International Journal of Creative and Open Research in Engineering and Management
ISSN: **3108-1754 (Online)**
Volume 02 Issue 06 June-2026 | Impact Factor: 3.5

Investigation on Structural Performance of Multi-Storeyed Building using Nonlinear Analysis

M. Soniya[1], ***Dr.* P. Poluraju**[2]

[1]M. Tech Scholar, Department of Civil Engineering, MGIT, Hyderabad, India
[2]Professor, Department of Civil Engineering, MGIT, Hyderabad, India

Corresponding Author Email: vymsoniya@gmail.com

How to Cite this Article:

Soniya, M. (2026). Investigation on Structural Performance of Multi-Storeyed Building using Nonlinear Analysis. International Journal of Creative and Open Research in Engineering and Management, <i>02</i>(6). https://doi.org/10.55041/ijcope.v2i6.109

License:

This article is published under the terms of the Creative Commons Attribution 4.0 International License (CC BY 4.0), which permits unrestricted use, distribution, and reproduction in any medium, provided the original author(s) and the source are credited.

© The Author(s). Published by International Journal of Creative and Open Research in Engineering and Management.

https://doi.org/10.55041/ijcope.v2i6.109

Abstract

This paper presents a comprehensive seismic performance evaluation of a G+15 reinforced concrete (RC) moment-resisting frame building located in Seismic Zone V of India, employing both Nonlinear Static (Pushover) Analysis and Nonlinear Dynamic Response Spectrum Analysis (RSA). The building, modelled in ETABS as a three-dimensional space frame, is designed as per IS 456:2000, IS 875 (Parts 1 & 2), and IS 1893:2016, with M30 concrete and Fe415/Fe345 steel reinforcement. Pushover analysis was performed in both the EQ-X and EQ-Y directions; the resulting capacity curves exhibited nearly linear elastic behavior throughout, with a maximum base shear of approximately 144,452 kN (X-direction) and global lateral stiffness values of 80.3 kN/mm (X) and 8,762 kN/mm (Y). The FEMA 440 Acceleration-Displacement Response Spectrum (ADRS) evaluation did not yield a performance point in either direction (ductility ratio $\mu = 1.0$), confirming elastic structural response. Response spectrum analysis produced maximum roof displacements of 52 mm (RSX) and 118 mm (RSY) at Story 15, with maximum inter-storey drift ratios of 4×10^{-6} and 8×10^{-6} — approximately 500 times below the IS 1893:2016 permissible limit of 0.004. The results collectively demonstrate that the structure meets all performance-based design objectives as per FEMA 356 and ATC-40 guidelines, with substantial safety reserves against seismic demand. The combined application of both analytical methods is confirmed as a robust framework for comprehensive seismic performance assessment of tall RC buildings in high seismic zones.

Keywords— Nonlinear Static Pushover Analysis; Response Spectrum Analysis; ETABS; Seismic Performance; Inter-Storey Drift; Base Shear; Plastic Hinge; FEMA 440; Performance-Based Design; IS 1893:2016

I. INTRODUCTION

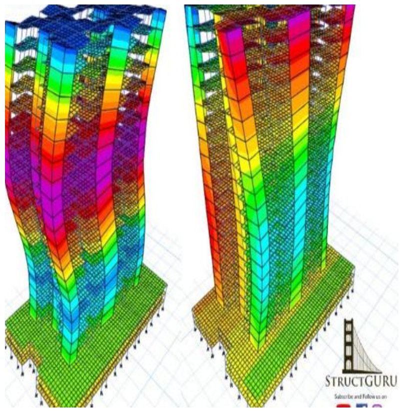
The rapid pace of urbanisation across India has driven unprecedented demand for vertical construction. Multi-storey reinforced concrete (RC) moment-resisting frames have become the structural system of choice for residential, commercial, and institutional occupancies in urban areas. As buildings grow taller, the influence of lateral forces — particularly those induced by seismic events — becomes increasingly dominant in the structural design process. The Indian subcontinent is among the world's most seismically active regions; approximately 59% of its landmass is classified as moderate-to-very-high seismic hazard under IS 1893:2016, placing a significant responsibility on structural engineers to develop safe and economical designs [1].

Traditional linear elastic analysis methods, such as the Equivalent Static Method and the linear Response Spectrum Method, assume that structural members remain within the elastic range under design loads and employ force-based acceptance criteria. However, during a major earthquake, structures inevitably experience inelastic deformations beyond the elastic limit. Linear analysis cannot capture the redistribution of forces as members yield, the progressive degradation of stiffness, the sequence of plastic hinge formation, or the ductility demands on individual members [4].

In response to these limitations, performance-based earthquake engineering (PBEE) has emerged as the modern framework for seismic design and assessment. PBEE shifts the focus from strength-based compliance to quantifying and controlling the actual performance of a building across multiple hazard levels. Nonlinear analysis is central to PBEE: it provides the means to model inelastic behavior explicitly and to predict structural response at levels of demand that may exceed the elastic range [5,6].

