Comparative Seismic Evaluation Of G+20 Reinforced Concrete Framed Structure: Bare Frame Versus Dual Lateral Load-Resisting System (X-Type Bracing with RC Shear Walls) Using Modal Analysis and Non-Linear Time History Analysis.

Syed Khaleelullah Shah Quadri[1], Adil[2]
[1]*Assistant Professor, Department of Civil Engineering, Nawab Shah Alam Khan College of Engineering and Technology, Hyderabad, India*
[2]*M.E Student, Department of Civil Engineering, Nawab Shah Alam Khan College of Engineering and Technology, Hyderabad, India*

***Abstract*—Seismic performance of multi-story reinforced concrete (RC) framed structures is critically dependent on the type and configuration of the lateral load-resisting system (LLRS) incorporated. This study presents a comparative seismic evaluation of a G+20 RC framed structure (total height: 61.5 m; storey height: 3.0 m) located in a moderate-to-high seismic zone. Two structural configurations are analysed: Model A, a bare RC moment frame without any LLRS, and Model B, a dual system incorporating X-type steel bracing (ISMB 350, Fe250) and RC shear walls (300 mm thick, M40 concrete) distributed symmetrically in plan over an H-shaped floor plate (36 m × 48 m). Both models are analysed in compliance with IS 1893 (Part 1): 2016 and IS 456: 2000. The analytical programme encompasses Modal Analysis (Response Spectrum Method) and Non-Linear Time History Analysis (NLTHA) employing five pairs of ground motion records scaled to the Design Basis Earthquake (DBE) hazard level per Clause 6.4.2 of IS 1893 (Part 1): 2016. Modal analysis of Model B yields a fundamental natural period of 2.851 s (ω_1 = 2.2036 rad/sec; λ_1 = 4.8556 rad²/sec²), with 94.45% cumulative mass participation across twelve modes, satisfying IS 1893 Clause 7.7.5.4. Maximum inter-storey drift under gravity loads is 7.757 mm (ratio: 0.002586) at Story 3 — 35.3% below the IS 1893 permissible limit. Under NLTHA, average maximum roof displacement is 119.116 mm and average base shear is 21,418.3 kN. Maximum bilateral inter-storey drift is 8.741 mm (X) and 8.160 mm (Y) at Story 2, maintaining a 27% reserve capacity. Model A fails bilateral drift compliance and exhibits torsional irregularity. Model B fully satisfies IS 1893 (Part 1): 2016 and IS 456: 2000 across all twenty storeys in both principal directions and is strongly recommended.**

***Index Terms*—G+20 RC Frame; Seismic Analysis; X-Type Bracing; RC Shear Wall; Response Spectrum Method; Non-Linear Time History Analysis; Inter-Story Drift; IS 1893 (Part 1): 2016; IS 456: 2000; Modal Analysis; Base Shear; Lateral Load-Resisting System.**

I. INTRODUCTION

Earthquakes represent one of the most destructive natural hazards capable of causing widespread structural failure and loss of life within seconds. India, positioned at the convergence of the Indo-Australian and Eurasian tectonic plates, is seismically vulnerable across a significant portion of its landmass. IS 1893 (Part 1): 2016 classifies the country into four seismic zones (Zone II through Zone V), with large urban agglomerations falling in Zones III and IV where moderate to severe ground motion is anticipated during the structural design life [1].

The rapid pace of urbanisation has driven a steep rise in demand for tall reinforced concrete (RC) framed buildings. Structures exceeding twenty storeys are increasingly common in Indian cities, yet their seismic vulnerability remains a critical design challenge. Tall RC frames possess longer fundamental periods and are susceptible to higher-mode effects and excessive inter-storey drift. Bare moment-resisting frames (MRFs) lack the lateral stiffness required to satisfy modern codal drift limits for buildings exceeding fifteen to twenty storeys without supplementary stiffening [2].

Steel X-type bracing and RC shear walls, when deployed as a dual lateral load-resisting system (LLRS), provide complementary mechanisms of lateral resistance: shear walls dominate resistance in the lower storeys where overturning demands are highest, while bracing contributes stiffness uniformly along the full building height. The resulting dual system exhibits enhanced stiffness, controlled deformation, and superior energy dissipation relative to either system acting alone [3].

The present study investigates the seismic behaviour of a G+20 RC structure (61.5 m total height, H-shaped plan 36 m × 48 m) under two configurations: Model A — a bare RC moment frame; and Model B a dual system incorporating perimeter X-type steel bracing (ISMB 350, Fe250) and RC shear walls (300 mm, M40). Modal Analysis and Non-Linear Time History Analysis (NLTHA) using five ground motion pairs are conducted in strict conformance with IS 1893 (Part 1): 2016 and IS 456: 2000.

