

HEIGHT-DEPENDENT WIND–SEISMIC GOVERNORSHIP IN RC FRAME-CORE BUILDINGS: A CODE-BASED PARAMETRIC STUDY

Himanshu Naresh Selukar¹, Mr. Vivek Shukla², Dr. Jyoti Yadav³

Research Scholar, Department of Structural Engineering, Sarvepalli Radhakrishnan University,
Bhopal¹

Guide, Department of Structural Engineering, Sarvepalli Radhakrishnan University, Bhopal²

Head of the Department (HOD), Department of Structural Engineering, Sarvepalli Radhakrishnan
University, Bhopal³

ABSTRACT

Tall reinforced concrete (RC) buildings must resist two dominant lateral actions, earthquake ground motion and strong wind, and the governing action depends on height, stiffness and seismic hazard. This empirical study analyses how these demands change with building height through a data-driven, code-based parametric investigation. Five RC frame-core models with 10, 15, 20, 25 and 30 storeys, each on a 24 m x 24 m plan, were analysed using the seismic provisions of IS 1893 (Part 1):2016 and the wind provisions of IS 875 (Part 3):2015. Fundamental periods, seismic coefficients, design base shear, wind base shear, overturning moments and serviceability drifts were computed and organised into five tabulated datasets. The results show that wind base shear grows from 1052 kN to 4550 kN as height increases from 10 to 30 storeys, whereas Zone V seismic base shear grows only from 2588 kN to 3406 kN because the lengthening period reduces spectral acceleration. Wind therefore governs beyond about 20 to 25 storeys in Zone V and beyond 10 to 15 storeys in Zone III. Top wind displacement rises from 0.4 mm to 45.1 mm, yet all serviceability limits are satisfied. The conclusions confirm that the governing design criterion shifts with height and that core stiffness is the key control parameter for slender towers. These findings form the basis of the design recommendations presented in the conclusion. Wind-to-seismic demand ratios are reported so that engineers can interpolate the results to intermediate heights, and the study is intended as a screening tool for preliminary design.

Keywords: High-rise buildings¹; Reinforced concrete²; Seismic analysis³; Wind load⁴; Base shear⁵; Storey drift⁶; Data analysis⁷

1. INTRODUCTION

1.1 Background of Tall RC Buildings

Rapid urbanisation has pushed reinforced concrete (RC) towers to heights once associated mainly with steel and composite construction [1], [2]. Concrete offers mass, stiffness, fire resistance and local material availability, which make it attractive for residential and commercial high-rise development in India and many other countries [3], [2]. Early high-rise structures were governed largely by gravity loads, but modern towers are proportioned by lateral action. In growing Indian cities, vertical development on constrained urban sites has further increased this trend. Yet tall buildings are slender, and their lateral response is governed by stiffness, damping and the dynamic characteristics of the structure rather than by gravity capacity alone [4], [1].

1.2 Lateral Load Actions

Two lateral actions dominate the design of such buildings. Earthquake loads arise from ground acceleration and are estimated through a design response spectrum and a response reduction factor [5], [6], [7]. Wind loads arise from turbulent atmospheric flow and are estimated from basic wind speed, terrain roughness and height factors [8], [9], [10]. Because seismic zones and basic wind speeds vary across India, the relative importance of the two actions can differ from one site to another. Both actions increase with height, but they do so differently: seismic demand depends on the period of vibration, whereas wind demand depends on the velocity profile and the exposed area. Codes therefore impose drift and displacement limits to protect non-structural elements and occupant comfort [11], [12], [13]. Designers must decide which action governs, because this choice affects member sizes, detailing and cost [14], [15].

1.3 Objectives and Contribution

Despite extensive research, practitioners often lack a compact, data-driven comparison showing where the governing lateral action changes with height in a given seismic zone. Several studies classify structural systems or review wind and seismic effects separately [16], [17]. This study addresses that gap. The objectives are: (i) to quantify fundamental periods, seismic coefficients and wind pressure profiles for five building heights; (ii) to compare seismic and wind base shear and overturning moments; (iii) to check top and storey drifts against code limits; and (iv) to identify the height at which wind governs for two seismic zones. The study therefore asks a practical question: at what height does wind replace seismic action as the governing criterion, and how does this depend on the seismic zone? The paper contributes a reproducible parametric dataset, five analytical tables and a critical comparison with earlier work.