Two principal methods of nonlinear analysis are in widespread use for seismic assessment of buildings. Nonlinear Static (Pushover) Analysis involves the monotonic application of a laterally increasing load pattern until a target displacement is reached, yielding the capacity curve — a plot of base shear versus roof displacement — which provides direct insight into ductility, overstrength, failure mechanisms, and plastic hinge distribution. Nonlinear Dynamic Response Spectrum Analysis (RSA), based on modal superposition, captures higher-mode effects and provides realistic estimates of storey displacements, inter-storey drifts, and force distributions across the building height [3,7].

The present study employs both methods in a comparative framework for a G+15 RC moment-resisting frame building in Seismic Zone V of India. The building is modelled in ETABS, analyzed per IS 456:2000 and IS 1893:2016, and evaluated against performance criteria from FEMA 356, ATC-40, and FEMA 440. The combined application of pushover and RSA provides a holistic seismic performance picture that neither method alone can deliver.



3D ETABS Model — Building Layout

Fig. 1. Three-Dimensional ETABS Model of the G+15 RC Building

II. LITERATURE REVIEW

The investigation of structural performance of multi-storey buildings under seismic loading has attracted substantial research interest over the past three decades. The concept of pushover analysis was formalised in the 1970s and 1980s as a practical alternative to computationally expensive nonlinear time-history analysis. Saiidi and Sozen (1981) demonstrated that a simplified single-degree-of-freedom (SDOF) representation could adequately capture inelastic displacement demand, forming the theoretical backbone of the Capacity Spectrum Method later codified in ATC-40 [8]. Krawinkler and Seneviratna (1998) established the conditions under which pushover analysis provides accurate seismic demand estimates, confirming that for regular, first-mode-dominated structures — the class studied here — conventional pushover analysis is a reliable assessment tool [4].

ATC-40 (1996) introduced the Capacity Spectrum Method (CSM), in which the building's pushover capacity curve is converted to ADRS format and intersected with the seismic demand spectrum to determine the performance point, with performance levels of Immediate Occupancy (IO), Life Safety (LS), and Structural Stability (SS) [5]. FEMA 356 (2000) extended these concepts by introducing the Displacement Coefficient Method (DCM) and providing tabulated plastic hinge acceptance criteria for RC beams and columns, which are adopted as default hinge properties in ETABS [6]. FEMA

© 2026 The Author(s). Published by IJCOPE Journal. Website: https://ijcope.org/

440 (2005) proposed an improved equivalent linearisation procedure using period-dependent effective damping and stiffness [7] — the method employed in the present study.

Sivakumar et al. (2013) demonstrated that buildings with a soft ground storey exhibit inherently higher vulnerability, with significantly increased drift demands and column force demands, underscoring the importance of nonlinear analysis in identifying weak storeys [9]. Ranjit et al. (2015) showed that shifting a soft storey to higher floors increases base shear and displacement, with natural period varying from 2.571 s (soft storey at 4th floor) to 2.366 s (16th floor) [10]. Junwon et al. (2015) performed a seismic performance evaluation of a 12-storey RC moment-resisting frame with shear walls using RSA and nonlinear time-history analysis, finding that inter-storey drift ratios generally satisfied codified limits and that the combined use of RSA and nonlinear static methods was a robust evaluation strategy [11]. Mehmet Inel and Hayri Baytan Ozmen (2006) investigated the sensitivity of pushover results to plastic hinge properties in ETABS and concluded that default FEMA 356 hinge properties are adequate for design-level assessment of regular RC frames [12]. Mwafy and Elnashai (2001) found that pushover analysis generally gives conservative estimates of building capacity compared to nonlinear dynamic collapse analysis, validating its use as a reliable screening tool [13].

A review of existing literature identifies several specific gaps that the present study addresses: (i) most pushover studies focus on buildings of 10 storeys or fewer, with fewer studies on G+15 RC frames under Indian seismic Zone V conditions; (ii) studies comparing pushover and RSA on the same ETABS model for Zone V ($Z = 0.36$) with IS 1893:2016 are limited; (iii) the effect of a purely elastic pushover response ($\mu = 1.0$) in the context of IS code compliance and FEMA 440 evaluation has not been extensively discussed for tall, stiff frames.