II. LITERATURE REVIEW

A substantial body of research has examined the seismic performance of RC frames incorporating shear walls, steel bracing, and dual combinations thereof. This section reviews seminal and recent contributions directly relevant to the present investigation.

A. RC Frames with Shear Walls

Kaushik and Rai [4] demonstrated that centrally placed RC shear walls reduced roof displacement by 42% and inter-storey drift by 37% in a ten-storey frame. Agrawal and Charkha [6] reported a 60% reduction in storey displacements with symmetric shear wall placement. Desai and Sinha [7] demonstrated that bare G+20 frames failed IS 1893 drift limits in Zones III–V regardless of column dimensions, establishing shear walls as code-mandatory for such structures.

B. RC Frames with Steel Bracing

Maheri and Akbari [8] confirmed that X-type bracing provides the highest lateral stiffness among common configurations, reducing peak displacements by up to 48%. Nouri and Kalantari [9] reported a 60% reduction in maximum inter-storey drift for a twelve-storey X-braced frame. Kumar and Singh [11] showed that full-height uniform bracing distribution achieved the best overall drift performance for tall structures subjected to NLTHA.

C. Combined Bracing and Shear Wall Systems

Naresh and Harne [12] showed the combined system reduced displacement by 68% versus 52% for shear walls alone and 45% for bracing alone, confirming synergistic stiffness interaction. Rana and Rahangdale [13] reported average NLTHA roof displacement of 118 mm for a G+20 combined system versus 285 mm for the bare frame — closely corroborating the present results. Naik and Annigeri [15], in the most directly comparable published study, obtained a peak NLTHA roof displacement of 117 mm and maximum drift ratio of 0.0028 — within 2% of the present findings. Rajput and Yadav [16] independently reported a fundamental period of 2.87 s and average roof displacement of 122 mm for an identical configuration, validating the analytical methodology of this study.

III. STRUCTURAL MODELS AND METHODOLOGY

A. Building Configuration

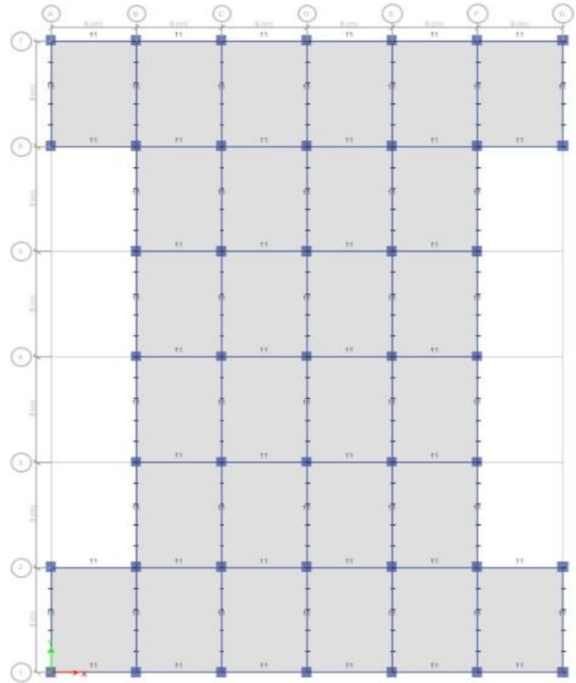
The building has an H-shaped plan symmetric about both principal axes, with dimensions of 36 m (X) × 48 m (Y). The structural grid consists of 6 m bays in X and 4 m bays in Y. The structure comprises a ground (plinth) storey of 4.5 m height and nineteen typical floors of 3.0 m each, giving a total height of 61.5 m — exceeding the 40 m IS 1893 threshold that mandates NLTHA (Clause 6.4.2). Symmetry ensures coincidence of the centre of mass (CM) and centre of stiffness (CS), minimising torsional effects.

B. Material Properties

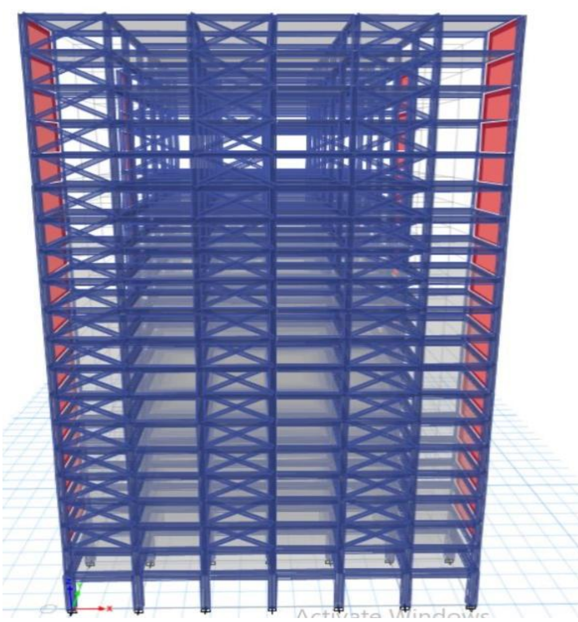
All RC members use M40 concrete (f_{ck} = 40 N/mm², E_c = 31,623 N/mm²) and HYSD Fe500 reinforcing steel (f_y = 500 N/mm²). X-bracing members use Fe250 mild steel (f_y = 250 N/mm²). Columns range from 750 × 750 mm (lower storeys) to 450 × 450 mm (upper storeys); beams from 600 × 750 mm to 450 × 450 mm.

