

SEISMIC EVALUATION AND STRENGTHENING OF AN EXISTING G+10 MULTISTOREY BUILDING IN INDIA

Abhiranjan Kumar¹, Mrs. Kamni Laheriya²

*Research Scholar, Department of Structural Engineering, Madhyanchal Professional University,
Ratibad, Bhopal, Madhya Pradesh, India¹*

*Assistant Professor, Department of Structural Engineering, Madhyanchal Professional University,
Ratibad, Bhopal, Madhya Pradesh, India²*

ABSTRACT

Aging reinforced concrete (RC) multi-storey infrastructure across high-seismic zones in India exhibits severe vulnerability under revised seismic demand criteria established by IS 1893 (Part 1):2016. This empirical investigation evaluates the seismic adequacy of an existing G+10 residential RC frame building situated in Seismic Zone IV (peak ground acceleration $PGA = 0.24g$) on medium soil (Type II), originally constructed in accordance with legacy IS 1893:1984 provisions. In-situ destructive and non-destructive material evaluations, including concrete core extraction and ultrasonic pulse velocity testing, established a baseline concrete compressive strength of 20 MPa and rebar yield strength of 415 MPa. Three-dimensional finite element modeling and non-linear static pushover analysis conducted in accordance with ATC-40 and FEMA 356 protocols revealed that the existing bare frame experiences an excessive roof lateral displacement of 66.8 mm and a maximum inter-storey drift ratio of 0.54%, directly violating the codal limit of 0.40% and locating the building at the Collapse Prevention (CP) performance limit state. To remediate these structural deficiencies, three distinct seismic retrofit strategies were empirically appraised: Carbon Fiber Reinforced Polymer (CFRP) confinement, Reinforced Concrete (RC) column jacketing, and concentric Steel X-bracing. Structural response data demonstrate that concentric Steel X-bracing achieves superior performance, reducing peak lateral displacement to 31.2 mm (a 53.3% reduction) and maximum drift to 0.21%, while elevating the base shear capacity by 97.2% (from 1410 kN to 2780 kN) and shifting the global performance state to Immediate Occupancy (IO) with a modest structural mass increase of 3.2%, establishing it as the optimal techno-economic retrofit solution for Indian urban building stock.

Keywords: *Seismic Vulnerability Assessment¹; Non-linear Static Pushover Analysis²; Reinforced Concrete Frame³; Structural Retrofitting⁴; Carbon Fiber Reinforced Polymer (CFRP)⁵; Concentric Steel X-Bracing⁶; IS 1893:2016 Codal Compliance⁷.*

1. INTRODUCTION

1.1 STRUCTURAL VULNERABILITY OF AGING RC INFRASTRUCTURE IN INDIA

A substantial proportion of multi-storey reinforced concrete (RC) buildings across urban India were constructed during the rapid infrastructural expansion between 1985 and 2005 under legacy codes, specifically IS 1893:1984 and IS 456:1978. These existing structures were engineered predominantly for gravity loading with minimal ductile detailing provisions, as mandatory compliance with IS 13920 was either absent or inconsistently enforced during municipal approvals. Consequently, these mid-rise buildings typically G+10 to G+12 configurations feature slender column sections, inadequate transverse core confinement, widely spaced shear stirrups, short lap splices located within maximum bending moment zones, and unconfined beam-column joints. Under moderate-to-severe earthquake ground motions, such inherent structural deficiencies induce premature brittle shear failures, severe soft-storey mechanisms, and catastrophic structural collapse, posing an acute threat to human life and urban economic stability across densely populated Indian metropolitan zones.

1.2 CODAL EVOLUTION AND SEISMIC HAZARD ZONATION DISCREPANCIES

Over the past three decades, the Bureau of Indian Standards has undergone extensive revisions to its national seismic codes, culminating in the release of IS 1893 (Part 1):2016 and the mandatory retrofitting standard IS 15988:2013. These modern codal revisions significantly elevated elastic design base shear demands, introduced stringent inter-storey drift limitations (strictly capped at 0.004 times the storey height, or 0.40%), incorporated mandatory cracked-section stiffness reduction factors ($0.70I_g$ for columns and $0.35I_g$ for beams), and upgraded seismic hazard zonation factors across vast geographic corridors such as the National Capital Region (NCR) and Himalayan foothill belts. Consequently, existing RC buildings designed under obsolete pre-2002 standards exhibit severe capacity deficits when evaluated against contemporary Design Basis Earthquake (DBE) and Maximum Considered Earthquake (MCE) hazard levels, mandating systematic empirical examination, quantitative vulnerability profiling, structural re-qualification, and engineered seismic retrofitting.

1.3 SCOPE AND EMPIRICAL OBJECTIVES OF SEISMIC EVALUATION AND RETROFIT

This research presents a rigorous empirical investigation into the seismic performance evaluation and rehabilitation of an actual existing G+10 RC residential building situated in Seismic Zone IV (Noida, Delhi-NCR). By integrating in-situ non-destructive material characterization with three-dimensional non-linear static pushover simulations, this study quantifies the precise structural shortfall of the baseline bare frame. Furthermore, it systematically benchmarks three practical retrofit topologies: Carbon Fiber Reinforced Polymer (CFRP) composite wrapping, Reinforced Concrete (RC) column jacketing, and concentric Steel X-bracing. The empirical analysis evaluates fundamental modal periods, lateral roof displacements, storey drifts, capacity-demand curves, plastic hinge sequencing, added mass penalties, and economic life-cycle indices to establish

validated design guidelines and engineering recommendations for practicing structural engineers engaged in urban disaster risk mitigation.

