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**Case Report**

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# Comparative seismic evaluation of RC column jacketing and CFRP wrapping for an existing low-rise RC building in Nepal

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## Abstract

Much of the reinforced concrete (RC) building stock of Nepal was built without seismic detailing or reliable control of concrete quality, and replacing it is not realistic. Two retrofit schemes were therefore applied to the same deficient building, so that performance and cost could be judged on the same basis. A four-storey (G+3) residential RC frame in Lalitpur was surveyed, rebound-hammer tested and modelled in ETABS. The mean in-situ strength of about 14.9 MPa gave M15 concrete with Fe415 reinforcement in the as-built model. Under equivalent static seismic loading, sixteen columns failed the axial-flexure check and the ratio of maximum to minimum storey displacement reached 3.13, against a torsional irregularity limit of 1.5. The schemes were RC jacketing of those columns from 230 × 230 to 430 × 430 mm with M25 concrete and a new Fe500 cage, and single-ply carbon fibre reinforced polymer (CFRP) confinement. Jacketing took the axial capacity of the reference column from 377.5 to 2514.4 kN, multiplied storey stiffness by three to five, cut the maximum roof displacement by 75% and brought every torsional irregularity ratio below 1.5. CFRP took the same capacity to 817.8 kN and cleared the member-level deficiency, but storey stiffness stayed within about 1% of the as-built value, so displacement, drift and torsional response barely moved. At indicative 2026 Kathmandu rates CFRP cost NPR 437,169 against NPR 823,923 for jacketing. Only jacketing satisfies every code check on this building, though CFRP remains attractive where added mass or occupant disruption governs.

**Keywords** Seismic retrofitting · RC column jacketing · CFRP confinement · Torsional irregularity · Storey drift · ETABS

## Acknowledgements

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# 1 Introduction

Nepal lies along the convergent boundary between the Indian and Eurasian plates, and the seismic hazard along the Himalayan arc is among the highest in South Asia. The 2015 Gorkha earthquake (M 7.8) damaged or destroyed several hundred thousand buildings and showed how badly the existing urban RC stock performs under strong shaking. Much of that stock was built incrementally by owners and local masons with no engineering supervision, and even the engineered buildings were usually proportioned for gravity loads, with lateral resistance treated as an afterthought.

The same deficiencies recur: low grade concrete, columns no wider than the infill walls, longitudinal reinforcement provided without capacity design, and transverse reinforcement spaced too widely to confine the core or prevent shear failure. Under lateral load these frames develop large storey drift, and where the column layout is unsymmetrical they develop significant torsion as well. Both effects push demand into a small number of ground storey columns, and those columns are usually where collapse starts.

Demolishing and rebuilding this stock is neither economically nor socially feasible, so retrofitting is the practical route to risk reduction. RC jacketing enlarges the member and adds reinforcement, raising axial capacity, flexural capacity and stiffness together. Externally bonded fibre reinforced polymer (FRP) confines the existing core without changing the member size, so the compressive strength and the deformation capacity of the concrete rise while the added mass and the added stiffness are close to zero. Steel jacketing, shear walls and bracing sit between the two in how invasive and how expensive they are.

Choosing between them is not a purely technical decision. Owners of small residential buildings in Kathmandu Valley care about cost, about how long the property is unusable, and about the floor area lost to enlarged columns. An engineer therefore needs to know whether a technique restores member capacity, whether it corrects the global irregularities that govern the seismic response, and what the price difference comes to.

This study addresses that question for one representative building. A G+3 residential RC frame in Lalitpur was assessed in the field, modelled in ETABS and checked against the seismic provisions of NBC 105 (DUDBC 2020). The same set of deficient columns was then retrofitted twice, once by RC jacketing designed to IS 15988:2013 (BIS 2013) and once by CFRP confinement designed to ACI 440.2R-08 (ACI Committee 440 2008), and the three models (as-built, jacketed and CFRP wrapped) were compared for member capacity, storey stiffness, displacement, drift, torsional irregularity, the strong column weak beam hierarchy and floor mass. A quantity take-off and rate analysis on a common basis completes the comparison. What the work adds is a like-for-like comparison of a section enlargement technique against a confinement technique on one Nepalese low-rise frame, with the cost worked out on the same basis rather than omitted or reported separately.

## 2 Previous studies and research gap

Pradhan et al. (2016) retrofitted the earthquake damaged Central Library building of Kathmandu University. The structure was modelled in SAP2000, the flexure deficient columns were identified from the axial-flexure (P-M-M) demand to capacity ratio, and RC column jacketing was designed to IS 15988:2013. That paper sets out the assessment sequence used here: model the as-built structure, isolate the members that fail the interaction check, and size the jacket for those members alone.

Adhikari et al. (2021) examined a commercial building in which two storeys had been added to an original G+3 frame. The extended structure was analysed in ETABS 2017 and 52 columns were found to fail under the increased demand. Nine of them were strengthened by RC jacketing to IS 15988:2013 and the remaining 43 by CFRP wrapping to ACI 440.2R-08. That study is one of the few to apply both techniques within a single building, although the two were assigned to different members rather than compared on the same members, so their relative effect on the global response could not be isolated.

