ANALYSIS AND DESIGN OF RC TALL BUILDING SUBJECTED TO WIND AND EARTHQUAKE LOADS

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ABSTRACT

Consideration of site specific lateral loading due to wind or earthquake loads along with vertical gravity loads is important for finding the behavior of the tall buildings. As the height of a building becomes taller, the amount of structural material required to resist lateral loads increases drastically. The design of tall buildings essentially involves a conceptual design, approximate analysis, preliminary design and optimization, to safely carry gravity and lateral loads. The design criteria are strength, serviceability and human comfort. The aim of the structural engineer is to arrive at suitable structural schemes, to satisfy these criteria. In the present study, the limit state method of analysis and design of a 3B+G+40-storey reinforced concrete high rise building under wind and seismic loads as per IS codes of practice is described. Safety of the structure is checked against allowable limits prescribed for base shear, roof displacements, inter-storey drifts, accelerations prescribed in codes of practice and other relevant references in literature on effects of earthquake and wind loads on buildings.

Keywords: Tall buildings, seismic loads, wind loads, response spectrum method, gust factor method.

Introduction

In general, for design of tall buildings both wind as well as earthquake loads need to be considered. Governing criteria for carrying out dynamic analyses for earthquake loads are different from wind loads. According to the provisions of Bureau of Indian Standards for earthquake load, IS 1893(Part 1):2002, height of the structure, seismic zone, vertical and horizontal irregularities, soft and weak storey necessitates dynamic analysis for earthquake load. The contribution of the higher mode effects are included in arriving at the distribution of lateral forces along the height of the building. As per IS 875(Part 3):1987, when wind interacts with a building, both positive and negative pressures occur simultaneously, the building must have sufficient strength to resist the applied loads from these pressures to prevent wind induced building failure. Load exerted on the building envelope are transferred to the structural system and they in turn must be transferred through the foundation into the ground, the magnitude of the wind pressure is a function of exposed basic wind speed, topography, building height, internal pressure, and building shape.

The main objective of this study is to carry out the analysis of 3B+G+40 multi stored residential building against earthquake and wind loads as per Indian standard codes of practice IS 1893(Part 1):2002 and IS 875(Part 3):1987. First, the sensitivity of base shear of the building with respect to the location of the building at different wind zones in India is investigated. The wind loads and earthquake loads on the building are calculated assuming the building to be located at Mumbai. The member forces are calculated with load combinations for Limit State Method given in IS 456: 2000 and the members are designed for the most critical member forces among them. The building is subjected to self weight, dead load, live load as per IS 875(Part 1, Part 2):1987. Safety of the structure is checked against allowable limits prescribed for base shear, roof displacements, inter-storey drifts and accelerations in codes of practice and other references in literature on effects of earthquake and wind loads on buildings.

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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 1.

Table 1 General features of the model structure

Length	Width	Height					Thickness		Beam schedule
		(including basement)	BF (3 nos.)	GF	TF	RF*	Slab	SW	
30.485	27	148.9	3.2	5	3.5	1.6	0.15	0.26	B1 : 0.3x1.2 B2 : 0.3x0.7
Note: Basement Floor(BF),Ground Floor (GF), Typical Floor(TF), Refuge Floor (RF), Shear Wall (SW), *10th, 20th and 30th floors are designed as refuge floors; All units are in 'm'									

The building has been provided with wall type RCC columns having rectangular, L, Tee and Star shapes of varying sizes. The grades of reinforcing steel and concrete used in the building are assumed to be of Fe500 and M35 respectively. The material properties used for concrete and steel are given in the Table 2. The dimensions of the rectangular, L, Tee and Star sections are given in Table 3 and the prismatic general section properties of L, Tee and Star column sections given as input to STAAD.Pro building model are given in Table 4. The plan of the building model indicating the Beam and Column schedule is shown in Figure 1.