Table 1. Summary of Key Literature Reviewed

Author(s) & Year	**Focus Area**	**Key Findings & Relevance**
Krawinkler & Seneviratna (1998)	Pros and cons of pushover analysis	Pushover adequate for first-mode-dominated regular structures; higher-mode methods needed for irregular buildings.
ATC-40 (1996)	Capacity Spectrum Method (CSM)	Introduced ADRS-based performance point evaluation; defined IO, LS, SS performance levels.
FEMA 356 (2000)	Prestandard for seismic rehabilitation	Tabulated plastic hinge acceptance criteria for RC members; performance levels IO, LS, CP.
FEMA 440 (2005)	Improvement of nonlinear static procedures	Improved equivalent linearisation method; more accurate performance point determination — used in this study.
Sivakumar et al. (2013)	Soft storey seismic vulnerability	Soft ground storey increases drift and column demand; highlights need for nonlinear analysis.
Junwon et al. (2015)	12-storey RC MRF with shear walls	Inter-storey drift satisfied codified limits; RSA + nonlinear static combination is robust.
Inel & Ozmen (2006)	Plastic hinge properties in ETABS	Default FEMA 356 hinges adequate for design-level assessment of regular RC frames.
Mwafy & Elnashai (2001)	Pushover vs. dynamic collapse	Pushover gives conservative capacity estimates; valid as a screening tool for regular structures.

III. METHODOLOGY

3.1 Structural Model Description

The building considered in this study is a G+15 storey regular RC moment-resisting frame. The plan configuration is a symmetric 4-bay × 4-bay grid with uniform bay widths of 5 m in both the X- and Y-directions, yielding total plan dimensions of 15 m × 15 m. Each storey has a uniform height of 3 m, giving a total building height of 45 m. This regular plan geometry was chosen to avoid torsional irregularities and to ensure first-mode-dominated response, consistent with the assumptions inherent in conventional pushover analysis. Columns are arranged at every grid intersection (16 columns per floor), and beams span between adjacent column nodes in both principal directions.

The structural materials adopted are defined in accordance with relevant Indian Standards. Concrete grade M30 is used for all structural members, with characteristic compressive strength f_{ck} = 30 N/mm² and modulus of elasticity $E_c = 5000\sqrt{f_{ck}} \approx 27{,}386$ N/mm² as per IS 456:2000. HYSD steel bars of grade Fe415 (f_y = 415 N/mm²) are used as longitudinal reinforcement for beams and columns, while Fe345 (f_y = 345 N/mm²) bars are adopted for confinement reinforcement. The foundation is modelled as fully fixed supports at the base of all ground-floor columns, restraining all six degrees of freedom — a conservative boundary condition appropriate for well-designed raft or deep pile foundations in Zone V construction. A rigid diaphragm (Diaphragm D1) is assigned to each storey level. The structural parameters are summarised in Table 2.

Table 2. Summary of Structural Model Parameters

Parameter	Value / Description
Building Type	RC Moment-Resisting Frame (MRF)
Number of Storeys	G + 15 (Ground + 15 upper floors)
Plan Configuration	Regular symmetric — 4 bays × 4 bays
Bay Width (X & Y)	5 m × 5 m (uniform)
Total Plan Dimension	15 m × 15 m
Typical Storey Height	3 m
Total Building Height	45 m (15 storeys × 3 m)
Concrete Grade	M30 (f_{ck} = 30 N/mm²)
Longitudinal Reinforcement	Fe415 (HYSD415, f_y = 415 N/mm²)
Confinement / Stirrups	Fe345 (HYSD345, f_y = 345 N/mm²)
Beam Section	450 mm × 650 mm (B0.45 × 0.65)

Parameter	Value / Description
Column Section	800 mm × 500 mm (C0.8 × 0.5)
Slab Thickness	150 mm (rigid diaphragm)
Foundation Condition	Fixed supports at base
Analysis Software	ETABS (CSI Berkeley)

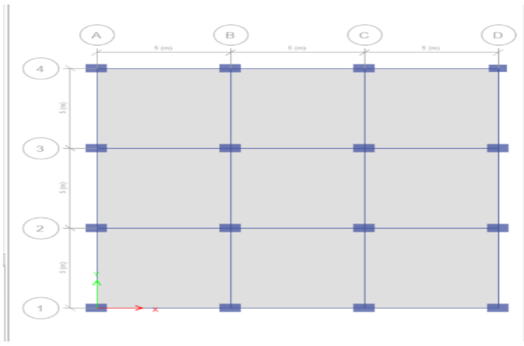


Fig. 2. Ground Floor Plan Layout (4×4 grid, 5 m bay spacing)

3.2 Loading

Gravity loads comprise the self-weight of all structural members (automatically computed by ETABS based on member dimensions and material density) plus superimposed dead load (SDL) of 1.5 kN/m² on all floor slabs in accordance with IS 875 Part 1. Live load (LL) is applied as 2 kN/m² on typical floors and 1.5 kN/m² on the roof slab per IS 875 Part 2. Wind loads in X and Y directions are applied as equivalent static lateral loads per IS 875 Part 3. Seismic loads are defined using the IS 1893:2016 Response Spectrum function, with parameters as listed in Table 3.