Dingorkar and Srivastava (2016) compared RC jacketing with FRP wrapping for a deficient RC column and reported axial strength gains of roughly 117 to 119% for jacketing and 22 to 74% for FRP, depending on the number of plies and the column considered. Their numbers are the benchmark used later to check the member level gains obtained here.

Dasgupta (2018) reviewed the use of FRP for strengthening concrete members and set out the mechanics of confinement, in which the wrap resists the lateral dilation of the concrete and thereby raises its compressive strength and ultimate strain. Das (2023) applied the same principles to the seismic retrofit of RC columns and reported the improvement in ductility that confinement produces. Sharma and Singh (2018) examined the seismic behaviour of RC buildings stiffened by shear walls, a system level intervention that reduces drift by adding stiffness rather than by strengthening individual members.

Two observations follow from this body of work. First, most studies evaluate a single retrofit technique, so the reader cannot tell how much of the reported improvement belongs to the technique and how much to the building. Second, where FRP is used, the discussion usually stops at member capacity and says nothing about storey stiffness, drift or torsional irregularity, which are the quantities that govern acceptance under NBC 105. The present study applies both techniques to the same sixteen deficient columns of the same building, re-analyses each retrofitted model in full, and reports member level and system level response together with a comparable cost estimate.

## 3 Building description and condition assessment

### 3.1 As-built configuration

The building is a four-storey (G+3 with roof) residential RC frame in Lalitpur, Bagmati Province. It is a moment resisting frame with 230 × 230 mm columns, 230 × 350 mm beams and a 125 mm solid slab, laid

out on grid lines 1 to 4 and A to D. Storey height is about 2.90 m and the total height above plinth level is 8.61 m. The as-built data were collected during site visits through measurement and photographic records, together with information provided by the owner on the construction history of the building, since no design drawings or mix design records were available. Table 1 summarises the configuration and Figs. 1 and 2 show the ground floor plan and the ETABS model.

**Table 1** Configuration and material data of the as-built building

| Parameter | Value |
|---|---|
| Building type | Residential, G+3 with roof |
| Location | Lalitpur, Bagmati Province, Nepal |
| Plinth area | 832 sq. ft. |
| Storey height | About 2.90 m (total height 8.61 m) |
| Structural system | RC moment resisting frame |
| Column / beam size | 230 × 230 mm / 230 × 350 mm |
| Slab thickness | 125 mm RC solid slab |
| Concrete / steel grade | M15 / Fe415 |
| Column reinforcement | 4 bars of 12 mm diameter |
| Soil type / seismic zoning factor | Type D / 0.35 |

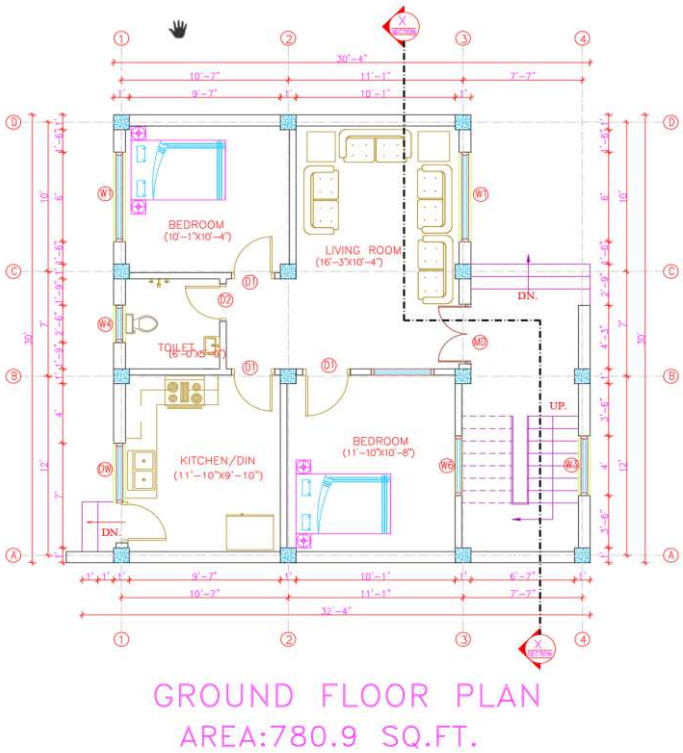



**Fig. 1** Ground floor plan of the building, in which section X-X marks the frame line used for the deformed shape and force diagram comparisons

