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Seismic Strengthening of a Deficient Pre-Code G+4 RC Frame by Concrete Column Jacketing: An ETABS Evaluation under IS 1893:2016

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Abstract

A large fraction of India's residential reinforced concrete (RC) building stock was constructed before modern seismic provisions, leaving frames structurally deficient under current codes. This study evaluates concrete column jacketing as a practical retrofit technique for a representative deficient G+4 RC frame located in Pune, Maharashtra. The building has a 20 m x 12 m plan, a total height of 15.5 m, and 300 mm x 300 mm external columns typical of pre-code construction. The original frame was modelled in ETABS v23.2.0 and analysed as an ordinary moment-resisting frame (OMRF, R = 3.0) under IS 1893 (Part 1):2016 equivalent-static seismic loading for Zone III, Soil Type II (Z = 0.16). The original frame violates the IS 1893:2016 inter-storey drift limit of 0.004h at four of five storeys in both principal directions, reaching maximum drift ratios of 0.006474 (X) and 0.005963 (Y), approximately 1.6 times the permissible value. A 75 mm M30 reinforced-concrete jacket was then applied to all columns, enlarging external columns from 300 x 300 mm to 450 x 450 mm, and the strengthened building was re-analysed as a special moment-resisting frame (SMRF, R = 5.0). Post-jacketing, maximum drift fell to 0.002931 (X) and 0.003086 (Y), reductions of 54.7% and 48.2% respectively, achieving full code compliance at every storey. The fundamental period reduced from 0.717 s to 0.613 s, and the dead load increased by only 6.9%. Concrete column jacketing is an effective, practical intervention for restoring seismic code compliance in pre-code Indian RC frames.

Keywords: Seismic Strengthening; Column Jacketing; RC Frame Retrofitting; IS 1893:2016; Inter-Storey Drift; ETABS; Ordinary Moment-Resisting Frame; Special Moment-Resisting Frame

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Introduction

Reinforced concrete (RC) frame construction dominates the residential and institutional building stock across India. A large fraction of this stock was erected between the 1970s and the 1990s, before the introduction of IS 1893:2002 and the current IS 1893 (Part 1):2016. Those buildings were proportioned primarily for gravity loads, with little allowance for earthquake forces, and typically lack the ductile column detailing, joint confinement, and lateral load-resisting capacity that modern seismic design demands [1], [2]. The consequences of this deficiency have been documented repeatedly: the Latur (1993), Bhuj (2001), Sikkim (2011), and Nepal-India border (2015) earthquakes each caused widespread partial or total collapse of pre-code RC frames. The Bhuj event alone caused the partial or total collapse of more than 230,000 buildings, the majority of them RC frames with under-confined columns and weak beam-column joints. Such events have established the seismic strengthening of existing RC buildings as a public-safety priority, particularly in Seismic Zones III, IV, and V of IS 1893:2016.

The specific problem motivating this study is a deficient G+4 RC frame building located in Pune, Maharashtra, which falls in Seismic Zone III with zone factor Z = 0.16 and Soil Type II. The building, with a plan of 20 m x 12 m and a total height of 15.5 m, was constructed with 300 mm x 300 mm external columns, a size below the minimum for ductile detailing under IS 13920:2016. Analysed in ETABS v23.2.0 as an ordinary moment-resisting frame (OMRF, R = 3.0) under the equivalent-static provisions of IS 1893:2016, the original frame violates the code inter-storey drift limit of 0.004h at four of its five storeys in both principal directions. The maximum drift reaches 0.006474 in the X-direction and 0.005963 in the Y-direction, about 1.6 times the allowable value, confirming that the building requires seismic strengthening before it can be considered code-compliant.

Several strengthening techniques are available for pre-code RC frames: concrete jacketing of columns, steel bracing, RC shear walls, and fibre-reinforced polymer wrapping. Concrete jacketing is selected for this study for three reasons rooted in the specific deficiency of the building and in Indian construction practice. First, it directly addresses the source of the deficiency by enlarging and confining the undersized columns, increasing both lateral stiffness and ductility. Second, it uses ordinary concrete, reinforcement, and formwork that are readily available across Indian cities. Third, the jacket can be applied storey by storey while the building remains occupied, minimising disruption. The IITK-GSDMA (2007) guidelines, which govern the seismic evaluation and strengthening of existing Indian buildings, recognise these attributes and recommend concrete jacketing as a member-level intervention for column-deficient frames [3].

