	5.400	m
length*depth*width*unit weight = (36.981-5.4)*0.23*(0.355-0.125)*25	41.766	KN
Along numeric grid		
Length =	44.904	m
width of column =	5.400	m
length*depth*width*unit weight = (44.904-5.4)*0.3*(0.45-0.125)*25	96.291	KN
Total load =	**138.057**	KN
Column		
no. of column =	12.000	no
size*no. of column*floor ht*unit weight = 0.45*0.45*12*3.2*25	**194.400**	KN
Wall		
Outer wall		
9" wall without opening =	9.850	m
Load = 9.85*0.23*(3.2-0.45)*19.2	119.618	KN
9" wall with opening (35% opening) =	27.143	m
Load = 27.143*0.23*(3.2-0.45)*19.2*0.65	214.256	KN
Inner wall		
9" wall without opening =	6.654	m
Load = 6.654*0.23*(3.2-0.45)*19.2	80.806	KN
4" wall with opening (12% opening) =	32.436	m

Load = 32.436*0.102*(3.2-0.45)*19.2*0.88	153.725	KN
Total load on wall = 119.618 + 214.256 + 80.806 + 153.725	**568.405**	KN
Staircase Dead Load	**84.680**	kN
Shear Wall(50% consideration)	**437.248**	
Total Dead load for ground floor (W1) = 341.469 + 115.589 + 194.400 + 568.405+163.905	1928.164	KN
2. Dead load of first floor		
Slab		
Total area of slab =	142.262	sq.m
Slab area*unit weight*thickness = 142.262*25*0.125	444.569	KN
Slab area*unit weight*thickness = 142.262*1.5	213.393	
Beam		
Total load =	138.057	KN
Column		
Total load =	194.4	KN
Wall		
Outer wall		
9" wall without opening =	9.85	m
Load =	119.618	KN
9" wall with opening (35% opening) =	30.88	m
Load = 30.88*0.23*(3.2-0.45)*19.2*0.65	243.754	KN

Inner wall		
9" wall without opening = 9'	2.743	m
Load = 2.743*0.23*(3.2-0.45)*19.2	33.311	KN
4" wall with opening (12% opening) =	33.58	m
Load = 33.58*0.102*(3.2-0.45)*19.2*0.88	159.147	KN
Parapet wall = 4"	11.861	m
Load = 11.861*0.102*1.2*19.2	27.874	KN
Total load on wall = 119.618 + 243.754 + 33.311 + 159.147 + 27.874	583.704	KN
Staircase Dead Load	**84.680**	
Total Dead load for first floor (W2) = 444.569 + 115.589 + 194.4 + 583.704+213.393	1636.335	KN
3. Dead load calculation of second floor		
Slab		
Total area of slab =	142.462	sq.m
Slab area*unit weight*thickness = 142.462*25*0.125	**444.569**	KN
Floor Finish		
Slab area*unit weight*thickness = 142.262*1.5	**213.393**	
Beam		
Total load =	**138.057**	KN
Column		
Total Load =	**194.4**	KN

Wall		
Total load =	**583.704**	KN
Staircase Dead Load	**84.680**	
Total Dead load for second floor (W3) = 444.569 + 115.589 + 194.4 + 583.704+213.393	1658.802	KN
4. Dead load calculation for third floor		
Slab		
Total area of slab =	140.01	sq.m
Slab area*unit weight*thickness = 140.01*25*0.125	**437.531**	KN
Floor Finish		
Slab area*unit weight*thickness = 140.01*1.5	**210.015**	
Beam		
Total load =	**138.057**	KN
Column		
Total load =	**194.4**	KN
Wall		
Total load =	**583.7**	KN
Staircase Dead Load	**84.680**	
Total Dead load for third floor (W4) = 437.531 + 115.589 + 194.4 + 583.704+210.015	**1625.919**	KN
5. Dead load calculation for fourth floor		

Slab		
Total area =	140.04	sq.m
Slab area*unit weight*thickness = 140.01*25*0.125	437.531	KN
Floor Finish		
Slab area*unit weight*thickness = 140.01*1.5	210.015	
Beam		
Along alphabetic grid		
Total length =	36.981	m
width of column =	3.6	m
length*depth*width*unit weight = (36.981-3.6)*0.23*(0.355-0.125)*25	44.146	KN
Along numeric grid		
Length =	44.904	m
width of column =	3.6	m
length*depth*width*unit weight = (44.904-3.6)*0.3*(0.45-0.125)*25	100.679	KN
Total load on beam = 44.146 + 77.187	121.333	KN
Column		
no. of column =	8	
size*no. of column*floor ht*unit weight = 0.45*0.45*8*3.2*25	129.6	KN
Wall		
Outer wall		
9" wall without opening =	7.67	m
Load = 7.67*0.23*(3.2-0.45)*19.2	93.144	KN
9" wall with opening (35% opening) =	22	m
Load = 22*0.23*(3.2-0.45)*19.2*0.65	173.659	KN
Inner wall		

4" wall with opening (12% opening) =	19.43	m
Load = 19.43*0.102*(3.2-0.45)*19.2*0.88	92.085	KN
Parapet wall = 4"	11.861	m
Load = 11.861*0.102*1.2*19.2	27.874	KN
Total load on wall = 93.144 + 173.659 + 92.085 + 27.874	386.762	KN
Staircase Dead Load	**84.680**	
Total Dead load for fourth floor (W5) = 437.531 + 121.333 + 129.6+ 386.762+210.015	1369.921	KN
6. Dead load calculation for fifth floor		
Slab		
Total area =	87.584	sq.m
Slab area*unit weight*thickness = 87.584*25*0.125	273.7	KN
Slab area*unit height = 87.584*1.5	131.376	
Beam		
Along alphabetic grid		
Total length =	24.229	m
length*depth*width*unit weight = 24.229*0.23*(0.355-0.125)*25	32.043	KN
Along numeric grid		
Total length =	17.176	m
length*depth*width*unit weight = 17.176*0.3*(0.45-0.125)*25	41.867	KN

Total load on beam = 32.043 + 32.098	64.141	KN
Parapet wall = 4"	37.26	m
Load = 37.26*0.102*1.2*19.2	87.564	KN
Lift load =	32.000	KN
Total Dead load for fifth floor (W6) = 273.7 + 64.141 + 87.564+131.4	588.781	**KN**

Floor to floor height= 10'6" = 3.2 m		
Height of riser = 7” = 0.178 m		
Width of tread = 0.254 m		
Width of first flight = 4'4" = 1.32 m		
Width of second flight = 1.219 m		
Width of third flight = 4'4" = 1.320 m		
Width of landing = 4'4" = 1.320 m		
No. of riser on first flight = 5		
No. of riser on second flight = 8		
No. of riser on first flight = 5		
No. of tread on first flight = 4		
No. of tread on first flight = 7		
No. of tread on first flight = 4		
Thickness of waist slab =0. 125mm		
Unit weight of slab = 25 KN/m^3		
Let, thickness of floor finish=0.04m		

Staircase dead load calculation:
Load calculation:
On stairs
Area of concrete per step = 1/2*Riser*Tread = ½*0.178*0.254 = 0.023 m^2
Load on waist slab = sqrt(R^2 + T^2)*D = 0.039 m^2
Floor finishing = (Riser + Tread) * Thickness= (0.178 + 0.254)*0.04 = 0.017 m^2
Total Area of Concrete(per step) = Step area+Waist slab area+floor finish area
= 0.023+0.039+0.017
= 0.079 m^2
Per step load on stair = Unit weight* Total concrete area = 0.079*25 =1.975 KN/m

Load of step per meter on stair = Per step load *1/ size of tread = 1.975*1/0.254= 7.776 KN/m
Load of first flight 4'4" width on stair= 7.776*1.320 = 10.264KN/m
Load of second flight 4' width on stair= 7.776*1.219 = 9.479 KN/m
Load of third flight 4'4" width on stair= 7.776*1.320 = 10.264 KN/m
Total dead load on stairs = 10.264+9.479+10.264 = 30.007 KN/m
On Landing:
Load on slab = Unit weight* width of landing*depth of waist slab
=25*1.32*0.125 =4.125 KN/m
Floor Finishing =Unit weight*width of landing * thickness of floor finish
=25*1.32*0.04= 1.32 KN/m
Load = 4.125+1.32 = 5.445 KN/m

5.2 Live load calculation:

For Staircase:
Length of first flight = 3’4” = 1.016 m
Width of first flight = 4’4” = 1.320 m
Load = 4.023 KN
Length of second flight = 6” = 1.828 m
Width of second flight = 4” = 1.219 m
Load = 6.685 m
Length of first flight = 3’4” = 1.016 m
Width of first flight = 4’4” = 1.320 m
Load = 4.023 KN
Landing = 4’4” + 3’10” = 2.489 m
Width = 4’4” = 1.32m
Intensity = 3 KN/sq.m
Landing*width*intensity = 2.489*1.32*3 = 9.85644 KN
Total load = 4.023 + 6.685 + 4.023 + 9.85644 = 24.588 KN
For live load , take 30% = 0.3 * 24.588 = 7.3764468

Live load calculation according to NBC:

S.N	Live load for Specific floor	Calculated value	Live load of staircase	Total live load = 7.38 + CV	Unit
1	Ground floor	109.27*3*0.3 **= 98.343**	7.38	105.723	KN
2	First floor	142.262*3*0.3 = 128.0358	7.38	135.4158	KN
3	Second floor	142.462*3*0.3 = 128.2158	7.38	135.5958	KN
4	Third floor	140.01*3*0.3 = 126.009	7.38	133.389	KN

5	Fourth floor	140.04*3*0.3 **= 126.036**	7.38	133.416	KN
6	Fifth floor	87.584*3*0.3 **= 78.826**	7.38	78.826	KN

Lift Load calculation								
1	Machine weight	470	kg					
2	Counter weight require	1250	kg					
3	Lift Car/Cabin weight	1000	kg					
4	Capacity of lift	6	persons					
5	Total weight carried by the car	1480	kg					
6	Dynamics tension in the cable	27106.23	N					
7	Statics tension in the cable	27300	N					
8	Hence, tension in the cable is	27300	N					
9	Total Dead load including machine weight	32	KN					
10	Area of lift Slab	3.356	m^2					
11	Total Dead load of Lift applicable	**9.535161**	KN/m^2					

5.3 Lump mass calculation for each floor:

Lump mass calculation	Total load	Unit
Ground floor =	1643.961	KN
First floor =	1651.153	KN
Second floor =	1658.802	KN
Third floor =	1648.383	KN
Fourth floor =	1500.79	KN
Fifth floor =	846.962	KN

5.4 Base shear calculation

Base shear according to NBC:

Total seismic weight (W)= $\sum Wi$ = 9586.209 KN

seismic zone = 0.36

For residential building (From Table 8, 7.2.3) Is 1893 (part I)2016
Here ,
Importance factor(I) = 1
Response reduction factor R = SMRF/ other = 5
Natural fundamental time period of building = $0.075*h^{0.75}*1.25$= 0.750070311 sec
For the following details,
Location of building = Kathmandu
Type of structure = Moment Resisting Concrete Frame
Type of building = Reinforced Concrete Moment Resisting Frame
Base Shear = Ah* W = 1258.669KN

		Horizontal Base Shear Coefficient for Equivalent Static Method			
		Horizontal Base Shear Coefficient at the ULS State **(Cdu=C(T1)/(Ru*Ωu)**	0.1313		
		Horizontal Base Shear Coefficient at the SLS State **(Cds=Cs(T1)/Ωs)**	0.1260		
		Exponent Related to the Structural Period **(K)** (Clause 6.3)	1.1250		

4. STRUCTURAL ANALYSIS

Structural Analysis deals with the prediction of performance of a given structure under stipulated loads and other external effects. The performance characteristics of interest are stresses and stress resultants such as axial forces, shear forces, bending moments, deflections and support reactions.
The analysis of the building was done by the estimation of dimensions of various structural members such as slab, beam, column, staircase, foundation, and basement wall with the help of preliminary design. And different types of loads such as vertical load (Dead + finishes, and Live) and earthquake load were calculated. Earthquake being pre-dominant, only its effect was taken for lateral loads.
For the structural analysis of the structure, mainly four load cases are considered which are mentioned below:
1. Dead load(DL)
2. Live load(LL)
3. Earthquake load in X direction (EQX)
4. Earthquake load in Y direction (EQY)
5. Dynamic Load (Response Spectrum)

With the help of ETABS, element stresses in beams and column were calculated in the provision of rigid diaphragm system.

ETABS is a programme for linear, non-linear, static and dynamic analysis, and the design of building systems. From an analytical standpoint, multistorey buildings constitute a very special class of structures and therefore deserve special treatment. The concept of special programmes for building type structures was introduced over 40 years ago and resulted in the development of the ETABS series of computer programmes

4.1 Features and Benefits of ETABS

- The input, output and numerical solution techniques of ETABS are specifically designed to take advantage of the unique physical and numerical characteristics associated with building type structures. As a result, this analysis and design tool expedites data preparation, output interpretation and execution throughput.