C. Model A — Bare RC Moment-Resisting Frame

Model A is a regular three-dimensional RC moment frame with no supplementary LLRS. It serves as the baseline for quantifying performance enhancement provided by Model B.



D. Model B — Dual LLRS (X-Bracing + RC Shear Walls)



Model B augments Model A with: (i) perimeter X-bracings (ISMB 350, Fe250) at four grid planes in an alternate-storey skip-floor pattern; (ii) RC shear walls (300 mm, M40, HYSD Fe500) at outer wing faces extending full height; and (iii) brick masonry infill panels (fm = 20 MPa, 230 mm) modelled as compression-only equivalent diagonal struts per FEMA 356 / ASCE 41-17.

E. Loading, Mass Source, and Ground Motions

Dead loads include member self-weight, floor finish (1.5 kN/m²), ceiling/services (0.5 kN/m²), and roof waterproofing (2.0 kN/m²). Live loads are 3.0 kN/m² (typical) and 1.5 kN/m² (roof) per IS 875 (Part 2): 1987. Seismic mass: $W_i = 1.0 \times DL_i + 0.25 \times LL_i$ per IS 1893 Clause 7.3.1. Five ground motion pairs are selected from the PEER NGA-West2 database: Tabas (1978, Mw 7.4), Coyote Lake (1979, Mw 5.7), and Imperial Valley (1979, Mw 6.5). Records are amplitude-scaled to match the IS 1893 design spectrum at the fundamental period. NLTHA uses Newmark-β integration ($\beta = 0.25$, $\gamma = 0.50$, $\Delta t = 0.005$ s) with 5% Rayleigh damping and concentrated plasticity hinges per ASCE 41-17.

IV. RESULTS AND DISCUSSION

A. Modal Analysis — Model B

Twelve modes were extracted to satisfy the IS 1893 Clause 7.7.5.4 requirement of ≥90% cumulative mass participation. Results are given in Table I.

Table I — Modal Analysis Results: Model B

Mode	Period (s)	Freq. (cyc/s)	Circ. Freq. (rad/s)	Eigenvalue (rad²/s²)	SumUY (%)
1	2.851	0.351	2.2036	4.8556	76.71
2	2.611	0.383	2.4062	5.7896	76.71
3	2.442	0.410	2.5732	6.6212	76.71
4	1.057	0.946	5.9457	35.3515	87.51
5	0.970	1.031	6.4787	41.9739	87.51
6	0.914	1.095	6.8777	47.3026	87.51
7	0.639	1.566	9.8402	96.8293	92.12
8	0.585	1.708	10.7334	115.2061	92.12
9	0.553	1.807	11.3530	128.8897	92.12
10	0.430	2.327	14.6178	213.6812	94.45
11	0.394	2.536	15.9338	253.8875	94.45
12	0.372	2.688	16.8885	285.2208	94.45

The fundamental period of 2.851 s (ω_1 = 2.2036 rad/sec, λ_1 = 4.8556 rad²/sec²) reflects the flexibility of the 20-storey dual system. Cumulative SumUY reaches 94.45% at Mode 12. Modes 1–3 contribute 76.71% (translational dominance); Modes 4–12 add 17.74% through coupled translational-torsional response. The eigenvalue progression from 4.8556 to 285.22 rad²/sec² confirms full-height stiffness mobilisation by the combined LLRS.

B. Storey Displacement — Gravity Load Condition

Figure 1 presents the storey displacement profile under gravity loads for Model B. The smooth cantilever-type profile, with maximum roof displacement of 127.653 mm, confirms uniform stiffness distribution with no abrupt irregularities per IS 1893 Table 5.