The aim of this study is to evaluate and improve the seismic performance of the deficient G+4 RC frame by concrete jacketing of its columns, and to assess the effectiveness of the technique through ETABS analysis conforming to IS 1893:2016. The specific objectives are: (1) to model the deficient frame as an OMRF with R = 3.0 under Zone III, Soil Type II loading and confirm the storeys that fail the drift limit; (2) to apply a 75 mm M30 reinforced-concrete jacket to the deficient columns and re-analyse the strengthened building as a special moment-resisting frame (SMRF, R = 5.0); (3) to compare modal periods, design base shear, inter-storey drift, lateral displacement, and structural weight for the original and jacketed configurations; and (4) to assess whether jacketing, together with the accompanying reclassification from OMRF to SMRF, restores compliance with the IS 1893:2016 drift limit of 0.004h.

The contribution of this work is practical rather than theoretical. Although concrete jacketing is well established at the level of individual members, relatively few studies quantify, storey by storey, how column jacketing modifies the full seismic response of a complete, representative pre-code Indian frame under the current IS 1893:2016 provisions. This study addresses that gap by providing a storey-by-storey quantification of jacketing effectiveness for a frame that is directly representative of the large pre-code residential building stock of Pune and similar Indian Zone III cities.

Literature Review

The published work on seismic strengthening of reinforced concrete frames falls into three principal technique groups: column and joint jacketing experiments; global retrofit and comparison studies; and bracing and shear-wall alternatives. A fourth group, FRP wrapping, is noted for completeness [4], [5].

Column and joint jacketing experiments. The foundational experimental evidence for concrete jacketing as a member-level intervention was established by Ersoy, Tankut, and Suleiman [6], who tested columns under axial and cyclic combined loading and showed that jackets applied to unloaded columns restored near-monolithic strength, stiffness, and energy dissipation. Rodriguez and Park [7] extended this finding to previously damaged columns and confirmed large gains in flexural strength and ductility that exceeded the original undamaged performance. Alcocer and Jirsa [8] demonstrated that jacketing non-ductile beam-column connections relocated inelastic action away from the joint and restored shear strength and stiffness under bidirectional cyclic loading. The importance of interface preparation was quantified by Júlio, Branco, and Silva [9], who reported a 20 to 30 per cent capacity loss when the substrate was inadequately prepared relative to a monolithic equivalent, and by Vandoros and Dritsos [10], who identified a roughened substrate with drilled-and-anchored dowels as the most effective field-applicable detail. Karayannis, Chalioris, and Sirkelis [11] showed that 50 to 60 mm jackets raised exterior joint shear strength by 40 to 55 per cent and shifted failure from brittle joint-shear to ductile beam flexural hinging. In the Indian context, Kaliyaperumal and Sengupta [12] combined slant-shear interface tests with column tests at IIT Madras and provided detailing recommendations consistent with IS 13920, directly supporting jacketing in an IS-code framework. Bousias, Spathis, and Fardis [5] compared jacketing against FRP wrapping on columns with the lap-spliced smooth reinforcement common in pre-code Indian buildings and found that concrete jackets produced the largest and most uniform improvements in strength, stiffness, and deformation capacity. Lampropoulos and Dritsos [13] and Achillopoulou, Pardalakis, and Karabinis [14] confirmed through numerical and experimental studies that, with adequate interface treatment, composite action approaching monolithic behaviour can be achieved even in squat, shear-sensitive members. Thermou, Pantazopoulou, and Elnashai [15] provided the analytical basis for the full-composite single-section idealisation by developing a dual-section model that quantifies how interface slip governs stiffness and strength gain.

Global retrofit and comparison studies. Sugano [16] reviewed rehabilitation techniques after the 1995 Kobe earthquake and distinguished local member-level interventions such as jacketing from global system additions such as shear walls. Thermou and Elnashai [2] formalised this distinction by classifying interventions according to their effect on stiffness, strength, and ductility, and showed that each class addresses a different deficiency type. Türker, Türkmen, and Akpinar [17] quantified the performance-versus-weight trade-off, reporting that shear walls achieved approximately 92 per cent drift reduction against 78 per cent for X-bracing but with a 20 per cent dead-load penalty against only 4 per cent for bracing. Mazza [18] concluded that, under near-fault excitation, conventional bracing remains the most cost-effective global retrofit for buildings outside very-high seismic zones.

