

Comparative Seismic Analysis of Fixed-Base on Base-Isolated G+20 Storey RC Building with X-Type Steel Bracing as per IS 1893 (Part 1): 2016

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ABSTRACT

This study presents a comparative seismic evaluation of a G+20 storey reinforced concrete (RC) building in Seismic Zone V, Gujarat, under three configurations: a fixed-base system (Model A), a base-isolated system using pure rubber bearings (Model B), and a hybrid system combining base isolation with X-type steel bracing at Storeys 17-20 (Model C). Three-dimensional models were analysed under Equivalent Static Analysis (ESA), Response Spectrum Analysis (RSA), and Non-Linear Time History Analysis (NLTHA) using five spectrum-compatible ground motions, per IS 1893 (Part 1): 2016. Modal analysis showed only marginal period elongation under isolation (1.511 s to 1.521 s), so Models A and B recorded nearly identical NLTHA roof displacements (130.28 mm), exceeding the $H/500 = 120$ mm limit; isolation alone yielded modest base-shear (0.98-2.03%) and drift (3.9%) reductions. ESA over-predicted and RSA under-predicted the NLTHA demand, confirming NLTHA as essential for reliable assessment. The hybrid configuration (Model C) reduced peak roof displacement to 117.56 mm (9.8% reduction, $H/500$ -compliant) and critical inter-storey drift by 44.1%, at an 11.7% base-shear increase relative to Model B. These findings were corroborated by a supplementary pushover and Incremental Dynamic Analysis (IDA) study, confirming that isolation shifts the governing limit state from superstructure collapse (Model A, 0.40 g) to isolator displacement capacity (Models B and C, 0.08 g). The study concludes that the hybrid base-isolation-with-bracing system alone satisfies the drift limit, the $H/500$ criterion, and a net base-shear reduction relative to the fixed-base design, establishing it as an effective seismic protection strategy for tall RC buildings in high-seismic zones.

Keywords: Seismic Base Isolation; Non-Linear Time History Analysis; Incremental Dynamic Analysis; Pushover Analysis; X-Type Steel Bracing; Inter-Storey Drift Ratio; IS 1893 (Part 1):2016; High-Rise RC Building; ETABS

1. Introduction

Earthquakes remain among the most destructive natural hazards confronting the built environment, releasing enormous energy in seconds and imposing sudden dynamic demands on structures that are conventionally proportioned for gravity loads. Reinforced concrete (RC) frame construction is the dominant structural typology across urban India, and the rapid vertical growth of Indian cities has produced an increasing stock of G+20 and taller RC buildings in moderate-to-high seismic zones. At such heights (approximately 60-63 m for a G+20 building), higher-mode contributions become significant, the fundamental period grows long and sensitive to stiffness degradation, and cumulative roof displacement can become large even when individual storey drift ratios appear moderate - conditions under which the simplified assumptions of code-level static analysis begin to break down.

India's seismicity, driven by the ongoing collision of the Indian and Eurasian tectonic plates, has produced a long record of damaging earthquakes, from the 1897 Shillong and 1934 Bihar-Nepal events to the 2001 Bhuj earthquake, which alone caused the collapse of hundreds of RC frame buildings. IS 1893 (Part 1): 2016 divides the country into four seismic zones (II-V) with Zone V, covering north-east India and parts of Gujarat, carrying the highest design PGA of 0.36g. The 2016 revision introduced updated response spectra, refined response-reduction factors, and dedicated provisions (Part 6) for base-isolated structures, providing the design framework adopted throughout the present study.

Seismic base isolation - a passive control strategy that introduces a flexible, energy-dissipating layer between the foundation and superstructure - elongates the fundamental period of a structure and shifts its response into a region of lower spectral acceleration on the design spectrum, while providing supplemental hysteretic damping. The strategy has been used worldwide since the 1970s, but its effectiveness diminishes in tall buildings: overturning-moment demand on the isolators increases with height, the fixed-base period of a tall RC building may already be 2-3 s (limiting the achievable elongation), and the absolute roof displacement - the sum of

isolator displacement and accumulated superstructure drift - can approach or exceed serviceability limits such as the IS 1893 permissible drift of $H/500$. Steel X-type bracing, by contrast, is an economical, high-stiffness-to-weight lateral system that controls storey drift through axial action; when applied selectively at storeys where deformation demand concentrates, it can act as a targeted stiffness-enhancement device without a full-building retrofit.

Motivated by this complementary behaviour, the present study develops and compares three configurations of a G+20 RC building in Seismic Zone V - fixed-base (Model A), pure rubber-bearing base-isolated (Model B), and a hybrid system combining isolation with X-type steel bracing at Storeys 17-20 (Model C) - under ESA, RSA and NLTHA in strict accordance with IS 1893 (Part 1): 2016, and cross-validates the principal findings using a supplementary pushover and Incremental Dynamic Analysis (IDA) study. No unified comparative study addressing all three configurations within this framework for a G+20 RC building has been reported in the available literature, establishing the motivation for the present investigation.