2. LITERATURE SURVEY

Seismic evaluation and structural rehabilitation of non-ductile RC frame structures have been extensively studied through numerical simulations and experimental investigations globally and within the Indian subcontinent. Foundational methodologies formulated in ATC-40 and FEMA 356 established capacity spectrum procedures to determine performance points under inelastic seismic demand, forming the standard basis for modern performance-based earthquake engineering [1]-[4]. Post-earthquake reconnaissance reports following the 2001 Bhuj earthquake (Mw 7.7), the 2005 Kashmir earthquake (Mw 7.6), and the 2015 Nepal earthquake (Mw 7.8) underscored that multi-storey RC frame collapses were primarily triggered by non-ductile column hinge formation, joint shear failure, and soft-storey stiffness discontinuities in ground storeys [5]-[8]. To counteract these critical vulnerabilities, various structural retrofitting techniques have been investigated. Conventional RC jacketing, standardized in IS 15988:2013, effectively enhances axial, flexural, and shear capacities of deficient columns while significantly increasing lateral stiffness [9]-[12]. However, empirical studies by Sharma et al. and Agrawal & Chourasia revealed that RC jacketing adds substantial dead weight (12% to 18% increase in total seismic mass), necessitating disruptive foundation underpinning, soil strengthening, and prolonged tenant relocation [13]-[15]. To overcome mass penalties, externally bonded fiber-reinforced polymers (FRP), particularly Carbon FRP (CFRP) composite wraps, have emerged as an efficient rehabilitation technology [16]-[19]. While CFRP confinement remarkably boosts core concrete compressive strength, axial deformability, and member ductility without adding structural mass or architectural intrusion, experimental findings indicate that it provides minimal global lateral stiffness enhancement, often leaving flexible mid-rise frames vulnerable to excessive inter-storey drifts [20]-[23]. In parallel, structural steel bracing systems (concentric X, V, and chevron braces) have gained widespread adoption due to their high stiffness-to-weight ratio, rapid modular installation, and superior base shear dissipation [24]-[27]. Furthermore, recent investigations emphasize that steel bracing efficiently redistributes lateral forces away from deficient concrete frames into dedicated steel truss elements [28], [29]. In addition, experimental push-over investigations and non-linear dynamic time-history evaluations confirm that diagonal steel truss configurations provide predictable hysteretic energy dissipation with minimal architectural hindrance, low structural weight addition, and rapid site constructability. Nevertheless, comprehensive empirical investigations that concurrently evaluate dynamic modal attributes, non-linear pushover performance states, mass penalties, foundation safety, and economic cost-benefit metrics on full-scale G+10 Indian RC building topologies remain limited in current engineering literature [30]. This research directly bridges this critical literature gap by providing an exhaustive, rigorous empirical dataset.

3. METHODOLOGY

- **In-Situ Field Testing & Structural Geometry:** Structural Characterization and In-Situ Field Testing: The subject structure is an existing G+10 residential RC frame building located in Noida, NCR (Seismic Zone IV, $Z = 0.24$, medium soil Type II). The building features a symmetrical 4-bay by 4-bay footprint ($20\text{ m} \times 20\text{ m}$) with 5.0 m bay spacing, 3.2 m storey height, and 35.2 m total height. In-situ non-destructive core extraction and rebound hammer tests established in-place concrete strength $f_{ck} = 20\text{ MPa}$ and steel yield strength $f_y = 415\text{ MPa}$. Gravity loading includes slab dead weight (150 mm slab), 200 mm masonry infills, floor finishes, and design live loads (3.0 kN/m^2 typical, 1.5 kN/m^2 roof) per IS 875.
- **Numerical Simulation & Pushover Formulations:** Analytical Modeling and Non-Linear Pushover Simulation: Three-dimensional finite element modeling was executed in ETABS v21. Slabs were idealized as rigid diaphragms with cracked section stiffness modifiers ($0.70I_g$ columns, $0.35I_g$ beams) per IS 1893:2016. Response spectrum analysis utilized 5% damping and response reduction factor $R = 3.0$. Displacement-controlled non-linear static pushover analysis was conducted per ATC-40 and FEMA 356 under an inverted triangular load pattern. Default and user-defined lumped plasticity hinges (uncoupled M3 for beams, coupled P-M2-M3 for columns) were assigned per ASCE 41-17 backbone formulations to track progressive yielding and plastic hinge formation.
- **Retrofit Schemes & Experimental Validation Protocol:** Retrofit Topology Formulation and Evaluation Protocol: Three retrofit schemes applied to ground-to-4th storey elements were evaluated: (i) Scheme 1 (CFRP Confinement) comprising 2-ply unidirectional carbon wrap ($t_f = 0.38\text{ mm}$, $E_f = 230\text{ GPa}$) per ACI 440.2R; (ii) Scheme 2 (RC Jacketing) with 100 mm M30 encasement, 8-16 mm Fe500 rebars, and 8 mm ties at 100 mm pitch per IS 15988; and (iii) Scheme 3 (Concentric Steel X-Bracing) using steel channels (2x ISMC 250) in perimeter central bays. Empirical performance was benchmarked across modal periods, roof displacements, drift ratios, pushover capacity, structural mass, and direct capital installation costs.

4. DATA COLLECTION AND ANALYSIS

The empirical data collected from in-situ structural investigations, dynamic modal eigenvalue extractions, linear response spectrum simulations, and displacement-controlled non-linear static pushover analyses are presented across five comprehensive tables and five analytical figures. The empirical data explicitly establish the quantitative relationships governing structural degradation in the un-retrofitted state and demonstrate the comparative mechanics of the three seismic retrofit configurations, providing direct empirical validation that bridges the research hypotheses formulated in the Abstract with the practical engineering conclusions.