Bracing and shear-wall alternatives. Maheri and Sahebi [19] established that steel X-bracing converts a flexure-dominated frame into a shear-dominated lateral system with approximately four times the lateral strength of the unbraced frame, making the brace-to-frame connection the governing detail. Massumi and Tasnimi [20] confirmed that a properly anchored connection is essential to realise the full brace strength. Youssef, Ghaffarzadeh, and Nehdi [21] reported approximately 90 per cent inter-storey drift reduction and only marginal dead-load increase for concentric internal X-bracing under cyclic loading. TaeWan, Lee, and Choi [22] showed that eccentric bracing provides ductility advantages but greater fabrication complexity, positioning concentric X-bracing as the conventional default. Khoukhi, Faqih, and Bouchair [23] studied braced RC frames under IS 1893:2016 in Zone IV and reported drift reductions of 78 per cent for X-bracing, 65 per cent for V-bracing, and 60 per cent for chevron arrangement. Strafaci, Marini, and Belleri [24] extended the shear-wall approach to deficient existing walls, demonstrating that high-performance RC jackets combining high-strength concrete with steel fibres improve seismic response through material upgrading.

Although concrete jacketing is well established at the level of individual members and joints, relatively few studies quantify, storey by storey, how much column jacketing improves the seismic response of a complete, representative pre-code Indian building under the current IS 1893:2016 provisions. The present study addresses this gap by applying concrete jacketing to a deficient G+4 RC frame in Pune (Zone III) and reporting its full performance vector against the code drift limit.

Methodology

Building Description

The structure studied is a regular G+4 reinforced concrete frame of residential occupancy, located in Pune, Maharashtra. Its plan dimensions are 20.0 m x 12.0 m and its total height above plinth is 15.5 m (Table 3.1). The ground storey is 3.5 m high and the four upper storeys are each 3.0 m. The frame is laid out on a 4 x 5.0 m grid in the X-direction and a 3 x 4.0 m grid in the Y-direction, giving 20 columns per floor with no soft storey, re-entrant corner, or mass irregularity. The building is representative of pre-code residential construction in Pune in which column sizing was governed by gravity loads alone.

Table 1. *Building geometry*

Parameter	Value
Plan dimensions	20.0 m x 12.0 m
Number of storeys	5 (Ground + 4 upper floors)
Ground-storey height	3.5 m
Typical-storey height	3.0 m
Total height above plinth	15.5 m
Bays in X-direction	4 bays at 5.0 m
Bays in Y-direction	3 bays at 4.0 m
Columns per floor	20

Occupancy	Residential (I = 1.0)
Location	Pune, Maharashtra (Zone III)
Soil type	II (Medium)

Materials and Deficient Member Sizes

Material grades for the original frame and for the strengthening intervention are given in Table 3.2. The original frame uses M25 concrete (f_{ck} = 25 N/mm², E_c = 25,000 N/mm²) with Fe500 reinforcing steel (f_y = 500 N/mm², E_s = 200,000 N/mm²). The jacket concrete is M30 (f_{ck} = 30 N/mm², E_c = 27,386 N/mm²).

Table 2. *Material properties*

Component	Grade	Characteristic strength (N/mm²)	Modulus of elasticity (N/mm²)
Concrete (original frame)	M25	f_{ck} = 25	E_c = 25,000
Concrete (jacket)	M30	f_{ck} = 30	E_c = 27,386
Reinforcing steel	Fe500	f_y = 500	E_s = 200,000

The original member sizes are listed in Table 3.3. External columns are 300 mm x 300 mm and internal columns 350 mm x 350 mm, both in M25 concrete. Primary beams in the X-direction are 230 mm x 400 mm, secondary beams in the Y-direction 230 mm x 350 mm, and all floor slabs are 125 mm thick. The 300 mm x 300 mm external column dimension is below the minimum required for ductile detailing under IS 13920:2016, so the original configuration is classified as an ordinary moment-resisting frame (OMRF) with response reduction factor R = 3.0.

