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# Structural Analysis and Design of High Rise Building Using Etabs

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**ABSTRACT:** The behaviour of the structures during seismic and wind loads definitely has a major role, not only from structural engineering point of view, but also safety of humans living in the structure. Consideration of site specific lateral loading due to wind or earthquake loads along with vertical gravity loads is important for finding the behaviour of the tall buildings. The design of tall buildings essentially involves a conceptual design, approximate analysis, preliminary design and optimization, to safely carry gravity and lateral loads. In the present study, the analysis and design of a 3B+G+40 multi-storey residential building for earthquake and wind loads is carried out as per Indian standard codes of practice IS 1893 :2002 and IS 875 :1987.

**KEYWORDS:** Structural Analysis, Response Spectrum, Gust Factor, Earthquake Loads, Wind Loads.

## I. INTRODUCTION

The member forces because of earthquake observed the use of reaction spectrum technique want to be increased with the aid of using a component and the RCC participants want to be designed as consistent with is 456: 2000 the use of restriction country technique. The significance of protection of the shape is checked in opposition to allowable limits prescribed for inter-storey drifts, base shear, accelerations and roof displacements because of earthquake and wind masses. Hence, this observe is done to discover the significance of protection of the shape is checked in opposition to allowable limits prescribed for inter-storey drifts, base shear, accelerations and roof displacements because of earthquake and wind masses in codes of exercise and different courses pronounced in literature. In this literature the outline of constructing version and the modelling of 3B+G+forty multi saved residential constructing were described.

## II. LITERATURE REVIEW

*Azlanand Suhana (2008)* investigated the performance of reinforced concrete buildings with the comparison on the effect of earthquake and wind loads for selected seven existing buildings from West and East Malaysia. From this study the authors noted that, the Earthquake Static Equivalent Analysis (ESEA) is mostly governed the lateral load design, rather than Static Wind Analysis (SWA) and Earthquake Dynamic Response Spectrum Analysis (EDRSA). The maximum internal forces under wind and earthquake linear analysis has similar manners of Failure at beams and column elements. The maximum axial forces at most of the building frames exceeded the design capacity of the column.

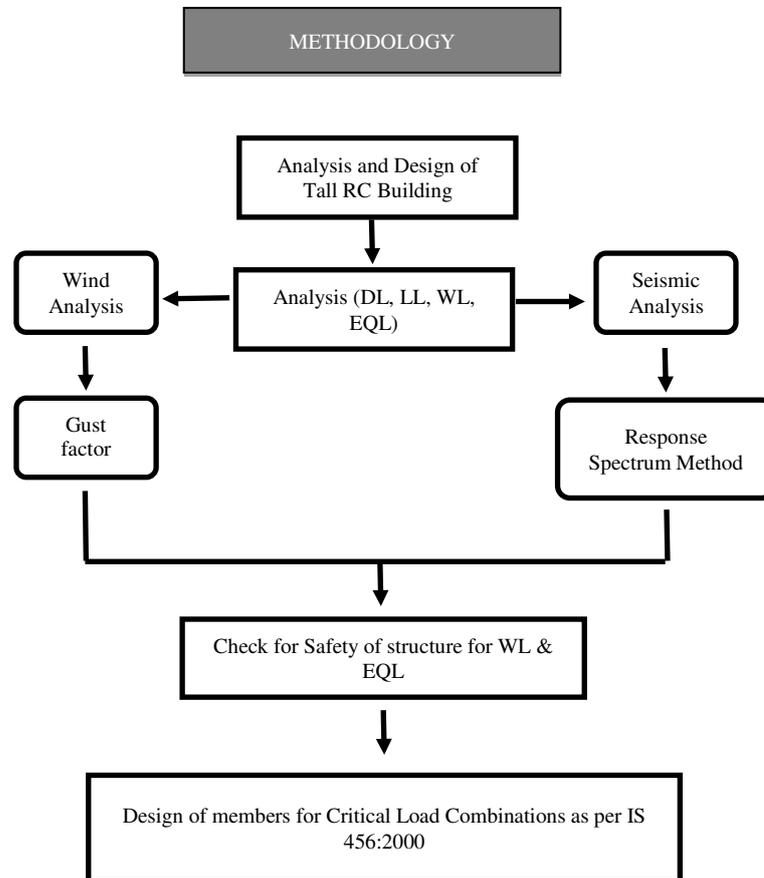
*Dean Kumar and Swami (2012)* presented a comparative study of present IS 875 (Part 3)-1987 code and their proposed draft code. The gust factor decreases with the height because as the height of the frame increases the fundamental frequency decreases, and the gust pressures computed using proposed draft code are more compared to the present IS code because of the introduction of the importance factor.

*Halder and Dutta (2010)* conducted a comparative study of the wind forces obtained by Indian standards and that by American standards. This study includes an exhaustive comparison of the wind forces obtained by force coefficient based static analysis and Gust factor based dynamic analysis.

*Swati and Vipul (2012)* analysed a G+11 building subjected to wind load by using STAAD.Pro. From this analysis, it is shown that, as the wind speed increases, moments ( $M_y$  &  $M_z$ ) and forces ( $F_y$  &  $F_z$ ) also increases according to the terrain category, and displacement increases as the wind speed increases depends upon the degree of permeability of cladding to the flow of air and terrain category.



III. METHODOLOGY OF PROJECT WORK



IV. DESCRIPTION OF BUILDING MODEL

In the present study, an RCC 3B+G+40 storeyed residential building model and its loading conditions are taken from INSDAG report (2007). The general features of the building model and beam sections used in the building are given in Table 3.1 and material properties are given in Table 3.2.

Table 3.1 General features of the model structure

Length	Width	Height					Thickness		Beamschedule
		(including basement)	BF (3nos.)	GF	TF	RF*	Slab	SW	
30.485	27	148.9	3.2	5.0	3.5	1.6	0.15	0.26	B-1:0.3x1.2 B-2:0.3x0.7
Note: Basement Floor (BF), Ground Floor (GF), Typical Floor (TF), Refuge Floor (RF), Shear Wall (SW), *10 <sup>th</sup> , 20 <sup>th</sup> and 30 <sup>th</sup> floors are designed as Refuge floors.									
All units are in 'm'									

The building has been provided with wall type RCC columns for analysis of the 3D model of the building (3B+G+40) as shown in Figure 3.1 using ETABS Software, the L (C3) and T (C4) shaped columns as given in Table 3.4. This is because centroid of L, T sections are eccentric to center line of other beams connecting them. The Star, L and Shaped column sections are shown in Figure 3.1. The plan of the building with these sections for analysis is shown in Figure 3.1. The grades of reinforcing steel and concrete used in the building are assumed to be of Fe500 and M60 respectively.