1.1 Objectives

(1) To develop three-dimensional finite-element models of a G+20 RC building with fixed-base (Model A) and base-isolated (Model B) configurations and evaluate their seismic response using ESA, RSA and NLTHA in accordance with IS 1893 (Part 1): 2016.

(2) To characterise the dynamic behaviour of Models A and B through fundamental period, modal mass participation and energy dissipation.

(3) To compare the seismic performance of Models A and B in terms of storey displacement, inter-storey drift and storey/base shear, and quantify the effectiveness of base isolation alone.

(4) To assess the performance of the hybrid system (Model C) under NLTHA and validate the ETABS results using pushover and Incremental Dynamic Analysis.

2. Literature Review

A structured review of recent studies on seismic ground-motion selection, base isolation, and steel bracing was undertaken to position the present study relative to the existing body of knowledge. Table 1 summarises the principal findings and the specific gap each study leaves open relative to the objectives of the present work.

Table 1. Summary of reviewed literature and identified research gaps.

Author(s) & Year	Key Finding	Gap Addressed by Present Study
Bhandari et al. (2025)	NLTHA of an LRB-isolated 5-storey building; up to 46% base-shear, 65% drift and 66% displacement reduction.	Extends isolator influence to a G+20 tall building.
Hasanoglu et al. (2024)	Energy- and acceleration-spectra-compatible ground-motion selection method reduced drift dispersion.	Adopts the underlying record-selection rationale for the 5-record NLTHA suite used here.
Raghukanth et al. (2024)	PSHA-based spectrum-compatible ground-motion database for Indian urban centres.	Supplies scaling framework; present study adds structural response evaluation.
Patel et al. (2024)	Review incl. E-Defence shake-table validation of LRB force reduction.	Provides a building-specific, IS-1893-based quantitative study.
Benaoum et al. (2023)	HDRB+FVD hybrid cut base shear >80% but increased isolator displacement by 47%.	Present hybrid (isolation + bracing) avoids the displacement penalty via a stiffness-based remedy.
Furtado et al. (2022)	Review proposing isolation combined with supplementary stiffness devices for high-rise buildings.	Provides the first quantitative validation on a specific G+20 RC building.
Khoury et al. (2021)	NLTHA of a 44-storey LRB-isolated building; roof displacement remained a concern.	Proposes and tests an X-bracing remedy for the displacement concern.
Ghasemi & Talaeitaba (2020)	Isolated 20-24 storey RC buildings approached/exceeded $H/500$.	Confirms the $H/500$ issue and resolves it via hybrid bracing.
Rahimi & Maheri (2020)	X-braced RC frames (4-, 8-, 12-storey): bracing reduces displacement, drift & shear.	Applies bracing to an already base-isolated tall building.
Alghuff et al. (2019)	ESA vs RSA on 75 m / 24 m RC buildings; RSA 35-65% lower shear than ESA.	Confirms ESA over-prediction; extends to isolation, bracing & NLTHA.
Abdi et al. (2019)	Optimum DCR ranges for X-braced frames (columns 0.4-0.5; braces 0.8-0.9).	Provides design basis, integrated here with base isolation.
Kakpure & Mundhada (2016)	ETABS static/dynamic comparison for G+10 & G+25; RSA recommended beyond 10 storeys.	Confined to fixed base; present study adds isolation, bracing & NLTHA.
Cancellara & De Angelis (2016)	3-D nonlinear dynamic analysis of RC + hybrid isolation/dampers.	Present study examines a stiffness-based (bracing) hybrid instead of a

		damping-based one.
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Viewed collectively, the reviewed literature validates the ground-motion selection approach and the individual effectiveness of base isolation and X-bracing, but no study combines all three configurations - fixed-base, pure isolation, and isolation-plus-bracing - within a single IS 1893 (Part 1): 2016 framework for a G+20 RC building using ESA, RSA and NLTHA together with a supplementary IDA cross-check, which is the specific contribution of the present work.

3. Materials and Methods

3.1 Building Description and Structural Configuration

A G+20 storey RC building (plan dimensions 20 m x 35 m, total height $H = 60$ m at 3 m storey height) located in Gujarat, Seismic Zone V ($PGA = 0.36g$, design wind speed 47 m/s per IS 875 Part 3) was modelled as a three-dimensional frame in ETABS. Three configurations were analysed: Model A (conventional fixed-base building), Model B (fixed-base geometry with Pure Rubber Bearing isolators introduced at the base), and Model C (Model B with X-type ISMB 350 steel bracing introduced at Storeys 17-20). A centrally located RC shear-wall core and a regular 5 m x 5 m bay grid were adopted for all models, with a rigid diaphragm assigned at every floor level. Member cross-sections were varied over the height to reflect the reduction of gravity and seismic demand with elevation (Table 2).

Table 2. Cross-sectional properties of columns and beams (varying with storey range).