Table 3. *Original (deficient) member sizes*

Member	Section	Material
External columns	300 x 300 mm	M25
Internal columns	350 x 350 mm	M25
Primary beams (X)	230 x 400 mm	M25
Secondary beams (Y)	230 x 350 mm	M25
Slab (all floors)	125 mm thick	M25

Loading and Load Combinations

Gravity loads are computed in accordance with IS 875 Parts 1 and 2. Dead loads include member self-weight (computed by ETABS), floor finish at 1.5 kN/m², terrace waterproofing at 2.0 kN/m², partition allowance at 1.0 kN/m², external wall loads at 12.0 kN/m, and internal wall loads at 6.0 kN/m. Live loads are 2.0 kN/m² on residential floors and 1.5 kN/m² on the inaccessible terrace.

Seismic loading follows IS 1893 (Part 1):2016. The building is located in Seismic Zone III (Z = 0.16), on Soil Type II (Medium), with importance factor I = 1.0 and 5 per cent damping. The response reduction factor is R = 3.0 for the original OMRF and R = 5.0 for the strengthened SMRF. The design horizontal seismic coefficient is computed as:

$$A_h = (Z / 2) \times (I / R) \times (S_a / g)$$

where S_a/g is read from the IS 1893:2016 design spectrum for the appropriate period and soil type. The design base shear is $V_B = A_h \times W$, where the seismic weight W is the full dead load plus 25 per cent of the live load per IS 1893:2016 Clause 7.4.2. Thirteen load combinations are generated per IS 1893:2016 Clause 6.3, including 1.5(DL + LL); 1.2(DL + LL +/- EQx +/- EQy); 1.5(DL +/- EQx +/- EQy); and 0.9 DL +/- 1.5 EQ.

ETABS Modelling

Both configurations are built and analysed in ETABS v23.2.0 (Computers and Structures, Inc.). The model grid, five storeys, materials, frame sections, and 125 mm area sections are defined first. A rigid diaphragm is assigned at each floor level and all column bases are fixed. Dead, live, and seismic load patterns and the thirteen load combinations are applied. The equivalent-static method governs design forces and storey drifts for this regular building of 15.5 m height in Zone III; a modal analysis is run concurrently to extract natural periods. The original deficient model is analysed first, then the concrete-jacketed model is created from it and re-analysed under identical loading so that changes in response are attributable solely to the jacketing.

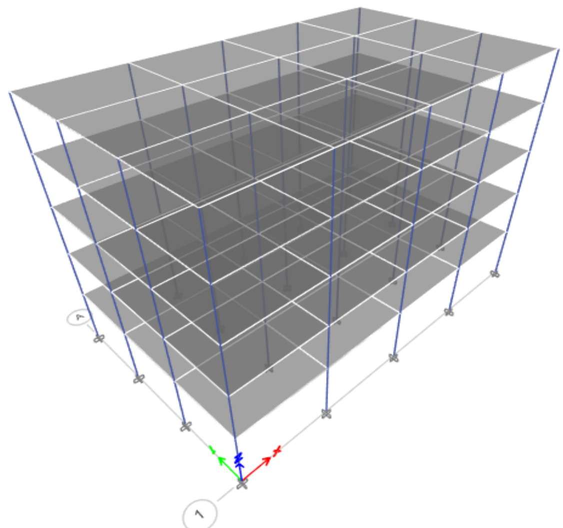

Fig. 1. *ETABS v23.2.0 three-dimensional model of the deficient G+4 RC frame.*

Concrete Jacketing Scheme

Concrete jacketing is applied to all columns of the deficient frame. The strengthening configuration is given in Table 3.4. A 75 mm thick M30 reinforced-concrete jacket is cast around every column: external columns increase from 300 mm x 300 mm to 450 mm x 450 mm and internal columns from 350 mm x 350 mm to 500 mm x 500 mm. New Fe500 longitudinal bars are anchored to the existing reinforcement through drilled-and-epoxied dowels, and transverse ties are provided at 75 mm spacing over the confined end zones in accordance with IS 13920:2016. The existing column surface is roughened before casting. In the ETABS model each jacketed column is represented as a single enlarged section, assuming full composite action between the original core and the new jacket, consistent with the interface preparation specified. The ductile confinement provided by the jacket satisfies the detailing requirements of IS 13920:2016, which reclassifies the frame as a special moment-resisting frame with R = 5.0.