- The need for special purpose programmers has never been more evident as Structural Engineers put non-linear dynamic analysis into practice and use the greater computer power available today to create larger analytical models.
- Over the past two decades, ETABS has numerous mega-projects to its credit and has established itself as the standard of the industry. ETABS software is clearly recognized as the most practical and efficient tool for the static and dynamic analysis of multistory frame and shear wall building

4.2 ETABS Result

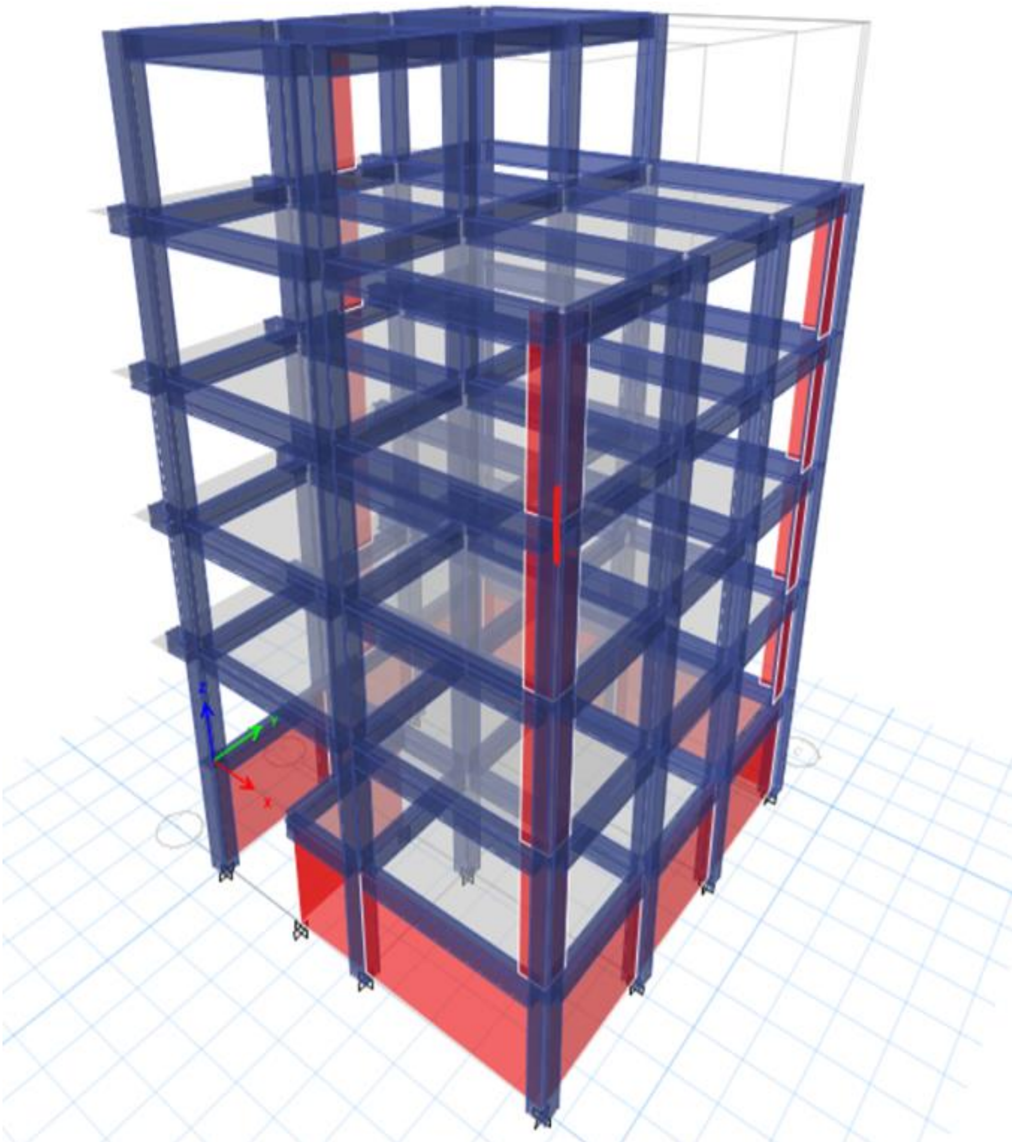


Figure 1: 3D modelling of building

Concrete Frame Design

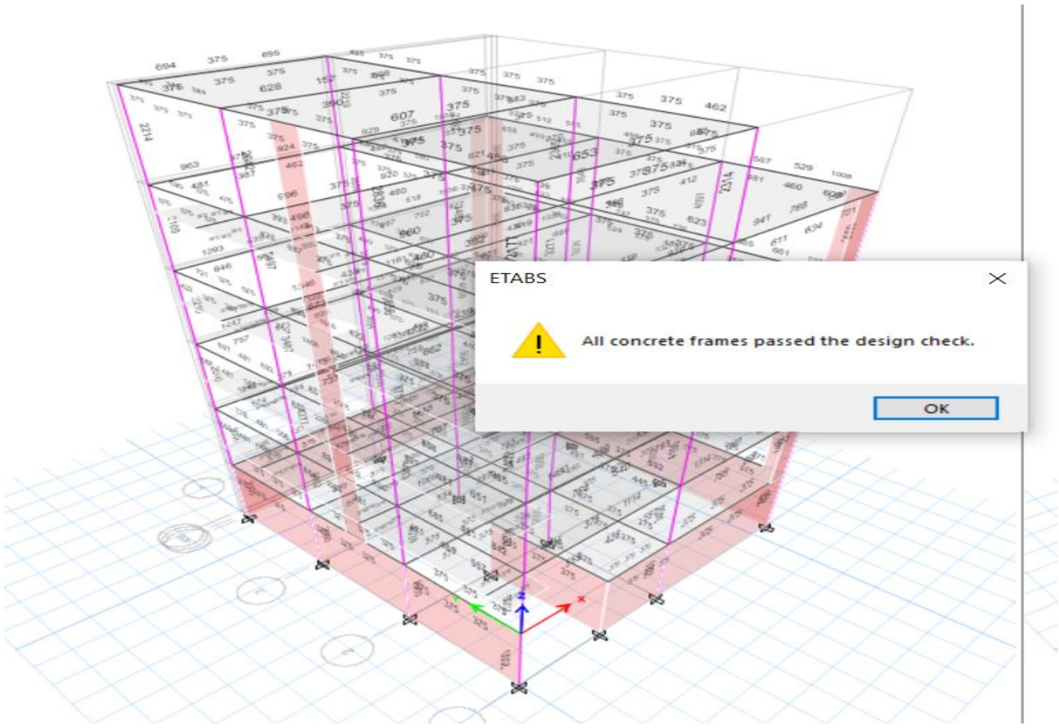


Figure 2:Concrete frame design

Load Cases:

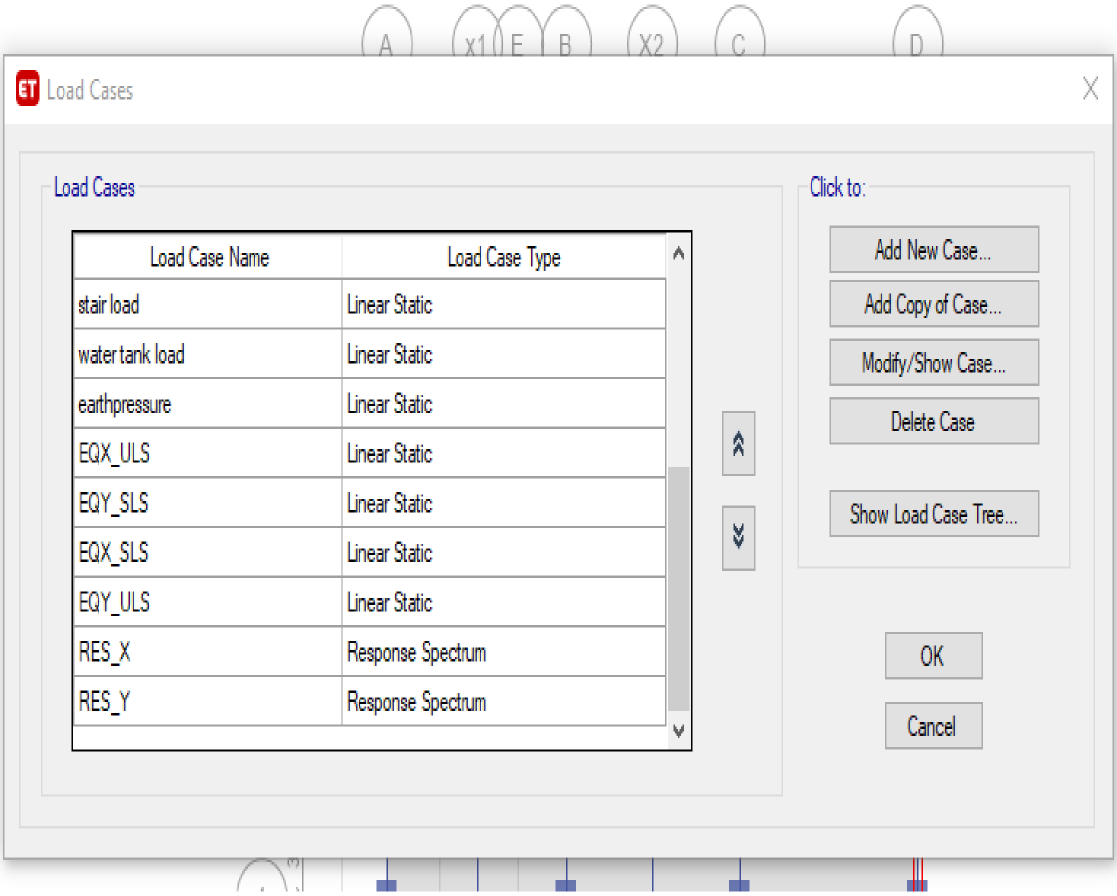


Figure 3:Load Cases (NBC)

Response Spectrum Curve:

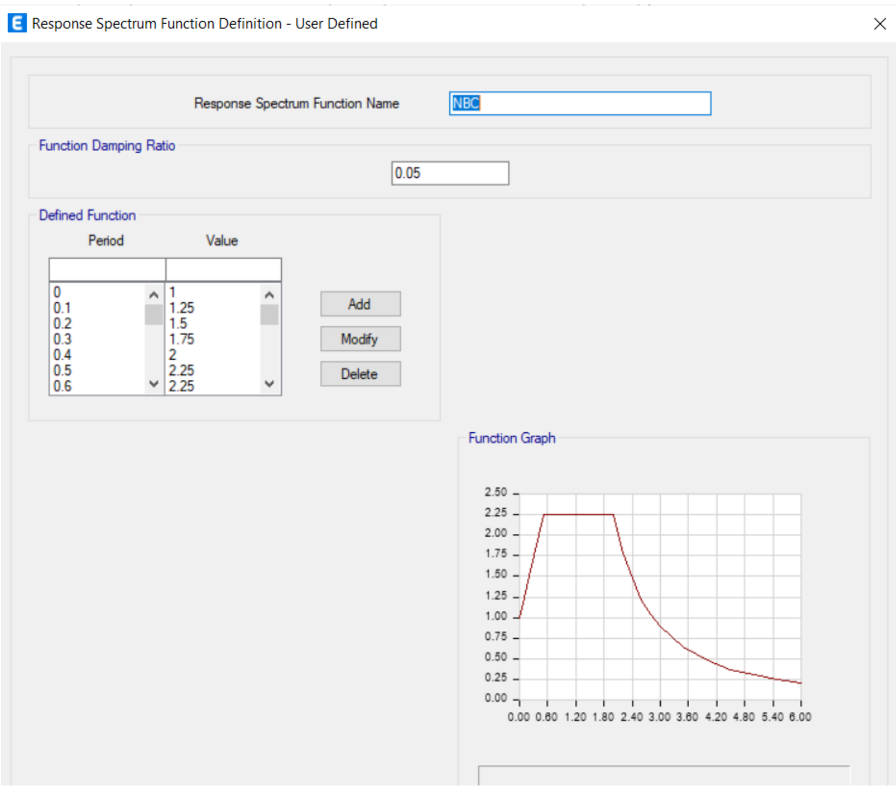


Figure 4:Response Spectrum Curve (NBC)

Load Combination

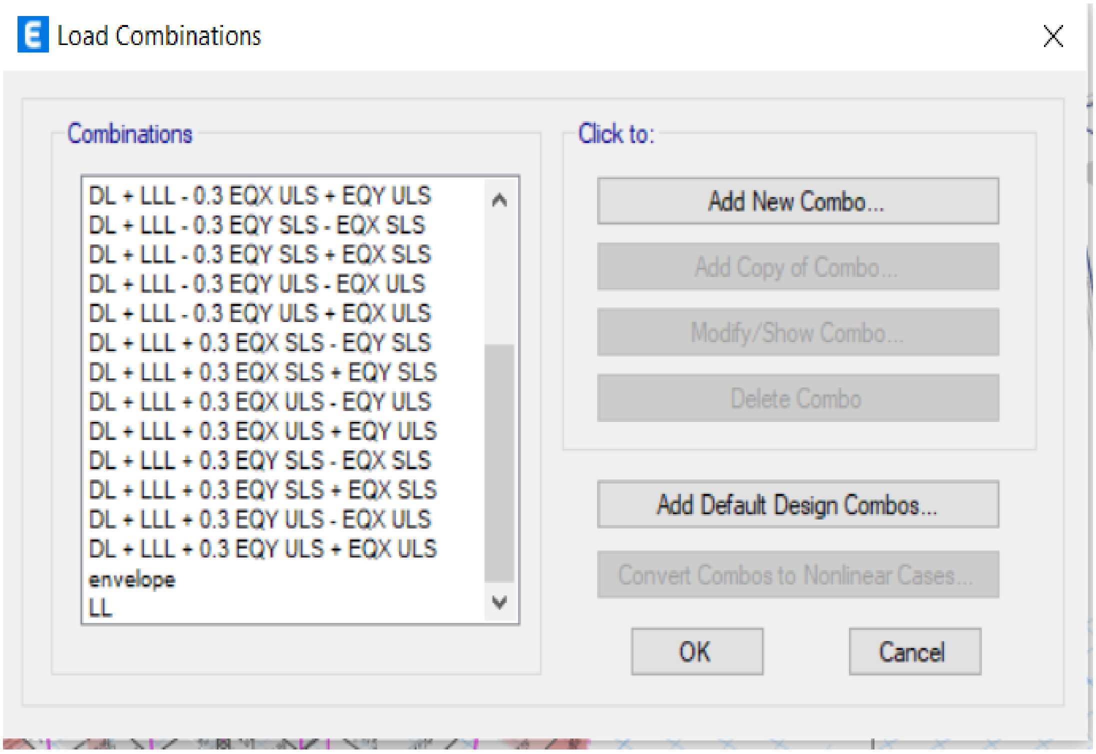
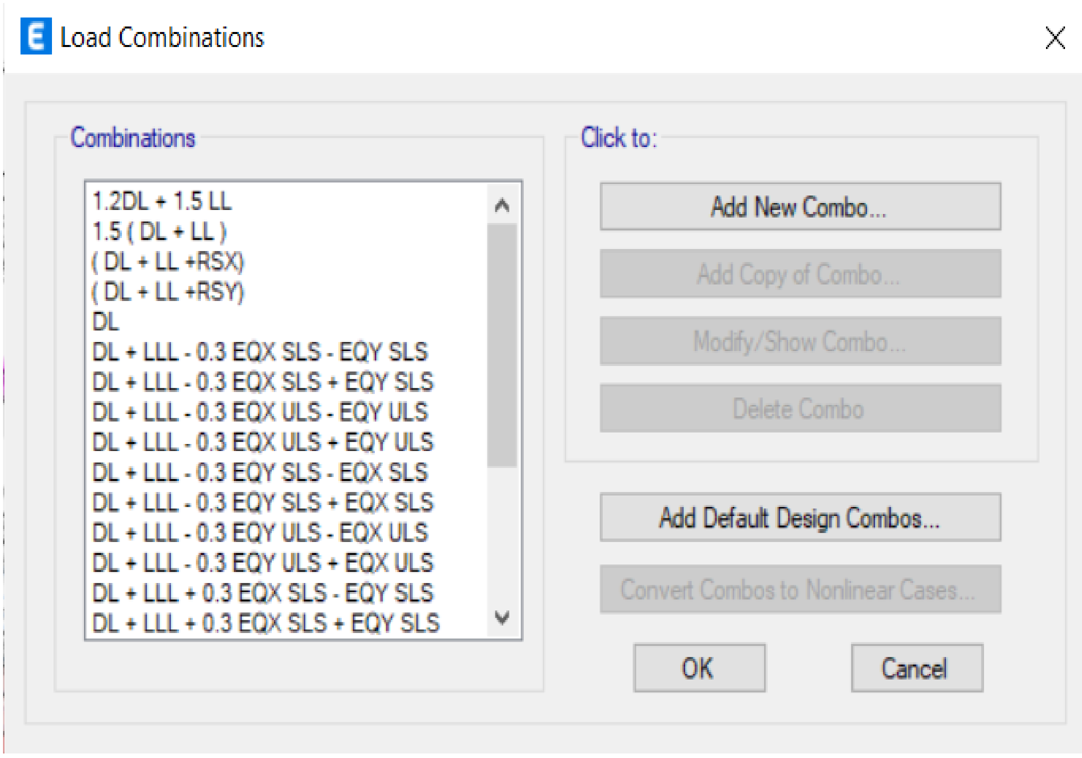