Member	Size	Grade	Storey Range
Beam	300 x 450 mm	M30	17-20
Beam	300 x 550 mm	M30	9-16
Beam	350 x 600 mm	M30	1-8
Column (edge)	450 x 450 mm	M40	17-20
Column (inner)	530 x 530 mm	M40	1-16
Column	600 x 600 mm	M40	9-16
Column	750 x 750 mm	M40	1-8
Slab	150 mm thick	M30	All floors

Table 3. Material properties adopted in the analysis (IS 456:2000 / IS 1893:2016).

Property	M30 Concrete	M40 Concrete	Fe500 Rebar
Unit weight (kN/m ³)	25	25	78.5
Elastic modulus (GPa)	27.39	31.62	200
Poisson's ratio	0.2	0.2	0.3
Characteristic strength (MPa)	30	40	500 (yield)

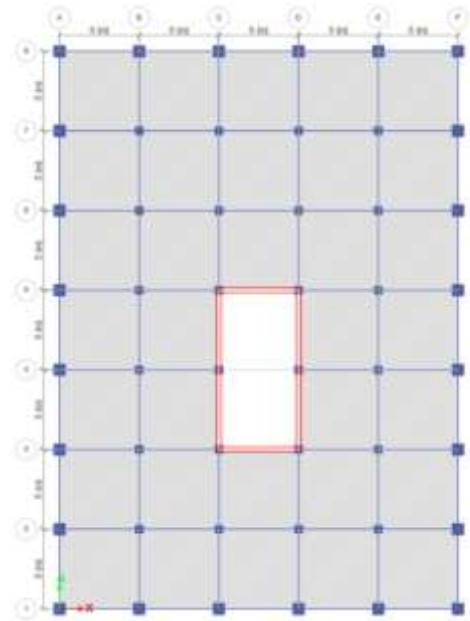


Fig. 1. Typical floor plan of the G+20 building showing the central RC shear-wall core (highlighted) and 5 m grid layout.

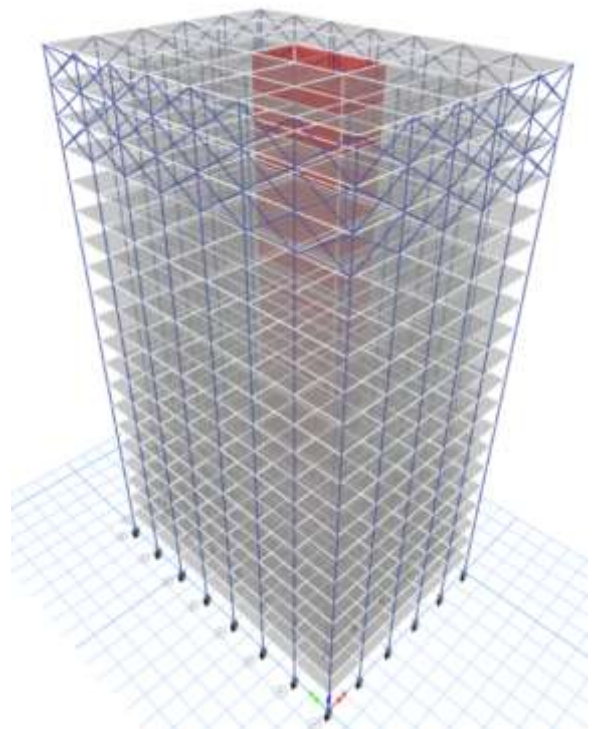


Fig. 2. Three-dimensional analytical model of the base-isolated G+20 building with central shear-wall core.

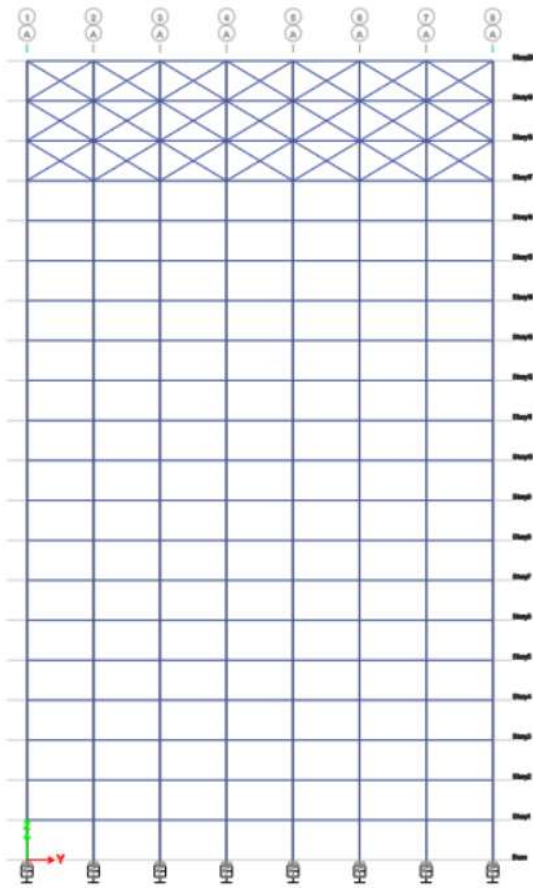


Fig. 3. Elevation of the building showing X-type steel bracing introduced at Storeys 17-20 (Model C).

