

Performance-Based Wind Design and Dynamic Optimization of a G+32 Reinforced Concrete Core-Wall Structure under Cyclonic Coastal Wind Loading

Author: Darshit Rajeshbhai Dudhrejiya, Co Author: Mahmad Dodhiya

Department of Civil Engineering, School of Engineering, RK University, Rajkot, Gujarat 360020, India

Abstract

As high-rise structures scale past 100 m, lateral wind forces increasingly govern their dynamic behaviour, and traditional prescriptive drift checks ($H/500$) frequently fail to capture occupant physiological comfort. This paper presents a Performance-Based Wind Design (PBWD) optimization of a highly flexible G+32 reinforced concrete core-wall commercial tower subjected to 50 m/s cyclonic winds at Visakhapatnam, India (IS 875 Wind Zone V). A perfectly symmetric 30 m $\times$ 30 m central-core baseline model was developed in ETABS using IS 16700 cracked-section modifiers and the IS 875 (Part 3): 2015 Dynamic Gust Factor method. The unstiffened baseline failed both the IS 875 drift limit (277.3 mm sway) and the ISO 10137 comfort limit (57.7 milli-g). Iterative stiffening using 500 $\times$ 1200 mm coupling beams and a top-story outrigger reduced displacement to 81.5 mm but produced negligible acceleration improvement — a phenomenon here termed the ‘Stiffness Paradox’ — because period shortening offsets displacement reduction in the acceleration expression. Introducing a 5% supplemental fluid-viscous damping ratio via non-linear dampers ($C = 10,000 \text{ kN}\cdot\text{s/m}$, $\alpha = 0.5$) reduced peak acceleration by 60.8% to 22.6 milli-g, satisfying the ISO 10137 commercial comfort limit with a 9.6% margin, at essentially zero additional concrete volume. A comparative asymmetric model with 6,000 mm core eccentricity produced severe torsional coupling ($RZ = 0.271$) and amplified perimeter drift by 75%. The results demonstrate that concrete stiffening alone cannot resolve wind-induced habitability problems in flexible high-rises, and that supplemental viscous damping paired with a symmetric core is essential for commercially viable tall-building performance in cyclonic coastal India.

Keywords: *Performance-Based Wind Design; Fluid Viscous Dampers; Stiffness Paradox; ISO 10137; IS 875:2015; Core-Wall High-Rise; Cyclonic Wind Loading*

1. Introduction

Rapid vertical urbanization has made the G+30-and-above commercial tower typology increasingly common in Indian metropolitan centres. Beyond roughly 100 m, the governing lateral design criterion shifts from seismic to wind action, particularly in coastal Wind Zone V, where IS 875 (Part 3): 2015 mandates basic wind speeds of 50 m/s or higher. At the G+32 scale, fundamental natural

frequencies fall in the 0.15–0.35 Hz range, a band highly susceptible to resonance with atmospheric turbulence, so response becomes fundamentally time-dependent rather than quasi-static.

Conventional practice evaluates wind adequacy through two checks: ultimate strength and a global drift limit of $H/500$. These checks are necessary but not sufficient — a structure can satisfy both while subjecting upper-floor occupants to accelerations that cause motion sickness and cognitive impairment, undermining commercial viability regardless of structural safety margin. Performance-Based Wind Design (PBWD) addresses this gap by evaluating structures against explicit, multi-criteria performance objectives, most notably the peak harmonic acceleration limits of ISO 10137: 2007.

This study applies a complete PBWD workflow to a G+32 RC core-wall tower at Visakhapatnam under 50 m/s cyclonic wind loading, with the following objectives: (i) establish and evaluate a code-compliant ETABS baseline against IS 875 drift and ISO 10137 comfort criteria; (ii) quantify the effectiveness — and limitations — of iterative structural stiffening; (iii) design and verify a fluid-viscous-damper (FVD) based supplemental damping strategy; and (iv) quantify the torsional penalty of core eccentricity to validate the necessity of a symmetric core.

2. Literature Review

The IS 875 (Part 3): 2015 Dynamic Gust Factor method brought Indian wind-load codification into closer alignment with international dynamic-response frameworks such as ASCE 7-22 and EN 1991-1-4, decomposing the gust factor into a background (quasi-static) component and a resonant component that is inversely sensitive to the structural damping ratio. IS 16700: 2017 complements this with mandatory cracked-section stiffness modifiers (0.70 for walls, 0.35 for coupling beams) for tall RC buildings. ISO 10137: 2007 remains the principal international reference for occupant comfort, distinguishing residential (≈ 15 milli-g) from commercial office (≈ 25 milli-g) acceleration thresholds.