Figure 5:Load Combination (NBC)



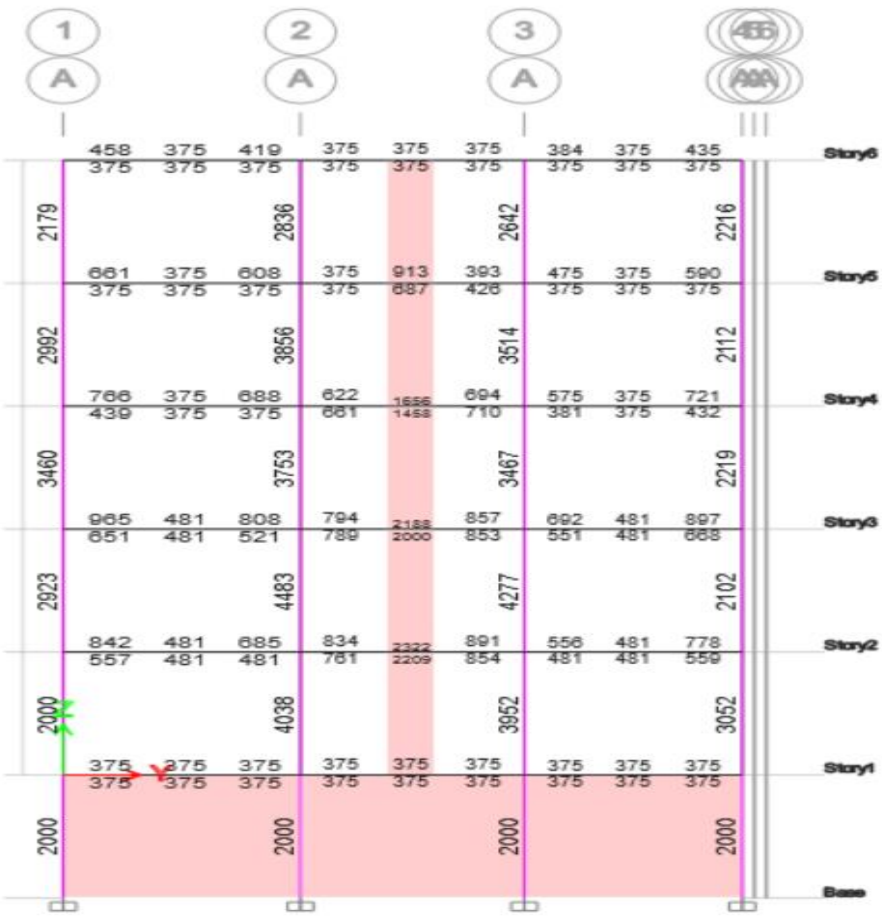


Figure 6 : Reinforcement for grid A - A

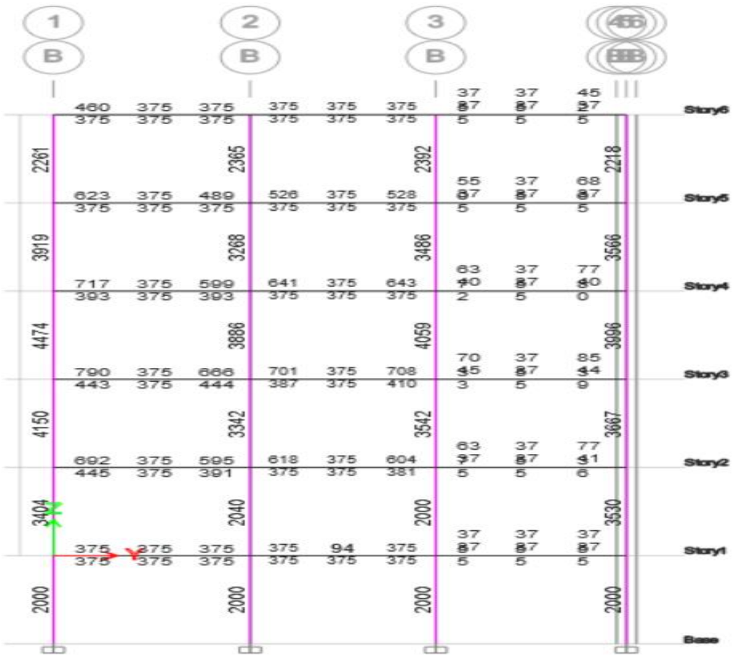


Figure 7: Reinforcement for grd B - B

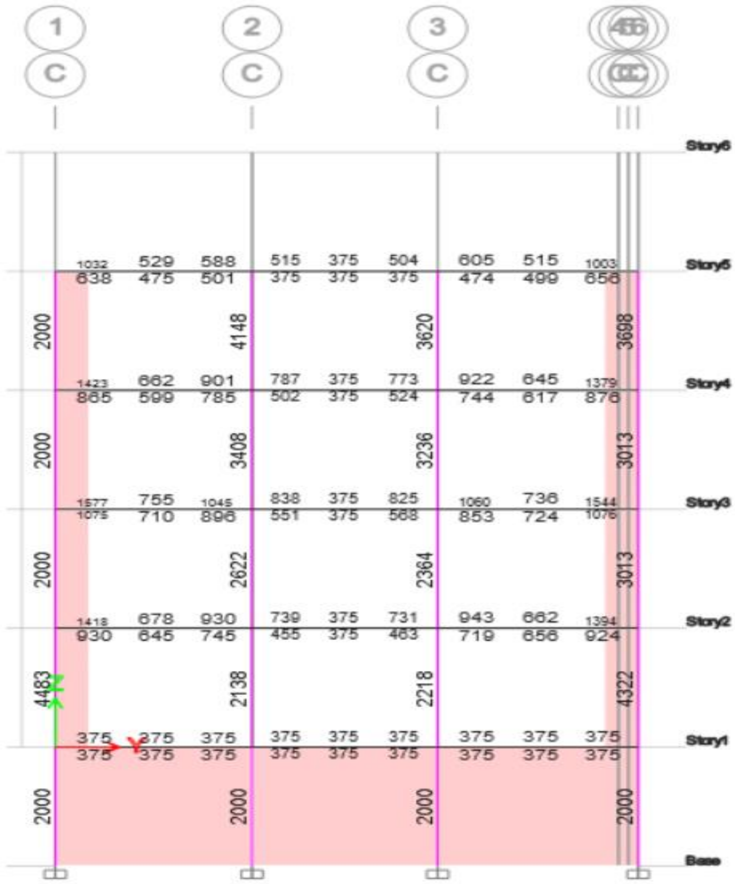


Figure 8: Reinforcement for grid C - C

Modal Analysis (By NBC)

TABLE: Modal Participating Mass Ratios						
Case	Model	Period	UX	UY	SumUX	SumUY
		sec				
Modal	1	1.069	0.621	0.00004	0.621	0.0000420
Modal	2	0.811	0.000	0.61960	0.621	0.6196000
Modal	3	0.691	0.004	0.00880	0.625	0.6284000
Modal	4	0.335	0.091	0.00010	0.716	0.6285000
Modal	5	0.25	0.000	0.08660	0.716	0.7152000
Modal	6	0.218	0.002	0.00050	0.718	0.7156000
Modal	7	0.182	0.036	0.00030	0.754	0.7160000
Modal	8	0.155	0.001	0.02600	0.755	0.7420000
Modal	9	0.122	0.000	0.01680	0.755	0.7588000
Modal	10	0.106	0.036	0.00000	0.791	0.7588000
Modal	11	0.081	0.000	0.03120	0.791	0.7900000
Modal	12	0.071	0.000	0.00002	0.791	0.7900000
Modal	13	0.06	0.028	0.00000	0.819	0.7900000
Modal	14	0.049	0.000	0.02400	0.819	0.8140000
Modal	15	0.039	0.001	0.00500	0.819	0.8191000
Modal	16	0.034	0.179	0.00001	0.998	0.8191000
Modal	17	0.031	0.000	0.17670	0.998	0.9958000
Modal	18	0.026	0.002	0.00380	1.000	0.9996000

Table 1: Modal participating mass ratios and frequency (NBC)

The model mass participation ratio is greater than 90 as per NBC. It is under criteria**.**

4.3 Eccentricity Check

TABLE: Centers Of Mass And Rigidity							
Story	Diaphragm	XCCM	YCCM	XCR	YCR	Eccentricity X	Eccentricity Y
		M	m	m	m		
Story1	D1	5.493	6.2856	5.1124	6.8872	3.537174721	-5.030100334
Story2	D2	4.9749	6.2701	5.2945	5.5441	-2.970260223	6.070234114
Story3	D3	4.985	6.2612	4.5768	5.5906	3.793680297	5.607023411
Story4	D4	5.0386	6.2666	4.2337	5.5464	7.480483271	6.02173913
Story5	D5	4.2115	6.3185	4.0919	5.5117	1.111524164	6.745819398
Story6	D6	2.6964	5.8885	3.3438	5.5831	-6.016728625	2.553511706

Table 2: Eccentricity check (NBC)

From table it can be observed that, eccentricity is les then 10%.