2. SURVEY OF RELATED WORK

Seismic design of RC buildings has developed from the concept of an equivalent static force and a response reduction factor, which allows ductile yielding to reduce design forces [7], [6]. Modal response spectrum analysis, described in standard dynamics texts, enables the designer to capture the contribution of higher modes [18], [19]. Capacity design and ductile detailing, presented in the work of Paulay and Priestley, are essential to avoid brittle failure of RC members [20], [21]. Indian practice implements these principles through IS 13920 for ductile detailing [22], while IS 1893 provides the seismic coefficient method used in this study [5].

Research on tall-building systems has concentrated on confinement and the behaviour of structural walls. The confined stress-strain model of Mander et al. remains a benchmark for estimating the ductility of core and column sections [23]. Wallace summarised the behaviour of structural walls and coupling beams from laboratory and earthquake observations, showing that walls provide the stiffness needed to control drift [24]. Gunel and Ilgin proposed a classification of tall-building structural systems [16], while Ali and Moon reviewed structural developments and current trends in tall buildings [1]. Smith and Coull and Taranath provide comprehensive design procedures for tall buildings in steel and concrete [4], [2].

Wind engineering for tall buildings has been shaped by Davenport's statistical framework, which treats wind as a random process with a known spectrum [10]. Holmes presented the principles of wind loading that underpin modern codes [9], while Kareem described numerical simulation approaches that account for the probabilistic nature of wind effects [25]. Irwin highlighted the challenges of super-tall buildings, where dynamic response can control the design [13]. Kareem, Kijewski and Tamura reviewed motion mitigation techniques for tall buildings [17]. Kwok, Hitchcock and Burton showed that perception of wind-induced motion can control occupant acceptability, an important serviceability consideration [26]. Taranath also treats wind and earthquake resistant design of buildings within one framework [27].

Wilson described three-dimensional static and dynamic analysis formulations suitable for multistorey systems [19]. International codes, including ASCE/SEI 7 and Eurocode 8, define seismic and wind provisions with differing treatments of importance, ductility and directionality [11], [28]. ACI 318 provides the reinforced concrete requirements used internationally [29], while IS 456 governs Indian practice for plain and reinforced concrete [3]. Naeim's handbook collects seismic design practice for buildings [30]. Despite this body of knowledge, comparative data-driven studies linking height, seismic zone and wind speed to the governing action remain limited, particularly for Indian conditions. The present study responds to this gap.

3. METHODOLOGY

A parametric, code-based study was conducted on five RC frame-core buildings with 10, 15, 20, 25 and 30 storeys. Each building has a square plan of 24 m × 24 m, a storey height of 3.0 m and a seismic weight of 12 kN/m² per

floor, including dead and reduced live load. Concrete grade M35 ($f_{ck} = 35$ MPa) with elastic modulus $E_c = 5000 \sqrt{f_{ck}} = 29.6$ GPa was assumed. Lateral stiffness was idealised as an equivalent cantilever with effective second moment of area $I_{eff} = 0.7 \times 0.02 \times I_g$, where $I_g = B^4/12$, to account for cracking.

Seismic demand was computed with the equivalent static method of IS 1893 (Part 1):2016 [5]. The fundamental period of a bare RC moment-resisting frame was estimated as $T_a = 0.075 h^{0.75}$, where h is height in metres. The spectral acceleration coefficient S_a/g was taken for medium soil. The design horizontal coefficient $A_h = (Z/2)(I/R)(S_a/g)$ was evaluated for Zone III ($Z = 0.16$) and Zone V ($Z = 0.36$), with importance factor $I = 1.0$ and response reduction factor $R = 5$ for special moment-resisting frames. The design base shear $V_B = A_h W$ was distributed to floors as $Q_i = V_B W_i h_i^2 / \sum(W_j h_j^2)$ [6].