Table 3. Seismic Parameters as per IS 1893:2016

Seismic Parameter	Value
Seismic Zone	Zone V
Zone Factor (Z)	0.36
Importance Factor (I)	1.0
Soil Type	Type II (Medium Soil)
Response Reduction Factor (R)	5
Damping Ratio	5%
Design Spectrum	IS 1893:2016 (Part 1)
Modal Combination Method	CQC (Complete Quadratic Combination)
Directional Combination	SRSS

All load combinations are defined per IS 456:2000 and IS 1893:2016. The governing seismic combinations include 1.2(DL + LL ± EQX/EQY), 1.5(DL ± EQX/EQY), and 0.9DL ± 1.5EQX/EQY, covering all critical loading scenarios for Zone V design.

3.3 Nonlinear Static (Pushover) Analysis

Pushover analysis involves the application of a monotonically increasing lateral load pattern, distributed proportionally to the product of storey mass and first-mode shape amplitude, until a predetermined target displacement is reached or the structure reaches its ultimate capacity. The primary output is the capacity curve: a plot of total base shear (V) versus roof displacement (Δ). This is converted to the Acceleration-Displacement Response Spectrum (ADRS) format (Sa–Sd space) by dividing by the modal participation factor (Γ) and modal mass coefficient (m*): $Sd = \Delta/\Gamma$ and $Sa = V/(m^*\cdot g)$.

Plastic hinge (P-M2-M3) assignments follow default FEMA 356 (ASCE 41) properties in ETABS: flexural hinges (M3) for beams at 10% of member length from each end; axial-flexural interaction hinges (P-M2-M3) for columns at both ends. The nonlinear force-deformation backbone follows the FEMA multi-linear curve (A–B–C–D–E), with performance levels mapped as IO between B and C, LS between C and D, and CP between D and E.

Two pushover cases are defined: Push X (EQ-X direction) and Push Y (EQ-Y direction), both with an initial gravity load of 1.0DL + 0.25LL before lateral pushing begins. P-Δ (geometric nonlinearity) effects are activated. The monitored displacement is the roof displacement at Story 15, and the analysis is run for a maximum of 1,000 steps or until 1,800 mm of roof displacement is reached. Performance point determination uses the FEMA 440 Equivalent Linearisation method.

3.4 Nonlinear Dynamic Response Spectrum Analysis

Response Spectrum Analysis (RSA) is a frequency-domain dynamic analysis method that determines the maximum structural response by superposing the maximum responses of each mode of vibration, weighted by their modal participation factors. For mode i with circular frequency ωi, the maximum modal displacement is: $q_{i,max} = \Gamma i \cdot \varphi i \cdot Sa(Ti, \zeta)/\omega i^2$, where Γi is the modal participation factor and φi is the mode shape vector. Individual modal maxima are combined using the CQC method, which accounts for statistical correlation between closely spaced modes. Directional combination is performed using the SRSS method, as required by IS 1893:2016 Clause 7.8.4.

Modal analysis is performed to extract natural frequencies and mode shapes, with sufficient modes to capture at least 90% of the total seismic mass in each direction per IS 1893:2016 Clause 7.8.4.2. The RSA is performed for separate RSX and RSY load cases. The analysis model includes P-Δ effects to maintain consistency with the pushover analysis setup and enable direct comparison of results.

IV. RESULTS AND DISCUSSION

4.1 Nonlinear Static Pushover Analysis Results

Figure 3 presents the pushover capacity curve for the Push X load case (lateral loading in the EQ-X direction). The curve is essentially a straight line from the origin to maximum displacement, indicating that the building responded in a nearly linear elastic manner throughout the entire loading history, with no stiffness degradation, post-yield plateau, or strength reduction. The maximum base shear achieved was Vmax ≈ 144,452 kN at a monitored roof displacement of 1,800 mm (the imposed limit), yielding a global lateral stiffness of K = Vmax/Δmax = 144,452/1,800 ≈ 80.3 kN/mm.

Pushover Curve EQ-X Direction

Fig. 3. Pushover Capacity Curve Base Shear vs. Roof Displacement (EQ-X Direction), Vmax = 144,452 kN

Figure 4 presents the capacity curve for the Push Y load case. The maximum base shear in the Y-direction was Vmax ≈ 15,771,286 kN — approximately 109 times larger than in the X-direction — with a corresponding global lateral stiffness of K ≈ 8,762 kN/mm. This extreme directional anisotropy is attributable to the rectangular column section (800 mm × 500 mm) orientation: when the 800 mm dimension is aligned parallel to Y-direction loading, the strong-axis bending resistance is substantially higher. Like the X-direction result, the capacity curve is a perfect straight line with no evidence of yielding. The ductility ratio μ = 1.0 in both directions confirms purely elastic response.