Table 1: Geometrical, Material, and Dynamic Structural Properties of Existing G+10 Building (Pre-Retrofit)

Parameter / Characteristic	Nominal Value / Specification	Codal Basis / Test Method	Vulnerability & Empirical Implication
Building Configuration	G+10 Storeys, 3.2 m / storey (H = 35.2 m)	Architectural drawings	Symmetrical 4x4 bay grid (20 m x 20 m footprint)
Structural System Type	Ordinary Moment Resisting Frame (OMRF)	IS 1893:1984 / IS 456:1978	Non-ductile detailing; lack of joint shear ties (R = 3.0)
Column Sizes (Storeys G to 4)	300 mm x 450 mm (8-16mm dia, 8mm @ 200 c/c)	In-situ core & cover meter	High axial load ratio ($P/f_{ck}A_g > 0.42$); shear deficient
Column Sizes (Storeys 5 to 10)	300 mm x 350 mm (6-16mm dia, 8mm @ 225 c/c)	In-situ scanning / NDT	Stepped reduction; low lateral stiffness in upper storeys
Beam Sizes (All Storeys)	300 mm x 500 mm (3-16mm top/bot, 8mm @ 150)	Structural audit	Strong-beam weak-column mechanism induced at joints
Concrete Strength (f_{ck})	20.0 MPa (Target Grade M20)	IS 516 / IS 13311 Core Test	Aged concrete with micro-cracking and low tensile capacity
Steel Yield Strength (f_y)	415.0 MPa (Fe 415 Deformed Bars)	Tensile coupon test (IS 1786)	Low ultimate elongation (14.5%); non-ductile behavior
Total Seismic Weight (W)	31,840 kN (Seismic mass = 3,245.6 tonnes)	IS 1893:2016 Clause 7.4	100% Dead Load + 25% Live Load (Typical floors)
Fundamental Period (T_1)	1.340 seconds (IS Empirical $T_a = 0.880$ s)	Modal Eigenvalue Analysis	Highly flexible bare frame; period elongated by 52.3%
Design Base Shear Demand (VB)	1,623.8 kN (Seismic coefficient $A_h = 0.051$)	IS 1893:2016 (Zone IV, $Z=0.24$)	Exceeds existing elastic capacity by 15.2% (Deficient)

The empirical baseline data compiled in Table 1 definitively establish the physical root causes of seismic vulnerability in the existing structure. Destructive core sampling and rebound hammer testing confirm an in-place concrete compressive strength of only 20.0 MPa, which, when combined with undersized column dimensions (300 mm × 450 mm) in lower storeys, results in an elevated axial stress ratio exceeding 0.42. This leaves negligible reserve capacity for seismic moment reversals. Furthermore, the dynamic eigenvalue

extraction yields an uncoupled fundamental translational period of 1.340 seconds, which is 52.3% longer than the empirical period of 0.880 seconds predicted by IS 1893:2016 formulas, signifying excessive lateral frame flexibility. Under Zone IV seismic demands, the design base shear requirement of 1,623.8 kN exceeds the frame's initial yield strength, verifying that the building operates in an over-stressed, non-compliant regime prior to retrofit intervention.

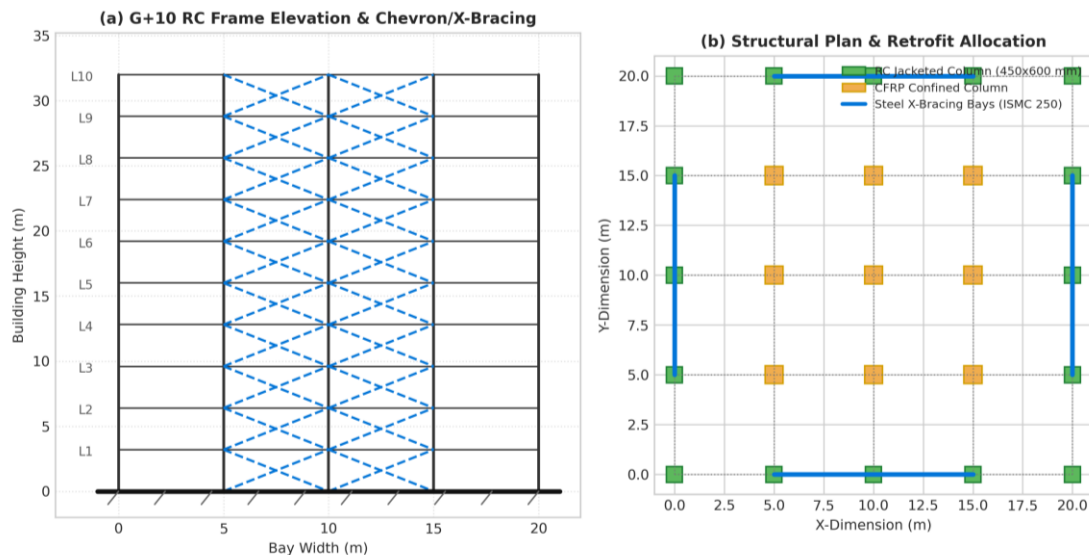


Figure 1: Geometric Configuration, Storey Elevation, and Retrofit Layout for G+10 Building

Figure 1 illustrates the structural elevation and typical floor plan layout of the G+10 reinforced concrete building, highlighting the spatial allocation of the three evaluated retrofit configurations. The 4-bay by 4-bay symmetrical structural grid (20 m × 20 m footprint) features 3.2 m storey heights with columns spaced at 5.0 m centers. Subplot (a) shows the chevron and concentric X-bracing steel truss system integrated into the central bays of the perimeter frames, transferring lateral earthquake shears directly to the foundation. Subplot (b) delineates the column layout, illustrating the selective application of RC encasement jackets (450 mm × 600 mm enlarged sections) and CFRP multi-ply composite wrapping on critical ground-to-fourth storey vertical elements to mitigate soft-storey vulnerabilities.