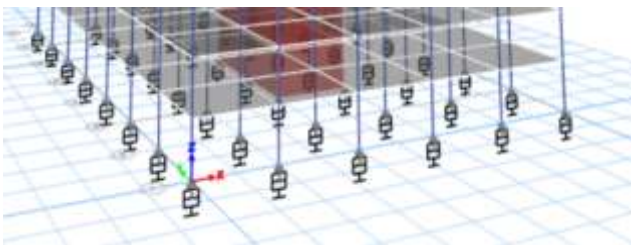


Fig. 4. Isometric detail of the pure-rubber-bearing isolators located beneath the columns at foundation level.

3.2 Loading and Mass Source

Dead load was computed automatically from member geometry and material density; a super-imposed dead load of 3.0 kN/m² and a live load of 4.0 kN/m² (IS 875 Part 2) were applied. The seismic mass was derived per IS 1893:2016 Cl. 7.3.1 using 100% dead load plus 25% of the live load, with the isolator mass included as additional point mass at the isolation level. Thirteen load combinations were defined per IS 456:2000 and IS 1893:2016, covering factored gravity and bidirectional seismic cases (e.g., 1.2(DL+LL+/-EQX), 1.5(DL+/-EQY), 0.9DL+/-1.5EQ).

3.3 Base Isolation System

Pure Rubber Bearing (PRB) isolators - relying on the inherent damping of natural rubber rather than a lead core - were modelled at the base of each column as linear link elements

with the properties listed in Table 4, targeting an isolated fundamental period of approximately 2.5 s.

Table 4. Base isolation properties (Pure Rubber Bearing).

Property	Value
Effective horizontal stiffness	800 kN/m
Vertical stiffness	2.0×10^6 kN/m
Shear modulus of rubber	0.6-1.0 MPa
Effective damping ratio	2-5%
Design displacement (MCE)	200 mm
Effective isolated period	2.5 s
Diameter of isolator	550 mm

3.4 X-Type Steel Bracing System

To control excessive upper-storey displacement identified in Model B (Section 4), X-type concentric bracing using ISMB 350 sections (Table 5) was introduced at Storeys 17-20 in Model C, modelled as pin-ended truss elements carrying axial force only.

Table 5. X-bracing properties (ISMB 350 section, Storeys 17-20).

Property	Value
Depth (d)	350 mm
Flange width (bf)	140 mm
Cross-sectional area	6671 mm ²
Moment of inertia, I _{xx}	136.0×10^6 mm ⁴
Material grade	Fe250 (IS 2062)
Yield strength, f _y	250 MPa

3.5 Ground Motion Selection

Thirteen spectrum-compatible ground-motion records were selected from the PEER NGA (Next Generation Attenuation) database, scaled to the site-specific hazard and applied in both principal directions for NLTHA; five representative records (TH1-TH5, Table 6) governed the primary comparative results reported in Section 4.

Table 6. Representative ground motion records used for NLTHA (PEER NGA database).

Record ID	Earthquake	Mw	PGA (g)
RSN77_SFERN_PUL164	San Fernando	6.6	0.21
RSN147_COYOTELK_G02-UP	Coyote Lake	5.7	0.18
RSN149_COYOTELK_G04-UP	Coyote Lake	5.7	0.24
RSN764_LOMAP_GOF160	Loma Prieta	6.9	0.64
RSN766_LOMAP_G02090	Loma Prieta	6.9	0.56

3.6 Analysis Procedure

Three analysis methods prescribed by IS 1893 (Part 1): 2016 were applied progressively in ETABS: (i) Equivalent Static Analysis (ESA), using the seismic coefficient method; (ii) Response Spectrum Analysis (RSA), a linear dynamic method combining modal responses by CQC and directional effects by SRSS, with Zone V parameters ($Z = 0.36$, $I = 1.5$,

$R = 5.0$, Type II soil, 5% damping fixed-base / 2% isolated); and (iii) Non-Linear Time History Analysis (NLTHA), solving the equations of motion at each time step under the five spectrum-compatible records to capture material and geometric (P-delta) non-linearity. Storey displacement, inter-storey drift and storey shear were extracted and compared across all three models and methods. Fig. 5-6 summarise the overall research methodology.