Table 4. *Strengthening configuration*

Parameter	Value
Jacket thickness	75 mm all round
Jacket concrete	M30
External columns	300 x 300 to 450 x 450 mm
Internal columns	350 x 350 to 500 x 500 mm
Jacket reinforcement	Fe500 longitudinal bars + ties at 75 mm
Response reduction, R	5.0 (SMRF)

Acceptance Criterion

Compliance is assessed against the inter-storey drift limit of IS 1893:2016 Clause 7.11.1, which requires that the inter-storey drift ratio (delta/h) shall not exceed 0.004 under the design earthquake with a load factor of 1.0. Drift values are extracted from the ETABS storey response tables for both the X and Y earthquake directions and compared storey by storey against this limit for both the original and jacketed configurations.

Results and Discussion

The five-response metrics (modal periods, design base shear, inter-storey drift, lateral displacement, and structural/seismic weight) are compared below for the original deficient frame (OMRF, R = 3.0) and the concrete-jacketed frame (SMRF, R = 5.0). All values are extracted from ETABS v23.2.0. Compliance with the IS 1893:2016 inter-storey drift limit of 0.004 times the storey height (Clause 7.11.1) is the primary acceptance criterion.

Modal Periods

The fundamental period of vibration is an inverse indicator of lateral stiffness: a stiffer structure vibrates at a shorter period. Table 5 lists the first three modal periods for both configurations.

Table 5. *First three modal periods (seconds)*

Mode	Original	Concrete Jacketing
Mode 1	0.717	0.613
Mode 2	0.688	0.584
Mode 3	0.640	0.536

Concrete jacketing reduces the fundamental period by 14.5 per cent (from 0.717 s to 0.613 s), with reductions of 15.1 per cent and 16.3 per cent in the second and third modes respectively. The consistent shortening across all three modes confirms that enlarging every column from 300 x 300 mm to 450 x 450 mm (external) and from 350 x 350 mm to 500 x 500 mm (internal) has measurably increased the global lateral stiffness of the frame.

Design Base Shear and Structural Weight

Table 6 presents the design seismic base shear ($V_B = A_h W$, IS 1893:2016 Cl. 7.6) and the dead load for each configuration.

Table 6. *Design base shear and dead load*

Quantity	Original (R = 3.0)	Concrete Jacketing (R = 5.0)
$V_{B,x}$ (kN)	601.5	449.5
$V_{B,y}$ (kN)	626.8	471.8
Dead load (kN)	11,155	11,926

Despite the greater mass of the jacketed frame, the design lateral base shear falls by about 25 per cent in both directions. This outcome follows directly from the R = 3.0 to R = 5.0 upgrade: since the design horizontal coefficient A_h is inversely proportional to R, increasing R from 3.0 to 5.0 reduces the design force by a factor of 0.6, more than offsetting the modest increase in seismic weight. The structural weight data are tabulated in Table 7 below.

Table 7. *Structural and seismic weight (kN)*

Configuration	R	Frame type	Dead load	Seismic weight	Change in DL vs original
Original	3.0	OMRF	11,155	11,891	-
Concrete Jacketing	5.0	SMRF	11,926	12,663	+6.9 %

The 75 mm M30 jacket increases the dead load by only 6.9 per cent, a modest penalty that reflects the relatively small concrete volume of a column-only intervention. This marginal increase in mass allows the existing foundation to carry the additional load without major modification in typical cases, which is one of the practical advantages of column jacketing over heavier global-system alternatives such as shear walls.

Inter-Storey Drift

The maximum inter-storey drift ratio in each direction is the governing compliance check against IS 1893:2016 Clause 7.11.1.

Table 8. *Maximum inter-storey drift (Δ/h)*

Direction	Original	Concrete Jacketing	IS limit	Reduction
X	0.006474 (FAIL)	0.002931 (PASS)	0.004	54.7 %
Y	0.005963 (FAIL)	0.003086 (PASS)	0.004	48.2 %

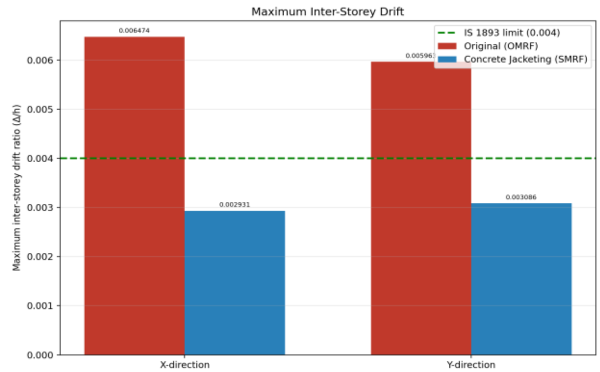

Fig. 2. *Maximum inter-storey drift, original vs jacketed (IS 1893:2016 limit 0.004).*

The original frame exceeds the drift limit in both directions, with peak values about 1.6 times the allowable. After jacketing, the maximum drift falls to 0.002931 in X and 0.003086 in Y, reductions of 54.7 per cent and 48.2 per cent respectively, bringing the frame within the IS 1893:2016 limit in both directions with a margin of about 25 per cent below the allowable value.