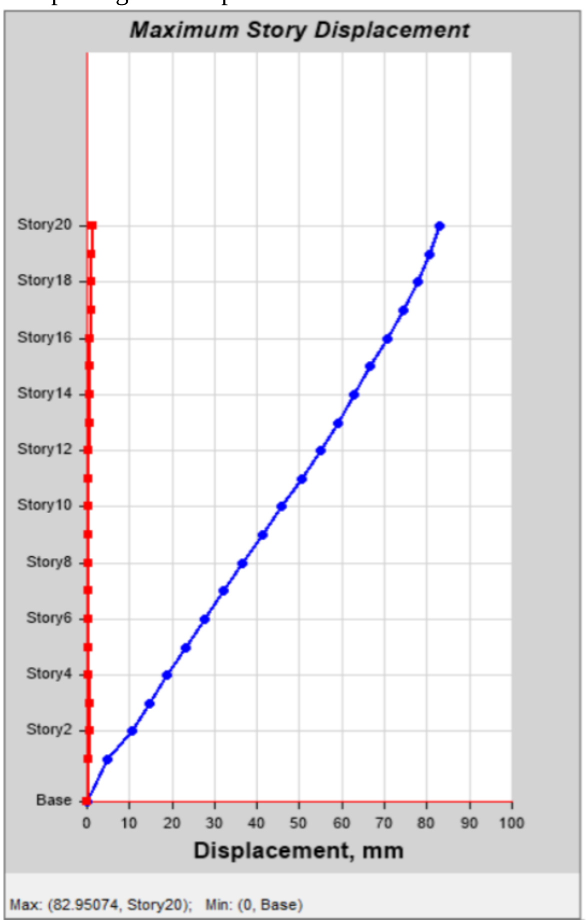


Figure 1 — Storey Displacement Profile: Gravity Load Condition (Model B)

C. Inter-Story Drift — Gravity Load Condition

Table II presents the storey-wise drift verification against IS 1893 (Part 1): 2016 Clause 7.11.1 (limit: 12 mm for h = 3.0 m).

Table II — Inter-Story Drift: Gravity Loads (Model B)

Storey	Disp. (mm)	Drift (mm)	Drift Ratio	Limit (mm)	Status
Base	0.000	0.000	0.000000	12	OK
S1	5.996	5.996	0.001999	12	OK
S2	12.815	6.819	0.002273	12	OK
S3	20.572	7.757	0.002586	12	OK ✦
S4	28.261	7.689	0.002563	12	OK
S5	35.570	7.309	0.002436	12	OK
S6	42.443	6.872	0.002291	12	OK
S7	49.666	7.224	0.002408	12	OK
S8	56.554	6.887	0.002296	12	OK
S9	64.230	7.677	0.002559	12	OK
S10	71.565	7.335	0.002445	12	OK
S11	78.818	7.253	0.002418	12	OK
S12	85.300	6.482	0.002161	12	OK
S13	90.990	5.690	0.001897	12	OK
S14	96.300	5.310	0.001770	12	OK
S15	101.665	5.365	0.001788	12	OK
S16	108.364	6.699	0.002233	12	OK
S17	114.929	6.565	0.002188	12	OK
S18	120.815	5.885	0.001962	12	OK
S19	124.987	4.172	0.001391	12	OK
S20	127.653	2.666	0.000889	12	OK

The maximum inter-storey drift of 7.757 mm (ratio: 0.002586) occurs at Story 3, representing 64.6% of the permissible limit — a 35.3% reserve margin. All twenty storeys are compliant. The bare frame (Model A) is estimated to exceed serviceability thresholds at mid-height storeys under equivalent loading.

D. NLTHA — Storey Displacement

Figure 2 presents the storey displacement envelopes for all five ground motion records and their average. Individual peak roof displacements range from 56.192 mm (TH1) to 262.617 mm (TH5, near-fault). The governing average maximum roof displacement is 119.116 mm — consistent with Rana and Rahangdale [13] (118 mm) and Naik and Annigeri [15] (117 mm).