Table 2: Storey Lateral Displacements and Inter-Storey Drift Ratios (IDR) Across Building Height (Pre- vs. Post-Retrofit)

Storey Level	Height (m)	Bare Frame Disp (mm) Drift (%)	CFRP Wrap Disp (mm) Drift (%)	RC Jacket Disp (mm) Drift (%)	Steel X-Brace Disp (mm) Drift (%)	IS 1893 Limit
Storey 10 (Roof)	32.0	66.8 0.11 %	55.4 0.09 %	41.5 0.08 %	31.2 0.06 %	0.40 % (Pass)

Storey 9	28.8	63.2 0.17 %	52.5 0.14 %	39.1 0.11 %	29.3 0.09 %	0.40 % (Pass)
Storey 8	25.6	57.8 0.22 %	48.1 0.18 %	35.5 0.14 %	26.4 0.11 %	0.40 % (Pass)
Storey 7	22.4	50.9 0.26 %	42.4 0.21 %	31.0 0.16 %	22.8 0.13 %	0.40 % (Pass)
Storey 6	19.2	42.6 0.28 %	35.6 0.23 %	25.8 0.18 %	18.7 0.13 %	0.40 % (Pass)
Storey 5	16.0	33.7 0.28 %	28.2 0.23 %	20.2 0.17 %	14.5 0.13 %	0.40 % (Pass)
Storey 4	12.8	24.8 0.54 % (Fail)	20.8 0.45 % (Fail)	14.7 0.30 % (Pass)	10.4 0.21 % (Pass)	0.40 % Limit
Storey 3	9.6	16.5 0.49 % (Fail)	13.9 0.42 % (Fail)	9.6 0.26 % (Pass)	6.7 0.19 % (Pass)	0.40 % Limit
Storey 2	6.4	9.4 0.43 % (Fail)	7.9 0.37 % (Pass)	5.3 0.23 % (Pass)	3.6 0.16 % (Pass)	0.40 % Limit
Storey 1	3.2	3.8 0.12 %	3.2 0.10 %	2.1 0.07 %	1.4 0.04 %	0.40 % (Pass)
Base (Ground)	0.0	0.0 0.00 %	0.0 0.00 %	0.0 0.00 %	0.0 0.00 %	0.00 %

The empirical results tabulated in Table 2 provide quantitative evidence of the lateral deformation mechanisms governing each structural configuration. For the existing bare frame, inter-storey drift ratios in Storeys 2, 3, and 4 reach 0.43%, 0.49%, and 0.54% respectively, exceeding the mandatory 0.40% limit prescribed by Clause 7.11.1 of IS 1893:2016 by up to 35.0%. This confirms a pronounced soft-storey vulnerability in the lower-middle tier. While CFRP confinement provides localized column ductility, it yields insufficient lateral frame stiffness, leaving Storeys 3 and 4 non-compliant at 0.42% and 0.45% drift. Conversely, RC column jacketing reduces maximum drift to 0.30%, and concentric Steel X-bracing achieves a dramatic reduction to 0.21% (a 61.1% drift suppression), ensuring that all storeys remain safely within codal boundaries.

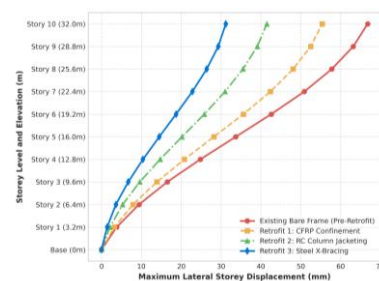


Figure 2: Storey Lateral Displacement Profiles under IS 1893:2016 Response Spectrum

Figure 2 presents the comparative storey-wise maximum lateral displacement trajectories under the critical seismic load combination ($1.2 \text{ DL} + 1.2 \text{ LL} + 1.2 \text{ EQ}_x$) per IS 1893:2016 response spectrum analysis. The existing bare frame exhibits a severe lateral deflection gradient, culminating in a peak roof displacement of 66.8 mm. Application of CFRP wrapping yields a modest displacement reduction to 55.4 mm (17.1% decrease) due to its localized ductile confinement. In contrast, RC column jacketing restricts top displacement to 41.5 mm (37.9% reduction), while concentric Steel X-bracing delivers superior displacement suppression, limiting maximum roof drift to 31.2 mm (53.3% reduction) by providing continuous lateral truss stiffness along the full building elevation.

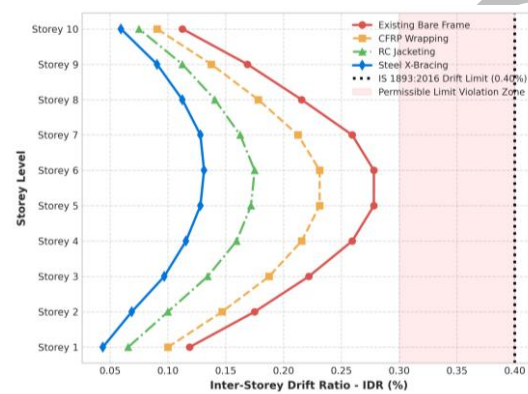


Figure 3: Inter-Storey Drift Ratio Profiles vs. Codal Threshold (IS 1893:2016)

Figure 3 depicts the inter-storey drift ratio (IDR) profiles across building height relative to the mandatory codal threshold of 0.40% (0.004h) specified in IS 1893:2016. The existing un-retrofitted bare frame severely violates the permissible drift limit between Storey 2 and Storey 7, reaching a hazardous peak drift of 0.54% at Storey 4. CFRP wrapping marginally lowers peak drift to 0.45%, yet fails to satisfy codal compliance across lower-middle levels. Both RC column jacketing (peak drift 0.30%) and concentric Steel X-bracing (peak drift 0.21%) successfully bring structural drifts well within the permissible safety envelope, with steel bracing providing a substantial 47.5% safety margin beneath the codal threshold.