Storey-wise Drift

Tables 9 and 10 show the storey-by-storey drift profiles in each direction.

Table 9. *Storey-wise drift, X-direction (Δ/h)*

Storey	Original	Concrete Jacketing	Limit
Terrace	0.003297	0.002193	0.004
3rd Floor	0.004948 (FAIL)	0.002700	0.004
2nd Floor	0.006123 (FAIL)	0.002931	0.004
1st Floor	0.006474 (FAIL)	0.002717	0.004
Ground	0.004592 (FAIL)	0.001414	0.004

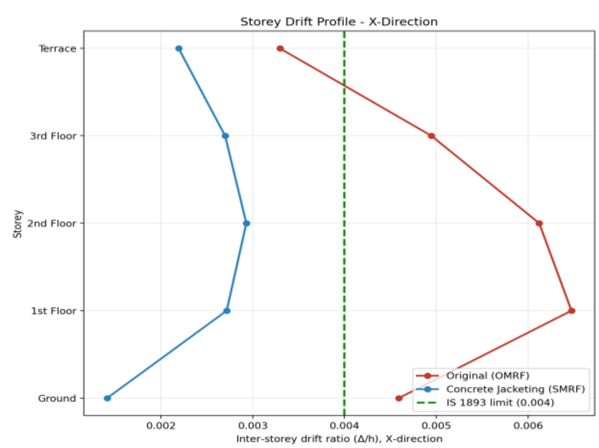

Fig. 3. *Storey-wise inter-storey drift profile, X-direction.*

Table 10. *Storey-wise drift, Y-direction (Δ/h)*

Storey	Original	Concrete Jacketing	Limit
Terrace	0.003569	0.002335	0.004
3rd Floor	0.005168 (FAIL)	0.002884	0.004
2nd Floor	0.005930 (FAIL)	0.003086	0.004
1st Floor	0.005963 (FAIL)	0.002767	0.004
Ground	0.004218 (FAIL)	0.001430	0.004

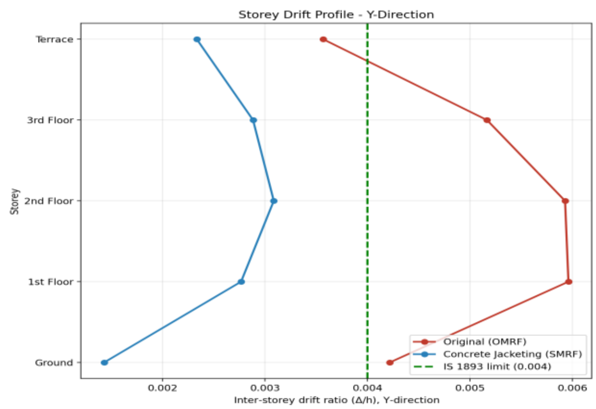

Fig. 4. *Storey-wise inter-storey drift profile, Y-direction.*

In the original frame, four of the five storeys fail the limit in both principal directions: Ground, 1st Floor, 2nd Floor, and 3rd Floor. The terrace level alone satisfies the limit in the original configuration. The maximum drift occurs at the first floor in each direction (0.006474 in X, 0.005963 in Y), which is the characteristic soft-lower-storey response of a bare moment frame with inadequate column stiffness. After jacketing, every storey in both directions satisfies the limit. The post-jacketing drift profile is also more uniform over the building height, peaking near 0.003 at mid-height and falling at the ground storey; this uniformity is the expected consequence of stiffening all columns consistently from foundation to roof.

Lateral Displacement

Table 11 gives the storey lateral displacements under the X and Y earthquake cases.