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4.4 Displacement Check

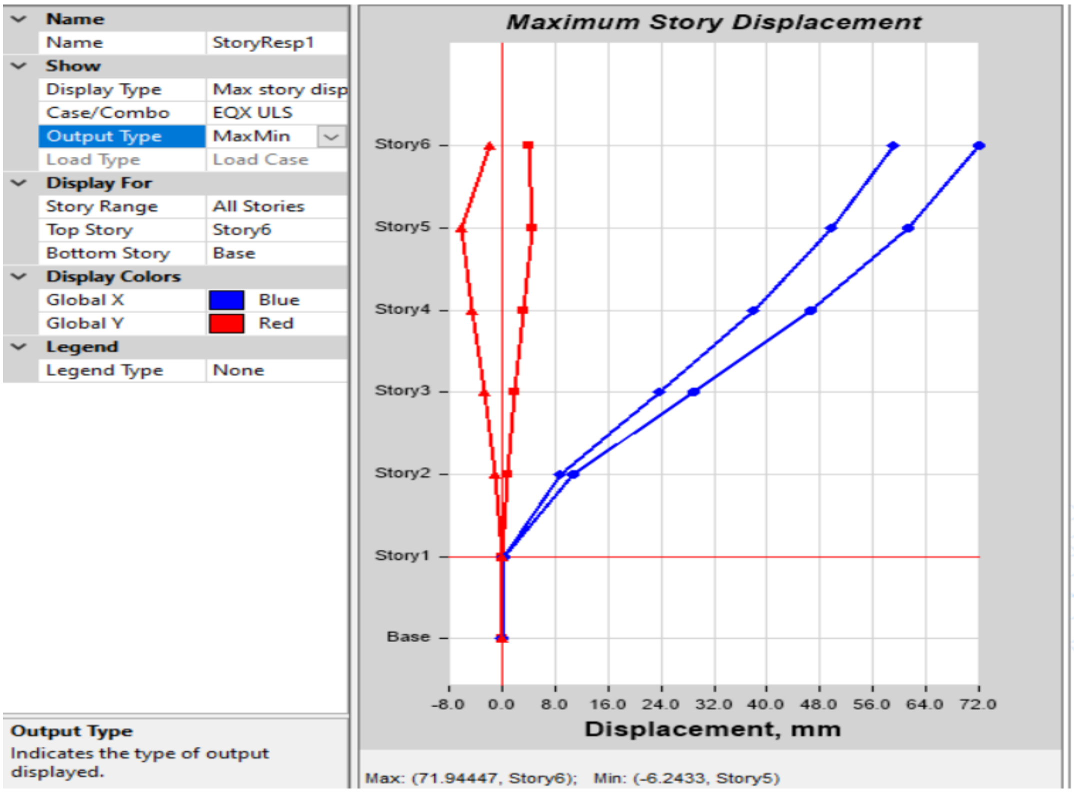

Figure 6: Displacement in EQX ULS

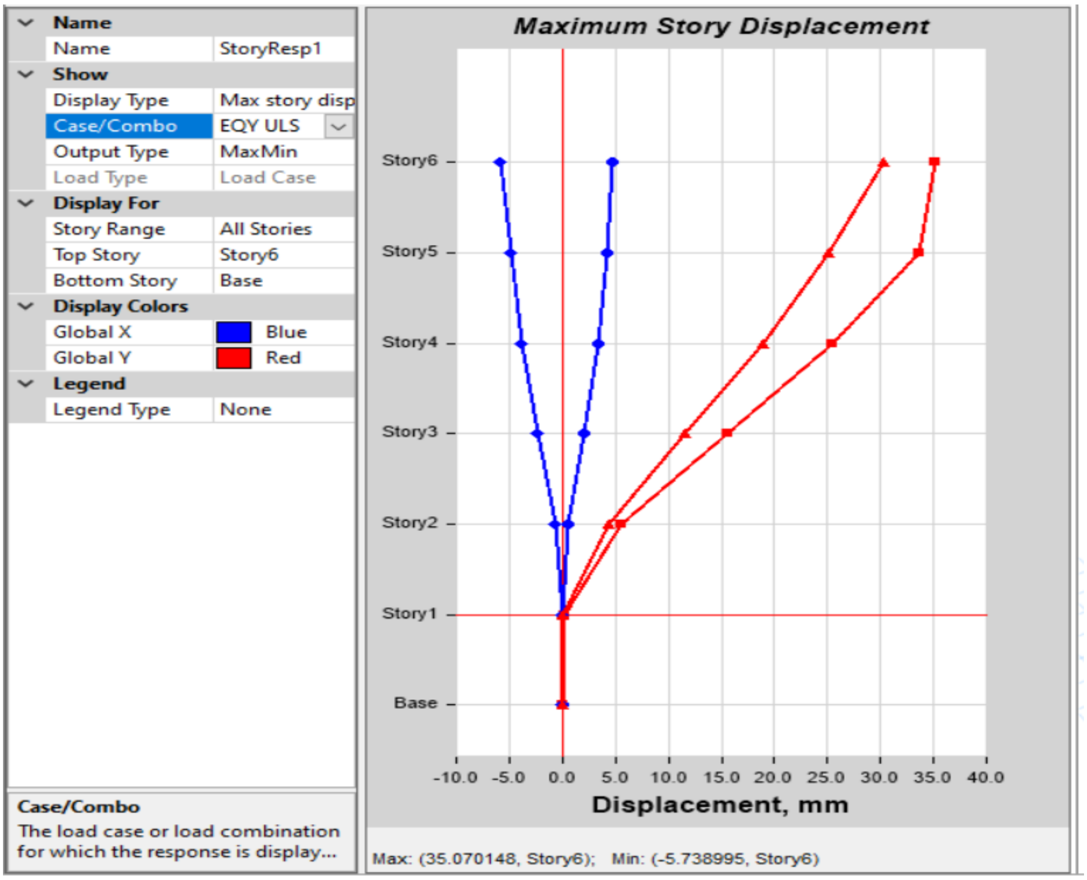

Figure 7 : Displacement in EQY ULS

Displacement Criteria for Equivalent Static Method :	
Design displacement from ETABS in ULS =	70.439 mm
Design displacement from ETABS in SLS =	36.81mm
Allowable ratio ULS =	0.025
Allowable dispalcement ULS = 0.025H/R_u =	100 mm
Result	OK
Allowable ratio SLS =	0.006
Allowable dispalcement SLS = 0.006H/Rs =	96 mm
Result	OK

Table 3 : Displacement check

The displacement is under criteria as per NBC.

4.5 Drift Check

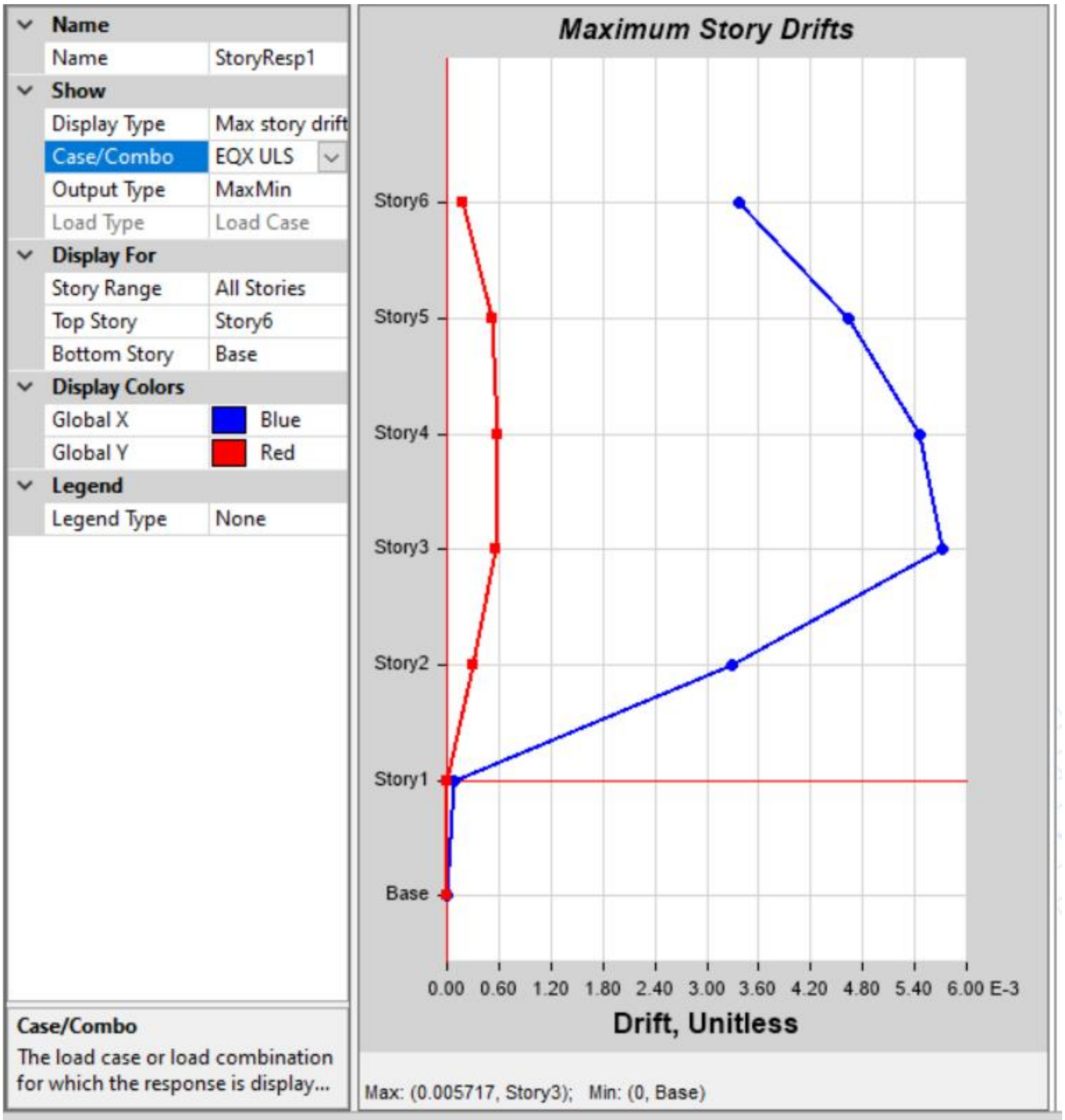

Figure 8 : Maximum drift due to EQX ULS

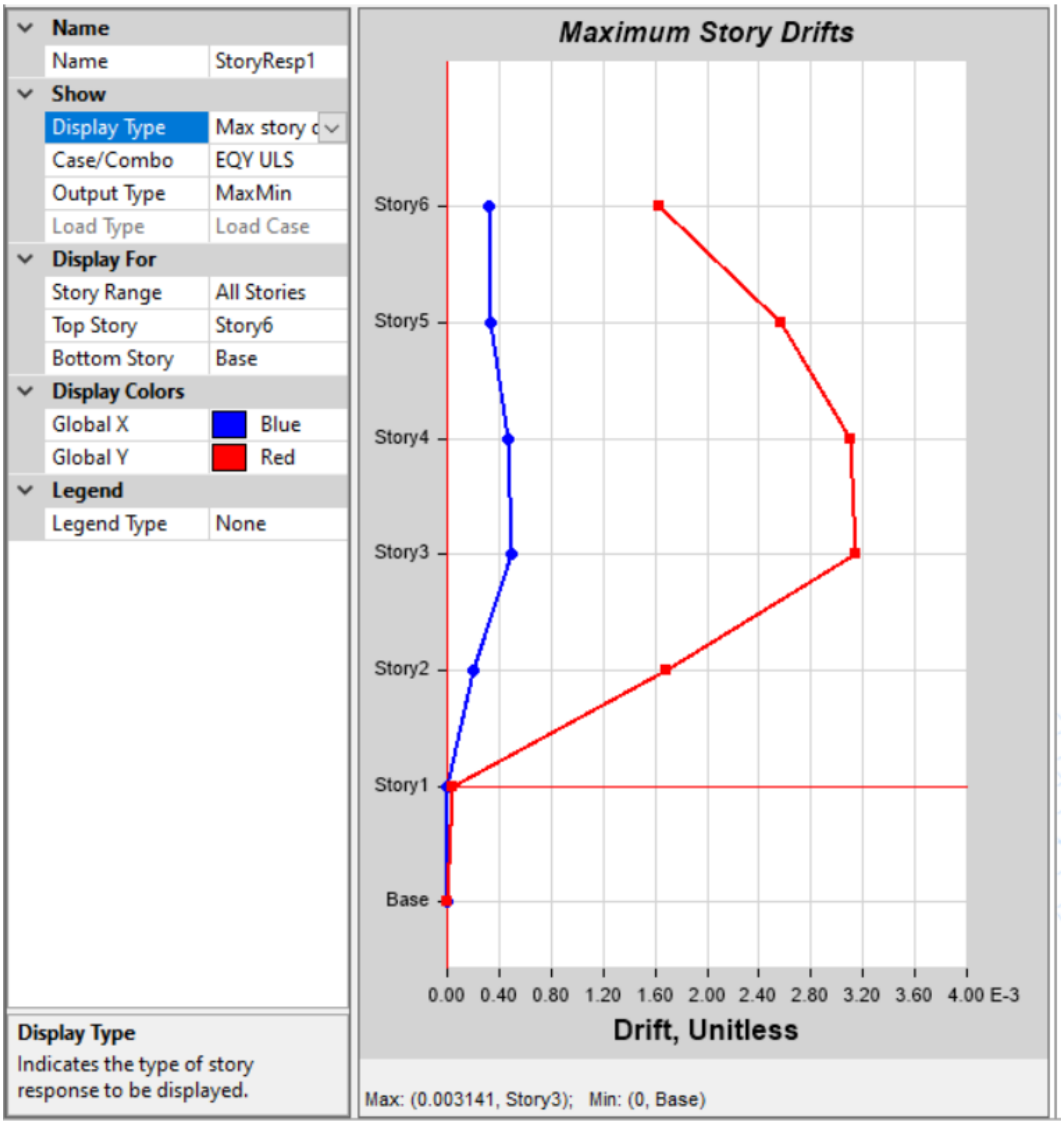

Figure 9 : Maximum drift due to EQY ULS

Drift Criteria for Equivalent Static Method :	
Design story drift from ETABS ULS =	0.005883
Allowable drift ULS = 0.025/R_u =	0.00625
Result	OK
Design story drift from ETABS SLS =	0.00342
Allowable drift ULS =	0.006
Result	OK

Table 4 : Drift criteria

The drift is under 0.00625 as per NBC.

4.6 Torsion check

□□□□□□□	□□□□	□□□□ □	□□□□ □	□□□□	□□□□□	□□□□□ □	□□□□ □

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5F	71.944	35.07	59.05	30.278	□□□□	□□□□	□□□□
4F	61.337	33.726	49.827	25.175	□□□□	□□□□	□□□□
3F	46.491	25.511	38.022	18.875	□□□□	□□□□	□□□□
2F	29.039	15.571	23.972	11.611	□□□□	□□□□	□□□□
1F	10.751	5.531	8.887	4.407	□□□□	□□□□	□□□□
□□□□□□□□□□□□ □□	0	0	0	0			

Table 5 : Torsion criteria

The ratio of Dmax / Dmin is under 1.5 in both EQX and EQY.

5. Chapter 5: STRUCTURAL DESIGN

5.1 Design of structural elements

The design section is the most important part. The design of the structural elements should be done for durability, construction and use in entire service life of the structure. The realization of design objectives requires compliance with clearly defined standards for materials, production, workmanship, and also maintenance and use of structure in service.

This chapter includes all the design process of sample calculation for a single element as slab, beam, column, staircase, basement wall, lift wall, shear wall, mat and isolated foundation.

1. Design of slab
2. Design of beam (Primary beam)
3. Design of column (Square column)
4. Design of staircase (dog legged & open-well staircase)
5. Design of basement wall
6. Design of lift wall
7. Design of mat foundation
8. Design of shear wall

5.2 Design of slab

RCC slabs are designed on the assumption that they consist of a number of series of beams having a width of one meter, even though it is cast monolithic. They are the plate elements forming floors and roofs of building and carrying distributed loads primarily by flexure. They may be supported by beams or walls or columns and may be used as the flange of T or L-beam. Slab may be simply supported or continuous over one or more supports and is classified according to the manner of support as: one way slabs spanning in one direction, two-way slab spanning in directions, circular slabs, and grid floor slabs resting directly on columns with no beams and grid floor and ribbed slabs.

Slabs are designed by using the same theories of bending and shear as are used for beams. The following methods of analysis are available:

1. Elastic analysis-idealization into strips or beams.
2. Semi empirical coefficients as given in the code, and
3. Yield line theory

Important information regarding the design of slab according to IS456:2000

1. Slab is designed for 1m wide strip
2. Ast min = 0.12% bD for deformed bars (Cl.26.5.2.1)
3. Cover = 25mm
4. If Ly/Lx < 2, two-way slab is designed

Compression reinforcement is used only in exceptional cases in a slab. Shear stresses are usually very low and shear reinforcements is never provided in slabs .It is preferred to increase the depth of a slab and hence reduce the shear stress rather than provide shear reinforcement. Temperature reinforcement is invariably provided at right angles to the main longitudinal reinforcement in a slab. There are two types of slabs described as follow:

Two-way slab

The slab is said to be spanning in two directions, when the length to breadth ratio falls below 2. The deflection and bending moments in a two-way slab are considerably reduced as compared to those in one- way slab. In a square slab, the two-way action is equal in each direction. In such case, main reinforcement is provided in both directions in the form of a mesh. Here, in this project two-way slabs are designed.