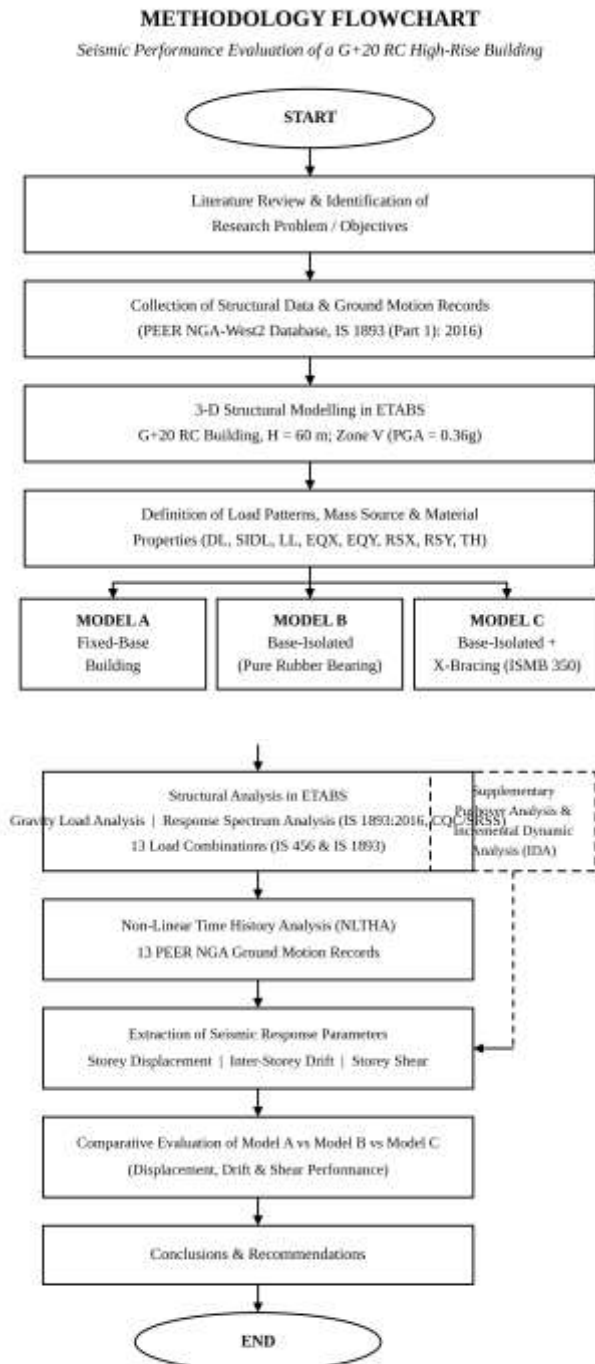


Figure 3.10: Methodology Flowchart

Fig. 5-6. Methodology flowchart from literature review and structural modelling through comparative evaluation and conclusions.

3.7 Supplementary Pushover and Incremental Dynamic Analysis (IDA)

To cross-validate the ETABS-based static/dynamic findings using an independent, simplified approach, a supplementary nonlinear static pushover and Incremental Dynamic Analysis (IDA), following Vamvatsikos and Cornell (2002), were carried out on reduced-order lumped-mass stick models of Models A, B and C (20 DOF for Model A; 21 DOF for Models B/C, including an isolator/raft DOF). Each of seven records was scaled to increasing intensity levels; the peak Engineering Demand Parameter (storey drift ratio for Model A, isolator displacement for Models B/C) was recorded at each level and plotted against the spectral intensity measure $S_a(T_1, 5\%)$ to construct IDA curves, from which median and 16th/84th-percentile capacities were derived. This supplementary study is indicative and intended to corroborate, not replace, the detailed ETABS results.

4. Results and Discussion

4.1 Modal Characteristics

Table 7 compares the first three modal periods and mass participation factors of the three models. Isolation elongated the fundamental period only marginally, from 1.511 s (Model A) to 1.521 s (Model B) - a 0.66% increase, far short of the three- to four-fold elongation expected of an effectively isolated system - while the addition of bracing in Model C reduced the period to 1.452 s relative to Model B, reflecting the added lateral stiffness of the upper-storey braces.

Table 7. Comparison of fundamental modal periods and mass participation (Modes 1-3).

Model	Mode 1: T (s)	MPF-1 (%)	Mode 2: T (s)	Mode 3: T (s)
A - Fixed base	1.511	66.54	1.176	1.139
B - Base isolated (PRB)	1.521	66.65	1.180	1.179
C - Isolated + X-bracing	1.452	67.76	1.137	1.107

4.2 Storey Displacement and H/500 Serviceability Compliance

ESA over-predicted the RSA displacement by approximately 15-18% while NLTHA under MCE-level ground motions produced roof displacements 3.5-4.5 times the RSA values, confirming that only NLTHA captures the true inelastic demand for this building class. Under NLTHA, Models A and B both recorded a peak positive roof displacement of 130.28 mm, exceeding the IS 1893 (Part 1): 2016 permissible limit of $H/500 = 120$ mm ($H = 60$ m). Because base isolation concentrates the majority of this displacement at the isolator level rather than reducing it, superstructure drift is protected even though the absolute roof displacement is essentially unchanged from the fixed-base case. Model C's targeted upper-storey bracing reduced the peak displacement to

117.56 mm - a 9.8% reduction that brings the response within the H/500 limit (Table 8).