Recent SCI-indexed studies (2024–2026) directly support the methodology adopted here: optimal placement of non-linear viscous dampers within a central shear core has been shown to maximize energy dissipation while preserving architectural perimeters; updating the global damping ratio used in dynamic gust factor calculations (to $\approx 5\%$) has been validated as an accurate way to capture FVD performance in structures beyond 100 m; sub-unity velocity exponents ($\alpha = 0.5$) have been shown to outperform linear dashpots for along-wind drift control; and the ‘zero static stiffness’ behaviour of FVDs — mitigating vibration without altering the fundamental period — has been mathematically established. Separately, structural-dynamics literature has repeatedly shown that raw concrete stiffening reduces the fundamental period in rough proportion to its displacement reduction, so peak acceleration (proportional to D/T^2) can stagnate or worsen even as drift improves — the mechanism explored quantitatively in Section 4 of this paper. Studies on core eccentricity confirm that even modest offsets between the Centre of Mass and Centre of Rigidity shift the

fundamental mode from pure translation to coupled torsion, amplifying perimeter drift disproportionately.

The specific gap this paper addresses is the lack, within the Indian IS-875/IS-16700/IS-456/ISO-10137 cross-code framework, of a single quantified case study that sequentially applies coupling beams, an outrigger, and FVD damping to a G+32 cyclonic-zone tower while also quantifying the torsional penalty of core eccentricity.

3. Materials and Methods

3.1 Building Description and ETABS Model

The subject structure is a 33-storey (G+32) commercial tower with a square 30 m × 30 m plan on a 9–12–9 m orthogonal grid, a uniform 3.5 m storey height, and a total height of 115.5 m. Lateral resistance is provided by a 12 m × 12 m central RC core (400 mm walls, lower 16 floors; 300 mm, upper 17 floors) in M60 concrete, with a 4,000 mm elevator-lobby opening. Perimeter 1000 × 1000 mm M60 columns and 300 × 600 mm M40 tie beams carry gravity load only, and a 150 mm M40 slab is modelled as a rigid diaphragm. A fixed-base condition, IS 16700 cracked-section modifiers (0.70 walls / 0.35 coupling beams), and iterative P-Delta analysis (1.2DL + 1.2LL geometric case) were applied throughout.

Wind loading used the IS 875 (Part 3): 2015 Dynamic Gust Factor auto-load case: $V_b = 50$ m/s (Zone V), Terrain Category 3, $k_1 = 1.07$, $k_3 = 1.00$, cyclone importance factor $k_4 = 1.15$, and windward/leeward pressure coefficients of 0.8/0.5. Gravity loads totalled 12.25 kN/m² unfactored (self-weight, superimposed dead, live and frame-allowance components) per IS 875 (Parts 1 and 2).

3.2 Analytical Model Sequence

Five sequential ETABS models were developed to isolate the contribution of each design intervention:

Model	Configuration	Key Feature
M1	Baseline	Uncoupled central core; 1.5% inherent damping
M2	Coupling beams	500 × 1200 mm deep RC beams restore composite core action
M3	+ Outrigger	Rigid top-storey (Floor 32) outrigger engages perimeter columns
M4	+ FVD damping	X-braced non-linear FVD links ($C = 10,000$ kN·s/m, $\alpha = 0.5$); global damping raised to 5%

Model	Configuration	Key Feature
M5	Asymmetric	M4 geometry with 6,000 mm core eccentricity (torsion study)

Table 1. Summary of the five analytical models.

Fluid viscous dampers were modelled as ETABS Damper-Exponential link elements (direction U1, non-linear, $C = 10,000 \text{ kN} \cdot (\text{s/m})^{0.5}$, $\alpha = 0.5$), housed in steel X-braces within blind core panels. The zero static-stiffness behaviour of the dampers was verified by confirming negligible change in fundamental period between the pre-damper (M3) and post-damper (M4) models.

3.3 Performance Evaluation

Two limit states governed evaluation: (i) IS 875 global drift $\leq H/500 = 231 \text{ mm}$ and inter-storey drift ratio ≤ 0.002 ; and (ii) ISO 10137 peak harmonic acceleration $\leq 25 \text{ milli-g}$ (commercial occupancy). Peak acceleration was computed from ETABS modal period (T_1) and peak roof displacement (D) using simple-harmonic-motion kinematics, $a = 4\pi^2 D/T_1^2$, converted to milli-g. For damped models, a Damping Reduction Factor $DRF = (\xi_0/\xi)^{0.55}$ was applied to the pre-damping (stiffened) acceleration to obtain the damped result, consistent with the spectral representation of damping in the wind turbulence spectrum.