Table 2 Material property

Property	Concrete	Steel
Young's Modulus, E (kN/mm^2)	21.718	205
Poisson's ratio(μ)	0.2	0.3
Density, ρ (Kg/m^3)	2549.291	7833.413

Table 3 Column schedule

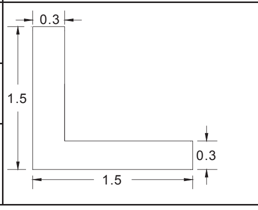
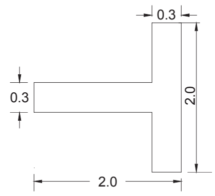
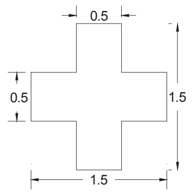
Column	C1	C2	C3	C4	C5
Basement to 5th floor	0.5x1.25	1.25x0.5			
6th to 10th floor	0.3x1.25	1.25x0.3			
11th floor to roof	0.3x1	1x0.3			
Note:C1 and C2 are rectangular sections, For all sections units are in 'm'					

Table 4 Prismatic general sections of Sections L, Tee and Star sections

C3				C4				C5			
A(m^2)	I_{yy}(m^4)	I_{zz}(m^4)	J(m^4)	A(m^2)	I_{yy}(m^4)	I_{zz}(m^4)	J(m^4)	A(m^2)	I_{yy}(m^4)	I_{zz}(m^4)	J(m^4)
0.81	0.159	0.159	0.023	1.11	0.403	0.204	0.033	1.25	0.151	0.151	0.104

MODELING OF LOADS

The basic loads considered in this study are dead load, live loads, earthquake loads and wind loads. The values of Dead loads (DL) are calculated from the unit weights as specified in IS 875 (Part 1): 1987. The live load (LL) intensities for the various areas of residential buildings are obtained from IS 875 (Part 2): 1987. The summary of dead load and live loads considered for the building is given in Table 5. In load combinations involving Imposed Loads (LL), IS 1893 (Part I):2002 recommends for loads up to and including *3* kN/m^2, *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. The earthquake loads are assigned in X and Y directions as EL_x and EL_y respectively as per IS 1893(Part 1): 2002.

ANALYSIS

Table 5 Dead Load and Live Loads

Load description	Value
Superimposed load on each floor	
▪ Finish load	1.25 kN/m^2
▪ Live load	2.0 kN/m^2
▪ Wall load with hollow bricks	
- 240 mm thick external wall	10.0 kN/m
- 125 mm thick internal wall KN/m	3.6 kN/m
Intensity of superimposed load over refuge floor	
▪ Finish load	1.25 kN/m^2
▪ Live load	4.0 kN/m^2
Additional service load over roof top	
▪ Water proofing load	3.0 kN/m^2
▪ Live load	1.5 kN/m^2
▪ Service load	4.0 kN/m^2
Overhead reservoir load (Column load at the column of shear wall)	500 kN

Analysis of 3B+G+40 storeyed RC building has been done considering the entire structure as a 3D moment resisting frame with brick infill panels using STAAD.Pro package. Beam and columns are considered as beam elements. The slabs are considered as plate elements. There are 2710 number of joints and 7585 elements in the STAAD.Pro analysis model of the structure. The main objective of modeling the whole structure as 3D model is to take into account the behavior of each and every component in space structure environment. 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. Plate elements are used for shear walls. From the analysis, the natural frequencies and time periods in first 20 modes are found to be 5.4, 4.0, 3.66, 1.94, 1.31, 1.28, 1.24, 1.14, 0.98, 0.97, 0.8, 0.77, 0.7, 0.61, 0.56,0.55, 0.54, 0.53, 0.47Hz and, 0.19, 0.25 , 0.27, 0.52, 0.77, 0.78,0.81,0.88,0.88, 1.02, 1.03, 1.24, 1.3, 1.42, 1.65, 1.8, 1.83, 1.84, 1.89 and 2.13 sec respectively.