4.3 Storey Shear

The NLTHA base shear for Model A reached 21,301.53 kN (positive) / -23,161.43 kN (negative), with a non-monotonic storey-shear profile - peaking at Storey 10 at approximately 64% of the base shear - reflecting higher-mode participation that ESA/RSA do not capture. Base isolation alone (Model B) reduced base shear by only 0.98-2.03% relative to Model A, a modest benefit attributable to the fixed-base building's already-moderate period (2.0-2.5 s) placing it on the descending branch of the design spectrum. The hybrid Model C increased base shear by 11.7% relative to Model B (to 23,304.02 kN), the expected trade-off of the added bracing stiffness attracting additional seismic force while controlling displacement.

4.4 Inter-Storey Drift

All three configurations satisfied the IS 1893 drift limit of 0.004 under every analysis method. Under NLTHA, the critical X-direction drift ratio reduced progressively across the three configurations: 0.00284 (Model A, Storey 17) to 0.00273 (Model B, Storey 16, -3.9%) to 0.001587 (Model C, Storey 18, -44.1% relative to Model A). A localised drift concentration of 0.002879 (Y-direction) was noted at the brace-termination transition storey (Storey 17) in Model C, still well within code limits but flagged for enhanced ductile detailing.

4.5 Consolidated Comparative Summary

Table 8 consolidates the governing NLTHA response parameters for all three configurations against the IS 1893 (Part 1): 2016 limits, showing that only the hybrid Model C achieves full compliance while simultaneously reducing base shear relative to the fixed-base design.

Table 8. Summary of governing seismic response parameters (NLTHA) and IS 1893 (Part 1):2016 compliance.

Parameter	Model A (Fixed Base)	Model B (Isolated, PRB)	Model C (Hybrid)	IS 1893 Limit
Peak roof displacement, +X (mm)	130.28 (Exceeds)	130.28 (Exceeds)	117.56 (Within limit)	H/500 = 120 mm
Peak roof displacement, -X (mm)	-112.31	-112.31	-107.63	H/500 = 120 mm
Peak base shear, +X (kN)	21,301.5	20,869.2 (-2.03%)	23,304.0 (+11.7% vs B)	Design capacity
Peak base shear, -X (kN)	-23,161.4	-22,933.4 (-0.98%)	-	Design capacity
Max inter-storey drift ratio, X	0.00284 (Storey 17)	0.00273 (Storey 16)	0.001587 (Storey 18)	≤ 0.004

		16, -3.9%)	18, -44.1%)	
Fundamental period, T1 (s)	1.511	1.521 (+0.66%)	1.452 (-4.54% vs B)	-
H/500 compliance (NLTHA)	No	No	Yes	H/500 = 120 mm
Overall IS 1893 compliance	Partial	Partial	Full	Full compliance required

4.6 Beam Force Response (Representative Member Check)

Fig. 7 presents the internal force and deformation response of a representative beam member under NLTHA for Model A, confirming that the assigned M30/Fe500 sections perform within serviceability limits under combined gravity and seismic demand.

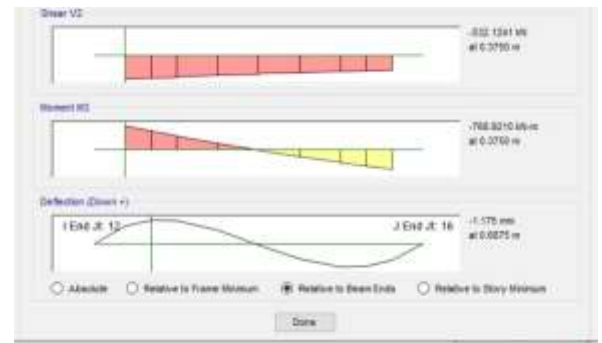


Fig. 7. Representative NLTHA beam force diagrams - Shear (V2), Moment (M3) and mid-span deflection profile (Model A).

4.7 Supplementary Pushover and Incremental Dynamic Analysis

The supplementary IDA study (Section 3.7) corroborates the ETABS findings under a family of dynamic time-histories. For Model A, the governing limit state is FEMA 356 Collapse Prevention drift (4%), reached at a median intensity of 0.40 g Sa(T1) with moderate record-to-record dispersion (16th/84th percentile 0.298-0.404 g). For Models B and C, the isolator design displacement (200 mm) governs, reached at a considerably lower, tightly-clustered median intensity of 0.08 g with negligible scatter, reflecting the isolator's near-linear-elastic behaviour; superstructure drift for both isolated models remains far below the Immediate Occupancy limit across the full intensity range tested (Table 9, Fig. 8-13).

Table 9. IDA comparative summary - governing limit state and median intensity-measure capacity.