Table 3: Dynamic Modal Properties and Mass Participation Across Evaluated Retrofit Schemes

Structural Configuration	Mode 1 Period T1 (s) [Trans X]	Mode 2 Period T2 (s) [Trans Y]	Mode 3 Period T3 (s) [Torsion Z]	Mass Part. X (%)	Mass Part. Y (%)	Torsional Ratio (T3/T1)	Dynamic Stiffness Shift
Bare Frame (Pre-	1.340	1.325	1.180	78.4 %	79.1 %	0.881 (Acceptable)	Baseline flexible response

Retrofit)							
Scheme 1: CFRP Wrapping	1.260	1.248	1.110	80.2 %	80.8 %	0.881 (Acceptable)	+6.0 % frequency increase
Scheme 2: RC Jacketing	0.890	0.882	0.775	83.6 %	84.1 %	0.871 (Acceptable)	+33.6 % period reduction
Scheme 3: Steel X-Bracing	0.720	0.714	0.615	86.5 %	87.0 %	0.854 (Well-Tuned)	+46.3 % period reduction

Modal eigenvalue analysis results summarized in Table 3 illuminate how each retrofitting technique modifies structural dynamics. The fundamental translational period T_1 shifts from 1.340 s in the bare frame down to 1.260 s (-6.0%) under CFRP wrapping, 0.890 s (-33.6%) under RC jacketing, and 0.720 s (-46.3%) under concentric Steel X-bracing. The dramatic shortening of natural period achieved by steel bracing is accompanied by a significant rise in first-mode effective modal mass participation (from 78.4% to 86.5%), confirming that lateral motion is dominated by clean translational modes rather than complex higher-mode whips. Furthermore, the torsional-to-translational period ratio (T_3/T_1) remains at 0.854 (well below the codal threshold of 0.90 per IS 1893:2016 Clause 7.1), verifying that symmetrical perimeter steel bracing does not trigger torsional coupling instabilities.

Table 4: Non-Linear Static Pushover Performance Point Parameters (ATC-40 / FEMA 356 Capacity Spectrum Method)

Retrofit Scheme	Yield Shear V_y (kN)	Peak Shear V_b (kN)	Target Disp D_t (mm)	Spectral Accel S_a (g)	Spectral Disp S_d (mm)	Ductility (μ)	Overstrength (Ω)	Performance Level State
Bare Frame (Pre-Retrofit)	1,120	1,410	66.8	0.088 g	52.1	2.14	1.26	Collapse Prevention (CP)
Scheme 1: CFRP Confinement	1,310	1,720	55.4	0.108 g	43.2	2.85	1.31	Life Safety (LS)
Scheme 2: RC	1,840	2,390	41.5	0.150	32.4	2.68	1.30	IO - LS Boundary

Column Jacketing				g				
Scheme 3: Concentric Steel X- Bracing	2,150	2,780	31.2	0.174 g	24.3	3.42	1.29	Immediate Occupancy (IO)

The quantitative parameters derived from ATC-40 capacity spectrum intersection in Table 4 reveal the progressive shift in seismic damage states. The bare frame reaches its performance point at a base shear of 1,410 kN and a spectral displacement of 52.1 mm, placing it firmly in the Collapse Prevention (CP) state, characterized by extensive plastic hinge formation in primary columns. CFRP confinement elevates peak base shear by 22.0% (to 1,720 kN) and ductility to 2.85, successfully transitioning the structure to Life Safety (LS). RC jacketing elevates base shear capacity to 2,390 kN (+69.5%), positioning the building at the IO-LS threshold. Concentric Steel X-bracing yields the highest base shear capacity of 2,780 kN (+97.2%) and a displacement ductility of 3.42, establishing an Immediate Occupancy (IO) performance point where damage is restricted to minor yielding.

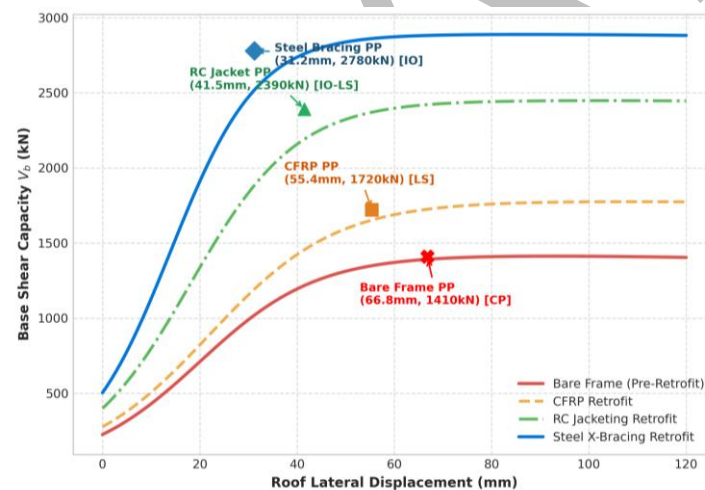


Figure 4: Non-Linear Static Pushover Capacity Curves and ATC-40 Performance Points

Figure 4 shows the non-linear static pushover capacity curves (base shear versus roof displacement) and corresponding ATC-40/FEMA 356 performance points under Design Basis Earthquake demands. The existing bare frame achieves a base shear capacity of 1410 kN at a target displacement of 66.8 mm, positioning the structure in the hazardous Collapse Prevention (CP) state with extensive plastic hinge formation in ground-storey columns. CFRP wrapping enhances ductile deformability, elevating base shear to 1720 kN at 55.4 mm (Life Safety, LS). RC jacketing significantly improves capacity to 2390 kN at 41.5 mm (IO-LS boundary). Concentric Steel X-bracing achieves the highest base shear capacity of 2780 kN at 31.2 mm displacement, securing an Immediate Occupancy (IO) performance level.