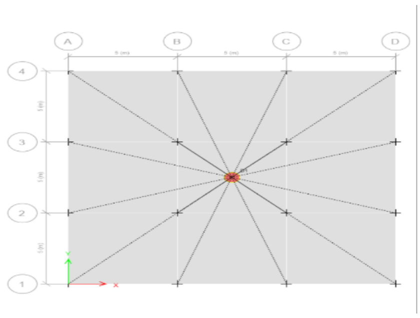


Fig. 4. Pushover Capacity Curve Base Shear vs. Roof Displacement (EQ-Y Direction), Vmax = 15,771,286 kN

Figures 5 and 6 present the FEMA 440 Acceleration-Displacement Response Spectrum (ADRS) plots for the Push X and Push Y cases, respectively. In each plot, the capacity curve (green) is compared against the seismic demand spectrum (red), with the effective period shown as the cyan period line and the highlighted capacity point as the blue dot. The FEMA 440 Equivalent Linearisation algorithm did not converge to a performance point in either direction (Point Found: No). The effective period derived from the secant stiffness was Teff ≈ 0.98 s (X-direction) and Teff ≈ 1.36 s (Y-direction). The absence of a performance point results

International Journal of Creative and Open Research in Engineering and Management
ISSN: **3108-1754 (Online)**
Volume 02 Issue 06 June-2026 | Impact Factor: 3.5

from the nearly linear capacity curve: since no post-yield plateau exists, the equivalent damping βeq remains at the elastic value (5%) throughout, and the reduced demand spectrum does not intersect the capacity curve within the plotted range. This is a structurally favourable outcome — the building has sufficient elastic stiffness and strength to resist the seismic demand without yielding, meeting or exceeding the Immediate Occupancy (IO) performance level.

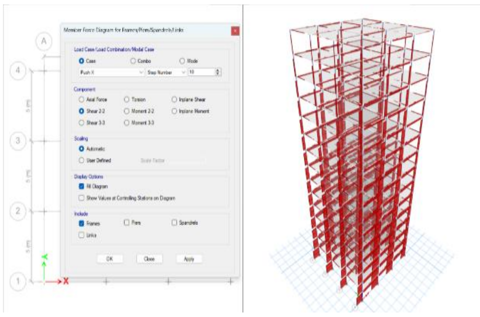
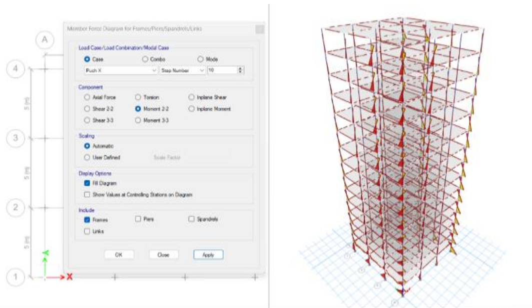


Fig. 5. FEMA 440 ADRS Plot Performance Point Evaluation (EQ-X Direction): Capacity curve (green), Demand spectrum (red), Period line (cyan), Capacity point (blue dot). Performance Point Not Found.



FEMA 440 ADRS EQ-Y

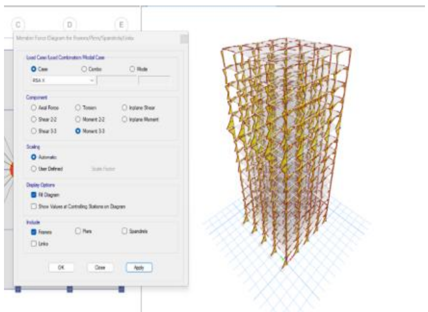
Fig. 6. FEMA 440 ADRS Plot Performance Point Evaluation (EQ-Y Direction). Performance Point Not Found.

Table 4. Summary of Nonlinear Static Pushover Analysis Results

Parameter	Push X (EQ-X)	Push Y (EQ-Y)
Maximum Base Shear (kN)	144,452	15,771,286
Maximum Roof Displacement (mm)	1,800	1,800
Global Lateral Stiffness K (kN/mm)	≈ 80.3	≈ 8,762
Effective Period Teff (s)	≈ 0.98	≈ 1.36
Ductility Ratio (μ)	1.0	1.0
Performance Point Found?	No	No
Observed Behavior	Linear Elastic	Linear Elastic

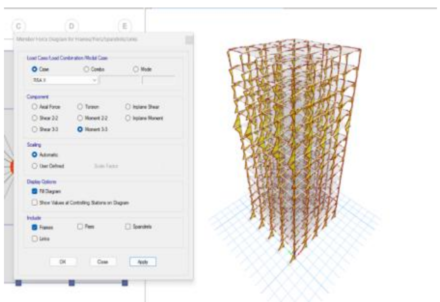
4.2 Nonlinear Dynamic Response Spectrum Analysis Results

Before presenting storey-level results, Figures 7 and 8 show the shear force diagram (V2-2) and bending moment diagram (M3-3) of the structure under the RSX load case. The shear force diagram reveals intense shear demands at beam-column joints throughout all storeys, with relatively uniform distribution up the building height. The moment diagram shows the characteristic triangular pattern with maximum moments at member ends, confirming that critical sections are at frame connections consistent with a well-designed strong-column weak-beam system.