Model	Governing Limit State	Median IM cap. (g)	16th pct. (g)	84th pct. (g)	T1 (s)
A	Collapse Prevention drift (4%)	0.40	0.298	0.404	1.511
B	Isolator design displacement (200 mm)	0.08	0.08	0.08	3.723

C	Isolator design displacement (200 mm)	0.08	0.08	0.08	3.716
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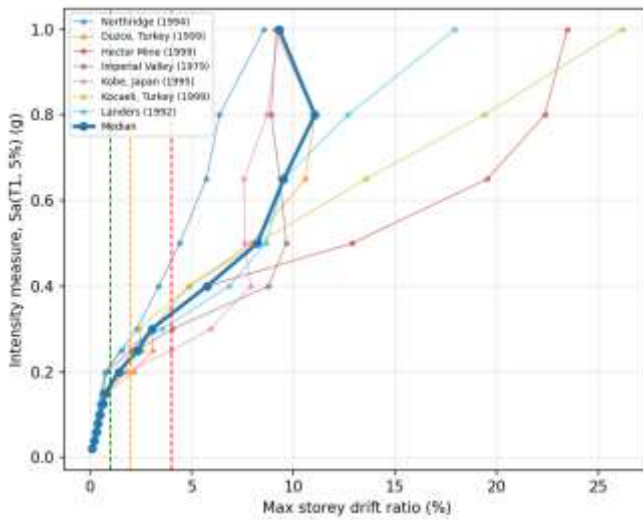


Fig. 8. IDA curves, Model A (fixed base) - maximum storey drift ratio vs. spectral intensity.

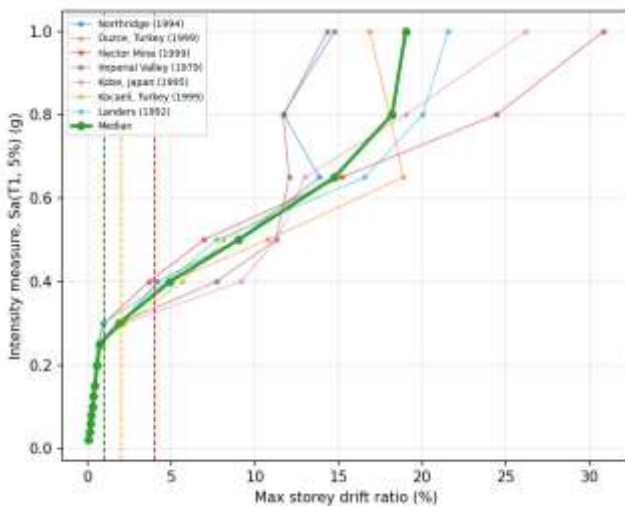


Fig. 9. IDA curves, Model B - superstructure storey drift ratio vs. spectral intensity.

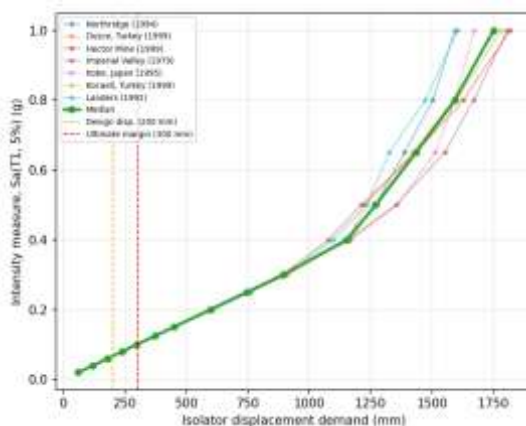


Fig. 10. IDA curves, Model B - isolator displacement demand vs. design (200 mm) and ultimate (300 mm) limits.

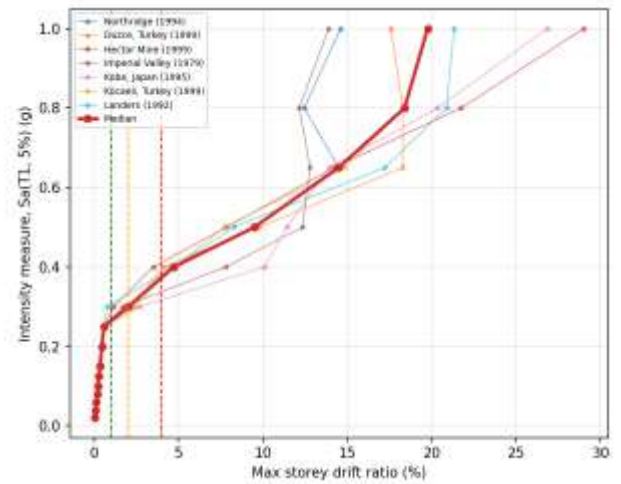


Fig. 11. IDA curves, Model C - superstructure storey drift ratio vs. spectral intensity.

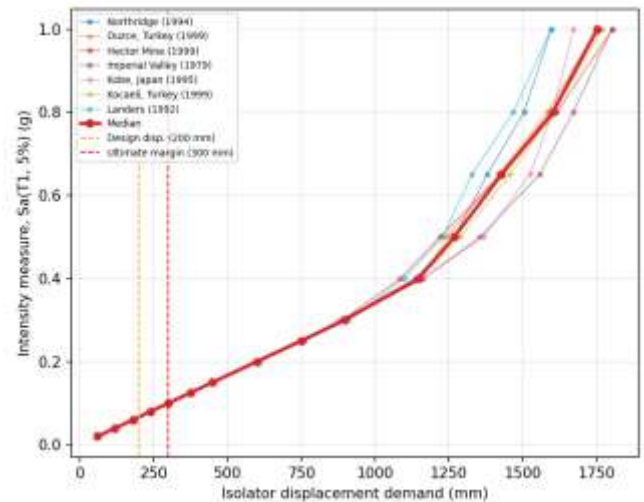


Fig. 12. IDA curves, Model C - isolator displacement demand vs. design/ultimate limits.