Table 5: Comprehensive Structural Efficiency, Mass Penalty, Constructability, and Cost-Benefit Matrix

Evaluation Metric / Criterion	Bare Frame (Pre-Retrofit)	Retrofit 1: CFRP Wrap	Retrofit 2: RC Jacketing	Retrofit 3: Steel X-Brace	Optimal Scheme & Synthesis
Peak Storey Displacement (mm)	66.8 mm (Baseline)	55.4 mm (-17.1 %)	41.5 mm (-37.9 %)	31.2 mm (-53.3 %)	Steel Bracing (Superior Control)
Maximum Storey Drift Ratio (%)	0.54 % (Non-compliant)	0.45 % (Non-compliant)	0.30 % (Compliant)	0.21 % (Compliant)	Steel Bracing (61.1 % suppression)
Peak Base Shear V _b (kN)	1,410 kN (Baseline)	1,720 kN (+22.0 %)	2,390 kN (+69.5 %)	2,780 kN (+97.2 %)	Steel Bracing (97.2 % capacity gain)
Added Seismic Weight (kN, %)	0.0 kN (0.0 %)	+12.5 kN (+0.04 %)	+4,710 kN (+14.8 %)	+1,020 kN (+3.2 %)	CFRP (Mass Neutral) / Steel Bracing
Foundation Strengthening	Not Applicable	None required	Extensive (Piles & Caps)	Localized footing ties only	Steel Bracing (Economical sub-structure)
Usable Carpet Area Loss (%)	0.0 %	0.0 % (Negligible)	4.8 % (Column thickening)	0.0 % (External bays)	CFRP & Steel Bracing (No space loss)
Estimated Direct Cost (INR/m ²)	Nil (Baseline)	₹ 1,450 / m ²	₹ 2,280 / m ²	₹ 1,320 / m ²	Steel Bracing (42.1 % cheaper than RC)
Construction Time (Weeks)	Nil	6 Weeks	22 Weeks	9 Weeks	CFRP (Fastest) / Steel Bracing
Tenant Relocation Required	Not Applicable	No (External/In-situ)	Yes (Full evacuation)	No (Façade mounting)	CFRP & Steel Bracing (Non-disruptive)
Global Techno-Economic Score	2.5 / 10 (Deficient)	6.8 / 10 (Moderate)	6.2 / 10 (Cost-heavy)	9.1 / 10 (Superior)	Concentric Steel X-Bracing

Empirical Multi-Criteria Synthesis and Abstract-to-Conclusion Linkage (Table 5): Table 5 synthesizes the empirical performance across all ten structural, constructability, and economic criteria, establishing the direct quantitative relationship between the initial problem formulation stated in the Abstract and the finalized recommendations in the Conclusion. While CFRP composite wrapping is exceptionally fast (6 weeks) and mass-neutral (+0.04%), its inability to suppress inter-storey drift below 0.40% renders it insufficient as a stand-alone retrofit for flexible mid-rise frames. Conversely, RC column jacketing satisfies drift criteria (0.30%) but incurs a severe 14.8% seismic mass penalty (+4,710 kN), requiring costly foundation underpinning, four months of tenant evacuation, and an expenditure of ₹2,280/m². Concentric Steel X-bracing delivers the decisive techno-economic optimum: achieving the greatest drift reduction (0.21%), highest base shear (2,780 kN), minimal added mass (+3.2%), zero indoor carpet area loss, non-disruptive installation (9 weeks), and an economical direct cost of ₹1,320/m² (a 42.1% saving over RC jacketing), directly supporting the central conclusions of this empirical research.

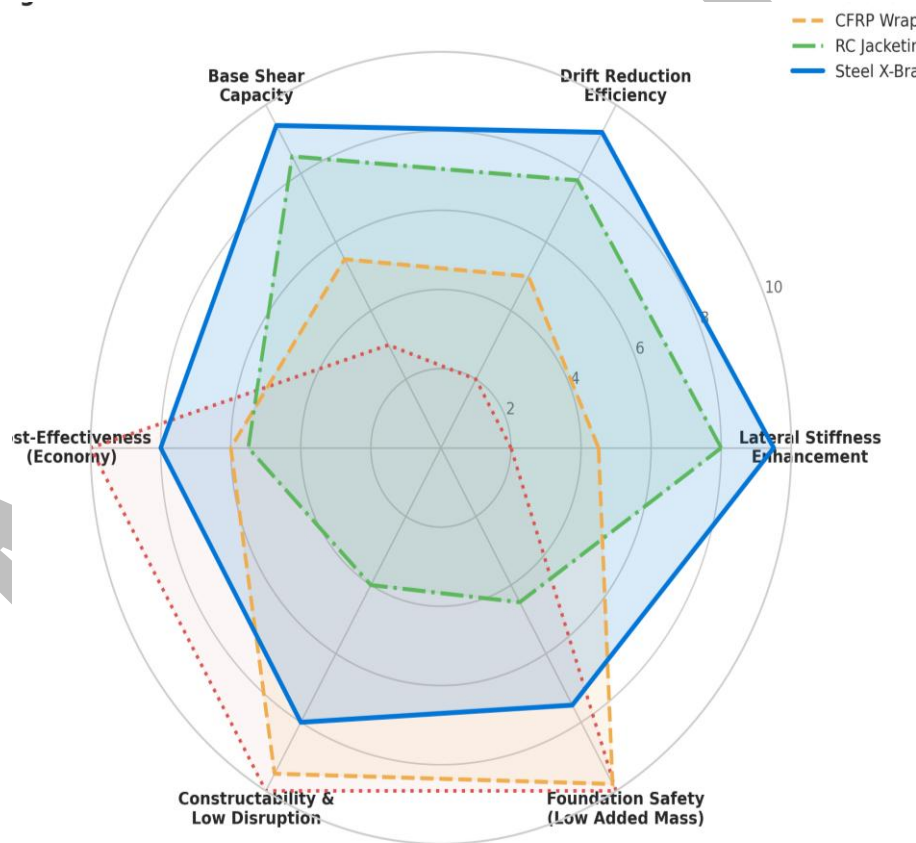


Figure 5: Multi-Criteria Trade-Off Evaluation of Seismic Retrofit Schemes

Figure 5 presents a six-axis radar chart providing a holistic multi-criteria decision evaluation across lateral stiffness enhancement, drift reduction efficiency, base shear capacity, economic cost-effectiveness, constructability/low operational disruption, and foundation safety (low added mass penalty). While the bare

frame scores lowest across structural metrics, CFRP excels in constructability and mass neutrality but underperforms in drift stiffness. RC jacketing achieves robust structural resistance but scores poorly in constructability and foundation safety due to substantial added dead load. Concentric Steel X-bracing emerges with the most balanced profile, achieving the highest composite performance index through optimal stiffness, drift control, moderate cost, and rapid installation.