Shear Force Diagram RSX

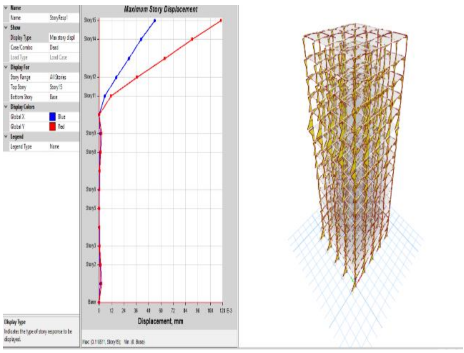
Fig. 7. Shear Force Diagram (V2-2) of the Building under RSX Loading



Bending Moment Diagram RSX

Fig. 8. Bending Moment Diagram (M3-3) of the Building under RSX Loading

Figure 9 presents the Maximum Story Displacement profile for both RSX (blue) and RSY (red) directions. Displacement increases monotonically with storey height from zero at the fixed base to a maximum at Story 15 — the expected cantilever-like behaviour of a fixed-base structure — with no abrupt changes, confirming a regular stiffness distribution without soft storey characteristics. Maximum roof displacement is approximately 52 mm (RSX, X-direction) and 118 mm (RSY, Y-direction) at Story 15. The building is approximately 2.3 times stiffer in the X-direction under dynamic loading, reflecting the directional anisotropy of the rectangular column section.



Maximum Story Displacement RSX and RSY

Fig. 9. Maximum Story Displacement Profile RSX (blue, max ~52 mm at Story 15) and RSY (red, max ~118 mm at Story 15)

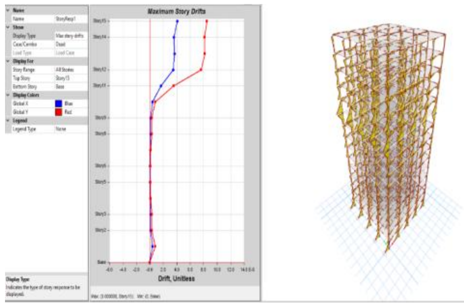
Table 5. Maximum Storey Displacement Results (Response Spectrum Analysis)

Storey	RSX Displacement (mm)	RSY Displacement (mm)
Base	0	0
Story 1	1.8	4.2
Story 3	9.8	22.6
Story 5	20.8	47.8
Story 7	32.1	74.0
Story 9	41.8	96.5
Story 11	47.9	110.3
Story 13	51.2	116.8
Story 15	52.0	118.0

Figure 10 presents the inter-storey drift profiles for Diaphragm D1 under both RSX and RSY load cases. Maximum drift values are approximately 4×10^{-6} (RSX, Story 5–6) and 8×10^{-6} (RSY, Story 5–6). Both values are dramatically lower than the IS 1893:2016 permissible inter-storey drift limit of 0.004 ($4{,}000\times10^{-6}$) a margin of approximately 500 times. Drift values increase from lower storeys, peak near mid-height (Stories 5–8), and reduce near the top a classic first-mode-dominated drift profile. The factor of approximately 2 between RSY and RSX drifts is consistent with the observed displacement ratio.

V. CONCLUSION

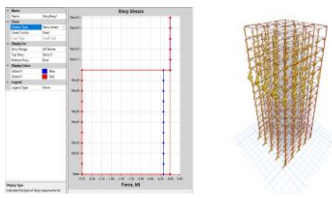
Summarize the key findings of your research and their implications for theory, practice, or policy. Highlight the significance of your contribution and suggest areas for future work. Avoid repeating sentences from the abstract.



Inter-Storey Drift Profile RSX and RSY

Fig. 10. Inter-Storey Drift Profile RSX (blue) and RSY (red) for Diaphragm D1. Maximum drift 8×10^{-6} well within IS 1893:2016 limit of 0.004.

Figure 11 presents the story shear distribution for both RSX and RSY directions. Story shear accumulates toward lower storeys as expected, with maximum base shear values of approximately 0.30 kN (RSX) and 2.70 kN (RSY). The shear is nearly constant from Story 15 to approximately Story 10, and increases sharply toward the base a step-like profile characteristic of buildings where most seismic mass is concentrated at upper storeys. The Y-direction base shear is approximately 9 times the X-direction value, consistent with the relative displacement and stiffness ratios.