Table 11. *Storey lateral displacement (mm)*

Storey	EQx Original	EQx Jacketed	EQy Original	EQy Jacketed
Ground	16.07	4.95	14.76	5.00
1st Floor	35.48	13.09	32.65	13.31
2nd Floor	53.85	21.88	50.44	22.56
3rd Floor	68.69	29.74	65.95	31.21
Terrace	78.11	36.08	76.66	38.22

The maximum terrace displacement of 78.1 mm in the X-direction is reduced to 36.1 mm by jacketing, a reduction of about 54 per cent. In the Y-direction, the corresponding reduction is from 76.7 mm to 38.2 mm, about 50 per cent. These reductions are consistent with the drift reductions and confirm the increase in global lateral stiffness. The post-jacketing roof-level displacements of about 36 to 38 mm are comfortable for the non-structural elements of a residential building.

Discussion

The results consistently show that concrete jacketing of the deficient columns is effective for this building under IS 1893:2016 loading. The fundamental period, the maximum inter-storey drift, and the terrace displacement all decrease by about 14 to 55 per cent, and the frame transitions from widespread drift violations to full code compliance with a comfortable margin.

The drift reduction reflects two simultaneous effects and not the stiffness gain alone. The original frame is assessed as an OMRF with R = 3.0, and the jacketed frame as an SMRF with R = 5.0. The R upgrade from 3.0 to 5.0 is justified because the 75 mm M30 jacket, with Fe500 longitudinal bars and ties at 75 mm spacing over the confined end zones, supplies the ductile confinement that IS 13920:2016 requires of a special moment-resisting frame; in addition, jacketing enlarges the column sections without altering the beams, which raises the column-to-beam flexural-strength ratio and thereby helps satisfy the strong-column/weak-beam requirement of IS 13920:2016. The reported reductions of 54.7 per cent in X and 48.2 per cent in Y therefore represent the realistic engineering outcome when a pre-code building of this type is retrofitted to IS 13920:2016 ductile-detailing standards through concrete jacketing: the combined benefit of greater stiffness and the lower design force permitted for a properly detailed SMRF.

Conclusions

1. The original deficient G+4 RC frame, modelled with 300 mm x 300 mm external columns as an ordinary moment-resisting frame (R = 3.0), fails the IS 1893:2016 inter-storey drift limit of 0.004h at four of its five storeys in both principal directions. The maximum drift of

0.006474 in X and 0.005963 in Y is approximately 1.6 times the allowable value, confirming that the building requires seismic strengthening.

2. Concrete jacketing of all columns using a 75 mm M30 reinforced-concrete jacket brings every storey within the IS 1893:2016 drift limit in both directions. The maximum inter-storey drift is reduced to 0.002931 in X and 0.003086 in Y, reductions of 54.7 per cent and 48.2 per cent respectively, leaving a margin of approximately 25 per cent below the allowable value.

3. Jacketing measurably increases the global lateral stiffness of the frame. The fundamental period decreases by 14.5 per cent from 0.717 s to 0.613 s, and the maximum lateral displacement at terrace level is reduced by approximately 50 per cent, from 78.1 mm to 36.1 mm in the X-direction and from 76.7 mm to 38.2 mm in the Y-direction.

4. The dead load increase associated with jacketing is only 6.9 per cent (11,155 kN to 11,926 kN), a modest penalty that the existing foundation can typically carry without major modification, reinforcing the practical advantage of this technique over global-system interventions that impose a substantially larger mass.

5. The reclassification of the frame from an ordinary moment-resisting frame (R = 3.0) to a special moment-resisting frame (R = 5.0), justified by the ductile detailing of the jacketed columns per IS 13920:2016, lowers the design base shear from 601.5 kN to 449.5 kN in X and from 626.8 kN to 471.8 kN in Y despite the increased mass, because A_h is inversely proportional to R; this reduction in design lateral force is an additional benefit of a code-compliant jacketing retrofit.

6. For deficient pre-code RC frames of this type in Indian Seismic Zone III (Z = 0.16, Soil Type II), concrete jacketing of the columns is an effective and practical strengthening technique: it restores full code compliance under IS 1893:2016, employs familiar materials and construction methods, adds little dead load, and can be executed storey by storey while the building remains occupied.

Future work may extend the study through non-linear pushover and time-history analyses using recorded Indian ground motions, and through a parametric comparison with steel bracing, RC shear walls, and FRP wrapping across a range of building heights and seismic zones.

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