Table 3.2 Material Property

Property	Concrete	Steel
Young's Modulus, E(kN/mm <sup>2</sup> )	21.718	205
Poisson's ratio( $\nu$ )	0.2	0.3
Density, $\rho$ (Kg/m)	2549.291	7833.413

Fig 3.1 Plan of Building Considered for Analysis

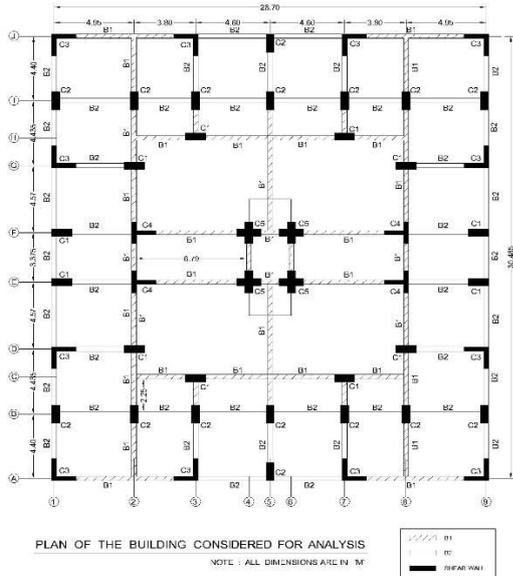


Fig 3.3 Model of Building in ETABS



Fig 3.5 Spring Coefficient values Considered for Analysis in ETABS

Fig 3.2 Plan of Building (Considered for analysis)

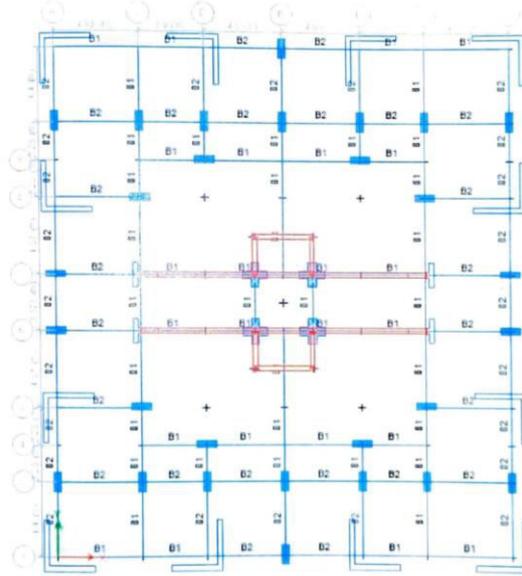
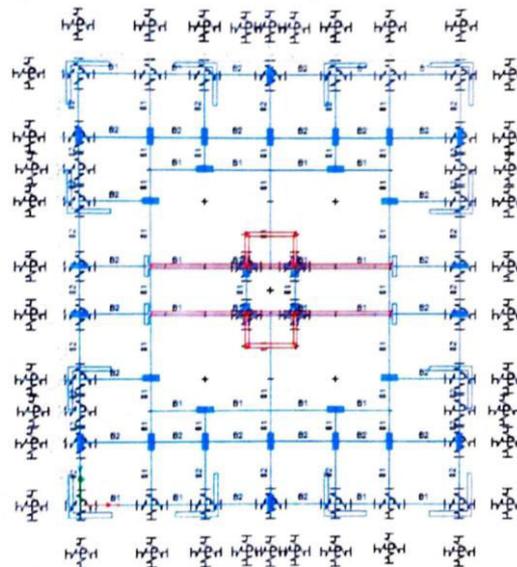


Fig 3.4 Plan of Building with Sections for analysis of Spring Values



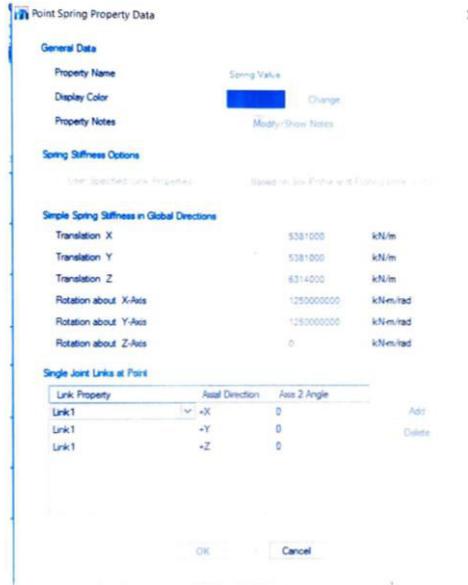


Table 3.3 Column Schedule

Column	C1	C2	C3	C4	C5
Basement to 5 <sup>th</sup> floor	0.5 x 1.25	1.25 x 0.5			
6 <sup>th</sup> to 10 <sup>th</sup> floor	0.3 x 1.25	1.25 x 0.3			
11 <sup>th</sup> to Top floor	0.3 x 1.0	1.0 x 0.3			

Note: C1 and C2 are rectangular sections. For all sections units are in 'm'

Analysis of 3B+G+40 storey RC building has been done considering the entire structure as a 3D moment resisting frame with brick infill panels using Beam and columns are considered as beam elements. The slab is modeled as an element to carry the live load as distributed pressure load. Lift well and staircase walls are modeled as shear wall to resist the lateral loads like wind and earthquake load.

**MODELING OF LOADS** The basic loads considered in this study are dead load, live loads, earthquake loads and wind loads. The summary of dead load and live loads considered for the building is given in Table 3.4. In load combinations involving Imposed Loads (LL), IS 1893 (Part 1):2002 recommends for loads up to and including 3 KN/m<sup>2</sup> 25% of the imposed load to be considered for seismic weight calculations. However to be conservative, in the present study, 100 % imposed loads are considered in load combinations.

Data for Response Spectrum Method	
Location	Mumbai
Zone	III
Zone factor (Z)	0.16
Importance Factor (I)	1.5
Response Reduction Factor (R)	3
Type of soil	Soft

Data for Gust Factor Method	
Wind zone	44m/s
Terrain category	1
Class of structure	C
Topography	Flat that is upward slope < 30
Life of structure	100 yrs
Gust Factor G	1.787
Force coefficient C <sub>r</sub>	1.38



**Dead Load and Live Loads**

Load Description	Value
Superimposed load on each floor	
• Finish load in KN/m <sup>2</sup>	1.25
• Live load in KN/m <sup>2</sup>	2.0
• Wall load with Hollow Bricks	
○ 240 mm thick external wall in KN/m	10.0
○ 125 mm thick internal wall in KN/m	3.6
Intensity of Superimposed load over Refuge Floor	
• Finish load in KN/m <sup>2</sup>	1.25
• Live load in KN/m <sup>2</sup>	4.0
Additional service load over Roof top	
• Waterproofing load in KN/m <sup>2</sup>	3.0
• Live load in KN/m <sup>2</sup>	1.5
• Service load in KN/m <sup>2</sup>	4.0
Overhead reservoir load (Common load at the column of shear wall) in KN	500

**V. ANALYSIS FOR EARTHQUAKE LOADS**

Structural analysis is a part of the detailed evaluation of an existing building or model of the new building to be designed. In static analysis, the vibration mode shapes or the time-wise variation of the quantities are not considered. In a dynamic analysis, time-wise variation of the quantities are considered. The different methods of analysis can be grouped as shown in Table 4.1.