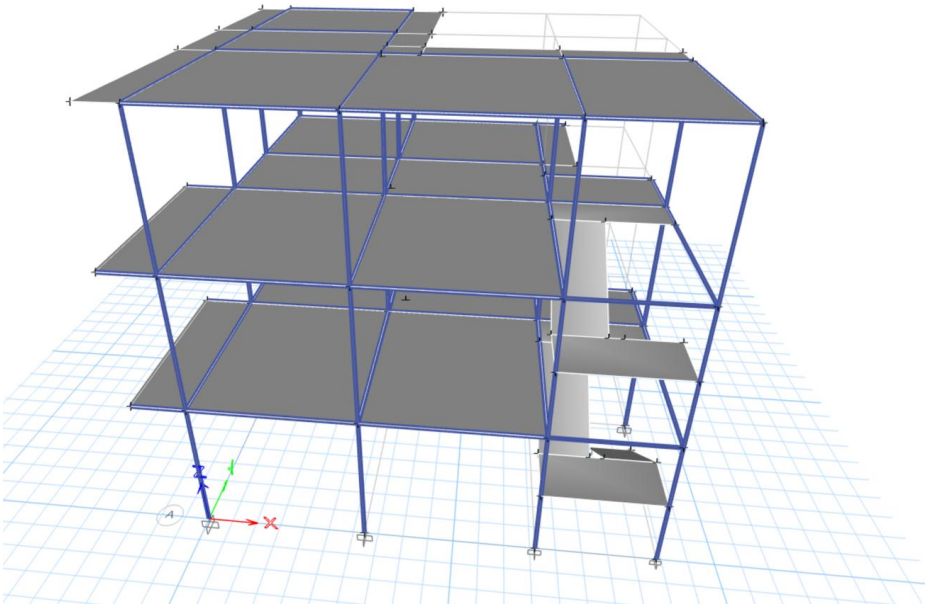


**Fig. 2** Three-dimensional ETABS model of the existing building

### 3.2 Rapid visual assessment

A rapid visual assessment was carried out before any analysis, with the aim of deciding whether the building was usable as it stood, retrofittable, or beyond repair. Structural components were inspected for cracking, spalling and deterioration, and the surface strength of representative members was checked with a rebound hammer. The seismic hazard at the site was taken from the zoning map of NBC 105, which gives a seismic zoning factor Z = 0.35 for this location. The outcome of the assessment was a non-ductile frame of low grade concrete in the highest hazard zone, for which detailed evaluation and retrofitting were recommended with life safety as the target performance level.

### 3.3 Non-destructive testing

Because no record of the original mix design existed, the in-situ concrete strength was estimated using a Schmidt rebound hammer. Ten readings were taken on each member, the highest and lowest were discarded, and the remainder were averaged. The rebound number correlates with surface hardness and hence, indirectly, with compressive strength; the accuracy of the correlation is of the order of 25%, so the results were interpreted conservatively. Table 2 gives the readings and Fig. 3 shows the test positions.

**Table 2** Summary of rebound hammer test results

| Member | Rebound number | Estimated strength (MPa) | Equivalent grade |
|---|---|---|---|
| Edge column C1 | 19.8 | 14.9 | About M15 |
| Centre column C5 | 20.2 | 15.3 | About M15 |
| Edge column C9 | 18.5 | 13.9 | About M15 |
| Beam B12 | 21.4 | 16.2 | About M15 |
| Mean of all members | 19.7 | 14.9 | M15 |