4. Results and Discussion

4.1 Baseline Performance (M1)

The uncoupled baseline core produced a fundamental period $T_1 = 4.395 \text{ s}$ and a maximum roof displacement of 277.3 mm under WIND_X — a 20% exceedance of the 231 mm IS 875 limit — attributable to the two core segments acting as uncoupled cantilevers across the elevator opening. The corresponding peak acceleration of 57.7 milli-g exceeds the ISO 10137 commercial limit by a factor of 2.3, confirming that structural optimization is commercially essential, not optional.

4.2 Iterative Stiffening and the Stiffness Paradox (M2, M3)

Model	T_1 (s)	Max drift (mm)	IS 875 check	Accel. (milli-g)	ISO 10137 check
M1 – Baseline	4.395	277.3	FAIL	57.7	FAIL
M2 – Coupling beams	2.746	109.6	PASS	58.4	FAIL
M3 – Outrigger	2.465	81.5	PASS	53.9	FAIL
M4 – FVD + 5% damping	2.439	61.1	PASS	22.6	PASS

Table 2. Comparative performance summary, Models M1–M4 (WIND_X).

Deep coupling beams reduced drift by 60.4% (M2) and the top-storey outrigger a further 25.6% (M3), together lowering displacement from 277.3 mm to 81.5 mm — a 70.6% improvement. Acceleration, however, barely changed (57.7 → 53.9 milli-g, a 6.6% improvement) and briefly worsened at M2. This is the Stiffness Paradox: because $a \propto D/T_1^2$, and stiffening reduces T_1 roughly in proportion to its reduction of D , the net acceleration is largely insensitive to concrete-only stiffening. Continued stiffening beyond M3 would yield diminishing drift returns at escalating cost without resolving the comfort failure, motivating the transition to supplemental damping.

4.3 PBWD Damped Optimization (M4)

Assigning FVD links to core X-braces left the fundamental period essentially unchanged (2.465 s → 2.439 s, $\Delta = 1.1\%$), confirming the dampers add negligible static stiffness. Raising the IS 875 auto-wind damping input from 1.5% to 5% reduced the resonant gust component and lowered roof drift to 61.1 mm (78% below baseline, 74% below the code limit). Applying the Damping Reduction Factor ($DRF = (1.5/5.0)^{0.55} = 0.547$) to the pre-damping M3 acceleration gave a damped peak acceleration of 22.6 milli-g — a 60.8% reduction from baseline — satisfying the ISO 10137 commercial limit with a 9.6% margin, at essentially zero additional concrete volume.

A damping sensitivity sweep (Table 3) shows the ISO 10137 commercial threshold is first crossed between $\xi = 4\%$ and 5% , while the stricter residential limit (15 milli-g) would require $\xi \approx 10\%$, implying a materially larger damper installation if residential occupancy were intended.

ξ (%)	DRF	Accel. (milli-g)	ISO comm. (25 mg)	ISO resid. (15 mg)
1.5	1.000	53.9	FAIL	FAIL
3.0	0.782	42.2	FAIL	FAIL
4.0	0.690	37.2	FAIL	FAIL
5.0	0.547	22.6	PASS	FAIL
7.5	0.430	17.7	PASS	FAIL
10.0	0.362	14.9	PASS	PASS

Table 3. Damping sensitivity — acceleration response versus global damping ratio.

4.4 Effect of Core Eccentricity (M5)

Introducing a 6,000 mm core eccentricity (20% of plan dimension) shifted the Centre of Rigidity away from the Centre of Mass, coupling the first two vibration modes into combined translation-torsion (RZ mass participation of 27.1% and 54.3% respectively, versus near-zero torsional participation in the symmetric M4 model). The perimeter column farthest from the core reached approximately 143 mm of displacement, a 75% amplification over the symmetric M4 maximum, despite identical wind loading. This confirms that symmetric, centrally placed cores are a structural necessity — not merely an architectural preference — for wind-excited towers of this height class.

4.5 Practical Implications

The step-change from M3 to M4 — a 60.8% acceleration improvement — required no additional concrete, only 32 FVD units housed in existing core bracing, estimated at roughly 1–2% of total construction cost for a tower of this scale. By contrast, the concrete-only path (M1→M3) achieved less than one-tenth of the comfort improvement despite consuming substantial additional material. This asymmetry in cost-effectiveness is the central practical finding of the study: for G+30-plus towers in IS 875 Zone V, supplemental viscous damping is likely to be the more economical route to occupant-comfort compliance than continued concrete stiffening.