**Table 4.1 Methods of structural analysis**

	Linear elastic	Non-linear elastic
Static	Equivalent static method	Pushover analysis
Dynamic	Response spectrum method	
	Linear time history analysis	Non-linear time history analysis

**RESPONSE SPECTRUM METHOD** As per Clause 7.8.2 of IS 1893 (Part 1):2002, for buildings having height greater than 90m, dynamic analysis with time history or response spectrum method need to be performed to obtain the design seismic force, and its distribution to different levels along the height of the building and to the various lateral resisting elements. Since the building taken for study is having a height of 148.9 m and it is more than 90m height, the dynamic analysis is required to be carried out. Here, dynamic analysis is carried out using response spectrum method.

**VI. ANALYSIS OF BUILDING FOR WIND LOADS**

The building considered for study has height to least lateral dimension ratio is 5.2 and it is more than 5 and, the natural frequency calculated using the formula given in IS 875 (Part 3):1987,  $T = 0.09H / \sqrt{d}$  [here, H is height of the building (139.3m) and d is the maximum base dimension of building (26.7m) in meters in a direction parallel to applied wind force is 0.412 Hz and by dynamic analysis of building using ETABS the natural frequency is 0.42 Hz are less than 1 Hz, the dynamic analysis of the building for wind loads need to be carried out. For considering dynamic effects in the present study, gust factor method given in IS 875 (Part 3):1987 is used.

**Basic wind speed and design wind pressure** The basic wind speed for any site shall be obtained from 1n Basic wind speed ( $V_b$ ) map of India in Fig.1 (IS 875 Part III - 1987) and shall be modified to include the following effects to get design wind speed,  $V_z$  at any height, Z for the chosen structure: (a) Risk level, (b) Terrain roughness and height of structure, and (c) Local topography factor. It can be mathematically expressed as follows:

$$V_z = V_b k_1 k_2 k_3 \tag{5.1}$$

- $V_z$  - Design wind speed at any height z in m/s,
- $k_1$  - Probability factor (risk coefficient)
- $k_2$  - Terrain roughness and height factor



$k_s$  - Topography factor

The wind pressure at any height above mean ground level shall be obtained by the following relationship between wind pressure and wind speed:

$$P_z = 0.6 V_z^2 \tag{5.2}$$

Where,

$P_z$  - Wind pressure in  $N/m^2$  at height  $z$

$V_z$  - Design wind speed in m/s at height  $z$ .

**5.3.2 LATERAL FORCE** The lateral force along wind load on a structure on a strip area ( $A_z$ ) at any height ( $z$ ) is given by:

$$F_z = A_z P_z C_f G \tag{5.3}$$

Where,

$F_z$  - Along wind load on the structure at any height  $z$  corresponding to strip area

$C_f$  - force coefficient for building

$A_z$  - Effective frontal area considered for the structure at height  $z$

$P_z$  - Design pressure at height  $z$  due to hourly mean wind obtained as  $0.6 V_z$

$G$  - Gust factor [peak load / mean load]

The gust factor can be given by the following equation,

$$G = 1 + g_r ( B(1+\Phi)^2 + SE ) \tag{5.4}$$

Where,

$g_r$  - Peak factor defined as ratio of the expected peak value to the root mean value of fluctuating load

$r$  - Roughness factor which is dependent on the size of the structure in relation to ground roughness.

$B$  - Background factor indicating a measure of fluctuating wind load

$SE$  - Measure of resonant component of fluctuating wind load

$S$  - Size reduction factor

$E$  - Measure of available energy in the wind stream at natural frequency of the structure

$\beta$  - Damping coefficient of structure

$g_r$  -  $B4$  And is to be accounted only for buildings less than 75m high in terrain category and for buildings less than 25m high in terrain category 3, and is to be taken as zero in all other cases.

**5.3.3 WIND INTENSITY ON BUILDING – GUST ENERGY FACTOR METHOD** The building is assumed to be located in Mumbai. The wind speed,  $V_B = 44$  m/s, the life of structure is assumed to be 100 years. The design wind speed,  $V_2$  is calculated using the equation (5.1) and design wind pressure,  $P_2$  is calculated using equation (5.2) and the variables  $k_1, k_2, k_3$  are assumed to be as given Table 5.1.

Table 5.1

$k_1$	$k_2$	$k_3$	$V_b$	$V_z$	$P_z$
0.07	2	0	44 m/s	$(44 \times 1.07 \times 1)k_2 = 47.08 k_2$	$0.6 \times V_z^2 = 1329.92k_2^2 N/m^2$

**Force coefficient calculation**

The width  $a$ , length  $b$ , height  $h$ , of the building taken for study are 26.7m, 30.18 m & 139.3 m respectively.

$$a/b = 26.8/30.18 = 0.8846$$

$$hb = 139.3/30.18 = 4.6150$$

For  $h/b > 1$ ,  $C_r$  value from IS 875 (Part 3) 1987, pg.39, Fig.4A is found to be 1.38.

**Gust factor calculation** The gust factor  $G$  is found using equation (5.4).

The variables  $g_r, r, L(h)$  is found from IS 875 (Part3) 1987, pg.39, Fig 8 for category I and height  $h = 139.3$  m as 0.68 & 2100 respectively. The damping coefficient  $\beta$  for reinforced concrete structure is 0.016 found from IS 875 (Part3) 1987, pg.52, Table.34. The variable  $\Phi$  is 0 for category I. Lateral correlation ( $C_y$ ) and longitudinal correlation ( $C_z$ ) constant which may be taken as 10 & 12 respectively in the absence of more precise load data.

$$\lambda = \frac{C_y b}{C_z h} = \frac{10 \times 30.18}{12 \times 139.3} = 0.1805$$

$$\frac{C_y b}{L(h)} = \frac{12 \times 139.3}{2100} = 0.796$$

For  $\lambda$  &  $\frac{C_y b}{L(h)}$ , background turbulence factor ( $B$ ) value from IS 875 (Part3) 1987, pg.50, Fig 9 is found to be 0.69.

$$\text{Reduced frequency, } F_o = \frac{C_y f_o h}{V(h)} = \frac{(12 \times 0.412 \times 139.3)}{48.49} = 14.207$$



For reduced frequency  $F_0$ , size reduction factor (S) value from IS 875 (Part3) 1987, pg.51, Fig 10 is found to be 0.12.

$$\text{The parameter } \frac{f_0 L(h)}{V(h)} = \frac{(0.415 \times 2100)}{48.49} = 17.84$$

For the parameter,  $\frac{f_0 L(h)}{V(h)}$ , Gust energy factor E value from IS 875 (Part3) 1987, pg.52, Fig 11 is found to be 0.08.