Wind demand followed IS 875 (Part 3):2015 [8]. A basic wind speed $V_b = 39$ m/s, assumed representative of inland Madhya Pradesh, was adopted with factors $k_1 = k_3 = k_4 = 1.0$. The height factor k_2 was approximated by the power law $(z/10)^{0.20}$ for suburban terrain, and the design pressure $p_z = 0.6 V_z^2$ was computed at each floor [9]. A force coefficient $C_f = 1.3$ was applied over a tributary height of 3 m. Displacements were obtained by double integration of the cantilever moment-curvature relation and checked against the wind limit $H/500$ and the seismic storey-drift limit of $0.004h$ [5].

4. DATA COLLECTION AND ANALYSIS

This section presents the data generated from the models described in Section 3. All values were computed with the stated equations and are reported in five tables, each followed by an explanation and supporting figures.

4.1 Model Geometry and Input Data

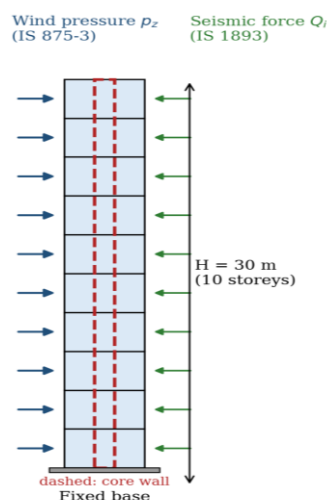


Figure 1: Idealised 10-storey elevation model with wind and seismic loads.

Figure 1 shows the idealised elevation used for all five models. Each building has a 24 m square plan, 3.0 m storeys, a central core wall shown with dashed lines and a fixed base. The blue arrows on the left represent wind pressure p_z , which increases with height because of the power-law wind profile, so the upper floors receive the largest lateral load. The green arrows on the right represent seismic floor forces Q_i , distributed in proportion to weight times height squared as prescribed by IS 1893. The schematic defines the load paths and the cantilever idealization used in the drift calculations.

Table 1: Geometric And Dynamic Properties Of The Models

Storeys	Height H (m)	Period T_a (s)	S_a/g	Seismic weight W (kN)
10	30	0.961	1.040	69,120
15	45	1.303	0.767	103,680
20	60	1.617	0.618	138,240
25	75	1.911	0.523	172,800
30	90	2.192	0.456	207,360

Table 1 shows that height increases linearly with storey count, whereas the code-based fundamental period increases nonlinearly, from 0.961 s for 10 storeys to 2.192 s for 30 storeys. Because the floor load is held constant at 12 kN/m², the seismic weight scales linearly from 69,120 kN to 207,360 kN. The period-height relation places the 30-storey model on the descending branch of the response spectrum, so its seismic coefficient is much lower than that of the low-rise models. These properties form the input to all subsequent calculations and establish the link between building height and dynamic response.

4.2 Seismic Demand Analysis

Table 2: Seismic Coefficients And Base Shear ($R = 5$, $I = 1.0$, Medium Soil)

Storeys	T_a (s)	A_h , Zone III	VB, Zone III (kN)	A_h , Zone V	VB, Zone V (kN)
10	0.961	0.01664	1,150	0.03745	2,588
15	1.303	0.01228	1,273	0.02763	2,864
20	1.617	0.00990	1,368	0.02227	3,078
25	1.911	0.00837	1,446	0.01883	3,255
30	2.192	0.00730	1,514	0.01643	3,406

Table 2 shows that the design horizontal coefficient A_h decreases from 0.01664 to 0.00730 in Zone III as the period lengthens. Consequently, base shear increases by only 32 per cent, from 1150 kN to 1514 kN, although the seismic weight triples. Zone V demand is 2.25 times the Zone III demand at every height because A_h is proportional to Z , ranging from 2588 kN to 3406 kN. This result indicates that seismic hazard, rather than height alone, determines the absolute magnitude of seismic demand in the study.