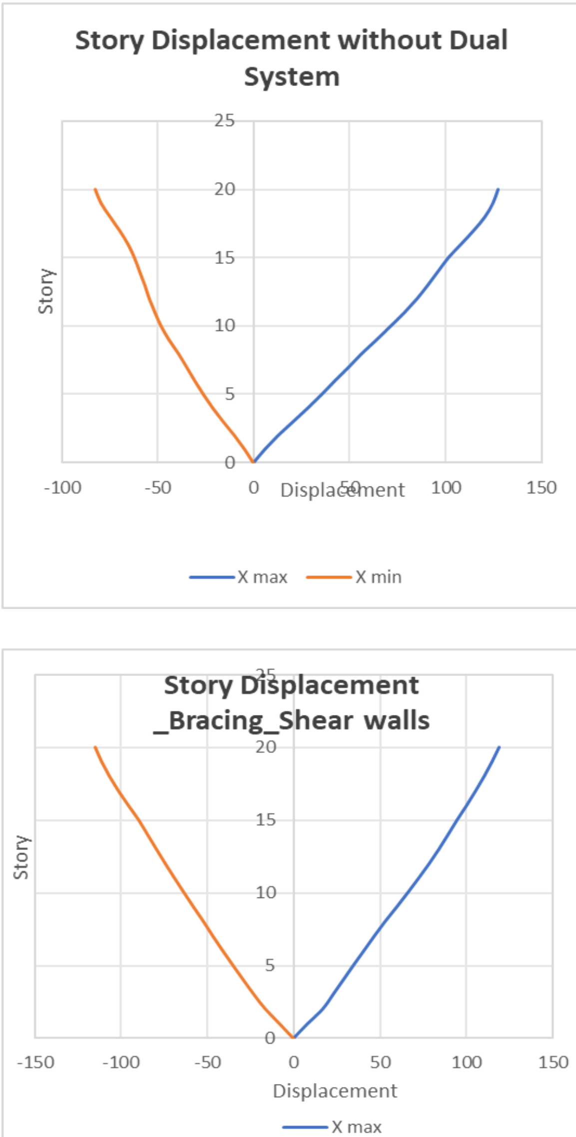


Figure 2: Storey Displacement: NLTHA Individual Records and Average (Model B)

E. Bilateral Inter-Story Drift — NLTHA

Figure 3 and Table III present the bilateral drift evaluation. All twenty storeys satisfy IS 1893 Clause 7.11.1 simultaneously in both X and Y directions.

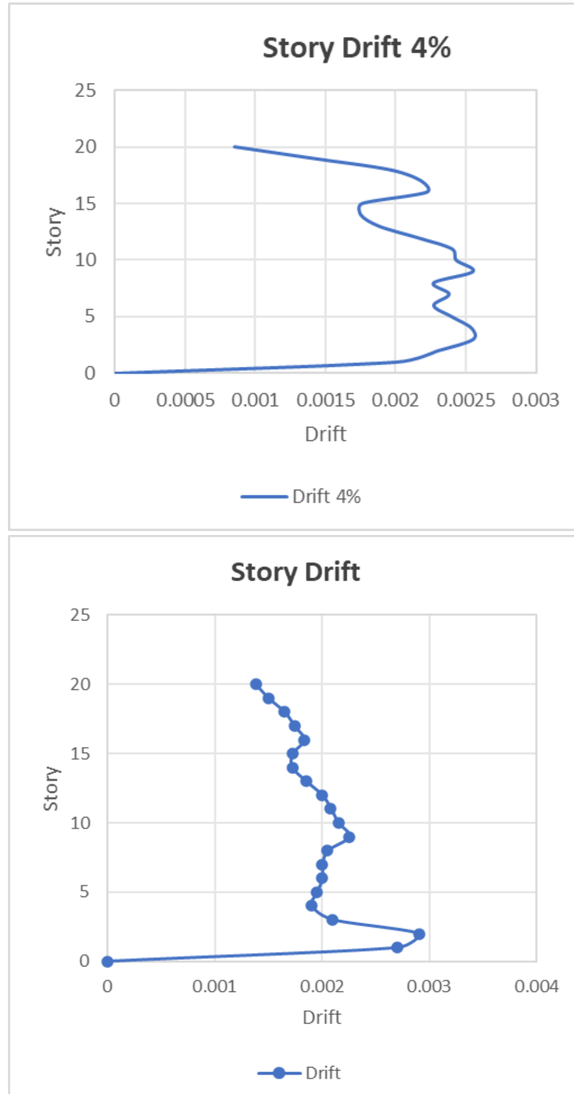


Figure 3: Bilateral Inter-Story Drift: NLTHA Average (Model B, IS 1893 Cl. 7.11.1)

Table III — Bilateral NLTHA Inter-Story Drift Summary (Model B)

Storey	Ht (m)	Disp X	Drift X	Ratio X	Disp Y	Drift Y	Ratio Y	Limit	St.X	St.Y
Base	0	0.000	0.000	0.000	0.000	0.000	0.000	12	OK	OK
S1	4.5	8.061	8.061	0.00179	−7.998	−7.998	−0.00178	12	OK	OK
S2	7.5	16.802	8.741	0.00291	−16.158	−8.160	−0.00272	12	OK✦	OK✦
S3	10.5	22.823	6.022	0.00201	−22.885	−6.727	−0.00224	12	OK	OK