Therefore the gust factor,

$$G = 1 + g_f r \sqrt{B(1 + \Phi)^2 + \frac{SE}{\beta}} = 1 + 0.068 \sqrt{0.69(1 + 0)^2 + \frac{(0.12 \times 0.08)}{\beta 0.016}} = 1.787$$

For the analysis purpose, the lateral force  $F_z$  is considered in  $\text{kN/m}^2$  and these wind intensities at various heights are given as input to the ETABS software as given Table 5.2.

**Table 5.2 Wind Intensity at various heights**

Height (m)	-8.5	2	15.5-19	22.5-29.5	33-48.6	52.1-99.2	102.7-139.3
Intensity ( $\text{kN/m}^2$ )	1.995	0.205	2.369	2.539	2.836	3.214	3.479

**5.4 ROOF DISPLACEMENT DUE TO WIND** The maximum value of deflection in serviceability limit condition obtained for wind loads from the finite element 3-D model of ETABS is  $268.9 \text{ mm}$ . According to BS 8110-Part 2: 1985 the maximum allowable deflection is calculated as  $h/500$ , where h is the storey height for single storey building. Therefore, maximum allowable deflection value for building height of  $139.3 \text{ m}$  is  $278.6 \text{ mm}$ . Since roof displacement found from analysis  $178.14 \text{ mm}$  in X - direction and  $156.47 \text{ mm}$  in Y - direction is less than allowable ( $278.6 \text{ mm}$ ). The design of structure is safe.

**5.5 PEAK INTER-STORY DRIFT DUE TO WIND** For the performance of the building envelope to be adequate, the peak inter-storey drift must not exceed  $1/300$  to  $1/400$  of the storey height under un factored loads, although this criterion may vary depending on type of cladding or glazing and cladding attachment details. In absolute terms, inter-storey drift should not exceed  $10 \text{ mm}$  unless special details allow non-structural partitions, cladding, or glazing to accommodate larger drift. However, this criterion must also be qualified, depending on specific building features (Simiu and Miyata, 2006). For the present problem, the maximum allowable drift is assumed to be  $1/400$  of storey height. For a storey height of  $3500 \text{ mm}$ , the allowable drift is  $8.75 \text{ mm}$  and for storey height of  $5000 \text{ mm}$ , the allowable drift is  $12.5 \text{ mm}$ . For the present problem, the peak inter-storey drift due to wind loads is found to be  $6.37 \text{ mm}$  in X - direction and  $5.29 \text{ mm}$  in Y - direction which is less than the allowable, so the design is safe as shown in Table 5.3.

**Table 5.3 Peak inter-storey drifts due to wind**

Storey height	Maximum allowable drift	Peak storey drift from ETABS analysis	
5000	$5000/400 = 12.5$	X-Direction	Y-Direction
3500	$3500/400 = 8.75$	6.37	5.29

All dimensions are in 'mm'

**5.6 PEAK ACCELERATIONS DUE TO WIND** In the case of wind load, serviceability limit states for tall buildings are specified in literature (Simiu and Scanlan, 1996) in terms of degree of discomfort caused by wind induced acceleration as given in Table 5.4.

**Table 5.4. Degree of human discomfort and the acceleration levels**

Degree of discomfort	Acceleration ( $\frac{3}{4}g$ )
Imperceptible	<0.5
Perceptible	0.5 to 1.5
Annoying	1.5 to 5
Very annoying	5 to 15
Intolerable	>15

In this study the a simple expression, equation (5.5) by Islam et al, 1990 is used for finding the acceleration levels caused by wind induced on a building in urban environment.

Acceleration at height z ( $\text{m/s}^2$ ),

$$A(z) = \frac{0.0116 B^{0.26} z U^{2.74}}{(k^{0.37} \epsilon^{0.5} M^{0.63})} \tag{5.5}$$



Where,

- U - Meanhourly wind speed at the top of the building (m/s)
- B - Plan dimension of building = 30.18 m
- M - Generalized mass of building (Kg) = 34364569.96 Kg = 337001.31kN
- K - Generalized stiffness (N/m)
- N - Frequency (Hz) =  $M(2\pi N)$
- $\epsilon$  - Damping ratio = 0.05

Using equation (5.5), the maximum acceleration at a height of 139.3 m is found to be 0.1096 (%g). Therefore the degree of discomfort due to peak acceleration is imperceptible as mentioned in Table 5.4.

### VII. CHARACTERISTIC LOAD

Table 6.1 Partial safety factor ( $\gamma_f$ ) for loads (IS 456:2000)

Load Combination	Limit state of collapse			Limit state of service ability		
	DL	L	L	DL	L	WL
DL+ LL	1.5	0.5	-	1.0	0.0	-
DL+ WL	1.5 or 0.9*	-	0.5	1.0	-	1.0
DL+ LL+ WL	1.2	0.2	0.2	1.0	0.8	0.8

Notes:

- DL= Dead load., LL= Imposed load or Live load., WL= Wind load
- While considering earthquake effects, substitute EL or WL
- For the limit states of serviceability, the values of  $\gamma_f$  given in this table are applicable for short term effects. While assessing the long term effects due to creep the dead load and that part of the live load likely to be permanent may only be considered.

(\*) This value is to be considered when stability against overturning or stress reversal is critical.