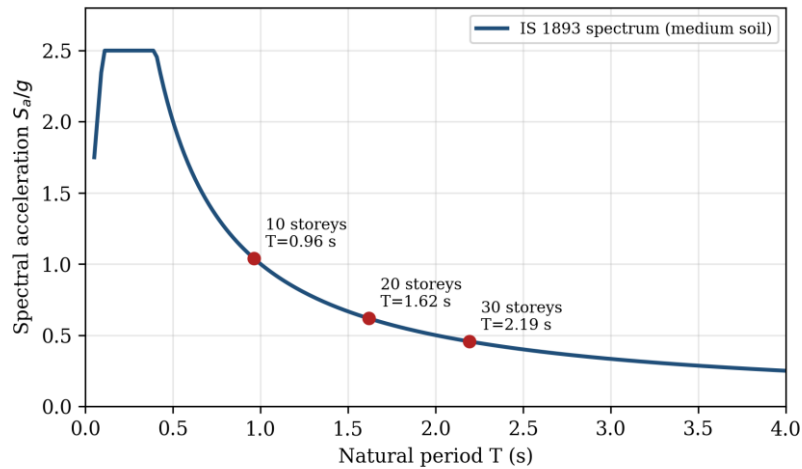


Figure 2: IS 1893 design response spectrum with the periods of the 10-, 20- and 30-storey models.

Figure 2 plots the design spectral acceleration coefficient S_a/g against natural period for medium soil. The curve has three branches: a linear rise up to 0.10 s, a plateau at 2.5 between 0.10 s and 0.40 s, and a hyperbolic decay of $1/T$ beyond 0.40 s. The markers show the estimated periods of the 10-, 20- and 30-storey models. The 10-storey building, with a period of 0.96 s, has a spectral coefficient of 1.04, whereas the 30-storey building, with a period of 2.19 s, falls to 0.46. This decay explains why seismic base shear grows slowly with height even though building mass increases proportionally.

4.3 Wind Demand Analysis

Table 3: Wind Profile Of The 30-Storey Model ($V_b = 39$ M/S, $C_f = 1.3$, Tributary Height 3 M)

Height z (m)	k2	Vz (m/s)	pz (Pa)	Floor wind force (kN)
10	1.000	39.0	913	85.4
15	1.084	42.3	1,073	100.5
30	1.246	48.6	1,416	132.6
45	1.351	52.7	1,666	155.9
60	1.431	55.8	1,869	174.9
75	1.496	58.4	2,043	191.2
90	1.552	60.5	2,198	205.7

Table 3 presents the wind profile of the 30-storey model at selected levels. The velocity V_z rises from 39 m/s at the reference height to 60.5 m/s at the roof, and the design pressure grows from 913 Pa to 2198 Pa. The floor wind force therefore increases from 85.5 kN near the base to 205.7 kN at the roof. The roof floor receives more than twice the force of the lower floors, so the upper storeys make the dominant contribution to overturning and displacement. This finding motivates the stiffness checks in Section 4.5.

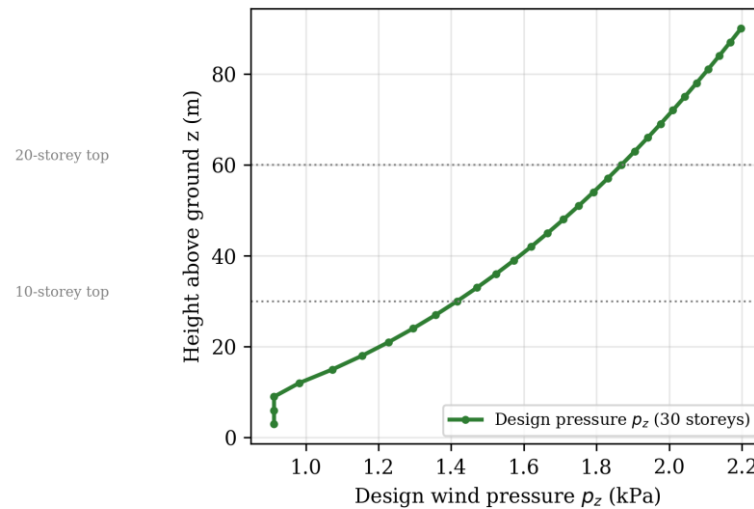


Figure 3: Design wind pressure variation with height for $V_b = 39$ m/s.