Slab ID S1	1.	**One short Edge Discontinuous** **Thickness of slab and durability consideration** Clear Spans L_x=3.353m L_y=4.927 m Now, $\frac{ly}{lx} = \frac{3.353}{4.927} = 1.469$ (Two Way Slab) Assume, depth (d) = $\frac{lx}{26*1.3} = \frac{3.353*1000}{33.8} = 99.201\ mm$ So, adopt d =105 mm Assuming clear cover=15 mm Providing 10 mm Ø bar Overall depth of slab, D= 110 + 15 + 10/2 = 125 mm	l_x= 3353mm l_y= 4927mm Design as Two Way Slab d = 105mm D = 125mm
	2.	**Design Load** Self -load of slab = 0.125 * 25 = 3.125 KN/m^2 Floor finishing load = 1.00 KN/m^2 Live load = 3.0 KN/m^2 Total load = DL+LL =3.125 + 3 + 1 =7.125 KN/m^2 Total factored load , w = 1.5*7.125 = 10.687 KN/m^2 Considering unit width of slab , w= 10.687 KN/m	
IS 456-2000 Table 26		**Moment Calculation** -ve Bending moment coefficient at continuous edge α_x^-= 0.0562, α_y^-= 0.037 +ve Bending moment coefficient at mid span α_x^+= 0.0426, α_y^+= 0.037	

IS 456-2000 Annex D.1.1 Annex D.1.1 Annex D.1.1 Annex D.1.1 IS SP-16 Cl.2.3,Table C	3.	For Short Span Support moment , Ms = α_x^- wl_x^2 = $0.0562*10.687*3.458^2$ = 7.182 KN-m Mid span moment , Mm = α_x^+ wl_x^2= $0.0426*10.687*3.458^2$ = 5.44 KN-m For Long Span Support moment , Ms = α_y^- wl_x^2= $0.037*10.687*3.458^2$ = 4.728 KN-m Mid span moment , Mm = α_y^+ wl_x^2= $0.028*10.687*3.458^2$ = 3.578 KN-m **Check for depth from Moment Consideration** Depth of Slab, d = $\sqrt{\frac{M_{max}}{0.133*25*b}} = \sqrt{\frac{7.182*10^6}{0.133*25*1000}}$ =47 mm < d=105 mm (OK) Adopt, d = 100 mm. Therefore, Overall depth, D =120 mm.	Provide d= 100 mm D = 120 mm
IS 456-2000 Annex G-1.1.b)	4.	**Calculation of Area of Steel** Min A_{st} = 0.12 % of bD = 0.12 x 1000 x 120 = 144mm^2 **Area of Steel along short span** **Area of Steel at support (Top Bars)** M_s =0.87*f_y*A_{stx}^-*(d - $\frac{f_y*A_{stx}-}{f_{ck}*b}$) $7.182*10^6$ =0.87*500*A_{stx}^- *($100 - \frac{500*A_{stx}}{25*1000}$) On solving this equation, A_{stx}^- = 171 mm^2< Min A_{stx}^- Providing 10 mm Ø bars @ 150 mm c/c $A_{stx, provided} = \frac{78.54*1000}{150} = 524$ $A_{stx, provided}$ =524 mm^2	$A_{stx, pro}$=524mm
IS 456-2000 Annex G G-1.1.b	5. i)	**Area of Steel Along Long Span** M_s =0.87*f_y*A_{sty}^-*(d - $\frac{f_y*A_{sty}-}{f_{ck}*b}$) $4.728*10^6$ =0.87*500* A_{sty}^-*(100- $\frac{500*A_{sty}}{25*1000}$) On solving this equation, A_{sty}^- = 112 mm^2 Providing 10 mm Ø bars @ 150 mm c/c $A_{sty,provided}$ = 524 mm^2 **Providing steel**	$A_{sty,pro}$=524 mm^2 Short span: 10mm bar @150mm c/c Long span: 10mm bar @ 150 mm c/c

IS 456-2000 Annex G-1.1.b		Provide 10 mm Ø bars @ 150 mm c/c along short span with alternate bent up. Provide 10 mm Ø bars @ 150 mm c/c along long span with alternate bent up.	Both side bar with alternate bent up
IS 456-2000 Table 19 cl.40.2	ii)	**Check for Shear** For x-direction i.e. short span Shear force at the face of the support, V = $\frac{w\,lx}{2}$= 10.687 x $\frac{3.353}{2}$=17.917 KN V_u = 17.917 KN Here, tension reinforcement of slab contribute in shear For $p_t = \frac{100*\frac{524}{2}}{1000*100}$=0.262 %, τ_v = 0.366 N/mm^2 τ_c = 0.366 N/mm^2 τ_c' =0.4225 N/mm^2> τ_v	Safe in shear
IS 456-2000 cl.23.2.1 IS 456-2000 Fig: 4	iii)	**Check for Deflection** Along short Span Since both ends are continuous, the basic value may be taken as 26 fs= $0.58\, f_y \frac{\text{Area of Steel Required}}{\text{Area of Steel Provided}}$ = 0.58 x 500 x $\frac{524}{171}$ = 113.1 N/mm^2 Pt= 0.4 % Modification factor (M.F.) = 2 $d_{per} = \frac{l_x}{\gamma*\alpha*\delta*\beta*\lambda} = \frac{3353}{2*26*1}$ =64.49 mm < dprovided So OK	Safe in deflection
IS 456-2000	iv)	**Check for Development Length** $L_d = \frac{\Phi\sigma_s}{1.6 \text{ x } 4 \text{ x } \tau_{bd}} = \frac{\emptyset*0.87*500}{1.6*4*1.2} = 56.6\emptyset$ $L_d \leq 1.3\frac{M_1}{V} + L_o$ $M_1 = 0.87*f_y*A_{sty}/2*(d - \frac{f_y*A_{sty}/2}{f_{ck}*b})$ = 0.87*500*524 /2 *(92 - $\frac{500*524/2}{25*1000}$) = 9.66*10^6 N-mm	Safe in L_d

cl.26.2.1 IS 456-2000 cl.26.2.1	 6. 7.	L_o =16ϕ (for U bent) Now $56.6\phi \le 1.3*(9.66*10^6)/(17.92*10^3) + 16\phi$ =17.25mm On solving 10 mm ≤ 17.25 mm **(OK)** **Torsion Reinforcement** Torsion reinforcement is provided as per IS 456-2000 cl .D.1.8, cl. D.1.9 **Curtailment of Reinforcement and Detailing** Curtailment is done by as per simplified method and detailing is done by as per Indian Standard Code IS SP 34	

	8.		

SLAB ID S2	 1.	**Two Adjacent Edges Discontinuous** **Thickness of slab and durability consideration** Clear Spans L_x=3.581 m L_y=4.927 m Now, $\frac{ly}{lx} = \frac{4.927}{3.581} = 1.376$ <2,(Two Way Slab) Assume, depth (d) = $\frac{lx}{26*1.3} = \frac{3.581*1000}{33.8} = 105.9\ mm$ So, adopt d =110 mm Assuming clear cover=15 mm Providing 10 mm Ø bar Overall depth of slab, D= 110 + 15 + 10/2 = 130 mm	 l_x= 3581 mm l_y=4927 mm Design as Two Way Slab
	2.	**Design Load** Self-load of slab = 0.130 * 25 = 3.25 KN/m^2 Floor finishing load = 1.00 KN/m^2 Live load = 3.0 KN/m^2 Total load = DL+LL =7.25 KN/m^2 Total factored load , w = 1.5*7.25 = 10.875 KN/m^2 Considering unit width of slab , b= 1000 mm	 d =110mm D=130mm
IS 456-2000 Table 26 IS 456-2000 Annex D.1.1 Annex D.1.1 Annex D.1.1 Annex D.1.1		**Moment Calculation** -ve Bending moment coefficient at continuous edge $\alpha_x^- = 0.0688$, $\alpha_y^- = 0.047$ +ve Bending moment coefficient at mid span $\alpha_x^+ = 0.0515$, $\alpha_y^+ = 0.035$ For Short Span Support moment , Ms = $\alpha_x^-\ wl_x^2 = 0.0688*10.875*3.711^2$ = 10.3 KN-m	

IS SP-16 Cl.2.3,Table C IS 456-2000 Annex G-1.1.b)	3.	Mid span moment , Mm = α_x^+ wl_x^2= $0.0515*10.875*3.711^2$ = 7.713 KN-m For Long Span Support moment , Ms = α_y^- wl_x^2= $0.047*10.875*3.711^2$ = 7.04 KN-m Mid span moment , Mm = α_y^+ wl_x^2= $0.035*10.875*3.711^2$ = 5.242 KN-m **Check for depth from Moment Consideration** Depth of Slab, d = $\sqrt{\frac{M_{max}}{0.133*25*b}} = \sqrt{\frac{10.3*10^6}{0.133*25*1000}}$ =55.65 mm< d=100 mm (OK) Adopt, d = 100 mm. Therefore, Overall depth, D =120 mm.	Provide d= 100 mm D = 120 mm
IS 456-2000 Annex G-1.1.b	4.	**Calculation of Area of Steel** Min A_{st} = 0.12 % of bD = 0.12 x 1000 x 120 = 144mm^2 Area of Steel along short span **Area of Steel at support (Top Bars)** M_s =0.87*f_y*A_{stx}^-*(d - $\frac{f_y*A_{stx}-}{f_{ck}*b}$) $10.3*10^6$ =0.87*500*A_{stx}^- *(100 – $\frac{500*A_{stx}-}{25*1000}$) On solving this equation, A_{stx}^- = 250 mm^2> Min A_{stx}^- Providing 10 mm Ø bars @ 150 mm c/c $A_{stx}^-{}_{,provided}$ =524 mm^2	$A_{stx}^-{}_{pro}$ =524 mm
IS 456-2000 Annex G-1.1.b IS 456-2000 Table 19	5. i)	**Area of Steel at Mid Span (Bottom Bars)** M_m =0.87*f_y*A_{sty}^+ *(d - $\frac{f_y*A_{sty}+}{f_{ck}*b}$) $7.04*10^6$ =0.87*500* A_{sty}^+*(100-$\frac{500*A_{stx}+}{25*1000}$) On solving this equation, A_{sty}^+ =106.64 mm^2 Adopt, A_{sty}^+= 168 mm^2 Providing 10 mm Ø bars @ 150 mm c/c $A_{sty}^+{}_{,provided}$ =524 mm^2 **Providing steel** Provide 10 mm Ø bars @ 150 mm c/c along short span with alternate bent up.	$A_{sty}^+{}_{pro}$=335.06 mm^2 Short span: 10 mm bar @150mm c/c Long span: 10

cl.40.2 IS 456-2000 cl.23.2.1 IS 456-2000 Fig: 4	ii)	Provide 10 mm Ø bars @ 150 mm c/c along long span with alternate bent up. **Check for Shear** For x-direction i.e. short span Shear force at the face of the support, $V = w\, l_x$ = 10.875 x $\frac{3.581}{2}$=19.472 KN V_u = 19.472 KN Here, tension reinforcement of slab contribute in shear For $p_t = \frac{100*\frac{524}{2}}{1000*110}$=0.262 %, τ_v = 0.262 N/mm^2 τ_c = 0.366 N/mm^2 $\tau_c{}'$ =0.476 N/mm^2> τ_v	mm bar @ 150 mm c/c Both side bar with alternate bent up Safe in shear
     IS 456-2000 cl.26.2.1	iii)	**Check for Deflection** Along short Span Since both ends are continuous, the basic value may be taken as 26 fs= $0.58\, f_y \frac{\text{Area of Steel Required}}{\text{Area of Steel Provided}}$ = 0.58 x 500 x $\frac{250}{524}$ = 138.36 N/mm^2 Pt= 0.4 % Modification factor (γ) = 2 $d_{per} = \frac{l_x}{\gamma*\alpha*\delta*\beta*\lambda} = \frac{3771}{2*26}$ =72.52 mm < d provided. So ok **Check for Development Length** $L_d = \frac{\Phi\sigma_s}{1.6 \text{ x } 4 \text{ x } \tau_{bd}}$ $L_d \leq 1.3\frac{M_1}{V} + L_o$ $M_1 = 0.87*f_y*A_{sty}/2*(d - \frac{f_y*A_{sty}/2}{f_{ck}*b})$ = 0.87*415*335.06 /2 *(90 - $\frac{500*335.06/2}{20*1000}$) = 8.65*10^6 N-mm L_o =16ϕ (for U bent) Now 45.5ϕ ≤ 1.3*(5.44*10^6)/(18.78*10^3) + 16ϕ =19.26 mm On solving 10 mm ≤ 19.25 mm **(OK)**	Safe in deflection Safe in L_d

IS 456-2000 cl.26.2.1	6.	**Torsion Reinforcement** Torsion reinforcement is provided as per IS 456-2000 cl .D.1.8, cl. D.1.9	
	7.	**Curtailment of Reinforcement and Detailing** Curtailment is done by as per simplified method and detailing is done by as per Indian Standard Code IS SP 34	
	8.		

5.3 Design of beam

Beam is a flexural member and supports the imposed load. It carry load by bending action. The beam may be rectangle, L and T section consisting of singly and doubly reinforcement.

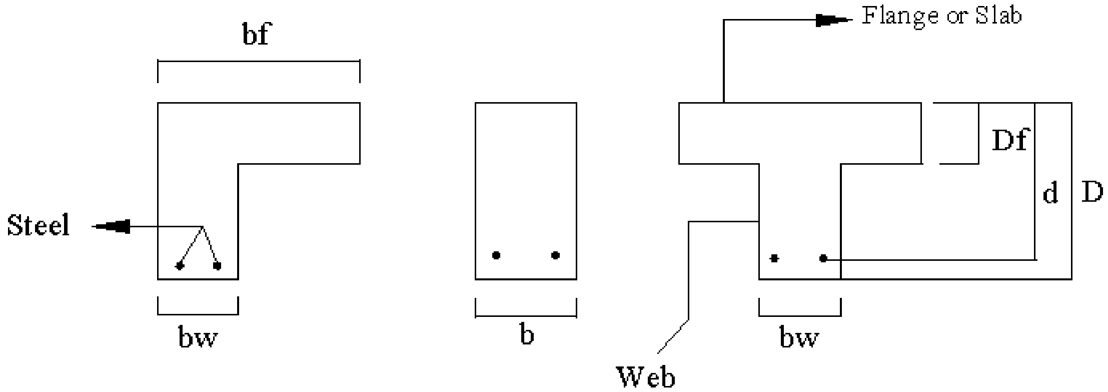

Figure 10: L-beam, rectangular beam and T-beam

Design of beams requires determination of the cross-sectional dimensions and reinforcement details to satisfy both serviceability and strength requirements. The serviceability requirement for deflection is controlled by effective span to effective depth ratio. Generally, depth of the beam is large and governed by the strength requirement. The spacing of reinforcement controls the serviceability requirement for crack. In beams, spacing of reinforcement bars are small and governed by the minimum spacing requirement than maximum spacing for crack control. The reinforcements are provided to satisfy strength requirements. The detailing of longitudinal and transverse bars should satisfy the bending, shear and bond requirements. The bending moment and shear are determined from the analysis generally based on the elastic theory.

Beams are designed for the worst condition. So the maximum values from the combination have been used for the design.