Table 6.2 Load combinations with the partial safety factors for limit state of collapse

1:1.5 (DL+LL)	2-3 : 1.2 (DL + LL ±EL <sub>x</sub> ) 5-6 : 1.2 (DL + LL ± EL <sub>x</sub> ) 16-17 : 1.2 (DL + LL ± WL <sub>x</sub> ) 18-19 : 1.2 (DL = LL± WL <sub>y</sub> )	7-8 : 1.5 (DL ± EL <sub>x</sub> ) 9-10: 1.5(DL± EL <sub>y</sub> ) 19-20 : 1.5 (DL ± WL <sub>x</sub> ) 20-21 : 1.5 (DL ± WL <sub>y</sub> )	11-12 : 0.9 DL± 1.5 EL <sub>x</sub> . 13-15 : 0.9 DL± 1.5 EL <sub>y</sub> 22-23 : 0.9 DL ± 1.5 WL <sub>x</sub> 24-25 : 0.9 DL ± 1.5 WL <sub>y</sub>
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**ANALYSIS DETAILS** For analysis of the 3D model of the building (3B+G+40) shown in Figure 3.1 using ETABS, the L (C3) and T (C3) and T (C4) shaped columns are assumed as equivalent rectangular and square shaped columns as given in table 3.4. This is because the centroid of L, T sections are eccentric to the center line of other beams connecting them. Using the forces from the analysis, the columns C3, C4 and C5 are designed as L,T and Star shaped columns using the hand book by Sinha, 1996 as shown in figure 6.2. The analysis details of different load cases for the columns C1,C2,C3,C4 and C5 are listed in tables 6.4a,6.4b,6.4c,6.4d. The analysis details of different load cases for the beams at 5<sup>th</sup>, 10<sup>th</sup> and roof floors are listed in table 6.5-6.10.

**COLUMN DESIGN** The design of columns is carried out for the critical load combinations from Tables A.1.1 and A.1.2. The critical load combinations and the design details of rectangular columns C1 and C2 are given in table 6.4a. The critical load combinations and the design details of L, T and *Star* shaped columns are given in tables 6.4b, 6.4c and table 6.4d respectively.



Table: 6.4 a Longitudinal Reinforcement for “C1” & “C2” Column design details

Storey	C	Pu (KN)	M <sub>ux</sub> (KNm)	M <sub>uy</sub> (KNm)	A <sub>s</sub> (mm <sup>2</sup> )	Reinforcement
24 - 40		514.37	449.31	464.62	2400	5#25
23		649.74	541.19	127.49	2410	5#25
22		762.92	34.05	187.13	3216	7#25
21		869.96	732.54	183.46	3730	8#25
20		952.05	983.98	126.05	2893	6#25
19		1041.37	699.04	167.94	5514	11#25
18		1121.91	871.65	14.35	6643	14#25
17		1211.04	484.47	109.73	7650	16#25
16		4989.44	605.65	62.55	8582	17#25
15	5	6528.78	622.24	71.58	9668	20#25
14		7983.92	101.43	311.01	10816	22#25
13		8672.19	332.46	26.94	11899	24#25
12		9291.14	27.78	497.62	13095	27#25
11		10202.56	71.10	382.26	13479	27#25
10		11026.14	138.26	28.03	13654	27#25
9		12884.30	87.62	95.82	13930	28#25
8		12969.92	87.74	305.48	15407	31#25
7		13932.76	332.72	116.68	16977	35#25
6		13554.97	83.74	646.55	18885	38#25
5	5	10885.62	121.44	8.92	8945	18#25
4		15634.90	5456.88	119.65	10721	22#25
3		17158.61	56.38	119.66	12671	26#25
2		17236.06	4286.21	68.32	14710	30#25
1		19466.28	1643.87	3.65	16574	34#25
B3		20035.95	5262.51	86.55	20560	38#25
B2		22921.31	4377.30	101.54	22583	42#25
B1		23003.36	5724.30	91.97	24463	46#25

NOTE: Load case(LC):25:0.9DL-1.5WLy;9:1.5(DL-ELy);8:1.5(DL+ELy);1:1.5(DL+LL);DL:Dead Load; LL:Live load;#:Numbers;φ:Diameter; 'C1' & 'C2' are rectangular columns.

Table:6.4b Longitudinal Reinforcement for ‘L-Section’ Column design details (C3)

Storey	C	Pu (KN)	M <sub>ux</sub> (KNm)	M <sub>uy</sub> (KNm)	A <sub>a</sub> (mm <sup>2</sup> )	Reinforcement
40.24		683.36	747.79	82.31	6480	8#32
23		789.27	208.49	138.37	6996	9#32
22		869.96	732.54	183.46	9861	12#32
21		891.72	111.08	638.15	12508	16#32
20		1041.37	699.04	167.94	14846	18#32
19		1121.91	871.65	14.35	17832	22#32
18		1211.04	484.47	109.73	20537	26#32
17	7	4989.44	605.65	62.55	27734	34#32
16		6528.78	622.24	71.58	11629	14#32
15		7983.92	101.43	311.01	31858	40#32
14		8672.19	332.46	26.94	30661	38#32
13		9291.14	27.78	497.62	33074	41#32
12		10202.56	71.10	382.26	35292	44#32
11		11026.14	113.81	1286.48	38026	47#32
10		12884.30	87.62	95.82	40631	51#32
9		12969.92	402.56	62.37	42956	53#32
8		13932.76	332.72	116.68	45022	56#32
7		13554.97	83.74	646.55	51212	64#32
6		10885.62	121.44	8.92	53381	66#32
5		15634.90	5456.88	119.65	50937	63#32
4		20035.95	5262.51	86.55	53381	66#32
3		22921.31	4377.30	101.54	55607	69#32
2		23003.36	5724.30	91.97	57088	71#32
1		27653.86	3970.63	357.61	58883	73#32
B3		31257.54	4025.78	374.62	58723	73#32
B2		33986.34	4044.28	356.32	60482	75#32



B1		35603.25	4095.29	378.21	62260	77#32
NOTE: Load case(LC):17:0.9DL-1.5WLy;9:1.5(DL-ELy);8:1.5(DL+ELy);7:1.5(DL-ELy);1:1.5(DL+LL);DL:Dead Load; LL:Live load;#:Numbers;φ:Diameter.						

Table:6.4c Longitudinal Reinforcement for 'T' Column design details (C4)