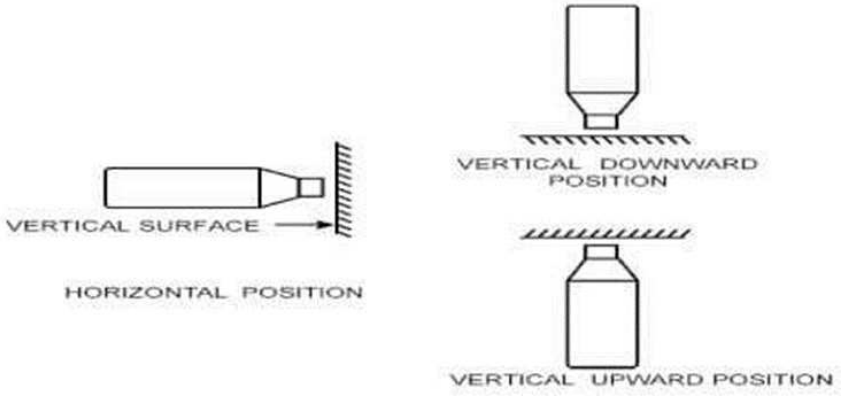


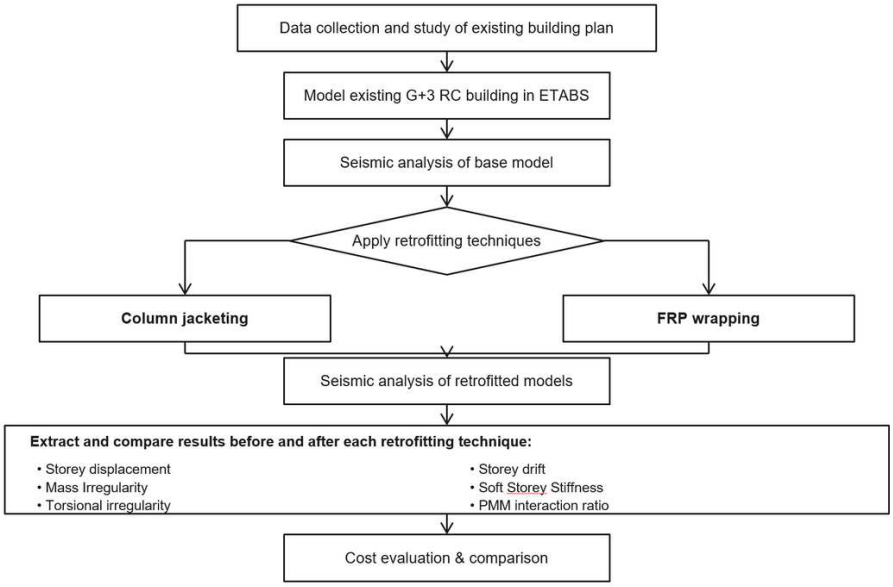

**Fig. 3** Rebound hammer test positions

The mean in-situ strength of about 14.9 MPa corresponds to M15 concrete, well below the M20 minimum required for RC frames in seismic regions by IS 456:2000 (BIS 2000) and by current Nepalese practice. M15 concrete with Fe415 reinforcement was therefore assigned to the as-built ETABS model. That figure shapes most of what follows. The sections are small, but the weak concrete is the main reason for the member deficiencies reported in Sect. 4.4.

# 4 Numerical model and seismic analysis

## 4.1 Idealization and modelling assumptions

The building was idealized in ETABS as an unbraced three-dimensional space frame. Beams and columns were modelled as line elements with rigid joints, and the floor slabs were modelled as shell elements assigned a rigid diaphragm so that the lateral force at each level is distributed to the columns in proportion to their stiffness. Supports were taken as fixed at the base; foundation flexibility and the adequacy of the existing footings were outside the scope of the study. Infill walls were not modelled as struts, and their weight was applied as a uniformly distributed load on the supporting beams. The analysis was linear elastic throughout, which is consistent with the equivalent static procedure adopted for design checking. The overall sequence of the work, from data collection to the cost comparison, is summarised in Fig. 4.




144

145 **Fig. 4** Research methodology adopted in the present study

146 **4.2 Gravity loading**

147 Dead load comprised the self weight of the frame and slab computed by the program, together with wall
148 loads applied on the beams, floor finishes, the staircase, the roof water tank and soil load where applicable.
149 Live load was taken as 2 kN/m² in rooms and 3 kN/m² on passages and the staircase. Shell loads were used
150 for the floor and roof live loads and frame loads for the walls, as illustrated in Fig. 5.