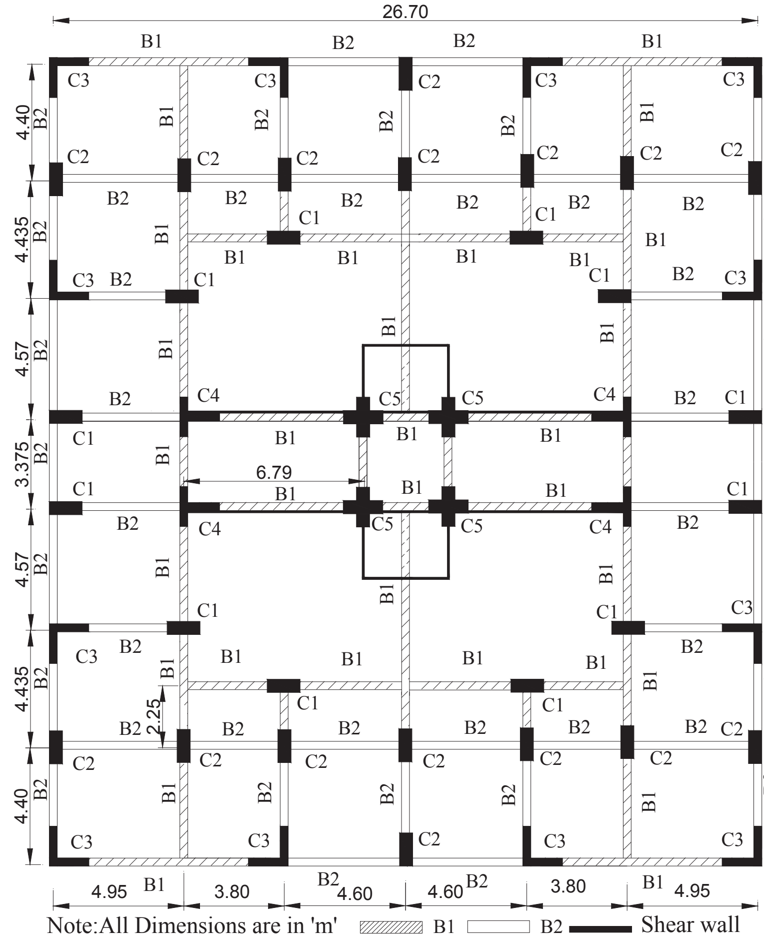


Figure 1. Plan of the model

Analysis of Building for Earthquake Loads as per IS 1893(Part-1):2002

The *equivalent static analysis* also known as the *equivalent lateral force procedure* or *seismic coefficient method* (SCM) is used for finding the response of the structures to earthquake loads. In this analysis, first the base shear along X- or Y- direction is calculated in terms of a spectral acceleration coefficient, the weight of the building and a few other variables. In seismic coefficient method, the total design seismic base shear (□$_B$) calculated as per clause 7.5.3 of IS 1893(Part 1): 2002. The plan of the 3B+G+40 storeyed RC building considered is unsymmetrical and it is assumed to be not sensitive to torsion. Since, the building is modeled as moment resisting frame with brick infill, the time periods in X- and Y-direction are calculated using the approximate formula given in Clause 7.6.2, $T_a = 0.09h/\sqrt{d}$, where, *h* is height of the building, in *m* and *d* is base dimension of the building at plinth level, in *m,* along the considered lateral force. For the present building, time period in X- and Y-direction is found to be 2.59*s* and 2.44*s* respectively. The data used for the seismic analysis are; importance factor, *I=1.5*, response reduction factor, *R=3*. The base shear values found using SCM for building located in Mumbai (zone III, zone factor, *z=0.16* and soft soil) are given in Table 6.

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 148.9m 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*.

The *response spectrum method* (RSM)also known as the "modal analysis procedure" is performed in accordance with the requirements of Clause 7.8.4, IS 1893(Part 1): 2002. The method is based on superposition of modes. Hence, free vibration modes are computed using eigen value analysis. The maximum value of a quantity (say λ_k) termed as the modal response, is obtained for each mode (say k^{th} mode). The number of modes considered is based on a quantity termed as the mass participation factor for each mode. Sufficient number of modes (*r)* to capture at least 90% of the total participating mass of the building (in each of the horizontal directions), should be considered in the analysis. In the present study, 20 modes are considered and corresponding participation of the building is 92%. The modal responses from all the considered modes are then combined together using either the square root of the sum of the squares (*SRSS*) method or the complete quadratic combination (*CQC*) method. The peak response quantities (for example, member forces, displacements, storey forces, storey shears, and base reactions) are combined as per *CQC* or SRSS method. In the present building, the natural periods of the building considered are found to be very closely spaced. So, a formulation known as the **C**omplete **Q**uadratic **C**ombination (**CQC**) based on the theory of random vibration and is also considered as the extension of SRSS method, is used for calculating earthquake loads for very closely spaced time periods. The base shear of building is found by response spectrum method using STAAD.Pro and the results are given in Table 6.