Storey	C	Pu (KN)	M <sub>ux</sub> (KNm)	M <sub>uy</sub> (KNm)	A <sub>a</sub> (mm <sup>2</sup> )	Reinforcement
40.24		6981.63	93.88	895.69	6480	8#32
23		7145.74	111.09	638.15	14510	18#32
22		9906.29	357.80	133.88	15070	19#32
21		9946.08	223.16	105.19	17832	22#32
20		11457.07	127.39	985.58	18099	23#32
19		12031.43	188.48	835.09	19493	24#32
18		14426.08	664.72	124.01	20537	26#32
17		15242.24	218.78	692.60	27734	34#32
16		15921.52	218.83	487.50	29819	37#32
15		16601.59	218.55	257.38	30661	38#32
14		16738.35	754.18	1.17	31858	40#32
13		17431.87	1118.88	101.32	33074	41#32
12		18016.89	1240.26	98.52	35292	44#32
11		19308.13	280.53	1113.87	37118	47#32
10		21892.29	1782.64	123.57	40631	51#32
9		22444.84	258.15	1200.10	42956	53#32
8		21172.95	204.94	1632.90	48787	61#32
7		21822.82	201.46	2350.21	19106	24#32
6		22461.77	195.86	3200.44	17344	22#32
5		21707.05	175.97	4812.94	18993	24#32
4		23692.95	177.38	5480.58	20376	25#32
3		24173.93	160.19	7068.66	21282	69#32
2		24831.16	152.96	9254.48	22861	71#32
1		27631.18	116.24	13697.56	23684	73#32
B3		29038.00	2812.65	12893.35	43512	54#32
B2		31439.56	2879.53	12890.03	46376	58#32
B1		32042.63	2812.72	12836.23	45621	57#32
NOTE: Load case(LC):17:0.9DL-1.5WLy;9:1.5(DL-ELy);8:1.5(DL+ELy);7:1.5(DL-ELy);1:1.5(DL+LL);DL:Dead Load; LL:Live load;#:Numbers;φ:Diameter.						

Table:6.4d Longitudinal Reinforcement for '+' Column design details (C5)

Storey	C	Pu (KN)	M <sub>ux</sub> (KNm)	M <sub>uy</sub> (KNm)	A <sub>a</sub> (mm <sup>2</sup> )	Reinforcement
40.24		792.88	150.44	97.30	10000	20#25
23		820.43	126.26	541.84	10000	20#25
22		869.96	732.54	183.46	10000	20#25
21		891.72	111.08	638.15	10000	20#25
20		1041.37	699.04	167.94	10000	20#25
19		1121.91	871.65	14.35	10000	20#25
18		1211.04	484.47	109.73	10000	20#25
17		4989.44	605.65	62.55	10000	20#25
16		6528.78	622.24	71.58	10000	20#25
15		7983.92	101.43	311.01	10000	20#25
14		8672.19	332.46	26.94	10780.97	24#25
13		9291.14	27.78	497.62	10000	20#25
12		10202.56	71.10	382.26	10000	20#25
11		11026.14	113.81	1286.48	10000	20#25
10		12884.30	87.62	95.82	10000	20#25
9		12969.92	87.74	305.48	10000	20#25
8		13932.76	332.72	116.68	10000	20#25
7		13554.97	83.74	646.55	10000	20#25



6		10885.62	121.44	8.92	10000	20#25
5		15634.90	5456.51	119.65	10799.22	20#25
4		20035.95	5262.51	86.55	10000	20#25
3		22921.31	4377.30	101.54	10000	20#25
2		24003.36	4524.30	91.97	10000	20#25
1		27653.86	4020.88	345.4	10000	20#25
B3		31257.54	4052.76	340.65	10000	20#25
B2		33986.34	4020.78	345.4	10000	20#25
B1		35603.25	4052.11	343.64	10000	20#25

NOTE: Load case(LC):17:0.9DL-1.5WL<sub>y</sub>;9:1.5(DL-EL<sub>y</sub>);8:1.5(DL+EL<sub>y</sub>);7:1.5(DL-EL<sub>y</sub>);1:1.5(DL+LL);DL:Dead Load; LL:Live load;#:Numbers; φ:Diameter.

**BEAM DESIGN**

The longitudinal and shear reinforcements details of beams for 5<sup>th</sup>, 10<sup>th</sup>, and roof floors are carried out using IS 456:2000 for the critical load combinations and the same are given in Table 6.5 – 6.10 respectively.

**Table 6.5 Beam Longitudinal Reinforcement – 5<sup>th</sup> Floor**

Moments			Reinforcements at 0 <sup>th</sup> node			Reinforcements at Mid Span			Reinforcements at 1 <sup>st</sup> node			
Direction	Grid ID	BT	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars	
Horizontal (X)	L <sub>1,0</sub> , L <sub>10,0</sub>	B1	5180.49	4#32φ	4#25φ	2590.24	2#32φ	2#25φ	5180.49	4#32φ	4#25φ	
	L <sub>1,0</sub> , L <sub>10,0</sub>	B2	4198.74	4#32φ	2#25φ	3216.99	2#32φ	2#25φ	4198.74	4#32φ	2#25φ	
	L <sub>2,0</sub> , L <sub>9,0</sub>	B2	2590.24	2#32φ	2#25φ	2590.24	2#32φ	2#25φ	2590.25	2#32φ	2#25φ	
	L <sub>3,0</sub> , L <sub>8,0</sub>	B1	5807.23	6#32φ	2#25φ	2590.24	2#32φ	2#25φ	5180.49	4#32φ	4#25φ	
	L <sub>4,0</sub> , L <sub>7,0</sub>	B2	3216.99	2#32φ	2#25φ	3216.99	2#32φ	2#25φ	3216.99	2#32φ	2#25φ	
	L <sub>5,0</sub> , L <sub>6,0</sub>	B1	6788.98	6#32φ	4#25φ	6788.98	6#32φ	4#25φ	7415.73	8#32φ	2#25φ	
	L <sub>5,0</sub> , L <sub>6,0</sub>	B2	4825.49	3#32φ	3#32φ	3216.99	2#32φ	2#25φ	4198.74	3#32φ	3#32φ	
	Vertical (Y)	L <sub>0,1</sub> , L <sub>0,9</sub>	B2	4825.49	3#32φ	3#32φ	4825.49	3#32φ	3#32φ	4825.49	3#32φ	3#32φ
		L <sub>0,2</sub> , L <sub>0,8</sub>	B1	12867.96	8#32φ	8#25φ	7415.73	8#32φ	2#25φ	12867.96	8#32φ	8#32φ
		L <sub>0,3</sub> , L <sub>0,7</sub>	B1	2590.24	2#32φ	2#25φ	1963.5	2#25φ	2#25φ	2590.24	2#32φ	2#25φ
L <sub>0,3</sub> , L <sub>0,7</sub>		B2	4198.74	4#32φ	2#25φ	4198.74	4#32φ	2#25φ	4198.4	4#32φ	2#25φ	
L <sub>0,4</sub> , L <sub>0,6</sub>		B1	3216.99	2#32φ	2#25φ	3216.99	2#32φ	2#25φ	3216.99	2#32φ	2#25φ	
L <sub>0,5</sub>		B1	4825.49	3#32φ	3#25φ	4198.74	4#32φ	2#25φ	4825.49	3#32φ	3#25φ	
L <sub>0,5</sub>		B2	3571.99	2#32φ	4#25φ	3571.99	2#32φ	4#25φ	3571.99	2#32φ	4#25φ	