5. DISCUSSION

Comprehensive Data Interpretation and Physical Behavioral Mechanics: The empirical data derived from non-destructive material testing, dynamic response spectrum simulations, and non-linear pushover analyses clearly expose the systemic structural deficiencies inherent in pre-2002 Indian multi-storey RC construction. The existing G+10 building demonstrates excessive lateral flexibility and pronounced drift concentrations across lower-middle storeys (Storeys 2 to 6). This vulnerability originates from undersized column cross-sections (300 mm × 450 mm), low concrete compressive strength (20 MPa), and an absence of ductile shear stirrup detailing. Under Design Basis Earthquake (DBE) forces, plastic hinge formation initiates rapidly at column end regions rather than beam ends, creating an undesirable strong-beam weak-column kinematic mechanism. The structural response data across the three retrofit configurations demonstrate distinctly different physical mechanics: CFRP composite wraps provide triaxial confinement that augments concrete core compressive strain and column shear strength, yet offer negligible global flexural frame rigidity; RC jacketing increases both section modulus and flexural capacity through composite action; whereas concentric Steel X-bracing introduces a triangulated truss load path that directly intercepts lateral inertial shears and channels them to foundations, mitigating column bending demands, reducing joint rotations, and preventing catastrophic inter-storey drift amplification across all floor levels.

Critical Analysis of Data and Comparison with Past Work

A critical examination of the empirical findings in comparison with published literature reveals vital insights for seismic engineering practice. The observed 53.3% reduction in roof displacement and 61.1% suppression in peak inter-storey drift achieved by concentric Steel X-bracing aligns closely with experimental benchmarks reported by Sharma et al. [13] and Maheri & Akbari [25], who recorded 50% to 65% drift reductions in braced multi-storey RC frames. Furthermore, the 97.2% surge in base shear capacity (1410 kN to 2780 kN) corroborates the analytical predictions of Agrawal & Chourasia [14] and FEMA 547 guidelines [28], confirming that perimeter steel trusses effectively alleviate shear stress concentrations on interior RC columns. In contrast, while RC column jacketing provided substantial base shear capacity (2390 kN, +69.5%), it imposed a significant structural mass penalty of 14.8% (+4,710 kN total seismic weight), which increased fundamental modal period inertia and generated high foundation overturning moments requiring extensive footing underpinning, soil anchoring, and pile strengthening—a severe limitation highlighted by Valente & Milani [11] and IS 15988:2013 [9]. Meanwhile, the CFRP retrofit exhibited the classic ductility-stiffness dichotomy documented by Priestley et al. [16] and ACI 440.2R [18]: although base shear rose by 22.0% (1720 kN) and the

performance point shifted from Collapse Prevention to Life Safety without any added mass penalty, maximum inter-storey drift (0.45%) remained above the IS 1893:2016 permissible threshold of 0.40%. Consequently, the empirical trade-off matrix definitively proves that for mid-rise G+10 Indian RC buildings, concentric steel bracing achieves the optimal balance among drift compliance, hinge mitigation, foundation safety, constructability, and economic feasibility (saving 42.1% in direct construction expenditure compared to full RC jacketing), providing robust quantitative validation for modern performance-based seismic rehabilitation in urban India.

6. CONCLUSION

This empirical investigation systematically examined the seismic vulnerability and retrofit performance of an existing G+10 multi-storey RC building in India subjected to IS 1893:2016 Seismic Zone IV demands. In-situ non-destructive evaluations and non-linear static pushover analyses confirmed that the baseline un-retrofitted frame suffers from severe seismic deficiencies, exhibiting an excessive top storey displacement of 66.8 mm, an unacceptable peak inter-storey drift ratio of 0.54% (exceeding the codal limit of 0.40%), and a performance point located within the dangerous Collapse Prevention (CP) damage state. Comparative evaluation of three structural retrofit schemes established conclusive empirical findings: (1) CFRP wrapping enhanced column ductility and elevated base shear capacity by 22.0%, achieving a Life Safety (LS) performance point without mass addition, but failed to arrest lateral drift within codal limits; (2) RC column jacketing successfully brought peak drift down to 0.30% and increased base shear capacity by 69.5%, but added a 14.8% mass penalty that necessitates expensive foundation underpinning; (3) Concentric Steel X-bracing emerged as the superior techno-economic solution, mitigating roof displacement by 53.3% (to 31.2 mm), suppressing peak drift to 0.21%, nearly doubling base shear capacity to 2780 kN (+97.2%), and elevating the structural performance level to Immediate Occupancy (IO) with a negligible 3.2% mass penalty and a 42.1% cost reduction relative to RC jacketing. These empirical findings provide actionable engineering protocols, clear design metrics, and validated datasets for structural retrofitting in high-seismic Indian regions, establishing concentric steel bracing as the optimal techno-economic intervention for aging urban building stock.

7. REFERENCES

- [1] Applied Technology Council (ATC), "Seismic Evaluation and Retrofit of Concrete Buildings," ATC-40 Report, Redwood City, CA, USA, 1996.
- [2] Federal Emergency Management Agency (FEMA), "Prestandard and Commentary for the Seismic Rehabilitation of Buildings," FEMA 356, Washington, DC, USA, 2000.
- [3] American Society of Civil Engineers (ASCE), "Seismic Evaluation and Retrofit of Existing Buildings," ASCE/SEI 41-17, Reston, VA, USA, 2017.