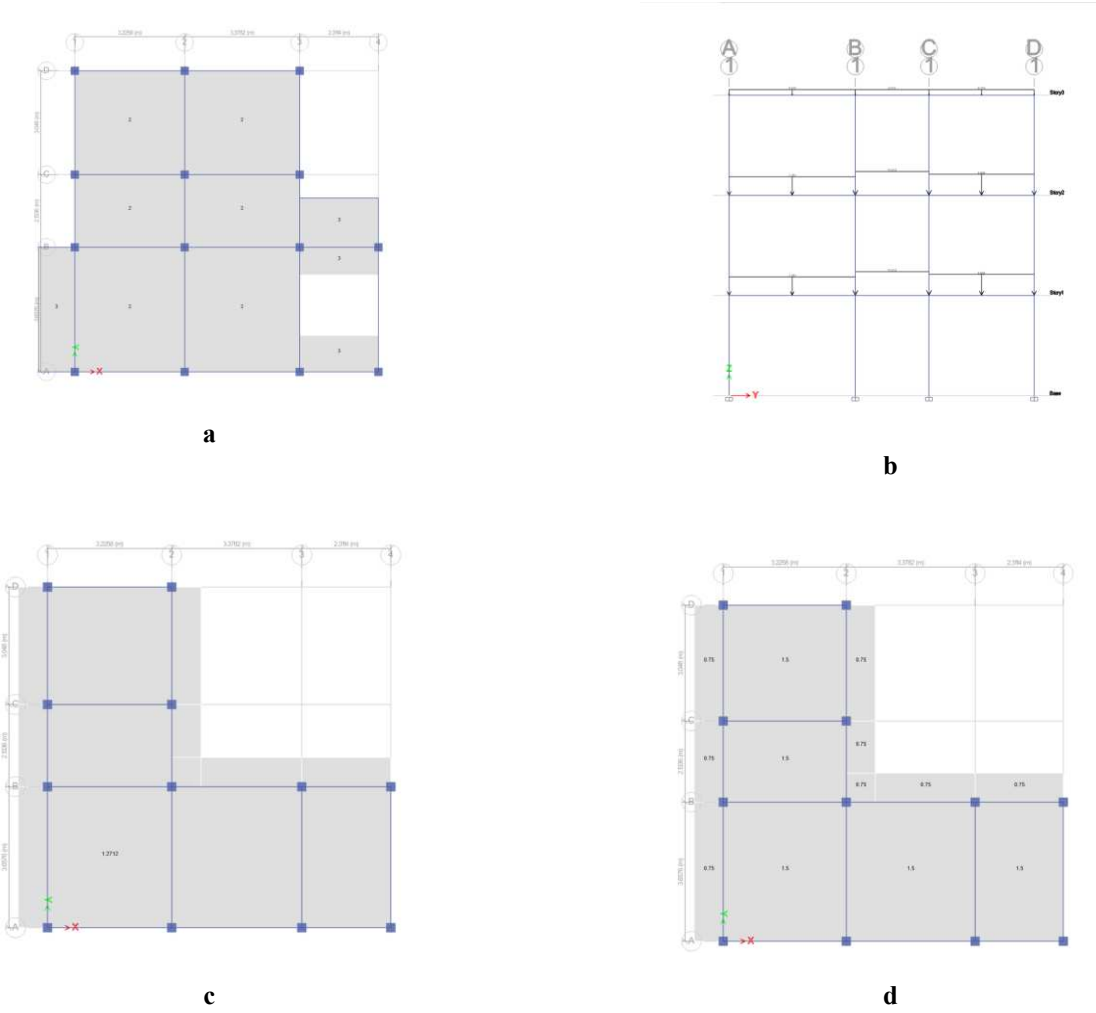



**Fig. 5** Application of gravity loads in the ETABS model: **a** live load on floor shells, **b** wall load on frames, **c** water tank load, **d** roof live load

### 4.3 Seismic loading and method of analysis

Seismic forces were computed by the equivalent static method of NBC 105 (DUDBC 2020) using a seismic zoning factor Z = 0.35 and soil type D; the same equivalent static framework is used in IS 1893 (Part 1) (BIS 2016a). The fundamental translational period of the as-built model was 0.471 s. NBC 105 permits the equivalent static method for all serviceability limit state checks, and at the ultimate limit state for buildings not taller than 15 m, with a period below 0.5 s, or regular and below 40 m in height. The present building satisfies these conditions on height and period, so the equivalent static method was adopted and the resulting base shear was distributed over the storeys as lateral forces. Modal response spectrum, elastic time history and nonlinear procedures were reviewed but not used, since they are not required for a structure of this size and the study is concerned with code compliance checks rather than with collapse simulation.

The design combinations followed NBC 105 and comprised the gravity cases 1.0DL + 1.0LL and 1.2DL + 1.5LL, together with the seismic combinations 1.0DL + 0.3LL or 0.6LL, combined with plus or minus the earthquake load in the X and Y directions at both the serviceability and the ultimate limit states. The live load factor of 0.3 was used for residential floor areas and 0.6 where the imposed load is higher.

### 4.4 Deficiencies identified in the as-built structure

The as-built model was checked in ETABS under the full set of seismic combinations. Sixteen columns failed the axial-flexure (P-M-M) design check and were flagged as over-stressed, twelve of them in the first storey and four in the second (Fig. 6). No beam failed the design check. The pattern is unsurprising: axial force and moment demand are largest at the base, the column section is small and the concrete is weak, so the interaction surface of the section is exceeded before that of the beams.

The building also failed the torsional irregularity check of NBC 105 at every storey and in both directions, with maximum-to-minimum displacement ratios between 1.73 and 3.13 against a limit of 1.5, and it failed the strong column weak beam requirement at several joints. Both are system level problems rather than member level ones, and that distinction governs the comparison between the two retrofit schemes.