BT: Beam type : L<sub>4,0</sub>, L<sub>7,0</sub> ; Where L<sub>4,0</sub> : L refers to layer, 4,0 : 4 and 0 indicates row and column numbers in Fig 6.2

**Table 6.6 Beam Longitudinal Reinforcement – 10<sup>th</sup> Floor**

Moments			Reinforcements at 0 <sup>th</sup> node			Reinforcements at Mid Span			Reinforcements at 1 <sup>st</sup> node		
Direction	Grid ID	BT	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars
Horizontal (X)	L <sub>1,0</sub> , L <sub>10,0</sub>	B1	4198.74	4#32φ	2#25φ	2590.24	2#32φ	2#25φ	4198.74	4#32φ	2#25φ
	L <sub>1,0</sub> , L <sub>10,0</sub>	B2	3216.99	2#32φ	2#25φ	2590.25	2#32φ	2#25φ	3216.99	2#32φ	2#25φ
	L <sub>2,0</sub> , L <sub>9,0</sub>	B2	1963.50	2#32φ	2#25φ	1963.50	2#32φ	2#25φ	1963.50	2#32φ	2#25φ
	L <sub>3,0</sub> , L <sub>8,0</sub>	B1	4198.75	4#32φ	2#25φ	1963.50	2#32φ	2#25φ	4198.74	4#32φ	2#25φ
	L <sub>4,0</sub> , L <sub>7,0</sub>	B2	3216.99	2#32φ	2#25φ	2590.25	2#32φ	2#25φ	3216.99	2#32φ	2#25φ
	L <sub>5,0</sub> , L <sub>6,0</sub>	B1	5180.49	4#32φ	4#25φ	5180.47	4#32φ	4#25φ	5180.49	4#32φ	4#25φ



	L <sub>5,0</sub> , L <sub>6,0</sub>	B2	3216.99	2#32Φ	2#32Φ	2590.24	2#32Φ	2#25Φ	3216.99	2#32Φ	2#32Φ
Vertical (Y)	L <sub>0,1</sub> , L <sub>0,9</sub>	B2	4198.74	4#32Φ	2#32Φ	4198.74	4#32Φ	2#25Φ	4198.74	4#32Φ	2#32Φ
	L <sub>0,2</sub> , L <sub>0,8</sub>	B1	9024.22	10#32Φ	2#25Φ	5180.49	4#32Φ	2#25Φ	9024.22	10#32Φ	2#32Φ
	L <sub>0,3</sub> , L <sub>0,7</sub>	B1	1963.50	2#25Φ	2#25Φ	804.25	2#16Φ	2#16Φ	1963.50	2#25Φ	2#25Φ
	L <sub>0,3</sub> , L <sub>0,7</sub>	B2	3216.99	2#25Φ	2#25Φ	3216.99	2#32Φ	2#32Φ	3216.99	2#32Φ	2#25Φ
	L <sub>0,4</sub> , L <sub>0,6</sub>	B1	1963.50	2#25Φ	2#25Φ	1963.50	2#25Φ	2#25Φ	1963.50	2#25Φ	2#25Φ
	L <sub>0,5</sub>	B1	4198.74	4#32Φ	2#25Φ	3216.99	2#32Φ	2#32Φ	4198.74	4#32Φ	2#25Φ
	L <sub>0,5</sub>	B2	3216.99	2#32Φ	2#25Φ	2590.24	2#32Φ	4#25Φ	2590.24	2#32Φ	2#25Φ

BT: Beam type ; L<sub>4,0</sub>, L<sub>7,0</sub> ; Where L<sub>4,0</sub>: L refers to layer, 4,0 : 4 and 0 indicates row and column numbers in Fig 6.2

**Table 6.7 Beam Longitudinal Reinforcement – Roof Floor**

Moments		Reinforcements at 0 <sup>th</sup> node			Reinforcements at Mid Span			Reinforcements at 1 <sup>st</sup> node			
Direction	Grid ID	BT	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars	A <sub>s</sub> (mm <sup>2</sup> )	Corner bars	Middle bars
Horizontal (X)	L <sub>1,0</sub> , L <sub>10,0</sub>	B1	1963.50	2#25Φ	2#25Φ	804.25	2#16Φ	2#16Φ	1963.50	2#25Φ	2#25Φ
	L <sub>1,0</sub> , L <sub>10,0</sub>	B2	1963.50	2#25Φ	2#25Φ	804.25	2#16Φ	2#16Φ	1963.50	2#25Φ	2#25Φ
	L <sub>2,0</sub> , L <sub>9,0</sub>	B2	804.25	2#16Φ	2#16Φ	402.12	1#16Φ	1#16Φ	804.25	2#16Φ	2#16Φ
	L <sub>3,0</sub> , L <sub>8,0</sub>	B1	3216.99	2#32Φ	2#32Φ	1963.50	2#25Φ	2#25Φ	3216.99	2#32Φ	2#32Φ
	L <sub>4,0</sub> , L <sub>7,0</sub>	B2	1963.50	2#32Φ	2#32Φ	804.25	2#16Φ	2#16Φ	1963.50	2#25Φ	2#25Φ
	L <sub>5,0</sub> , L <sub>6,0</sub>	B1	2590.24	2#32Φ	2#25Φ	1963.50	2#25Φ	2#25Φ	4198.74	4#32Φ	2#25Φ
	L <sub>5,0</sub> , L <sub>6,0</sub>	B2	1206.37	3#32Φ	3#32Φ	804.25	2#16Φ	2#16Φ	1963.50	2#25Φ	2#25Φ
	Vertical (Y)	L <sub>0,1</sub> , L <sub>0,9</sub>	B2	1963.50	2#25Φ	2#25Φ	1963.50	2#25Φ	2#25Φ	1963.50	2#25Φ
L <sub>0,2</sub> , L <sub>0,8</sub>		B1	5180.49	4#32Φ	4#25Φ	2590.24	2#32Φ	2#25Φ	5180.49	4#32Φ	4#25Φ
L <sub>0,3</sub> , L <sub>0,7</sub>		B1	804.25	2#16Φ	2#16Φ	402.12	1#16Φ	1#16Φ	804.25	2#16Φ	2#16Φ
L <sub>0,3</sub> , L <sub>0,7</sub>		B2	1963.50	2#25Φ	2#25Φ	1963.50	2#25Φ	2#25Φ	1963.50	2#25Φ	2#25Φ
L <sub>0,4</sub> , L <sub>0,6</sub>		B1	804.25	2#16Φ	2#16Φ	804.24	2#25Φ	2#25Φ	804.25	2#16Φ	2#16Φ
L <sub>0,5</sub>		B1	14476.46	8#32Φ	8#32Φ	4825.49	3#32Φ	3#32Φ	14476.46	8#32Φ	8#32Φ
L <sub>0,5</sub>		B2	1963.50	2#25Φ	2#25Φ	1963.50	2#25Φ	2#25Φ	3216.99	2#32Φ	2#32Φ