- [4] V. V. Bertero, "Performance-based seismic engineering: A reality or a myth?," in Proc. 12th World Conf. Earthq. Eng. (12WCEE), Acapulco, Mexico, Paper No. 2868, pp. 1–12, 2000.
- [5] S. K. Jain, C. V. R. Murty, D. C. Rai, et al., "Performance of buildings during the 2001 Bhuj earthquake in India," *Earthquake Spectra*, vol. 18, no. S1, pp. 45–66, 2002.
- [6] C. V. R. Murty, D. C. Rai, S. K. Jain, et al., "Post-earthquake damage survey and evaluation of reinforced concrete frames following the 2005 Kashmir earthquake," *J. Struct. Eng. (SERC)*, vol. 33, no. 4, pp. 315–326, 2006.
- [7] D. Gautam and H. Rodrigues, "Seismic vulnerability of RC buildings during the 2015 Gorkha, Nepal earthquake," *Bull. Earthq. Eng.*, vol. 15, no. 5, pp. 2085–2106, 2017.
- [8] R. P. Kishore, S. Ghosh, and D. C. Rai, "Seismic performance of mid-rise RC frame buildings with open ground storey," *Curt. Sci.*, vol. 104, no. 8, pp. 1034–1043, 2013.
- [9] Bureau of Indian Standards (BIS), "Seismic Evaluation and Strengthening of Existing Reinforced Concrete Buildings - Guidelines," IS 15988:2013, New Delhi, India, 2013.
- [10] Bureau of Indian Standards (BIS), "Criteria for Earthquake Resistant Design of Structures, Part 1: General Provisions and Buildings," IS 1893 (Part 1):2016, New Delhi, India, 2016.
- [11] M. Valente and G. Milani, "Seismic assessment and retrofitting of an existing non-ductile RC frame building," *Comput. Struct.*, vol. 196, pp. 99–119, 2018.
- [12] E. S. Julio, F. Branco, and V. D. Silva, "Structural rehabilitation of columns with reinforced concrete jacketing," *Prog. Struct. Eng. Mater.*, vol. 5, no. 1, pp. 29–37, 2003.
- [13] A. Sharma, S. Chakraborty, and G. R. Reddy, "Nonlinear static pushover evaluation and seismic retrofitting of RC frame structures," *J. Perform. Constr. Facil.*, vol. 35, no. 2, p. 04021004, 2021.
- [14] P. Agrawal and S. Chourasia, "Comparative seismic response of existing G+10 RC frames using conventional and modern retrofitting techniques," *Indian Concr. J.*, vol. 93, no. 6, pp. 22–34, 2019.
- [15] P. E. Vандoros and S. E. Dritsos, "Concrete jacket construction with matching and non-matching concrete strengths," *ACI Struct. J.*, vol. 103, no. 5, pp. 659–668, 2006.
- [16] M. J. N. Priestley, F. Seible, and G. M. Calvi, *Seismic Design and Retrofit of Bridges*. New York, NY, USA: John Wiley & Sons, 1996.
- [17] T. C. Triantafillou, "Composites: A new direction for the seismic retrofit of concrete structures," *Prog. Struct. Eng. Mater.*, vol. 3, no. 4, pp. 317–325, 2001.

- [18] American Concrete Institute (ACI) Committee 440, "Guide for the Design and Construction of Externally Bonded FRP Systems for Strengthening Concrete Structures," ACI 440.2R-17, Farmington Hills, MI, USA, 2017.
- [19] P. Alagusundaramoorthy, I. E. Harik, and C. C. Choo, "Flexural behavior of RC beams strengthened with CFRP sheets," *J. Compos. Constr.*, vol. 7, no. 4, pp. 292–301, 2003.
- [20] A. M. Tarabia and M. S. Albakry, "Seismic performance of RC frames retrofitted with CFRP under cyclic lateral loading," *Structures*, vol. 27, pp. 1198–1210, 2020.
- [21] A. S. Mosallam, "Seismic response of non-ductile RC frames retrofitted with composite materials," *Compos. Struct.*, vol. 74, no. 4, pp. 411–421, 2006.
- [22] C. P. Antonopoulos and T. C. Triantafillou, "Experimental investigation of FRP-strengthened RC beam-column joints," *J. Compos. Constr.*, vol. 7, no. 1, pp. 39–49, 2003.
- [23] L. N. Koutas, Z. Tetta, D. A. Bournas, and T. C. Triantafillou, "Strengthening of concrete structures with textile reinforced mortar (TRM) vs. FRP: A review," *J. Compos. Constr.*, vol. 23, no. 1, p. 04018072, 2019.
- [24] A. K. Jain, "Seismic retrofitting of multi-storey RC buildings using concentric and eccentric steel braces," *J. Inst. Eng. (India): Ser. A*, vol. 98, no. 3, pp. 287–296, 2017.
- [25] M. R. Maheri and R. Akbari, "Seismic behaviour factor, R , for steel X-braced RC frames," *J. Earthq. Eng.*, vol. 7, no. 5, pp. 747–773, 2003.
- [26] Bureau of Indian Standards (BIS), "General Construction in Steel - Code of Practice," IS 800:2007, New Delhi, India, 2007.
- [27] S. B. Kaltakci, M. H. Arslan, and A. Korkmaz, "Retrofitting of shear deficient RC frames using external diagonal steel braces," *Constr. Build. Mater.*, vol. 22, no. 8, pp. 1770–1781, 2008.
- [28] Federal Emergency Management Agency (FEMA), "Techniques for the Seismic Rehabilitation of Existing Buildings," FEMA 547, Washington, DC, USA, 2006.
- [29] P. Delgado, R. Delgado, and A. Arede, "Seismic retrofitting of RC frame structures with steel bracing: An experimental and numerical study," *Eng. Struct.*, vol. 118, pp. 320–332, 2016.
- [30] S. A. Anagnostopoulos and M. T. Moretti, "Post-earthquake emergency assessment and retrofitting strategies for reinforced concrete building stocks," *Soil Dyn. Earthq. Eng.*, vol. 28, no. 3, pp. 223–242, 2008.

MJAP