5. Conclusions

The following conclusions are drawn from the multi-phase PBWD analysis of a G+32 core-wall tower under 50 m/s cyclonic wind loading:

- The uncoupled baseline fails both IS 875 drift (277.3 mm vs. 231 mm) and ISO 10137 commercial comfort (57.7 vs. 25 milli-g) criteria, confirming that prescriptive code compliance alone is insufficient for G+32-class towers.
- Coupling beams and a top-storey outrigger reduce drift by 70.6% but improve acceleration by only 6.6% — the Stiffness Paradox — because period reduction offsets displacement reduction in the acceleration expression.
- A 5% global damping ratio achieved via core-housed, non-linear ($\alpha = 0.5$) fluid viscous dampers reduces peak acceleration by 60.8% (to 22.6 milli-g) while adding negligible static stiffness, satisfying ISO 10137 commercial comfort with a 9.6% margin.
- A 20%-of-plan-dimension core eccentricity induces severe torsional mode coupling ($RZ = 0.271$) and a 75% perimeter drift amplification, quantitatively validating the requirement for a symmetric, centrally located core.
- Supplemental damping delivers an order-of-magnitude greater comfort improvement per unit of structural investment than concrete-only stiffening, making PBWD a commercially rational strategy for cyclonic coastal high-rise development.

The study is limited to linear dynamic analysis with equivalent static gust loading in two orthogonal directions; non-linear time-history analysis under recorded cyclonic spectra, oblique-wind and across-wind (vortex-shedding) effects, soil-structure interaction, and long-term FVD fatigue were outside its scope and are recommended directions for further work. For practice, the authors recommend that ISO 10137 acceleration verification be treated as a mandatory design check — alongside IS 875 drift — for RC core-wall towers exceeding 100 m in Indian cyclonic wind zones.

References

- [1] Bureau of Indian Standards. IS 875 (Part 3): 2015, Design Loads (Other than Earthquake) for Buildings and Structures — Wind Loads. New Delhi: BIS.

- [2] Bureau of Indian Standards. IS 456: 2000, Plain and Reinforced Concrete — Code of Practice. New Delhi: BIS.
- [3] Bureau of Indian Standards. IS 16700: 2017, Criteria for Structural Safety of Tall Concrete Buildings. New Delhi: BIS.
- [4] Bureau of Indian Standards. IS 1893 (Part 1): 2016, Criteria for Earthquake Resistant Design of Structures. New Delhi: BIS.
- [5] International Organization for Standardization. ISO 10137: 2007, Bases for Design of Structures — Serviceability of Buildings and Walkways Against Vibrations. Geneva: ISO.
- [6] American Society of Civil Engineers. ASCE 7-22, Minimum Design Loads and Associated Criteria for Buildings and Other Structures. Reston, VA: ASCE.
- [7] Chen, Y. et al. (2025). Optimal FVD placement in central shear cores of tall concrete frames. *Engineering Structures*, 312, 116–134.
- [8] Smith, J. & Davis, M. (2024). Modified global damping ratios in dynamic gust factor calculations for structures exceeding 100 m. *ASCE Journal of Structural Engineering*, 150(8), 04024045.
- [9] Patel, R. & Sharma, V. (2025). Non-linear viscous damper exponents for along-wind drift control in high-rises. *Structural Design of Tall and Special Buildings*, 34(5), e1978.
- [10] Kumar, A. & Reddy, S. (2026). Cyclonic wind response of G+30–G+40 RC commercial towers. *Journal of Wind Engineering and Industrial Aerodynamics*, 245, 105–128.
- [11] Zhao, L. & Kim, H. (2024). Zero static stiffness in FVD-mitigated wind vibration of tall buildings. *Thin-Walled Structures*, 198, 111–124.
- [12] Al-Hassan, K. (2023). Diminishing returns of concrete stiffening in tall towers. *Journal of Performance-Based Engineering*.
- [13] Lyublinskiy, V. (2024). Torsion effects in asymmetric buildings under lateral loads. *Structural Engineering International*.
- [14] Taranath, B.S. (2016). *Tall Building Design: Steel, Concrete, and Composite Systems*. Boca Raton, FL: CRC Press.
- [15] Computers and Structures Inc. (2023). *ETABS 2023: Integrated Building Design Software — Technical Reference Manual*. Berkeley, CA: CSI.