Reference	Calculation	
Beam No.B18 (2^{RD} FLOOR 4 BC	**Rectangular Beam:** **Design data** Width(B)=355 mm Depth(D)=475 mm Hogging moment M_u(-ve)= 166.1875 KN-m Shear force (-V_u) = 122.663 KN Sagging moment M_u(+ve)= 73.7164 KN-m Shear force (+V_u) = 121.3768 KN Clear cover = 25 mm Assumed diameter of bar =20 mm Therefore, effective depth (d) = 475-25-$\frac{20}{2}$ =440 mm For Fe 500, x_m = 0.46d At end:	

Ultimate moment of resistance:

$M_{u,lim}$=0.36*f_{ck}*b*x_m*(d-0.42x_m)

= 0.36*25*355*0.46*440* (440-0.42*0.46*440)

= 229.562 KN-m

For Hogging moment,

M_u = 166.1875 KN-m

Here, $M_u < M_{u,\,lim}$

Therefore, the section must be designed as singly reinforced section.

Calculation of reinforcement:

For Hogging,

Area of tension steel corresponding to M_u,

$M_u = 0.87* f_y*A_{st1}*(d - \frac{f_y*A_{st1}}{f_{ck}*b})$

$166.1875*10^6 = 0.87*500*A_{st1}*(440-\frac{500*Ast1}{25*355})$

Therefore, A_{st1}= 995.05 mm^2

Provide, 16 mm diameter bars.

No. of bars = $\frac{Ast\ required}{Area\ of\ bars}$

$= \frac{995.05}{\frac{\pi*16^2}{4}}$

= 4.94 ≅ 5 nos.

$A_{st1,\,pro} = 5*\frac{\pi}{4} * 16^2$

= 1005.3 mm^2

For Sagging moment,

M_u = 64.4664 KN-m

Here, $M_u < M_{u,\,lim}$

Therefore, the section must be designed as singly reinforced section.

Calculation of reinforcement:

For Sagging,

Area of tension steel corresponding to M_u

$M_u = 0.87* f_y*A_{st2}^{-}*(d - \frac{f_y*A_{st2}}{f_{ck}*b})$

$73.7164*10^6 = 0.87*500*A_{st2}*(440-\frac{500*Ast2}{25*355})$

Therefore, A_{st2}= 406.28 mm^2

Provide,16mm diameter bars.

No. of bars = $\frac{Ast\ required}{Area\ of\ bars}$

$= \frac{406.28}{\frac{\pi*16^2}{4}}$

= 2.02 ≅ 3 nos.

$A_{st2,\,pro} = 3*\frac{\pi}{4} * 16^2$

= 603.185 mm^2

Check:

Reinforcement check

	Minimum reinforcement $A_0/(b*d) = 0.85/f_y$ $A_0 = (0.85*355*440)/500$ $=265.54\ mm^2 < A_{st,\ provided}$ Maximum reinforcement Maximum, Ast = 4% of b*D = 0.04*355*475 $= 7810\ mm^2 > A_{st,\ provided}$ ovide 5-16 mm diameter bars in top (Hogging) ovide 3-16 mm diameter bars in bottom (Sagging) **Development length:** **For Hogging,** $L_d \leq \frac{1.3\ M1}{V} + L_0$ $M_1 = 0.87*f_y*A_{st1}*(d - \frac{f_y*A_{st}}{f_{ck}*b})$ $=0.87*500*1005.3*(440-\frac{500*1005.3}{25*355})$ =167.65 KN-m V = 122.663 KN $L_d = \frac{0.87*fy*\emptyset}{4\tau bd}$ $= \frac{0.87*500*16}{4*1.4*1.6}$ = 776.786 mm Therefore, $776.786 \leq 1.3*\frac{167.65*10^6}{122.663*10^3}$ 776.786 mm ≤ 1777.257 mm Hence, Safe in development length. **For Sagging,** $L_d \leq \frac{1.3\ M1}{V} + L_0$ $M_1 = 0.87*f_y*A_{st2}*(d - \frac{f_y*A_{st}}{f_{ck}*b})$ $= 0.87*500*603.185*(440-\frac{500*603.185}{25*355})$ = 106.533 KN-m V = 121.3768 KN $L_o = 8\emptyset$ = 8*16 = 128 $L_d = \frac{0.87*fy*\emptyset}{4\tau bd}$ $= \frac{0.87*500*16}{4*1.4*1.6}$ = 776.785 mm Therefore, $776.785 \leq 1.3*\frac{106.533*10^6}{121.3768*10^3} + 128$ 776.785 mm ≤ 1269.016 mm	

Hence, Safe in development length.

Shear:

For Hogging,

Maximum shear force occurs in the face of support

Nominal shear stress

$\tau_v = V_u/b*d$

$$= \frac{122.663 * 10^3}{355 * 440}$$

$= 0.785$ N/mm^2

Percentage of tension steel,

$p_t = (100*A_{st1})/(b*d)$

$$= \frac{100 * 1005.3}{355 * 440}$$

= 0.643%

Design shear strength of M_{25} concrete

From IS 456:2000 Table 19

100 A_{st}/bd	τ_c, N/mm^2
0.50	0.49
0.643	X
0.75	0.57

Interpolating we get,

$\tau_c = 0.536$ N/mm^2

Maximum shear stress for M25 grade From IS 456:2000 Table 20,

$\tau_{c,\,max} = 3.1$ N/mm^2

Therefore, $\tau_{c,\,max.} > \tau_v > \tau_c$

Hence, shear reinforcement must be designed for shear value.

Provide 2-legged stirrups of 8mm Φ having an area of 100.53 mm^2.

The shear that must be contributed by vertical stirrups.

$V_{us} = V_u - \tau_c*b*d$

$=122.663*10^3 - 0.536*355*440$

= 38939.8 N

= 38.94 KN

Spacing of vertical stirrups

$S_v = (0.87*f_y*A_{sv}*d)/V_{us}$

$= \frac{0.87*500*100.53*440}{38.94*10^3}$

= 494.13 mm

Max. spacing as per minimum reinforcement,

$S_v = (0.87*f_y*A_{sv})0.4*b$

$= \frac{0.87*500*100.53}{0.4*355}$

= 307.96 mm

The spacing criteria are:

Should not be less than 100 mm
Should not be greater than 0.75d
Should not be greater than 355 mm
Adopt, 2- legged 8 mm diameter vertical stirrups @ 250 mm c/c.

For Sagging,

Maximum shear force occurs in the face of support
Nominal shear stress

$\tau v = V_u/b*d$

$$= \frac{121.3768 * 10^3}{355 * 440}$$

$= 0.777\ N/mm^2$

Percentage of tension steel,

$p_t = (100*A_{st2})/(b*d)$

$$= \frac{100 * 603.185}{355 * 440}$$

$= 0.386\%$

Design shear strength of M_{25} concrete
From IS 456:2000 Table 19

$100\ A_{st}/bd$	τ_c, N/mm^2
0.25	0.36
0.386	x
0.50	0.49

Interpolating we get,

$\tau_c = 0.431\ N/mm^2$

Maximum shear stress for M25 grade From IS 456:2000 Table 20,

$\tau_{c,\ max} = 3.1\ N/mm^2$

Therefore, $\tau_{c,\ max.} > \tau_v > \tau_c$

Hence, shear reinforcement must be designed for shear value.
Provide 2-legged stirrups of 8mm dia with an area of 100.53 mm^2.
The shear that must be contributed by vertical stirrups.

$V_{us} = V_u - \tau_c*b*d$

$= 121.3768*10^3 - 0.431*355*440$

$= 54054.6$ N

$= 54.055$ KN

Spacing of vertical stirrups

$S_v = (0.87*f_y*A_{sv}*d)/V_{us}$

$= \frac{0.87*500*100.53*440}{54.055*10^3}$

$= 355.96$ mm

Max. spacing as per minimum reinforcement,

$S_v = (0.87*f_y*A_{sv})0.4*b$

$= \frac{0.87*500*100.53}{0.4*355}$

$= 307.962$ mm

The spacing criteria are:
Should not be less than 100 mm
Should not be greater than 0.75d
. Should not be greater than 355 mm
Adopt, 2 - legged 8 mm diameter vertical stirrups @ 250 mm c/c.

Deflection

For Hogging

$d = l_{eff.}/(\alpha*\beta*\lambda*\delta*\gamma)$

For γ,

$$fs = 0.58\ f_y \frac{Area\ of\ Steel\ Required}{Area\ of\ Steel\ Provided}$$

$=0.58*500*\frac{995.05}{1005.3}$

$=247.45$ N/mm^2

$p_t = 0.515\%$

Therefore, $\gamma = 1.18$

$L_{effective} = 5385$mm

Therefore $d=\frac{5835}{1.1*26}$

$=204.02$ mm

< adopted depth(440mm) O.K.

For Sagging

$d = l_{eff.}/(\alpha*\beta*\lambda*\delta*\gamma)$

For γ,

$$fs = 0.58\ f_y \frac{Area\ of\ Steel\ Required}{Area\ of\ Steel\ Provided}$$

$=0.58*500*\frac{406.28}{603.185}$

$=195.332$ N/mm^2

$p_t = 0.26\%$

Therefore, $\gamma = 1.5$

$L_{effective} = 5835$ mm

Therefore, $d=\frac{5385}{1.5*26}$

$=150$ mm

< adopted depth(440mm) O.K.

2. **Rectngular Beam:**

Design data

Width(B)=355 mm
Depth(D)=475 mm

Hogging moment M_u(-ve)= 187.47 KN-m
Shear force $(-V_u)$ = 137.8 KN

Sagginng moment M_u(+ve)= 113.83 KN-m
Shear force (+V_u) = 113.85 KN
Clear cover = 25 mm
Assumed diameter of bar =20 mm
Therefore,effective depth (d) = 475-25-$\frac{20}{2}$
=440 mm
For Fe 500, x_m = 0.46d
At end:

Ultimate moment of resistance:

$M_{u,lim}$=0.36*f_{ck}*b*x_m*(d-0.42x_m)
= 0.36*25*355*0.46*440* (440-0.42*0.46*440)
= 229.562 KN-m
For Hogging moment,
M_u = 187.47 KN-m
Here, $M_u < M_{u,\,lim}$
Therefore, the section must be designed as singly reinforced section.

Calculation of reinforcement:

For Hogging,

Area of tension steel corresponding to M_u,
$M_u = 0.87 * f_y * A_{st1} * (d - \frac{f_y * A_{st1}}{f_{ck} * b})$
$187.47*10^6 = 0.87*500*A_{st1}*(440-\frac{500*Ast1}{25*355})$
Therefore, A_{st1}= 1148.30 mm^2
Provide, 16 mm diameter bars.
No. of bars = $\frac{Ast\ required}{Area\ of\ bars}$
= $\frac{1148.30}{\frac{\pi*16^2}{4}}$
= 5.72 ≅ 6 nos.
$A_{st1,\,pro} = 6*\frac{\pi}{4} * 16^2$
= **1206.38** mm^2
For Sagging moment,
M_u = 113.83 KN-m
Here, $M_u < M_{u,\,lim}$
Therefore, the section must be designed as singly reinforced section.

Calculation of reinforcement:

For Sagging,

Area of tension steel corresponding to M_u
$M_u = 0.87 * f_y * A_{st2}^{-} * (d - \frac{f_y * A_{st2}}{f_{ck} * b})$
$113.83*10^6 = 0.87*500*A_{st2}*(440-\frac{500*Ast2}{25*355})$
Therefore, A_{st2}= 648.585 mm^2
Provide,16mm diameter bars.

No. of bars = $\frac{Ast\ required}{Area\ of\ bars}$

= $\frac{648.585}{\frac{\pi*16^2}{4}}$

= 3.227 ≅ 4 nos.

$A_{st2,\ pro}$ = $4*\frac{\pi}{4}*16^2$

= **804.248** mm^2

Check:

Reinforcement check

Minimum reinforcement

$A_0/(b*d) = 0.85/f_y$

$A_0 = (0.85*355*440)/500$

$=265.54\ mm^2 < A_{st,\ provided}$

Maximum reinforcement

Maximum, Ast = 4% of b*D

= 0.04*355*475

$= 6745\ mm^2 > A_{st,\ provided}$

rovide 6-16 mm diameter bars in top (Hogging)

rovide 4-16 mm diameter bars in bottom (Sagging)

Development length:

For Hogging,

$L_d \leq \frac{1.3\ M1}{V} + L_0$

$M_1 = 0.87*f_y*A_{st1}*(d - \frac{f_y*A_{st}}{f_{ck}*b})$

$=0.87*500*1206.38*(440-\frac{500*1206.38}{25*355})$

=195.234 KN-m

V = 137.8 KN

$L_0 = \frac{0.87*fy*\emptyset}{4\tau bd}$

$= \frac{0.87*500*16}{4*1.4*1.6}$

= 776.786 mm

Therefore, $776.786 \leq 1.3*\frac{195.234*10^6}{137.8*10^3}$

776.786 mm ≤ 1416.8 mm

Hence, Safe in development length.

For Sagging,

$L_d \leq \frac{1.3\ M1}{V} + L_0$

$M_1 = 0.87*f_y*A_{st2}*(d - \frac{f_y*A_{st}}{f_{ck}*b})$

$= 0.87*500*804.248*(440-\frac{500*804.248}{25*355})$

= 138.082 KN-m

V = 113.85 KN

$L_0 = \frac{0.87*fy*\emptyset}{4\tau bd}$

B2 3RD STORY 2-3 A

$= \frac{0.87*500*16}{4*1.4*1.6}$

= 776.785 mm

Therefore, $776.785 \leq 1.3*\frac{138.082*10^6}{113.85*10^3}$

776.785 mm ≤ 1212.841 mm

Hence, Safe in development length.

Shear:

For Hogging,

Maximum shear force occurs in the face of support

Nominal shear stress

$\tau_v = V_u/b*d$

$$= \frac{137.8*10^3}{355*440}$$

= 0.882 N/mm^2

Percentage of tension steel,

$p_t = (100*A_{st1})/(b*d)$

$$= \frac{100*1206.38}{355*440}$$

= 0.772%

Design shear strength of M_{25} concrete

From IS 456:2000 Table 19

100 A_{st}/bd	τ_c, N/mm^2
0.5	0.49
0.772	x
0.75	0.57

Interpolating we get,

τ_c = 0.577 N/mm^2

Maximum shear stress for M25 grade From IS 456:2000 Table 20,

$\tau_{c,\,max}$ = 3.1 N/mm^2

Therefore, $\tau_{c,\,max.} > \tau_v > \tau_c$

Hence, shear reinforcement must be designed for shear value.

Provide 2-legged stirrups of 8mm Φ having an area of 100.53 mm^2.

The shear that must be contributed by vertical stirrups.