Table 6. Base shear (kN) of building

Zone	Soil Type	SCM		RSM	
		X direction	**Y** direction	**X** direction	**Y** direction
III	Soft	**8475.91**	**9032.7**	**3793.03**	**4362.78**

In response spectrum method, the design base shear (V_B) shall be compared with a base shear ($\tilde{V}_B$) calculated with seismic coefficient method. If V_B is less than □$_B$, all the response quantities (for example member forces, displacements, storey forces, storey shears and base reactions) need to be multiplied by a factor □$_B$ / V_B as per clause 7.8.2 of IS 1893(Part 1): 2002. In X-direction, V_B by response spectrum method is 3793.03 kN and □$_B$ calculated by seismic coefficient method is 8475.91kN. In Y-direction, V_B by response spectrum method is 4362.78 kN and □$_B$ calculated by seismic coefficient method is

9032.7kN. The earthquake loads are assigned in X and Y directions as EL_x and EL_y respectively as per IS 1893(Part 1): 2002, need to be multiplied by factors ($\square_B/V_B$) in X- and Y- direction are 2.2346 and 2.0704 respectively.

Storey Drift for seismic loads

As per Clause no. 7.11.1 of IS1893(Part 1):2002, the peak storey drift in any storey due to specified design lateral force with partial load factor of 1.0, shall not exceed 0.004 x h_s, where h_s is storey height (3500 mm). So maximum drift allowed= 0.004 x 3500 = 14 mm. From the response spectrum analysis, the peak storey drift in X-direction is found to be 10.35 mm. Since response quantities need to be multiplied by a factor, $\square_B$ / V_B =2.2346, the peak storey drift in X-direction is 23.14mm and it is more than the allowable.

The Lateral Wind Force (F_z) as per IS875 (Part 3):1987

According to the provisions of Bureau of Indian Standards for wind loads, IS 875 (Part 3):1987 dynamic analysis for wind load is suggested for closed buildings with height to minimum lateral dimension ratio of more than 5 or fundamental frequency of the building less than 1 Hz. It is suggested to check for wind induced oscillations and a magnification factor called gust response factor needs to be included in the dynamic effects of the wind. Since 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 ($i.e., T = 0.09H/\sqrt{d}$, here, H is height of the building and d is the maximum base dimension of building in metres in a direction parallel to applied wind force) is *0.412 Hz* and by dynamic analysis of building using STAAD.Pro the natural frequency is *0.17 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 IS 875 (Part 3):1987 is used.

The design wind speed, V_z at any height z is found by equation,

$$V_z = V_b k_1 k_2 k_3 \qquad (1)$$

where, V_b is basic wind speed in m/s, k_1 is probability factor (risk coefficient), k_2 is terrain roughness and height factor and k_3 is topography factor as per Clause 5.3.3.

The lateral force along wind load on a structure on a strip area (A_e) at any height, z is found by equation,

$$F_z = A_e P_z C_f G \qquad (2)$$

where, C_f is force coefficient for building, calculated from clause no 6.3.2.1(fig.4), A_e is effective frontal area considered for the structure at height z, P_z is design pressure at height z due to hourly mean wind obtained as $0.6V_z^2$(N/m^2), G is gust factor (peak load / mean load) as per Clause 8.3. The data considered for the guest factor method are wind speed, V_b=*44m/s*, force coefficient, C_f =*1.38,* K_1=*1.07,* K_2 is varying with height as per Terrain Category I, K_3=*1,* Life of the structure 100 yrs, Gust factor, G=*1.787.* For the analysis purpose, the lateral force F_z is considered in kN/m^2 and these wind intensities at various heights are given as input to the STAAD.Pro software as given in Table 7. From the analysis, the base shear and base moment due to wind load for wind speed zone *III* in X direction are found to be *13648.03 kN* and *205982.86 kN-m* respectively. Similarly in Y direction base shear and base moment are *12072.3 kN* and *161165.2 kN-m* respectively. The building is assumed to be located at different wind speed zones and corresponding base shear and base moment are found by gust factor method using STAAD.Pro and the results are given in Table 8**.**