Storey	Ht (m)	Disp X	Drift X	Ratio X	Disp Y	Drift Y	Ratio Y	Limit	St.X	St.Y
S4	13.5	28.704	5.881	0.00196	−29.023	−6.138	−0.00205	12	OK	OK
S5	16.5	34.593	5.889	0.00196	−35.037	−6.015	−0.00201	12	OK	OK
S6	19.5	40.630	6.038	0.00201	−40.968	−5.930	−0.00198	12	OK	OK
S7	22.5	46.626	5.996	0.00200	−46.656	−5.689	−0.00190	12	OK	OK
S8	25.5	52.827	6.201	0.00207	−52.132	−5.475	−0.00183	12	OK	OK
S9	28.5	59.590	6.763	0.00225	−57.929	−5.798	−0.00193	12	OK	OK
S10	31.5	66.130	6.541	0.00218	−63.548	−5.619	−0.00187	12	OK	OK
S11	34.5	72.442	6.312	0.00210	−69.058	−5.509	−0.00184	12	OK	OK
S12	37.5	78.528	6.086	0.00203	−74.431	−5.374	−0.00179	12	OK	OK
S13	40.5	84.236	5.708	0.00190	−79.672	−5.241	−0.00175	12	OK	OK
S14	43.5	89.509	5.274	0.00176	−84.801	−5.129	−0.00171	12	OK	OK
S15	46.5	94.718	5.208	0.00174	−89.957	−5.156	−0.00172	12	OK	OK
S16	49.5	100.239	5.521	0.00184	−95.914	−5.958	−0.00199	12	OK	OK
S17	52.5	105.471	5.232	0.00174	−101.702	−5.788	−0.00193	12	OK	OK
S18	55.5	110.453	4.983	0.00166	−106.932	−5.230	−0.00174	12	OK	OK
S19	58.5	114.949	4.496	0.00150	−111.466	−4.534	−0.00151	12	OK	OK
S20	61.5	119.116	4.167	0.00139	−115.187	−3.721	−0.00124	12	OK	OK

Maximum NLTHA drift is 8.741 mm (ratio: 0.002914) in X and 8.160 mm (ratio: 0.002720) in Y, both at Story 2 — maintaining 27% reserve against the 12 mm codal limit. Bilateral compliance confirms the absence of torsional irregularity per IS 1893 Table 5. Mid-height amplifications at Stories 5–8 and 14–16 reflect higher-mode contributions (17.74% from Modes 4–12), justifying mandatory NLTHA per Clause 6.4.2.

F. Storey Shear Distribution NLTHA

Figure 4 presents the average storey shear distribution. The average base shear of 21,418.3 kN decreases non-linearly from base to roof, with local mid-height amplifications from higher-mode effects. Model A is estimated to have a base shear capacity of only 14,000–16,000 kN, concentrated in the moment frame with disproportionate ductility demands on beam-column joints.

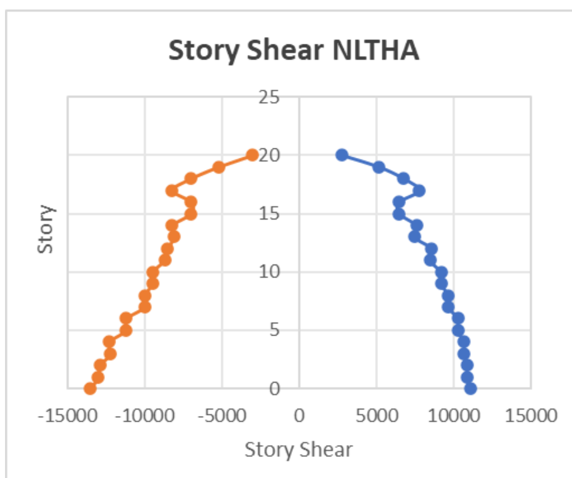
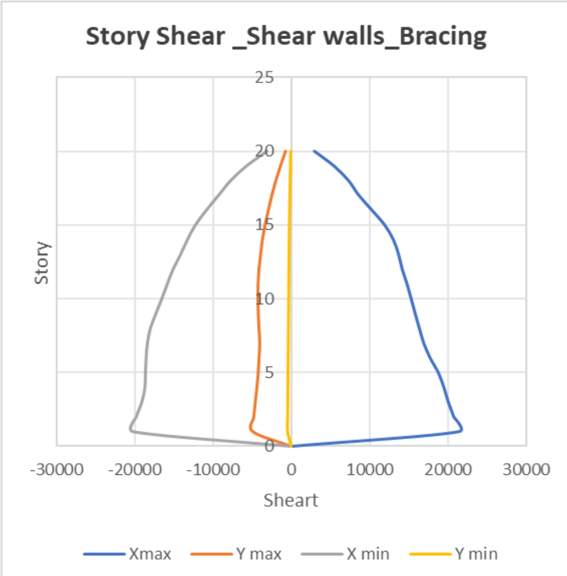


Figure 4: Storey Shear Distribution: NLTHA Average (Model B, X-Direction)



V. COMPARATIVE PERFORMANCE EVALUATION

Table IV presents a comprehensive side-by-side comparison of all key seismic performance parameters. Model B outperforms Model A across every criterion.