BT: Beam type ; L<sub>4,0</sub>, L<sub>7,0</sub> ; Where L<sub>4,0</sub>: L refers to layer, 4,0 : 4 and 0 indicates row and column numbers in Fig 6.2

**Table 6.8 Beam Design details- shear reinforcement-5th floor**

Direction	Gird ID	T	Shear reinforcement-2 legged		
			At 0 <sup>th</sup> node	At mid span	At 1 <sup>th</sup> node
Horizontal (X)	L <sub>1,0</sub> , L <sub>10,0</sub>	B1	10φ @ 220 c/c	8 φ @ 240 c/c	10 φ @ 220 c/c
	L <sub>1,0</sub> , L <sub>10,0</sub>	B2	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	L <sub>2,0</sub> , L <sub>9,0</sub>	B2	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	L <sub>3,0</sub> , L <sub>8,0</sub>	B1	12 φ @ 120 c/c	12 φ @ 120 c/c	12 φ @ 120 c/c
	L <sub>4,0</sub> , L <sub>7,0</sub>	B2	8 φ @ 2 30 c/c	8 φ @ 300 c/c	8 φ @ 240 c/c
	L <sub>5,0</sub> , L <sub>6,0</sub>	B1	12 φ @ 120 c/c	12 φ @ 120 c/c	12 φ @ 120 c/c
	L <sub>5,0</sub> , L <sub>6,0</sub>	B2	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	Vertical (Y)	L <sub>0,1</sub> , L <sub>0,9</sub>	B2	8 φ @ 300 c/c	8 φ @ 300 c/c
L <sub>0,2</sub> , L <sub>0,8</sub>		B1	12 φ @ 120 c/c	12 φ @ 120 c/c	12 φ @ 120 c/c
L <sub>0,3</sub> , L <sub>0,7</sub>		B1	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
L <sub>0,3</sub> , L <sub>0,7</sub>		B2	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c



	L <sub>0,4</sub> , L <sub>0,6</sub> L <sub>0,5</sub> L <sub>0,5</sub>	B1 B1 B2	8 φ @ 300 c/c 12 φ @ 120 c/c 8 φ @ 300 c/c	8 φ @ 300 c/c 12 φ @ 120 c/c 8 φ @ 300 c/c	8 φ @ 300 c/c 12 φ @ 120 c/c 8 φ @ 300 c/c
BT:Beam Type; 8 φ @ 300 c/c; L <sub>5,0</sub> , L <sub>6,0</sub> : where L <sub>5,0</sub> : L refer to layer, 5,0: 5 and 0 indicates row and column numbers in Fig 6.2					

**Table 6.10 Beam Design details- shear reinforcement-roof floor**

Direction	Gird ID	T	Shear reinforcement-2 legged		
			At 0 <sup>th</sup> node	At mid span	At 1 <sup>th</sup> node
Horizontal (X)	L <sub>1,0</sub> , L <sub>10,0</sub>	B1	10 φ @ 220 c/c	8 φ @ 240 c/c	10 φ @ 220 c/c
	L <sub>1,0</sub> , L <sub>10,0</sub>	B2	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	L <sub>2,0</sub> , L <sub>9,0</sub>	B2	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	L <sub>3,0</sub> , L <sub>8,0</sub>	B1	12 φ @ 120 c/c	12 φ @ 120 c/c	12 φ @ 120 c/c
	L <sub>4,0</sub> , L <sub>7,0</sub>	B2	8 φ @ 230 c/c	8 φ @ 300 c/c	8 φ @ 240 c/c
	L <sub>5,0</sub> , L <sub>6,0</sub>	B1	12 φ @ 120 c/c	12 φ @ 120 c/c	12 φ @ 120 c/c
	L <sub>5,0</sub> , L <sub>6,0</sub>	B2	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
Vertical (Y)	L <sub>0,1</sub> , L <sub>0,9</sub>	B2	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	L <sub>0,2</sub> , L <sub>0,8</sub>	B1	12 φ @ 120 c/c	12 φ @ 120 c/c	12 φ @ 120 c/c
	L <sub>0,3</sub> , L <sub>0,7</sub>	B1	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	L <sub>0,3</sub> , L <sub>0,7</sub>	B2	8 φ @ 230 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	L <sub>0,4</sub> , L <sub>0,6</sub>	B1	8 φ @ 300 c/c	8 φ @ 300 c/c	8 φ @ 300 c/c
	L <sub>0,5</sub>	B1	12 φ @ 120 c/c	12 φ @ 120 c/c	12 φ @ 120 c/c
	L <sub>0,5</sub>	B2	8 φ @ 150 c/c	8 φ @ 260 c/c	8 φ @ 300 c/c

**VIII. SUMMARY AND CONCLUSION**

In this project, the analysis and design of a 3B+G+40- storey RCC high rise building subjected to wind and seismic loads is carried out as per IS codes of practice. The basic structural model and its loading conditions are taken from the INSDAG report, INS/PUB/104 (2007). The building is modeled using ETABS software. Base shear of the building is found with seismic coefficient method ( $\Delta_B$ ) and response spectrum method ( $V_B$ ) as per IS 1893 ( Part 1):2002. As specified in the code, the member forces found due to earthquake in X- and Y- direction ( $EL_X$  and  $EL_Y$ ) using response spectrum method are multiplied by a factor ( $\Delta_B/V_B$ ). The analysis of the building due to wind load is carried out using gust factor method given is IS 875 (Part3): 1987 code. Safety of the structure is checked against allowable limits prescribed for inter-storey drifts, base shear, accelerations and roof displacements in the codes of practice for wind and earthquake loads on the buildings. For modelling ground floors of the building, the soil structure interactions is considered using the spring stiffness values given in the literature. Member forces were found due to twenty five load combinations with the partial safety factors for collapse given in IS 456:2000, and the members were designed for the most critical member forces among them. The critical member forces found from these load cases in the members are much higher than the member forces reported in INSDAG report. Longitudinal reinforcement in all the columns along the height of the building is documented in tabular form. Shear reinforcement details are not included in the thesis. The longitudinal and shear reinforcement details of beams in 5<sup>th</sup> floor are tabulated.

Future: **Further design details need to be provided for complete design of the building:**

In this thesis, the shear reinforcement in columns and beam-column joints need to be provided using the strong column and weak beam concept as per ductile code IS:1393 and the complete design details of beams other than 5<sup>th</sup> floor also need to be provided.



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