$V_{us} = V_u - \tau_c*b*d$

$= 137.8*10^3 - 0.577*355*440$

= 47672.6 N

= 47.6726 KN

Spacing of vertical stirrups

$S_v = (0.87*f_y*A_{sv}*d)/V_{us}$

$= \frac{0.87*500*100.53*440}{47.6726*10^3}$

= 403.62 mm

Max. spacing as per minimum reinforcement,

$S_v = (0.87*f_y*A_{sv})/0.4*b$

$= \frac{0.87*500*100.53}{0.4*355}$

= 307.962 mm

The spacing criteria are:

. Should not be less than 100 mm

Should not be greater than 0.75d

. Should not be greater than 355 mm

Adopt, 2- legged 8 mm diameter vertical stirrups @ 250 mm c/c.

For Sagging,

Maximum shear force occurs in the face of support

Nominal shear stress

$\tau v = V_u/b*d$

$$= \frac{113.85 * 10^3}{355 * 440}$$

= 0.728 N/mm^2

Percentage of tension steel,

$p_t = (100*A_{st2})/(b*d)$

$$= \frac{100 * 804.248}{355 * 440}$$

= 0.515%

Design shear strength of M_{25} concrete

From IS 456:2000 Table 19

$100\ A_{st}/bd$	τ_c, N/mm^2
0.5	0.49
0.515	x
0.75	0.57

Interpolating we get,

τ_c = 0.495 N/mm^2

Maximum shear stress for M25 grade From IS 456:2000 Table 20,

$\tau_{c,\ max}$=3.1 N/mm^2

Therefore, $\tau_{c,\ max.} > \tau_v > \tau_c$

Hence, shear reinforcement must be designed for shear value.

Provide 2-legged stirrups of 8mm dia with an area of 100.53 mm^2.

The shear that must be contributed by vertical stirrups.

$V_{us} = V_u - \tau_c*b*d$

$=113.85*10^3 - 0.495*355*440$

= 36531N

= 36.531 KN

Spacing of vertical stirrups

$S_v = (0.87*f_y*A_{sv}*d)/V_{us}$

$= \frac{0.87*500*100.53*440}{36.531*10^3}$

= 526.715 mm

Max. spacing as per minimum reinforcement,

$S_v= (0.87*f_y*A_{sv})/0.4*b$

$= \frac{0.87*500*100.53}{0.4*355}$

= 307.962 mm

The spacing criteria are:

- Should not be less than 100 mm
- Should not be greater than 0.75d
- Should not be greater than 355 mm

Adopt, 2- legged 8 mm diameter vertical stirrups @ 250 mm c/c.

Deflection

For Hogging

$d=l_{eff.}/(\alpha*\beta*\lambda*\delta*\gamma)$

For γ,

$fs=0.58\ f_y \frac{Area\ of\ Steel\ Required}{Area\ of\ Steel\ Provided}$

$=0.58*500*\frac{1148.30}{1206.38}$

$=276.038\ N/mm^2$

$p_t = 0.772\%$

Therefore, γ= 1.2

$L_{effective}$= 4038mm

Therefore $d=\frac{4038}{1.2*26}$

=129.423 mm

< adopted depth(440mm) O.K.

For Sagging

$d=l_{eff.}/(\alpha*\beta*\lambda*\delta*\gamma)$

For γ,

$fs=0.58\ f_y \frac{Area\ of\ Steel\ Required}{Area\ of\ Steel\ Provided}$

$=0.58*500*\frac{648.585}{804.248}$

$=233.870\ N/mm^2$

$p_t = 0.515\%$

Therefore, γ= 1.2

$L_{effective}$= 4038 mm

Therefore, $d=\frac{4038}{1.2*26}$

=129.423 mm

< adopted depth(440mm) O.K.

5.4 Design of column

A column may be defined as an element used primary to support axial compressive loads and with a height of a least three times its lateral dimension. The strength of column depends upon the strength of materials, shape and size of cross section, length and degree of proportional and dedicational restrains at its ends.

The longitudinal bars in columns help to bear the load in the combination with the concrete. The longitudinal bars are held in position by transverse reinforcement, or lateral binders. The binders prevent displacement of longitudinal bars during concreting operation and also check the tendency of their buckling towards under loads.

It is assumed to possess adequate safety against collapse. The limit state method of design of column is based on the behavior of structure at collapse ensuring adequate margin of safety. The serviceability limits of deflections and cracks are assumed to be satisfied as the column being primarily a compression member has very small deflections and cracks.

Assuming 25 mm dia bars with 40 mm clear cover and distributing reinforcement equally on four side. d' = 40 + 25 / 2 = 52.5 mm

Uniaxial moment capacity of the section about xx-axis

$$\frac{d'}{D} = \frac{52.5}{500} = 0.11$$

Chart for d'/D = 0.15 will be used.

Referring to *chart* # *,SP 16*

$$\frac{Mu}{fck \times bD^2} = 0.14$$

$$Mux1 = 0.14 \times 25 \times 500 \times 500^2 / 10^6$$
$$= 437.500 \text{ KNm}$$

$\therefore$ **Mux1=Muy1** **= 437.500 KNm** ($\because$The column is square.)

Calculation of Puz:

$$Puz = 0.45 \times fck \times Ac + 0.75 \times fy \times Asc$$
$$= 0.45 \times 25 \times (500^2 \times 0.985) + 0.75 \times 500 \times 0.015 \times (500)^2$$
$$= 4176.563 \text{ KN}$$

Now,

$$\frac{Pu}{Puz} = \frac{1346.280}{4176.563} = 0.322$$

By using *SP 16* the value of α_n is an exponent whose value depends on Pu/Puz as in table

Pu/Puz	α_n
≤ 0.2	1.0
≥ 0.8	2.0

Hence the value of α_n by interpolation is given as

$$\Rightarrow \alpha_n = 1 + \left(\frac{0.322 - 0.2}{0.80 - 0.2}\right) = 1.204$$

$$\frac{Mux}{Mux1} = \frac{2.240}{437.500} = 0.005$$

$$\frac{Muy}{Muy1} = \frac{13.520}{437.500} = 0.031$$

Check

As per *Clause 39.6, IS: 456-2000*

$$\left(\frac{Mux}{Mux1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy1}\right)^{\alpha_n} \leq 1.0$$

$$\left(\frac{Mux}{Mux1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy1}\right)^{\alpha_n} = (0.0051)^{1.204} + (0.031)^{1.204}$$
$$= 0.017 < 1$$

The column is safe. **Hence O.K.**

$\therefore$ Area of reinforcement reqd in column (Asc) = 1.50 % $\times$(500)^2 = 3750 mm^2

Let us provides 4 25 mm ∅ bar, giving area of steel = 1963.5 mm^2

& 8 – 20 mm ∅ bar, giving area of steel = 2513.3 mm^2

$\therefore$ Total Area of reinforcement provided in column (Asc) = 4476.8 mm^2 > Asc required. O.K

8.3 DESIGN OF TRANSVERSE REINFORCEMENT

As per *IS: 456-2000, Clause 26.5.3.2*

1. Diameter of lateral ties $\varnothing_T$

$\varnothing_T \geq \varnothing_L/4 = 25 / 4 = 6.3$ mm

$\varnothing_T \geq 6$ mm

Hence, provide mm ∅ lateral ties.

2. Spacing of lateral ties i.e. pitch distance

Pitch distance ≤ Least lateral dimension = 500 mm

≤ 16×smallest dia of longitudinal reinforcement = 16 × 20 = 320 mm

≤ 48×Lateral ties = 48 × 8 = 384 mm

≤ 300 mm

Hence, provide 300 mm pitch distance

Hence provide lateral ties of diameter 8 mm at a pitch of 300 mm i.e. 8 mm ∅ tie @ 300 mm

But from ductility requirement provide ties @ 100 mm C/C upto 1/6 of clear height equa 458.3 mm

or Largest column size = 500 mm.

Therefore provide lateral ties 8 mm ∅ @ 100 mm C/C up to 500 mm below and above the connection.

SAMPLE DESIGN OF COLUMN (Node number 9)

Load combination 3 = DL + LLL - 0.3 EQX ULS - EQY ULS

11.1. DESIGN PARAMETER

Design constants

fck = 25 N/mm^2

fy = 500 N/mm^2

Column size = 500 mm× 500 mm

Check for short or long column

As per *Clause 25.1.2, IS:456-2000*

$\frac{Lex}{D}$ and $\frac{Ley}{b}$ < 12, Short column

$\frac{Lex}{D}$ and $\frac{Ley}{b}$ > 12, Long column

As per *Clause 25.1.3, IS:456-2000*

Unsupported length L = 3200 – 450 = 2750 mm

Effective length Le = 0.80 × 2.75 = 2.200 m

Here, $\frac{Le}{D} = \frac{Le}{b} = \frac{2200}{500} = 4.400$ Let us design as Short Column.

From *Clause 25.4,* Mininmum eccentricity e_{min} is given by

$e_{min} \geq \frac{L}{500} + \frac{D}{30} \geq 20$ mm Where, L=unsupported length of column in mm

D=Lateral dimension of column in mm

$= \frac{2750}{500} + \frac{500}{30}$

= 22.167 mm > 20 mm

Hence minimum eccentricity e_{min} is taken as 22.17 mm

$\frac{e_{min}}{D} = \frac{22.167}{500} = 0.044 < 0.05.$ Hence O.K.

8.2 DESIGN DATA

Pu = 1780.840 KN

Mux = 15.592 KNm

Muy = 4.772 KNm

Let us assume percentage of reinforcement (pt) = 1.50 %

$\frac{pt}{fck} = \frac{1.5}{25} = 0.06$

$\frac{Pu}{fck \times bD} = \frac{1780.840 \times 1000}{25 \times 500 \times 500} = 0.285$

Assuming 25 mm dia bars with 40 mm clear cover and distributing reinforcement equally on four side. d' = 40 + 25 / 2 = 52.5 mm

Uniaxial moment capacity of the section about xx-axis

$$\frac{d'}{D} = \frac{52.5}{500} = 0.11$$

Chart for d'/D = 0.15 will be used.

Referring to *chart* # *,SP 16*

$$\frac{Mu}{fck \times bD^2} = 0.14$$

$$Mux1 = 0.14 \times 25 \times 500 \times 500^2 / 10^6$$
$$= 431.250 \text{ KNm}$$

∴ Mux1=Muy1 = 431.250 KNm (∵The column is square.)

Calculation of Puz:

$$Puz = 0.45 \times fck \times Ac + 0.75 \times fy \times Asc$$
$$= 0.45 \times 25 \times (500^2 \times 0.985) + 0.75 \times 500 \times 0.015 \times (500)^2$$
$$= 4176.563 \text{ KN}$$

Now,

$$\frac{Pu}{Puz} = \frac{1780.840}{4176.563} = 0.426$$

By using *SP 16* the value of α_n is an exponent whose value depends on Pu/Puz as in table

Pu/Puz	α_n
≤0.2	1.0
≥0.8	2.0

Hence the value of α_n by interpolation is given as

$$\Rightarrow \alpha_n = 1 + \left(\frac{0.426 - 0.2}{0.80 - 0.2}\right) = 1.377$$

$$\frac{Mux}{Mux1} = \frac{15.592}{431.250} = 0.036$$

$$\frac{Muy}{Muy1} = \frac{4.772}{431.250} = 0.011$$

Check

As per *Clause 39.6, IS: 456-2000*

$$\left(\frac{Mux}{Mux1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy1}\right)^{\alpha_n} \leq 1.0$$

$$\left(\frac{Mux}{Mux1}\right)^{\alpha_n} + \left(\frac{Muy}{Muy1}\right)^{\alpha_n} = (0.0362)^{1.377} + (0.011)^{1.377}$$
$$= 0.012 < 1$$

The column is safe. Hence O.K.

∴Area of reinforcement reqd in column (Asc) $= 1.50\ \% \times (500)^2 = 3750\ mm^2$

Let us provides 4 25 mm ∅ bar, giving area of steel $= 1963.5\ mm^2$

& 8 – 20 mm ∅ bar, giving area of steel $= 2513.3\ mm^2$

∴Total Area of reinforcement provided in column (Asc) $= 4476.8\ mm^2$ > Asc required. O.K.

8.3 DESIGN OF TRANSVERSE REINFORCEMENT

As per *IS: 456-2000, Clause 26.5.3.2*

1. Diameter of lateral ties $\varnothing_T$

$\varnothing_T \geq \varnothing_L/4 = 25 / 4 = 6.3$ mm

$\varnothing_T \geq 6$ mm

Hence, provide mm ∅ lateral ties.

2. Spacing of lateral ties i.e. pitch distance

Pitch distance	≤ Least lateral dimension	= 500 mm	
	≤ 16×smallest dia of longitudinal reinforcement	= 16 × 20	= 320 mm
	≤ 48×Lateral ties	= 48 × 8	= 384 mm
	≤ 300 mm		
Hence, provide	300 mm pitch distance		

Hence provide lateral ties of diameter 8 mm at a pitch of 300 mm i.e. 8 mm ∅ tie @ 300 mm

But from ductility requirement provide ties @ 100 mm C/C upto 1/6 of clear height equa 458.3 mm

or Largest column size = 500 mm.

Therefore provide lateral ties 8 mm ∅ @ 100 mm C/C up to 500 mm below and above the connection.

5.5 Design of staircase

Staircase is an inclined Structural system for the movement from one level to another. Since it is stepped, it is called staircase. The geometrical forms of staircase may be quiet different depending on the individual circumstances involved. The stairs and landing slab can angled in different form to get different types of staircase. A staircase behaves like an ordinary slab. It may span either in the direction of the steps or in the direction of going. Structurally, staircase may be classified largely into two categories, depending on the predominant direction in which the slab component of the stair undergoes flexure – stair slab spanning transversely and stair slab spanning longitudinally.

The design of staircase requires proportioning of its different components and determination of reinforcement and it’s detailing to satisfy both the serviceability and strength requirements. The design of staircase is made for serviceability requirements of deflection and cracks. The serviceability requirement of deflection is controlled by the effective span to effective depth.

ratio. The design of reinforcement is made to satisfy the strength requirements for moments and shears. The design for moment is made for maximum moments either by the working stress method or by the limit state method. The area of steel is expressed as diameter and spacing of bars. It is provided along the span of staircase and necessary curtailment is made wherever it is not required as in the case of edge-supported slabs.

Generally, the shear reinforcement is not required in staircase as the shear strength of concrete is much greater than the nominal shear stress. The shear strength of concrete in staircase is determined as in the case of edge- supported slab. The detailing of reinforcement in staircase shall be similar to that of the edge supported slab except at the junction of landing and flight of staircase where it should ensure that the reinforcement bars in tension tending to straighten out do not cause cracking in concrete.