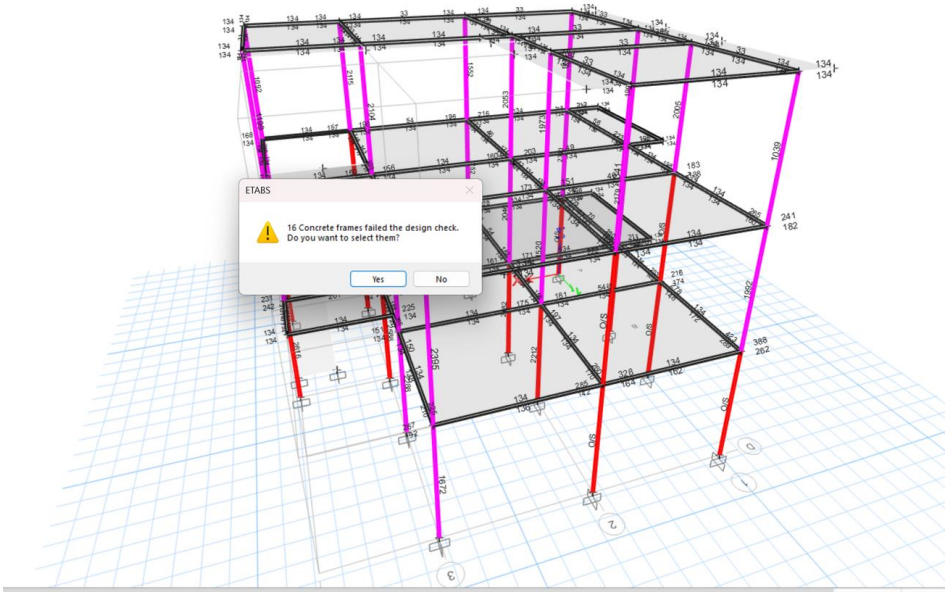



**Fig. 6** Over-stressed columns identified in the as-built model, shown in red

## 5 Retrofit design

The same sixteen columns were retrofitted in two independent ways, and each retrofitted model was analysed in full. No other member was modified, so any difference between the three models can be attributed to the intervention on those columns alone.

### 5.1 RC column jacketing

Jacketing was designed to IS 15988:2013 (BIS 2013), with detailing checked against IS 456:2000 (BIS 2000) and IS 13920:2016 (BIS 2016b). The existing 230 × 230 mm column was retained in place and a new jacket 100 mm thick was added on each face, giving a 430 × 430 mm section. The jacket concrete was specified as M25, at least 5 MPa stronger than the existing concrete as required by IS 15988, and the new longitudinal cage was Fe500 with eight bars of 20 mm diameter. Ties of 8 mm diameter were provided at

100 mm centres with 135 degree hooks; this spacing is governed jointly by the confinement requirement of IS 15988 and by the rule that tie spacing should not exceed half the least lateral dimension, which for the jacketed section is 215 mm. Table 3 lists the details and Fig. 7 shows the retrofitted section.

IS 15988:2013 requires that the concrete and steel areas obtained from the analysis of the jacket be increased to allow for imperfect composite action at the interface between old and new concrete:

$$A_c = 1.5\,A'_c\,, \quad A_s = 1.33\,A'_s \tag{1}$$

where $A'_c$ and $A'_s$ are the jacket concrete and steel areas required by analysis and $A_c$ and $A_s$ are the areas actually provided. The provided cage of eight 20 mm bars follows from applying Eq. (1) to the area demand reported by ETABS for the deficient columns.

Composite action between the old and the new concrete was assumed to be achieved by the construction detail rather than by analysis: the existing surface is chipped and roughened, an epoxy bonding agent is applied at the interface, and bent-down dowels welded to the existing longitudinal bars connect the two cages (Fig. 8). The jacket runs continuously through the beam and slab joints and is anchored 150 mm into the existing or enlarged footing, so that the increased axial force can be transferred to the foundation.

**Table 3** Reinforcement details of the jacketed columns

| Members | Section (mm) | Longitudinal bars | Ties | Materials |
|---|---|---|---|---|
| Storey 1 columns (12 nos.) | 230 to 430 (jacketed) | 8 bars of 20 mm (new cage) | 8 mm at 100 c/c | M25 + Fe500 |
| Storey 2 columns (4 nos.) | 230 to 430 (jacketed) | 8 bars of 20 mm (new cage) | 8 mm at 100 c/c | M25 + Fe500 |
| Existing core (retained) | 230 × 230 | 4 bars of 12 mm (original) | Existing ties | M15 + Fe415 |

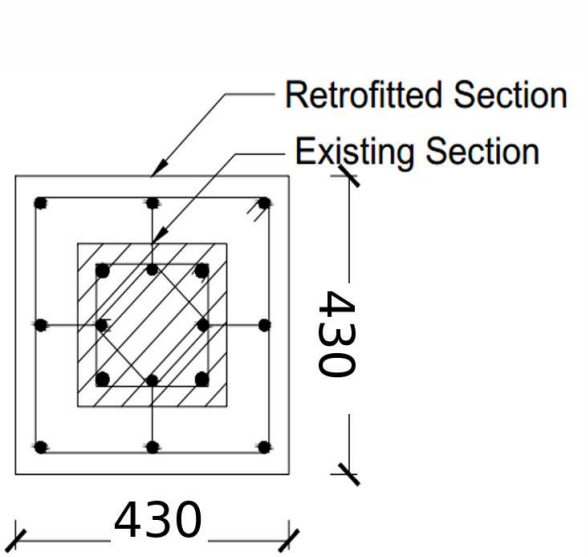
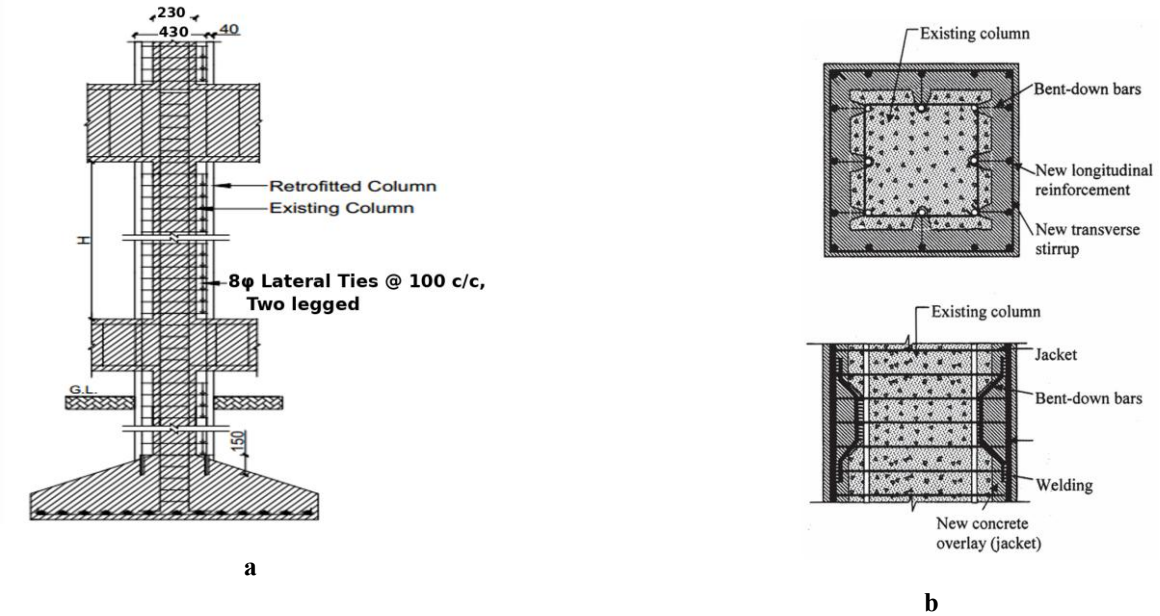