Table 7 Wind Intensity at various heights

Height (m)	5-8.5	12	15.5-19	22.5-29.5	33-48.6	52.1-99.2	102.7-139.3
Intensity (kN/m^2)	1.995	2.205	2.369	2.539	2.836	3.214	3.479

Roof displacement and Peak inter-story drift due to wind

According to BS 8110-Part 2: 1985 the maximum allowable deflection is calculated as $h_s/500$, where h_s is the storey height for single storey building. Therefore, maximum allowable deflection value for building height of *148.9m* is *297.8 mm*. The maximum value of deflection in serviceability limit condition obtained for wind loads from the finite element 3-D model of STAAD.Pro is *462.23 mm* is more than allowable (*297.8 mm*). The design of structure is not safe. For the performance of the building envelope to be adequate, the peak inter-story 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. For the present problem, the maximum allowable drift is assumed to be *1/400* of storey height. For a storey height of *3500 mm*, the allowable drift is *8.75 mm*. For the present problem, the peak inter-story drift due to wind loads is found to be *20.25 mm* which is more than the allowable, so the design is not safe.

Table 8 Base shear and Base moment of building located at different wind speed zones

Direction	Wind speed zone	*I*	*II*	*III*	*IV*	*V*	*VI*
	Basic wind speed(*m/s*)	33	39	44	47	50	55
X	Base shear (*kN*)	6658.51	9656.47	13648.03	15022.84	17437.93	21279.47
	Base moment (*kN-m*)	100493.38	145740.33	205982.86	226732.24	263181.94	321160.34
Y	Base shear (*kN*)	6083.5	8822.55	12072.3	13725.56	15931.76	19405.58
	Base moment (*kN-m*)	81214.77	117781.1	161165.2	183236.7	212689	259064.45
Note: The base shear values corresponding to building located in Mumbai (zone III) are underlined,							

Peak accelerations due to wind

In this study the simple expression, equation (3) 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 (m/s^2), $A(z) = 0.0116\ B^{0.26}\ z\ U^{2.74} / (K^{0.37}\ \xi^{0.5}\ \ M^{0.63})$ (3)

Where, U is mean hourly wind speed at the top of the building (*m/s*), B is plan dimension of building (*m*), M is generalized mass of the building (kilogram), K is generalized stiffness (*N/m*) = $M\ (2\pi N)^{2}$, N is frequency (*Hz*) and ξ is damping ratio. Using equation (3), the maximum acceleration at a height of *139.3m* is found to be *0.1096%g*. This is less than 0.5%g. Therefore the degree of discomfort is imperceptible (Simiu and Miyata, 2006).

Load Combinations

The variation in loads due to unforeseen increases in loads, constructional inaccuracies, type of limit state etc. are taken into account to define the design load. The design load is given by: design load = γ_f x characteristic load (Clause 36.4 of IS 456: 2000). Where, γ_f given Partial safety for loads for loads given in Table18 of IS 456: 2000 is given in Table 9.

After the computational model is developed and the loads are assigned, the model needs to be analyzed for the individual load cases. The internal forces in the members (such as bending moment, shear force and axial force) for earthquake and wind load cases are combined as per Table18 of IS 456: 2000, IS 1893(Part 1): 2002, Section 6.3 and IS 875 (Part 5):1987, Section 8.1 are given in Table 10. The internal forces in the members (such as bending moment, shear force and axial force) in the structure are found from twenty five dead load (DL), live load (LL), Earthquake load (EL), and wind load (WL) load combinations with the partial safety factors for limit state of collapse given in Table10. The members are designed for the most critical member forces among them.