Figure 3 shows how the design wind pressure varies with height for the 30-storey building. Below 10 m the pressure is held at 913 Pa because the power-law factor is referenced to 10 m. Above this level the pressure increases smoothly to about 2.2 kPa at the roof, a rise of roughly 141 per cent. Most of the wind load therefore acts on the upper half of the building, where the lever arm for overturning is largest. The horizontal dotted lines mark the roof heights of the 10- and 20-storey models, showing that shorter buildings experience a truncated version of the same profile.

4.4 Comparison of Governing Lateral Loads

Table 4: Wind Versus Seismic Demand And The Governing Action

Storeys	Wind VB (kN)	Seismic VB III (kN)	Seismic VB V (kN)	Wind/Seismic III	Wind/Seismic V	Governing (III / V)	Wind Mo (kN m)	Seismic Mo, V (kN m)
10	1,052	1,150	2,588	0.91	0.41	Seismic / Seismic	18,824	61,008
15	1,788	1,273	2,864	1.40	0.62	Wind / Seismic	47,628	99,790
20	2,625	1,368	3,078	1.92	0.85	Wind / Seismic	92,983	141,886
25	3,550	1,446	3,255	2.45	1.09	Wind / Wind	156,864	186,657
30	4,550	1,514	3,406	3.01	1.34	Wind / Wind	240,971	233,695

Table 4 compares wind and seismic demands and identifies the governing action. In Zone III, seismic design governs at 10 storeys (ratio 0.91), whereas wind governs from 15 storeys (ratio 1.40) and reaches 3.01 at 30 storeys. In Zone V, the ratio rises from 0.41 at 10 storeys to 1.34 at 30 storeys and crosses unity between 20 and 25 storeys. Overturning moments diverge more sharply: the wind moment is 0.31 times the seismic moment at 10

storeys but 1.03 times at 30 storeys. The seismic moment is smaller partly because design seismic forces are reduced by $R = 5$, whereas wind forces are not reduced.

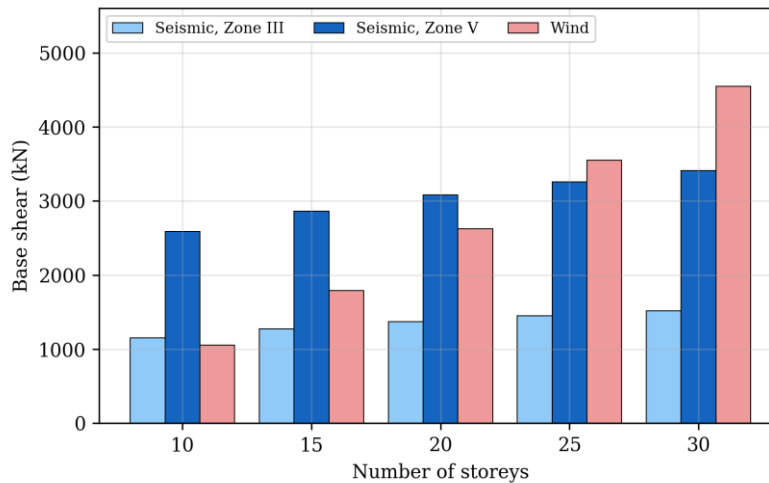


Figure 4: Seismic (Zones III and V) and wind base shear for 10 to 30 storeys.

Figure 4 compares seismic base shear for Zones III and V with wind base shear for the five building heights. For each height, the two seismic bars and the wind bar are shown side by side. At 10 storeys, the Zone III seismic demand of 1150 kN slightly exceeds the wind demand of 1052 kN, so seismic design governs. From 15 storeys onward, wind governs in Zone III. In the higher-hazard Zone V, seismic demand is greater at low heights, but wind demand overtakes it between 20 and 25 storeys. The figure therefore identifies a crossover height that depends on the seismic zone.

4.5 Serviceability Check

Table 5: Serviceability Check: Top Displacement And Storey Drift

Storeys	Wind top disp. (mm)	Limit H/500 (mm)	Utilisation (%)	Seismic top disp., Zone V (mm)	Max storey drift, wind	Max storey drift, seismic V	Status
10	0.4	60	0.7	1.4	0.00002	0.00007	Pass
15	2.2	90	2.5	5.2	0.00007	0.00016	Pass
20	7.8	120	6.5	13.1	0.00018	0.00030	Pass
25	20.4	150	13.6	26.8	0.00037	0.00050	Pass
30	45.1	180	25.1	48.2	0.00068	0.00075	Pass

Storey drift limit = 0.004 (IS 1893). All models satisfy both serviceability limits.