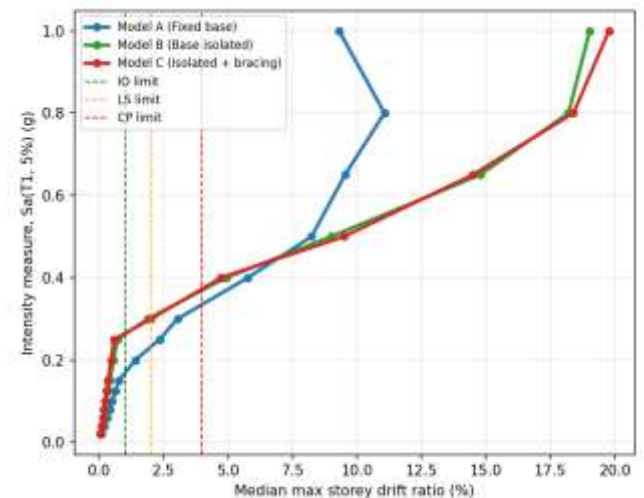


Fig. 13. Median IDA comparison, all three models superimposed - Models B/C remain near-elastic well beyond the intensity at which Model A reaches Collapse Prevention.

Taken together, the pushover/IDA results tell a consistent story: base isolation trades a structural (superstructure

collapse) limit state for a serviceability/maintenance one (isolator displacement capacity) - the intended and materially safer behaviour for life-safety purposes - while the X-bracing supplement in Model C provides a secondary, smaller benefit to superstructure drift control without altering this basic picture. This supplementary evaluation is best regarded as an indicative cross-check pending confirmation of the reinforcement schedule, column grid and bracing layout against the full ETABS model with real PEER NGA-West2 records.

5. Conclusions

1. ESA, RSA and NLTHA were performed on 3-D ETABS models of Models A and B per IS 1893 (Part 1): 2016. Peak roof displacement rose from 30.79-31.52 mm (RSA) to 130.28 mm (NLTHA, Model A), with a non-monotonic storey-shear profile (peak at Storey 10, 64% of base shear) reflecting higher-mode effects that ESA and RSA do not capture; the maximum drift ratio (0.00284) remained within the IS 1893 limit of 0.004 under all methods, confirming code compliance but establishing NLTHA as essential for this building class.
2. The fundamental period elongated only marginally, from 1.511 s (Model A) to 1.521 s (Model B, +0.66%), well short of the three- to four-fold elongation expected of effective isolation; modal mass participation (66.54-66.65% at Mode 1) was essentially unchanged, indicating energy dissipation remained governed by superstructure hysteresis rather than isolator damping.
3. Base isolation reduced NLTHA base shear by only 0.98-2.03% and drift by 3.9% relative to Model A, while peak roof displacement stayed unchanged at 130.28 mm - isolation redistributes demand to the isolator level rather than reducing it, lowering member-level damage but leaving global displacement demand essentially unaddressed.
4. Models A and B both exceed the $H/500 = 120$ mm limit at 130.28 mm; Model C reduces this to 117.56 mm (9.8% reduction, $H/500$ -compliant) and cuts critical drift by 44.1%, at an 11.7% base-shear cost relative to Model B. The supplementary pushover/IDA study confirms this pattern: Model A reaches Collapse Prevention drift at 0.40 g, while Models B/C are isolator-governed, reaching their 200 mm design displacement at a much lower, tightly clustered 0.08 g, with superstructure drift held well below Immediate Occupancy.

5. Model C is the only configuration to satisfy both the drift and $H/500$ criteria while reducing base shear relative to the fixed-base case, establishing the hybrid isolation-plus-bracing strategy as the recommended configuration for tall RC buildings in high-seismic zones such as Gujarat, Seismic Zone V.

5.1 Future Scope

Future work could extend the analysis to a wider range of building heights and to irregular plan/elevation configurations; systematically investigate alternative isolator types, damping ratios and target isolated periods; explore combined supplementary systems such as viscous dampers, buckling-restrained braces or tuned mass dampers; incorporate soil-structure interaction, particularly for soft-soil sites; conduct life-cycle cost and performance-based loss-estimation studies; and pursue experimental validation through scaled shake-table testing of the proposed hybrid configuration.

Funding

This research received no external funding.

Conflict of Interest

The authors declare that there is no conflict of interest.

Acknowledgement

The authors gratefully acknowledge **Deccan College of Engineering and Technology Hyderabad, India**, for providing the necessary research facilities and computational resources to carry out this study.

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