Table 9 Partial safety factor (γ_f)for loads(According to IS 456 : 2000)

Load combination	Limit state of collapse			Limit state of serviceability		
	DL	IL	WL	DL	IL	WL
DL + IL	1.5	1.5	-	1.0	1.0	-
DL+ WL	1.5 or 0.9*	-	1.5	1.0	-	1.0
DL+ IL + WL	1.2	1.2	1.2	1.0	0.8	0.8

Notes: (*) This value is to be considered when stability against overturning or stress reversal is critical.
1. DL = Dead load; IL = Imposed load or Live load; WL = Wind load
2. While considering earthquake effects, substitute EL for WL
3. For the limit states of serviceability, the values of γ_f given in this table are applicable for short tern 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.

Table 10 Load combinations with the partial safety factors for limit state of collapse

$1.5(DL + LL)$	$1.2(DL + LL \pm EL)$	$1.5(DL \pm EL)$	$0.9DL \pm 1.5EL$
While considering wind load effects, substitute EL with WL and they are in both X and Y directions, i.e., EL_x, EL_y, WL_x and WL_y			

While calculating earthquake loads, the member forces due to earthquake load (EL_x and EL_y) found using response spectrum method need to be multiplied by a factor ($\tilde{V}_B$ / V_B) as described in Section 4.1 of IS 1893(Part-1):2002. The critical load combination for each group of beam elements is the bending moment (*Mu*), Base shear (*Vu*) and corresponding load case from the present STAAD.Pro results and INSDAG report are listed in Table 11.

Table 11 Beam analysis results

Floor levels	Sub division	Load Case	STAAD.Pro results		INSDAG report	
			Mu(kN-m)	Vu (kN)	Mu(kN-m)	Vu (kN)
G1	B1	10	3525.77	2281.22	-	-
	B2	12	972.13	486.43	-	-
G2	B1	12	3842.93	2467.07	-	-
	B2	12	1090.90	544.41	256.84	197.70
G3	B1	12	3153.92	1574.83	1039.60	46.88
	B2	10	1000.76	474.50	442.97	160.44
G4	B1	12	3153.92	1574.83	1300.26	31.09
	B2	12	899.92	444.52	450.71	171.71
G5	B1	10	1102.74	471.30	820.03	104.64
	B2	10	1000.76	474.50	254.19	167.99
G6	B1	12	2019.68	925.75	921.63	3.92
	B2	12	855.62	387.94	206.92	145.53
G7	B1	10	2593.04	2651.60	-	-
	B2	12	718.16	330.11	-	-

Note:G1:Basement to GF;G2:GF, G3:GF to 5th floor; G4:5th floor to 10th floor; G5:10th floor to 20th floor; G6:20th floor to 30th floor, G7:30th to roof , GF: Ground floor, Load Cases: 10=1.5(DL+EL_x),12=1.5(DL+EL_z),

The critical load combination and corresponding load case for each group of column elements is the axial load; bending moment in X- and Y-directions (*Mux* and *Muy*) and percentage of steel (p) required for design from the present STAAD.Pro results and INSDAG report are listed in Table 12. The load cases considered for analysis are not mentioned in INSDAG report. From Tables 11-12, it is observed that the forces found from present analysis in beams and columns using STAAD.Pro are much higher than the results reported INSDAG report. The critical forces for both beams and columns are due to load combinations involving earthquake loads. The rectangular columns are designed using SP16 and L, Tee and Star shape columns are designed using the handbook by Sinha (1996). Some of the cases, the columns need to be redesigned because reinforcement required found to be much higher than maximum limit prescribed by IS 456, 2000 (4%).

Conclusions

The response of a tall building under wind and seismic load as per IS codes of practice is studied. Seismic analysis with response spectrum method and wind load analysis with gust factor method are used for analysis of a 3B+G+40-storey RCC high rise building as per IS 1893(Part1):2002 and IS 875(Part3):1987codes respectively. The building is modeled as 3D space frame using STAAD.Pro software. It is observed that the forces found from present analysis in beams and columns using STAAD.Pro are much higher than the results reported INSDAG report. The load cases considered for analysis are not mentioned in INSDAG report. Safety of the building is checked against allowable Limits prescribed for inter-storey drifts, base shear, accelerations and roof displacements in codes of practices and other references in literature. The structure is found to be wind and earthquake sensitive and the roof displacement and inter-storey drifts due to wind and earthquake are exceeding the limits prescribed. While designing, some of the beams and column sections, the limit on maximum percentage of reinforcement in the member is exceeding the maximum percentage of reinforcement in the member. To satisfy these limits, it is suggested to increase the grade of the concrete from M35 to M60 and the cross sections of the columns and beams are also need to be increased.