Table 5 summarises the serviceability results. Wind utilisation of the H/500 limit increases from 0.7 per cent at 10 storeys to 25.1 per cent at 30 storeys, a factor of roughly 36. Maximum storey drift ratios range from 0.00002 to 0.00068 under wind and from 0.00007 to 0.00075 under Zone V seismic forces, which stay below 19 per cent

of the 0.004 limit. Serviceability therefore does not govern this 24 m plan at the assumed stiffness, but the rapid growth of utilisation shows that the stiffness assumption must be verified in a detailed design.

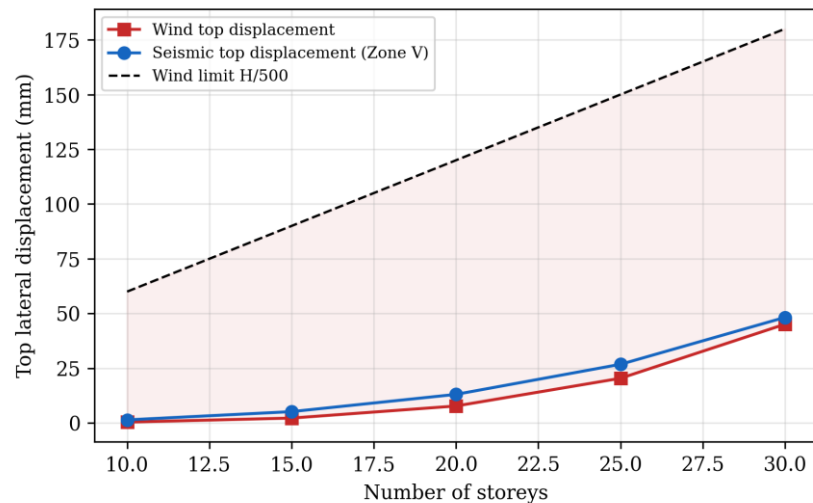


Figure 5: Top displacement under wind and Zone V seismic forces compared with the H/500 wind limit

Figure 5 compares the predicted top displacement under wind and Zone V seismic forces with the wind serviceability limit H/500. The wind displacement rises from 0.4 mm at 10 storeys to 45.1 mm at 30 storeys, while the allowable limit rises from 60 mm to 180 mm. The shaded band between the curve and the limit shows the remaining margin. The growth is nonlinear: displacement increases about 5.8 times between 20 and 30 storeys. At 30 storeys the Zone V seismic displacement of 48.2 mm slightly exceeds the wind displacement, so the two actions are comparable in serviceability terms for the tallest model.

5. DISCUSSION

The data reveal three main patterns. First, seismic demand is moderated by the lengthening period: A_h falls by 56 per cent in Zone III between 10 and 30 storeys, so the seismic base shear rises by only 32 per cent although the building weight triples. Second, wind demand rises steeply: the wind base shear increases 4.3 times (1052 kN to 4550 kN) and the overturning moment 12.8 times (18,824 to 240,971 kN m), because both the pressure profile and the lever arm increase with height. Third, the overturning comparison shows that wind moment exceeds the design seismic moment for a 30-storey tower, reflecting the reduction of seismic forces by $R = 5$ in the design, which is not applied to wind. These results indicate that the overturning check should precede the choice of detailing for towers above about 20 storeys.

The conclusions are sensitive to modelling choices. The cantilever idealisation neglects shear deformation of the frame and P-Delta effects, which typically increase lateral displacement in frame-dominated systems. The effective stiffness was assumed rather than derived from member sizing. Halving I_{eff} would double the top wind

displacement from 45.1 mm to about 90 mm at 30 storeys, which still satisfies the $H/500$ limit but would consume 50 per cent of the allowable displacement. A sensitivity check on the cracking factor, varied between 0.5 and 0.9 of I_g , would clarify this uncertainty and is recommended for future work. The empirical period $T_a = 0.075 h^{0.75}$ applies to bare frames and may differ from modal periods of infilled or core-wall buildings, and the power-law wind profile approximates the tabulated IS 875 factors. Because the study uses a single plan dimension and a fixed seismic weight, the results should be read as trends rather than as design values for a specific project.