Fig: Cross-section of Column after retrofitting

205

206 **Fig. 7** Cross section of the column after jacketing




207 **Fig. 8** Reinforcement and connector arrangement for the jacket, after IS 15988:2013 (BIS 2013): **a** longitudinal section of the
208 jacketed column, **b** bar and connector arrangement

209 **5.2 CFRP confinement**

210 The second scheme leaves the section unchanged and confines the existing concrete with one ply of carbon
211 fibre fabric wrapped around all four faces. The design followed ACI 440.2R-08 (ACI Committee 440 2008)
212 for a square 230 × 230 mm column of M15 concrete ($f'_c$ taken as 12.0 MPa) reinforced with four 12 mm
213 Fe415 bars, using a fabric of thickness $t_f = 1.5$ mm and elastic modulus $E_f = 220$ GPa.

214 The baseline axial capacity of the unwrapped column was computed from IS 456:2000 (BIS 2000), Cl.
215 39.3:

$$P_u = 0.4\, f'_c\, A_c + 0.67\, f_y\, A_{sc} \tag{2}$$

which gives $P_u$ = 377.54 kN. The design tensile strength and rupture strain of the fabric follow from the environmental reduction factor $C_E$:

$$f_{fu} = C_E\, f^*_{fu}\,, \quad \varepsilon_{fu} = f_{fu} / E_f \tag{3}$$

With $C_E$ = 0.85 the design values are $f_{fu}$ = 3230 MPa and $\varepsilon_{fu}$ = 0.01468. For confinement of a member the effective strain is limited by ACI 440.2R-08 to the lesser of the strain efficiency limit and 0.004:

$$\varepsilon_{fe} = min\,(\,\kappa_\varepsilon\, \varepsilon_{fu}\,,\, 0.004\,) \tag{4}$$

The 0.004 cap governs, so $\varepsilon_{fe}$ = 0.0040. The efficiency of confinement in a non-circular section is accounted for through the effectively confined area ratio and the shape factor:

$$A_e / A_c = 1 - [\,(b - 2r_c)^2 + (h - 2r_c)^2\,] / 3A_g - \rho_g \tag{5}$$

$$k_a = (\,A_e / A_c\,)\,(\,b / h\,)^2 \tag{6}$$

For the square section b = h, so $k_a = A_e / A_c$ = 0.583. The confining pressure and the confined concrete strength are then:

$$f_l = 2\, n\, E_f\, \varepsilon_{fe}\, t_f / h \tag{7}$$

$$f'_{cc} = f'_c + \psi_f\, (3.3)\, k_a\, f_l \tag{8}$$

Equation (7) gives $f_l$ = 11.48 MPa. The ratio $f_l / f'_c$ = 0.957 comfortably exceeds the minimum of 0.08 below which ACI 440.2R-08 considers confinement ineffective. With $\psi_f$ = 0.95, Eq. (8) gives $f'_{cc}$ = 32.98 MPa, which is 2.75 times the unconfined strength. Substituting the confined strength into the capacity expression:

$$P'_u = 0.4\, f'_{cc}\, A_c + 0.67\, f_y\, A_{sc} \tag{9}$$

gives $P'_u$ = 817.75 kN, an increase of 116.6% over the unwrapped column. Repeating the calculation for two plies doubles the confining pressure and gives 1258.0 kN, an increase of 233.2%. The single-ply solution was carried forward because it already exceeds the demand on the deficient columns.