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Table 12 Column analysis and design results

Group	SD	STAAD.Pro results						INSDAG report			
		Load case	Vu (kN)	Pu (kN)	Mux (kN-m)	Muy (kN-m)	P	Pu (kN)	Mux (kN-m)	Muy (kN-m)	p
G1	C1	14	1400.58	25016.52	449.22	6456.27	**8.46**	8922.68	616.64	24.03	2
	C2	13	1546.94	19904.76	3050.96	112.41	**8.04**	9687.11	599.68	119.44	2
	C3(L)	12	1143.66	24116.57	7139.35	388.10	**8.01**	-	-	-	2.53
	C4(T)	13	2808.87	30518.13	11885.62	227.17	**7.42**	24271.46	9314.06	3543.24	0.84
	C5(+)	12	1364.51	36044.39	5020.54	230.66	**4.39**	23875.86	8884.24	613.80	0.84
G2	C1	13	1391.52	14518.62	2463.59	1270.89	**8.05**	7453.05	542.50	52.56	2
	C2	13	1619.44	17462.90	2878.89	392.93	**8.05**	7752.16	332.25	85.12	2
	C3(L)	13	805.38	23575.67	1658.65	807.80	**4.47**	10241.97	894.37	678.91	2.53
	C4(T)	13	2505.49	17543.43	5333.28	404.76	0.84	11028.85	254.09	141.80	0.84
	C5(+)	11	1433.56	19576.92	439.49	2139.06	2.11	11028.85	254.09	141.80	0.84
G3	C1	13	2954.41	11871.53	4155.41	882.04	**8.88**	6142.98	438.16	56.98	2
	C2	13	1707.99	15121.90	2723.80	465.08	**8.37**	6511.62	112.78	106.30	2
	C3(L)	13	1338.88	18826.76	2885.81	720.12	**4.89**	10477.33	187.90	113.56	1.68
	C4(T)	13	3648.51	11975.92	6955.89	571.75	0.84	8822.55	144.26	55.90	0.84
	C5(+)	14	1077.65	7491.93	302.36	1956.19	0.84	8822.55	144.26	55.90	0.84
G4	C1	13	1843.08	9348.43	3193.52	316.40	**6.53**	5096.22	337.45	64.82	2
	C2	13	1112.01	12417.10	2124.52	230.22	**6.9**	4426.92	209.76	98.75	2
	C3(L)	13	730.45	14548.53	1226.14	197.17	0.84	8913.85	178.58	128.47	1.68
	C4(T)	12	1669.97	8502.29	2356.05	86.76	0.84	6873.38	143.67	54.13	0.84
	C5(+)	14	527.37	4955.46	44.31	726.67	0.84	6873.38	143.67	54.13	0.84
G5	C1	12	1718.97	5700.30	2156.83	275.65	**6.53**	3541.87	110.52	76.39	2
	C2	12	943.59	7034.39	1744.25	203.31	**6.09**	3522.55	125.92	94.73	2
	C3(L)	12	624.67	7444.87	1012.64	173.96	0.84	5184.09	246.74	125.23	1.68
	C4(T)	12	1491.87	5912.38	2056.85	166.48	0.84	4195.58	123.96	48.64	0.84
	C5(+)	10	680.49	7524.18	1915.20	622.50	0.84	4195.58	123.96	48.64	0.84

Note:G1:Basement to Ground floor;G2:Ground floor to 5thfloor;G3:5thfloor to 10thfloor; G4:10thfloor to 20thfloor; G5:20thfloor to roof ,SD: Sub Division, Load Cases: 10=1.5(DL+EL$_x$), 12=1.5(DL+EL$_z$), 13=1.5(DL-EL$_z$), 14=0.9DL+1.5EL$_x$; p=percentage of steel (100A$_s$/bd)