The results are broadly consistent with earlier studies. Reviews of tall-building development [1] and wind engineering for super-tall buildings [13] emphasise that wind effects grow in importance with height and slenderness, which agrees with the crossover heights identified here. Taranath's integrated treatment of wind and seismic design [27] provides a framework in which such a crossover can be examined for moderate-seismicity regions. The seismic reduction effect observed here corresponds to the ductility-based philosophy of Paulay and Priestley [20] and Moehle [21]. In contrast with the displacement-based view of Priestley et al. [15], which emphasises target drift for seismic design, the present data show that wind displacement is the more demanding check in the moderate-hazard Zone III. Compared with the probabilistic wind framework of Kareem [25], the deterministic code approach used here does not capture dynamic amplification explicitly, and the absence of an occupant-comfort assessment [26] is a limitation relative to recent literature. Such agreement is useful because it suggests that qualitative conclusions from earlier work hold when expressed as numerical crossover heights for a specific plan and seismic zone. The comparison supports the view that the present dataset is a plausible screening tool, while detailed dynamic analysis remains necessary for final design.

6. CONCLUSION

This empirical study analysed seismic and wind demands on five RC high-rise buildings from 10 to 30 storeys, using code-based parametric data organised into five analytical tables. The data show that the governing lateral action depends on both height and seismic hazard. In Zone III, seismic design governs at 10 storeys, whereas wind governs from 15 storeys, reaching a wind-to-seismic base shear ratio of 3.01 at 30 storeys. In Zone V, seismic demand governs up to about 20 storeys, and wind takes over between 20 and 25 storeys, reaching 1.34 times the seismic base shear at 30 storeys. Wind overturning moments grow faster than seismic moments and exceed them at 30 storeys. Serviceability checks show that top wind displacement rises from 0.4 mm to 45.1 mm, 25 per cent of the $H/500$ limit, and that storey drift remains below 19 per cent of the 0.004 limit in all cases. These results support the abstract: the crossover heights and the rapid growth of wind demand define when a designer must treat wind as the controlling action.

Three recommendations follow. First, preliminary designs of RC towers above about 15 storeys should check wind base shear and overturning moment before finalising seismic detailing. Second, core stiffness should be verified against $H/500$ with a frame model that includes shear deformation. Third, occupant comfort should be checked for slender towers in wind-dominated regions. Future work should include time-history analysis,

nonlinear hinge modelling and comfort acceleration checks, and should extend the study to other plan aspect ratios and to a wider range of basic wind speeds.

7. REFERENCES

- [1] M. Ali and K. S. Moon, "Structural developments in tall buildings: Current trends and future prospects," *Archit. Sci. Rev.*, vol. 50, no. 3, pp. 205-223, 2007.
- [2] B. S. Taranath, *Reinforced Concrete Design of Tall Buildings*. Boca Raton, FL, USA: CRC Press, 2010.
- [3] *Plain and Reinforced Concrete: Code of Practice, IS 456:2000*, Bureau of Indian Standards, New Delhi, India, 2000.
- [4] B. S. Smith and A. Coull, *Tall Building Structures: Analysis and Design*. New York, NY, USA: Wiley, 1991.
- [5] *Criteria for Earthquake Resistant Design of Structures, Part 1: General Provisions and Buildings, IS 1893 (Part 1):2016*, Bureau of Indian Standards, New Delhi, India, 2016.
- [6] A. K. Chopra, *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, 5th ed. Boston, MA, USA: Pearson, 2017.
- [7] N. M. Newmark and E. Rosenblueth, *Fundamentals of Earthquake Engineering*. Englewood Cliffs, NJ, USA: Prentice-Hall, 1971.
- [8] *Code of Practice for Design Loads (Other than Earthquake) for Buildings and Structures, Part 3: Wind Loads, IS 875 (Part 3):2015*, Bureau of Indian Standards, New Delhi, India, 2015.
- [9] J. D. Holmes, *Wind Loading of Structures*, 3rd ed. Boca Raton, FL, USA: CRC Press, 2015.
- [10] A. G. Davenport, "The application of statistical concepts to the wind loading of structures," *Proc. Inst. Civ. Eng.*, vol. 19, no. 4, pp. 449-472, 1961.
- [11] *Minimum Design Loads and Associated Criteria for Buildings and Other Structures, ASCE/SEI 7-16*, American Society of Civil Engineers, Reston, VA, USA, 2017.
- [12] *Criteria for Structural Safety of Tall Concrete Buildings, IS 16700:2017*, Bureau of Indian Standards, New Delhi, India, 2017.
- [13] P. A. Irwin, "Wind engineering challenges of the new generation of super-tall buildings," *J. Wind Eng. Ind. Aerodyn.*, vol. 97, no. 7-8, pp. 328-334, 2009.
- [14] *National Building Code of India 2016, Part 6: Structural Design*, Bureau of Indian Standards, New Delhi, India, 2016.
- [15] M. J. N. Priestley, G. M. Calvi, and M. J. Kowalsky, *Displacement-Based Seismic Design of Structures*. Pavia, Italy: IUSS Press, 2007.
- [16] A. Gunel and H. Ilgin, "A proposal for the classification of structural systems of tall buildings," *Build. Environ.*, vol. 42, no. 7, pp. 2667-2675, 2007.