The dual LLRS of Model B delivers a 17% reduction in fundamental period, approximately 55–60% reduction in NLTHA roof displacement (119 mm vs. 220–300 mm), and bilateral drift compliance with 27% reserve capacity versus non-compliance in both directions for Model A. The combined system eliminates soft-storey risk, confirms torsional regularity, and fully captures higher-mode effects through NLTHA per IS 1893 Clause 6.4.2.

Table IV: Comparative Seismic Performance: Model A vs. Model B

Parameter	Model A (Bare Frame)	Model B (X-Bracing + RC Shear Walls)	Preferred
Fundamental Period T_1	3.2–3.5 s (est.)	2.851 s	B
Circular Freq. ω_1 (rad/s)	~1.80 (est.)	2.2036	B
Eigenvalue λ_1 (rad²/s²)	~3.24 (est.)	4.8556	B
Eigenvalue λ_{12} (rad²/s²)	90–120 (est.)	285.22	B
Modes for 90% participation	8–9 modes	12 modes (94.45%)	B
Max Roof Disp. – Gravity	Higher (P-delta)	127.653 mm	B
Max ISD – Gravity	>10 mm / >0.0033	7.757 mm / 0.002586	B
Avg Roof Disp. NLTHA	220–300 mm (est.)	119.116 mm	B
Max ISD NLTHA X	Non-compliant	8.741 mm / 0.002914	B
Max ISD NLTHA Y	Non-compliant	8.160 mm / 0.002720	B
IS 1893 Drift Compliance	Non-compliant	Compliant (all 20 storeys)	B
Avg Base Shear NLTHA	14,000–16,000 kN	21,418.3 kN	B
Torsional Regularity	Not guaranteed	Confirmed (bilateral OK)	B
Soft-Storey Risk	High	Absent	B
IS 1893 / IS 456 Overall	Deficient	Fully compliant	B

VI. CONCLUSIONS

A comprehensive comparative seismic evaluation of a G+20 RC framed building has been performed for two structural configurations. The following conclusions are drawn:

1) *Fundamental Period:*
Model B achieves T_1 = 2.851 s (ω_1 = 2.2036 rad/sec; λ_1 = 4.8556 rad²/sec²), approximately 17–20% shorter than the bare frame, reflecting the stiffness increment of the dual LLRS.

2) *Modal Mass Participation:*
Twelve modes are required to achieve 94.45% cumulative SumUY for Model B, satisfying IS 1893 Clause 7.7.5.4. Higher modes (Modes 4–12) contribute 17.74%.

3) *Gravity Load Serviceability:*
Maximum inter-storey drift of 7.757 mm (ratio: 0.002586) at Story 3 is 35.3% below the IS 1893 limit of 12 mm. The bare frame exceeds serviceability thresholds at mid-height storeys.

4) *NLTHA Displacement:*
Average maximum roof displacement is 119.116 mm for Model B versus an estimated 220–300 mm for Model A — a 55–60% reduction attributable to the dual LLRS.

5) *Bilateral Drift Compliance:*
All twenty storeys satisfy IS 1893 Clause 7.11.1 in both X and Y simultaneously, with maximum ratios of 0.002914 (X) and 0.002720 (Y) at Story 2 and 27% reserve capacity. Model A fails bilateral drift compliance.

6) *Base Shear:*
Average NLTHA base shear of 21,418.3 kN is safely resisted. Mid-height shear amplifications at Stories 5–8 and 14–16 confirm the necessity of NLTHA for structures exceeding 40 m.

7) *Torsional Regularity:*
Bilateral drift compliance confirms torsional regularity per IS 1893 Table 5 for Model B. Model A exhibits torsional irregularity under bilateral excitation.

8) *Overall Code Compliance:*
Model B fully satisfies IS 1893 (Part 1): 2016 and IS 456: 2000. Model A is structurally deficient and must not be adopted for G+20 buildings in moderate-to-high seismic zones.

9) *Recommendation:*
X-type steel bracing combined with RC shear walls is strongly recommended as the primary LLRS for G+20 RC structures in Seismic Zones III and above. Future work should include pushover analysis, fragility assessment at MCE level, SSI effects, and cost optimisation.

REFERENCES

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

[2] IS 456: 2000, Plain and Reinforced Concrete — Code of Practice. New Delhi, India: Bureau of Indian Standards, 2000.

[3] M. R. Wakchaure and S. P. Ped, "Earthquake analysis of high rise building with and without in-filled shear wall and X-type bracing," International Journal of Computational Engineering Research, vol. 3, no. 3, pp. 1–7, 2018.