Design of Staircase (Open Well Staircase)

Floor to floor height= 10'6" = 3.2004 m

Height of riser(R) = 7" = 0.178m

Width of tread (T)= 10"= 0.254m
Width of first flight = 4'4"= 1.320m
Width of second flight = 4'= 1.219m
Width of third flight = 4'4"= 1.320m
Thickness of floor finish (Dff) = 0.040m
size of column = 1'6" = 0.457m
Width of landing = 4'4"= 1.320m
No. of riser on first flight = 5
No. of riser on second flight = 8
No. of riser on first flight = 5
No. of tread on first flight = 4
No. of tread on first flight = 7
No. of tread on first flight = 4
Thickness of waist slab = 125mm = 0.125m
Unit weight of slab = 25 KN/m^2
No. of riser on first flight = 6
Grade of Concrete fck = 25 N/mm^2
Grade of steel = 500 N/mm^2

For first flight

Effective Length (leff) = 3.683 m
For second flight
Effective length (leff) = 3'9"/2+3'9"+6'0"+4'6"+9"/2= 5.0284 m
Effective Depth(d) = leff / αβγδλ = 184.15 mm
Provide, d = 200 mm = 0.2 mm
overall depth(D) = d+20+16/2 = 228mm = 0.228m

LOAD CALCULATION

For first and third flight
Concrete area per slab
Step = (1/2)*R*T = 0.022606 m^2
Waist slab = D*sqrt(R^2+T^2) = 0.070716765 m^2
Floor Finish = (R+T) Dff = 0.01728 KN/m^2
Total Area of concrete = 0.110602765 m^2
Load on per step = 2.765069116 KN/ m^2
Load on per 'm' area = 10.88609888 KN/ m^2
Live load = 3 KN/ m^2
load on 1.320m width of stair = 14.36965052 KN/ m^2
Live load on 1.320m = 3.96 KN
Total load = 18.32965052 KN/ m^2
Factored load = 27.49447578 KN/ m^2
Load at Landing
Slab load = γ*b*D = 5.7 KN/ m^2
floor finish = 5*Dff = 1 KN/ m^2

Total load = 5.7+1+3 = 9.7 KN/ m^2
Factored load = 1.5*total load = 1.5* 9.7 = 14.55 KN/ m^2
Load at 1.320m Landing = 1.320*factored load = 19.206 KN/ m^2
load on left landing = 16.63065 KN/ m^2
Reaction support
RA = 37.2 KN
RB =39 KN
Maximum Bending moment is at distance of 1.90m from A
Maximum bending moment = 34.95 KN/m
Now,
From IS Code
Mlim = $0.133*fck*b*d^2$
$34.95*106 = 0.133*25*1000*d^2$
d = 102.52 < dassume
adopt d=110mm
overall depth D = 138 mm
For steel
Mlim = 0.87fyAst(d-FyAst/fckb)
34.95*106 = 0.87*500*Ast(110-500(Ast)/25*1000)
Astreq = 867.11 mm^2
For spacing
Astmin = 0.12% of bD = 165.6 mm^2
Spacing = 231.7583698 mm
Provide Main bar 16mmφ@200mmc/c
Astpro = 1004.8 mm^2
Check for shear
τmax = Vmax/b*d
Provide Distribution Bar 10mmφ@175mm
For 10mm bar
Spacing = 474.27 mm
Astpro = 448.79 mm^2
Check for shear
τv = Vu/bd = 0.282609
from table 19
τc = 0.741 > τv
Hence, ok
For second flight
Effective Depth(d) = leff/αβγδλ = 251.42 mm
Adopt, d = 260 mm = 0.26m
Overall depth, D = 288 mm = 0.288 m

LOAD CALCULATION
For middle flight

Concrete area per slab
Step = (1/2)*R*T = 0.022606 m^2
Waist slab = D*sqrt(R^2+T^2) = 0.08932644 m^2
Floor Finish = (R+T) Dff = 0.01728 KN/ m^2
Total Area of concrete = 0.12921244 m^2
Load on per step = 3.230310988 KN/ m^2
Load on per 'm' area = 12.7177598 KN/ m^2
Live load = 3 KN/ m^2
load on 1.219m width of stair = 15.50294919 KN/ m^2
Live load on 1.219m = 3.657 KN
Total load = 19.15994919 KN/ m^2
Factored load = 28.73992379 KN/ m^2

Load at Landing
Slab load = γ*b*D = 7.2 KN/ m^2
floor finish = 25*Dff = 1 KN/ m^2
Total load = 7.2+1+3 = 11.2 KN/ m^2
Factored load = 1.5*total load = 16.8 KN/ m^2
Load at 1.219m Landing = 1.219*factored load = 20.4792 KN/ m^2
load on left landing = 22.39944 KN/ m^2
Reaction support
RA = 54.29 KN
RB = 56.23 KN
Maximum Bending moment is at distance of 1.90m
Maximu bending moment = 66.813 KN/m
Now,
From IS Code
Mlim = 0.133*fck*b*d^2
66.813*106 = 0.133*25*1000*d^2
d = 141.75 < dassume
adopt d=150mm
overall depth, D =178 mm
For steel
Mlim = 0.87fyAst(d-FyAst/fckb)
66.813*106 = 0.87*500*Ast(150-500(Ast)/25*1000)
Astreq = 1223.57 m^2
For spacing
Astmin = 0.12% of bD = 213.6 mm^2
Spacing = 164.2407055 mm
Provide Main bar 16mmφ@150mmc/c
Astpro = 1339.733333 mm^2
Check for shear
Tmax = Vmax/b*d

Provide Distribution Bar 10mmφ@175mm
For 10mm bar
Spacing = 367.69 mm
Astpro = 448.79 mm
Check for shear
τv = Vu/bd = 0.315899
from table 19
τc = 0.70908 > τv
Hence, ok.

5.6 Design of foundation

Foundation is the basic and the lowermost part of structure which provides a uniform base for the superstructure and transfers the load of the same down to the soil below it uniformly.

Selection of foundation type
Design Of Footing

TABLE: Joint Reactions

Joint	**Output Case**	**Case Type**	**F3**	**Unfactored**
Text	Text	Text	KN	KN
4	(DL+LL)	Combination	2039	1694.40
18	(DL+LL)	Combination	2372	1325.73
22	(DL+LL)	Combination	2773	494.33
6	(DL+LL)	Combination	3446	497.28
20	(DL+LL)	Combination	2968	839.62
24	(DL+LL)	Combination	2567	561.86
7	(DL+LL)	Combination	3167	1338.05
9	(DL+LL)	Combination	2963	1553.17
11	(DL+LL)	Combination	2594	2100.58
1	(DL+LL)	Combination	2698	3021.74
3	(DL+LL)	Combination	3285	3125.83
5	(DL+LL)	Combination	2728	439.73
Total Unfactored Load				22400.00

Geometrical C.G x =5.62 m
y = 5.99 m
Total Load = 22400KN
Calculation of C.G of Load
Taking a moment of all loads about A-A axis

Load (KN)	Distance (m)	Load (KN)	Distance (m)	Load (KN)	Distance (m)	Total KN-m
2039.00	0	2963.00	5.3848			

3446.00	0	3285.00	5.3848			
3167.00	0	2773.00	10.77			
2698.00	0	2567.00	10.77			
2372.00	5.3848	2594.00	10.77			
2968.00	5.3848	2728.00	10.77			177227.693

Centre of Load about A-A axis (X) =5.27m

Taking a moment of all loads about 1-1 axis

Load (KN)	Distance (m)	Load (KN)	Distance (m)	Load (KN)	Distance (m)	Moment KN-m
2039.00	0	2968.00	4.0386	2594.00	7.8486	
2372.00	0	2567.00	4.0386	2698.00	11.557	
2773.00	0	3167.00	7.8486	3285.00	11.557	
3446.00	4.04	2963.00	7.85	2728.00	11.557	205414.88

Centre of Load about 1-1 axis (Y) = 6.11m

Eccentricities

ex = -0.35m

ey = 0.12m

Other Data

$I_{xx} = 1714.5\ m^4$

$I_{yy} = 1434.1\ m^4$

Area(A) = $137.13 m^2$

Moment Mx = 22400*-0.35 = -7736.20 KN-m

Moment My = 22400*0.12 = 2767.2 KN-m

Direct Soil Pressure = 22400/137.13 = 163.35 KN/m^2

Soil Pressure Calculation:

Soil pressure at different points is given by:

q =	**P**	±	**My**	**X**	±	**Mx**	**Y**	KN/m2
	A		**Iy**			**Ix**		
	88.38	±	**0.06007309**	**X**	±	**-3.00857**	**Y**	KN/m2
A2	163.35	±	1.929609744	-5.27	±	-4.51183	-1.240	179.11
		±			±			
A3	163.35	±	1.929609744	-5.27	±	-4.51183	2.570	161.92
		±			±			
A4	163.35	±	1.929609744	-5.27	±	-4.51183	6.110	177.76
		±			±			
B1	163.35	±	1.929609744	0.11	±	-4.51183	-5.270	166.54
		±			±			
B2	163.35	±	1.929609744	0.11	±	-4.51183	-1.240	168.73
		±			±			

B3	163.35	±	1.929609744	0.11	±	-4.51183	2.570	151.54
		±			±			
B4	163.35	±	1.929609744	0.11	±	-4.51183	6.110	135.57
		±						
C1	163.35	±	1.929609744	5.49		-4.51183	-5.270	176.53
		±						
C2	163.35	±	1.929609744	5.49		-4.51183	-1.240	158.35
		±						
C3	163.35	±	1.929609744	5.49		-4.51183	2.570	141.16
		±						
C4	163.35	±	1.929609744	5.49		-4.51183	6.110	125.19

Calculation of Bending Moment:

Along Numeric grid

For Strip 1-1			**W**	**L**	**M(KN-m)**	**Maximum Bending Moment**
	Maximum Moment=		179.00	4.38	343.40	

Along Alphabetic grid						
For Strip A-A			**W**	**L**	**M(KN-m)**	**Maximum Bending Moment**
	Maximum Moment=		179.000	5.38	528.00	528.00

For a corner column A-1(load = 1848.0KN)

Perimeter (bo)=2*(d/2+600)

= d+1200mm

$\tau v = \frac{Vu}{bo*d}$

$1848 * \frac{1000}{d(d+1200.0)} = 1.118\text{mm}$

$$d^2 + 1200.0d - 1652951.6 = 0$$

d = 818.78 mm or -2018.78 mm

Effective depth (d) = 850 mm

For a central column B-2(Greatest load = 2296.00KN)

Perimeter (bo)= 4*(d+600)

= 4d+2400 mm

$\tau v = \frac{Vu}{bo*d}$

$2296 * \frac{1000}{d(4d+2400.0)} = 1.118$mm

$4d^2 + 2400.0d - 2053667.263 = 0$

d = 476.799 mm or -1076.79 mm

Effective depth (d) = 500 mm

Overall Depth (D) = 550.0 mm

Reinforcement in the long direction i.e along Numeric grid

Bending Moment BM = $0.87\ fy * Ast\ \left(d - \frac{fy*Ast}{fck*b}\right)$

$356.0*10^6 = 0.87 * 500 * Ast\ (500 - \frac{500*Ast}{25*1000})$

$8.700*Ast^2 - 217500.0*Ast + 356*10^6 = 0$

Solving this equation, as quadratic equation which is quadratic in Ast

Ast = 23239.202 mm^2 or 1760.79 mm^2

Taking smaller area of steel i.e. Ast

Hence, provide 16 mm φ bar @ 150 mm c/c having area 1760.79 mm^2 at top and bottom in both direction.

Minimum reinforcement in slab = 0.12 %

= 0.12/100 *550*1000

= 660mm

Reinforcement in the long direction i.e. along Alphabetic grid

Bending Moment BM = $0.87\ fy * Ast\ \left(d - \frac{fy*Ast}{fck*b}\right)$

$528.00*10^6 = 0.87 * 500 * Ast\ (500 - \frac{500*Ast}{25*1000})$

$8.700*Ast^2 - 304500*Ast + 528.00*10^6 = 0$

Solving this equation, as quadratic equation which is quadratic in Ast

Ast = 22275.49 mm^2 or 2724.50 mm^2

Taking smaller area of steel i.e. Ast

Hence, provide 16 mm φ bar @ 100 mm c/c having area 2800 mm^2 at top and bottom in both direction.

6. Results and Discussions

The seismic analysis of the six-storey residential building was performed using ETABS, with loads determined according to NBC 105:2020 and IS 1893:2016. The results showed that the building satisfied the displacement, drift, and torsional checks prescribed by both codes, demonstrating good seismic performance.

The total base shear obtained was 1258.67 KN, distributed among storeys based on mass and height. Maximum displacement under seismic load was about 70 mm, which was within the allowable limits. Inter-storey drift values were also found to be acceptable, ensuring that the structure would remain safe and functional during strong earthquakes.

This project demonstrates the practical application of earthquake-resistant design principles in Nepal's challenging seismic environment. The fundamental period of 0.75 seconds confirms the structure's appropriate stiffness characteristics for a 6-story residential moment frame building. The dynamic analysis approach provides more accurate seismic demand assessment compared to equivalent static methods, particularly important for mid-rise buildings in high seismic zones.

The structural design confirmed that slabs behaved as two-way systems, beams and columns carried loads safely within their design capacities, and the mat foundation was effective for medium soil conditions. Overall, the building was designed with sufficient ductility and safety margins to prevent collapse, even under severe seismic loading.

In conclusion, the analysis validated that the adopted design provisions and detailing practices ensure the structural safety and resilience of the building in Kathmandu's seismic environment. The study highlights the importance of adhering to NBC 105:2020 for safer construction in Nepal.

7. Conclusion

This project successfully carried out the seismic analysis and design of a six-storey residential building in Kathmandu using ETABS. The work covered preliminary design, load estimation, base shear calculation, and structural modeling in accordance with NBC 105:2020 and IS 1893:2016. The design process ensured that all major structural elements—slabs, beams, columns, staircases, and foundation—were proportioned and detailed as per relevant codes.

The study highlighted the importance of code-based seismic design in Nepal's earthquake-prone context. By following the limit state design philosophy and applying ductile detailing provisions, the building was designed to withstand seismic forces without collapse, thereby ensuring safety and resilience. This demonstrates the effectiveness of modern structural engineering practices in achieving safe, functional, and durable residential buildings. Also comparing other codes like IS, reinforcement and sectional size are higher increasing the overall cost.

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