Story Shear Distribution RSX and RSY

Fig. 11. Story Shear Distribution RSX (blue, max ~0.30 kN at base) and RSY (red, max ~2.70 kN at base)

Table 6. Inter-Storey Drift Ratios — RSX and RSY Directions

Storey	RSX Drift ($\times10^{-6}$)	RSY Drift ($\times10^{-6}$)	IS 1893 Limit
Story 1	0.60	1.40	4000
Story 3	1.57	3.60	4000
Story 5	1.87	4.27	4000
Story 6 (Max)	1.90	4.40	4000
Story 8	1.77	4.07	4000
Story 10	1.13	2.67	4000
Story 12	0.73	1.40	4000
Story 15	0.07	0.13	4000

4.3 Comparative Analysis Pushover vs. Response Spectrum Analysis

Table 7 presents a side-by-side comparison of key performance metrics from both analysis methods. The most significant common finding is that the G+15 RC building responds in the elastic range under the applied IS 1893:2016 seismic loading. Pushover analysis yielded a perfectly linear capacity curve with $\mu = 1.0$ and no performance point convergence; RSA gave extremely small inter-storey drifts (maximum 8×10^{-6}) and roof displacements of 52–118 mm both well within the elastic range.

The substantial difference in absolute base shear magnitudes between pushover and RSA is expected and physically meaningful: pushover analysis applies a monotonically increasing force to push the structure to the target displacement (a capacity measure), while RSA reports the maximum dynamic force demand from the design ground motion (a demand measure). The much smaller RSA shear values reflect the actual force demand from the IS 1893:2016 design spectrum.

The directional asymmetry consistently observed in both analyses — Y-direction showing larger displacements, higher shear, and greater drift — reflects the rectangular column section orientation: the 800 mm column dimension oriented in the X-direction provides strong-axis bending resistance for Y-direction lateral loads, making the Y-direction more critical for deformation demand. This highlights the importance of bidirectional seismic analysis even for nominally symmetric buildings.

Table 7. Comparison of Pushover and Response Spectrum Analysis Results

Performance Metric	Pushover Analysis	Response Spectrum Analysis
Method Type	Nonlinear Static	Nonlinear Dynamic (modal)
Max. Lateral Displacement (mm)	1800 (imposed)	118 (RSY) / 52 (RSX)
Max. Base Shear (kN)	144,452 (X) / 15,771,286 (Y)	2.7 (RSY) / 0.3 (RSX)
Max. Inter-Storey Drift	Not separately quantified	8×10^{-6} (RSY) / 4×10^{-6} (RSX)
IS 1893:2016 Drift Limit (0.004)	—	✓ Safe (margin ~500×)
Performance Point	Not Found (Elastic, $\mu = 1.0$)	N/A (RSA method)
Ductility Demand	$\mu = 1.0$ (Elastic)	Elastic response confirmed
Critical Direction	Y (higher stiffness & shear)	Y (higher displacement & shear)
Evaluation Framework	FEMA 356/440, ATC-40	IS 1893:2016, CQC/SRSS

V. CONCLUSIONS

This study investigated the seismic performance of a G+15 RC moment-resisting frame building in Seismic Zone V of India using Nonlinear Static Pushover Analysis and Nonlinear Dynamic Response Spectrum Analysis, both implemented in ETABS. The following conclusions are drawn:

(1) The G+15 RC moment-resisting frame building designed per IS 1893:2016 for Seismic Zone V demonstrates excellent seismic performance under the design-level earthquake, remaining within the elastic range under both static and dynamic nonlinear analysis,

confirming adequate structural strength and stiffness for Zone V design forces.

(2) The inter-storey drift ratios from response spectrum analysis (maximum 8×10^{-6} in the Y-direction) are approximately 500 times below the IS 1893:2016 permissible limit of 0.004, confirming structural safety with respect to deformation and serviceability with a very substantial margin.

(3) The pushover capacity curves exhibit nearly linear elastic behaviour throughout the full 1,800 mm applied displacement range. The ductility ratio $\mu = 1.0$ and the absence of a FEMA 440 performance point confirm that the structure does not enter the inelastic range under the design-level seismic demand. This is a structurally favourable outcome, indicating that the building meets or exceeds the Immediate Occupancy (IO) performance level.

(4) Significant directional anisotropy was observed in both analyses: the Y-direction consistently showed approximately 2.3 times larger displacements, 9 times higher base shear, and twice the inter-storey drift compared to the X-direction. This arises from the rectangular column section orientation (800 mm × 500 mm) and underscores the necessity of bidirectional seismic analysis even for nominally symmetric buildings.

(5) The global lateral stiffness values (80.3 kN/mm in X, 8,762 kN/mm in Y) reflect the high inherent stiffness of the well-proportioned RC frame, which is the primary reason the structure remained elastic under the applied seismic demand level.

(6) The combined application of nonlinear static pushover analysis and response spectrum analysis is confirmed as a robust and comprehensive seismic performance evaluation framework. Pushover analysis reveals the global capacity envelope, lateral stiffness, and theoretical failure hierarchy, while RSA quantifies realistic dynamic displacement and drift demands. Together, they provide a complete PBEE assessment that fulfils the requirements of FEMA 356, ATC-40, and IS 1893:2016.