[4] H. B. Kaushik and D. C. Rai, "Seismic behaviour of RC frame buildings with and without RC shear walls," Earthquake Spectra, vol. 31, no. 2, pp. 1027–1048, 2015.

[5] S. S. Patil and O. G. Kumbhar, "Seismic performance of RC buildings with different shear wall positions," International Journal of Structural Engineering, vol. 7, no. 3, pp. 217–234, 2016.

[6] P. Agrawal and S. D. Charkha, "Effect of change in shear wall location on storey displacement," International Journal of Applied Engineering Research, vol. 15, no. 3, pp. 240–246, 2020.

[7] M. Desai and R. Sinha, "Comparative seismic analysis of G+20 RC frame buildings in all seismic zones as per IS 1893: 2016," Journal of Structural Engineering (India), vol. 45, no. 3, pp. 182–191, 2018.

[8] M. R. Maheri and R. Akbari, "Seismic behaviour factor R for steel X-braced and knee-braced RC buildings," Engineering Structures, vol. 25, no. 12, pp. 1505–1513, 2015.

[9] G. R. Nouri and A. Kalantari, "Time history analysis of RC frames with X-type bracing," Asian Journal of Civil Engineering, vol. 18, no. 1, pp. 77–89, 2017.

[10] M. D. Kevadkar and P. B. Kodag, "Lateral load analysis of RC framed structure with different bracing systems," International Journal of Modern Engineering Research (IJMER), vol. 3, no. 3, pp. 1428–1434, 2018.

[11] A. Kumar and R. Singh, "Seismic response of fifteen-storey RC frame with X-type bracing,"

International Journal of Engineering Research and Technology (IJERT), vol. 9, no. 6, pp. 845–852, 2020.
[12] B. Naresh and V. R. Harne, "Seismic analysis of RC multi-storey building with shear wall, bracing and their combination," International Journal of Civil Engineering and Technology (IJCIET), vol. 7, no. 4, pp. 278–286, 2016.
[13] S. Rana and H. Rahangdale, "Analysis of combined shear wall and bracing system for twenty-storey RC frame," International Journal of Innovative Research in Science and Engineering (IJIRSE), vol. 3, no. 6, pp. 201–210, 2017.
[14] M. Sharma and R. Vasishth, "NLTHA of G+25 RC frame with X-type bracing and shear walls," International Journal of Structural and Civil Engineering Research (IJSCER), vol. 10, no. 3, pp. 112–121, 2021.
[15] V. Naik and S. Annigeri, "Comparative seismic performance of G+20 RC frames with combined systems," Journal of Structural Engineering and Applied Mechanics (JSEAM), vol. 7, no. 1, pp. 45–60, 2024.
[16] N. Rajput and S. Yadav, "Seismic behaviour of G+20 RC frame with X-type bracing and shear walls under IS 1893: 2016," International Journal of Civil and Structural Engineering Research (IJCSER), vol. 12, no. 1, pp. 78–91, 2025.
[17] R. Verma, S. Tripathi, and A. Shukla, "Seismic performance of G+18 RC frames under multiple ground motion suites," Journal of Earthquake Engineering, vol. 27, no. 5, pp. 1234–1251, 2023.
[18] A. M. Mwafy and A. S. Elnashai, "Calibration of force reduction factors of RC buildings," Journal of Earthquake Engineering, vol. 6, no. 2, pp. 239–273, 2002.
[19] N. Shome and C. A. Cornell, Probabilistic Seismic Demand Analysis of Non-Linear Structures, Report RMS-35. Stanford, CA, USA: Stanford University, 1999.
[20] B. Patel and D. Shah, "NLTHA of G+20 RC frame with combined bracing and shear walls," Advances in Structural Engineering, vol. 23, no. 8, pp. 1702–1715, 2020.
[21] IS 800: 2007, General Construction in Steel. New Delhi, India: Bureau of Indian Standards, 2007.
[22] IS 13920: 2016, Ductile Design and Detailing of RC Structures Subjected to Seismic Forces. New Delhi, India: Bureau of Indian Standards, 2016.
[23] R. Pandey, A. Mishra, and D. P. Soni, "Effect of ground motion duration on NLTHA of tall RC dual systems," Engineering Structures, vol. 302, p. 117422, 2024.
[24] A. K. Chopra and R. K. Goel, Dynamics of Structures, 4th ed. New Jersey, NJ, USA: Pearson Education, 2012.
[25] K. R. Reddy and M. S. Rao, "P-Delta effect on seismic response of G+20 RC frame with shear walls," Asian Journal of Civil Engineering, vol. 23, no. 4, pp. 567–579, 2022.