- [17] A. Kareem, T. Kijewski, and Y. Tamura, "Mitigation of motions of tall buildings with specific examples of recent applications," *Wind Struct.*, vol. 2, no. 3, pp. 201-251, 1999.
- [18] R. W. Clough and J. Penzien, *Dynamics of Structures*, 2nd ed. New York, NY, USA: McGraw-Hill, 1993.
- [19] E. L. Wilson, *Three-Dimensional Static and Dynamic Analysis of Structures*, 3rd ed. Berkeley, CA, USA: Computers and Structures, Inc., 2002.
- [20] T. Paulay and M. J. N. Priestley, *Seismic Design of Reinforced Concrete and Masonry Buildings*. New York, NY, USA: Wiley, 1992.
- [21] J. P. Moehle, *Seismic Design of Reinforced Concrete Buildings*. New York, NY, USA: McGraw-Hill, 2015.
- [22] *Ductile Design and Detailing of Reinforced Concrete Structures Subjected to Seismic Forces: Code of Practice*, IS 13920:2016, Bureau of Indian Standards, New Delhi, India, 2016.
- [23] J. B. Mander, M. J. N. Priestley, and R. Park, "Theoretical stress-strain model for confined concrete," *J. Struct. Eng.*, vol. 114, no. 8, pp. 1804-1826, Aug. 1988.
- [24] J. W. Wallace, "Behavior, design, and modeling of structural walls and coupling beams: Lessons from laboratory testing and seismic observations," *Int. J. Concr. Struct. Mater.*, vol. 6, no. 1, pp. 3-18, 2012.
- [25] A. Kareem, "Numerical simulation of wind effects: A probabilistic perspective," *J. Wind Eng. Ind. Aerodyn.*, vol. 96, no. 10-11, pp. 1472-1497, 2008.
- [26] K. C. S. Kwok, P. A. Hitchcock, and M. D. Burton, "Perception of vibration and occupant comfort in wind-excited tall buildings," *J. Wind Eng. Ind. Aerodyn.*, vol. 97, no. 7-8, pp. 368-380, 2009.
- [27] B. S. Taranath, *Wind and Earthquake Resistant Buildings: Structural Analysis and Design*. New York, NY, USA: Marcel Dekker, 2005.
- [28] *Design of Structures for Earthquake Resistance, Part 1: General Rules, Seismic Actions and Rules for Buildings*, EN 1998-1:2004, European Committee for Standardization (CEN), Brussels, Belgium, 2004.
- [29] *Building Code Requirements for Structural Concrete*, ACI 318-19, American Concrete Institute, Farmington Hills, MI, USA, 2019.
- [30] F. Naeim, Ed., *The Seismic Design Handbook*, 2nd ed. Norwell, MA, USA: Kluwer Academic Publishers, 2001.