(7) Nonlinear analysis provides a substantially more realistic representation of structural behaviour than linear methods by explicitly accounting for material nonlinearity, geometric (P-Δ) effects, and inelastic deformations — capabilities that are essential for performance-based seismic assessment of tall RC buildings in high seismic zones.

IV. RECOMMENDATIONS AND FUTURE WORK

Based on the findings of this study, the following directions are recommended for future research:

• Incremental Dynamic Analysis (IDA) on the same building model to quantify the collapse margin ratio and develop fragility curves under increasing earthquake intensity levels.

• Nonlinear Time-History Analysis (NLTHA) using a suite of ground motion records compatible with the IS 1893:2016 design spectrum for Zone V to capture the full dynamic response and compare with the RSA results presented here.

• Parametric study varying the number of storeys (G+10, G+15, G+20, G+25) for the same plan and zone conditions to develop a performance comparison across building heights.

• Investigation of Soil-Structure Interaction (SSI) effects, as the fixed-base assumption may be unconservative for structures on soft or medium soils.

• Study of the influence of infill masonry walls on seismic performance, as infills are known to significantly alter lateral stiffness, natural period, and force distribution in RC frames.

• Extension to buildings with plan or vertical irregularity (L-shape, soft storey, mass irregularity) to assess how the pushover–RSA comparison changes for non-regular structures.

REFERENCES

Indian Standard Codes

[1] Bureau of Indian Standards (BIS). IS 1893 (Part 1):2016 Criteria for Earthquake Resistant Design of Structures (Sixth Revision). New Delhi: BIS, 2016.

[2] Bureau of Indian Standards (BIS). IS 456:2000 — Plain and Reinforced Concrete — Code of Practice (Fourth Revision). New Delhi: BIS, 2000.

[3] Bureau of Indian Standards (BIS). IS 875 (Part 1):1987 — Dead Loads; IS 875 (Part 2):1987 — Imposed Loads; IS 875 (Part 3):2015 — Wind Loads. New Delhi: BIS.

International Guidelines and Standards

[4] Krawinkler, H., and Seneviratna, G.D.P.K. (1998). Pros and Cons of a Pushover Analysis of Seismic Performance Evaluation. Engineering Structures, 20(4–6), 452–464.

[5] Applied Technology Council (ATC). ATC-40: Seismic Evaluation and Retrofit of Concrete Buildings. Redwood City, CA: ATC, 1996.

[6] Federal Emergency Management Agency (FEMA). FEMA 356: Prestandard and Commentary for the Seismic

Rehabilitation of Buildings. Washington, DC: FEMA, 2000.

[7] Federal Emergency Management Agency (FEMA). FEMA 440: Improvement of Nonlinear Static Seismic Analysis Procedures. Washington, DC: FEMA, 2005.

[8] Saiidi, M., and Sozen, M.A. (1981). Simple Nonlinear Seismic Analysis of R/C Structures. Journal of the Structural Division, ASCE, 107(5), 937–953.

Journal Articles

[9] Sivakumar, P., Senthil, R., and Perumal, L. (2013). Analytical Study on Seismic Effect of Soft Storey Columns in Multistorey Buildings. International Journal of Civil and Structural Engineering, 3(3), 599–610.

[10] Ranjit, P., Patel, S., and Bhatt, C. (2015). Performance Based Analysis of Multistoried Building with Soft Storey at Different Levels. International Journal of Engineering Research and Technology (IJERT), 4(6), 1183–1188.

[11] Junwon, S., Mehrdad, F., and Abolhassan, A. (2015). Seismic Performance Evaluation of Multistory Reinforced Concrete Moment Resisting Frame Structure with Shear Walls. International Journal of Earthquake Engineering and Hazard Mitigation, 3(2), 46–56.

[12] Inel, M., and Ozmen, H.B. (2006). Effects of Plastic Hinge Properties in Nonlinear Analysis of Reinforced Concrete Buildings. Engineering Structures, 28(11), 1494–1502.

[13] Mwafy, A.M., and Elnashai, A.S. (2001). Static Pushover versus Dynamic Collapse Analysis of RC Buildings. Engineering Structures, 23(5), 407–424.

[14] Chopra, A.K., and Goel, R.K. (2002). A Modal Pushover Analysis Procedure for Estimating Seismic Demands for Buildings. Earthquake Engineering and Structural Dynamics, 31(3), 561–582.

[15] Papanikolaou, V.K., and Elnashai, A.S. (2005). Evaluation of Conventional and Adaptive Pushover Analysis I: Methodology. Journal of Earthquake Engineering, 9(6), 923–941.

[16] Reddy, G.R., Soni, R.K., and Bhatt, S. (2015). Analysis of Multi-Storeyed Structures Using ETABS. International Journal of Innovative Research in Science, Engineering and Technology, 4(6), 4048–4055.

Software Reference

[17] Computers and Structures, Inc. (CSI). ETABS Extended Three-Dimensional Analysis of Building Systems, Version 19. Berkeley, CA: CSI, 2020.