### 5.3 Representation of the retrofits in the ETABS model

The jacketed columns were modelled explicitly. The section was redefined as 430 × 430 mm with M25 concrete and the Fe500 cage, so both the geometric properties and the material properties of the member change in the model, and the composite core and jacket are treated as a single homogeneous section. This is the usual simplification for jacketed members and is acceptable here because the connector detail is designed to develop full composite action.

ETABS has no element that represents an externally bonded composite sheet, so the CFRP retrofit was introduced through an equivalent material property rather than as a discrete composite layer. The column geometry and reinforcement were left unchanged and the concrete compressive strength of the wrapped

members was raised from $f'_c$ to the confined value $f'_{cc}$ = 32.98 MPa obtained from Eq. (8). The modulus of elasticity of the concrete was kept at the as-built value, and that was deliberate. Confinement is a passive mechanism, mobilised only once the concrete begins to dilate laterally, so the wrap has almost no influence on the initial elastic stiffness of the member. Keeping the original modulus therefore reproduces how a wrapped column behaves under service level lateral load, and it explains why the CFRP model behaves globally like the as-built model, as Sect. 6 shows. The mass of the fabric and adhesive, of the order of 1 to 2 kg per square metre, was neglected.

This representation has a clear limitation. The model is linear elastic and the wrap appears only as an increased compressive strength, so the analysis captures the gain in member capacity but not the gain in ductility and energy dissipation that confinement provides. That gain would show up in a nonlinear analysis as a longer, flatter descending branch on the concrete stress-strain curve and as greater plastic rotation capacity at the column ends, and it is the main advantage of FRP that this study cannot quantify.

# 6 Results and discussion

## 6.1 Member capacity and the axial-flexure check

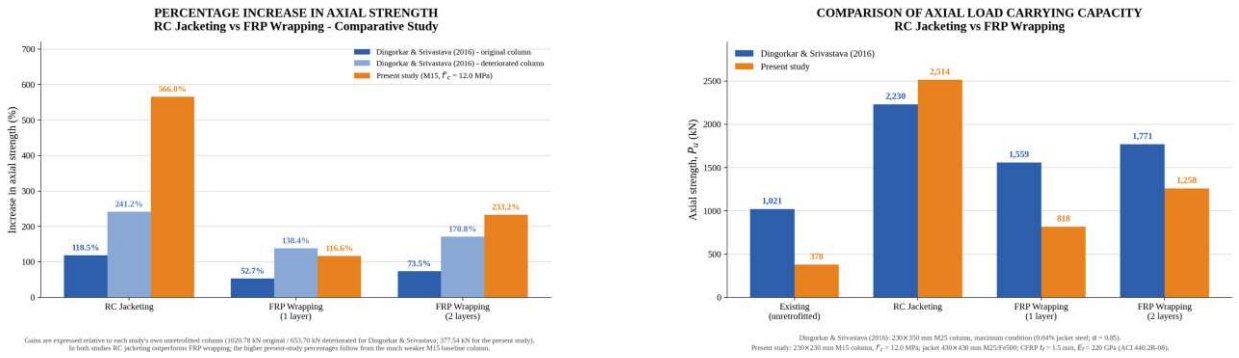
After jacketing, all sixteen previously over-stressed columns pass the P-M-M check, with demand to capacity ratios at or below 1.0, and the over-stressed flags disappear from the design output for the frames along the checked grid lines. The CFRP wrapped model also clears the member level check, since raising $f'_c$ from 12.0 to 32.98 MPa expands the axial-moment interaction surface of the section enough to contain the demand.

The size of the two gains differs sharply, as Table 4 shows. Jacketing multiplies the axial capacity of the reference column by 6.7, and the reported value of 2514.35 kN is consistent with a section analysis that sums four contributions: the M25 jacket concrete, the retained M15 core, the new Fe500 cage and the original Fe415 bars. Two mechanisms act together here. The concrete area increases by a factor of 3.5, from 52,900 to 184,900 mm², and the added area is of higher grade, while the longitudinal steel area rises from 452 to 2966 mm² in total, most of it at the higher yield strength. CFRP works on one variable only. With the section area and the reinforcement fixed, the whole gain comes from the higher effective compressive strength of the confined core.

**Table 4** Axial capacity of the reference 230 × 230 mm column before and after retrofit

| Case | Existing $P_u$ (kN) | Retrofitted $P_u$ (kN) | Increase (%) |
|---|---|---|---|
| Existing column (M15, 230 × 230 mm) | 377.54 | Reference | Baseline |
| CFRP wrapping, 1 ply (ACI 440.2R-08) | 377.54 | 817.75 | +116.6 |
| CFRP wrapping, 2 plies (ACI 440.2R-08) | 377.54 | 1258.0 | +233.2 |
| RC jacketing to 430 mm, M25 + Fe500 (IS 15988:2013) | 377.54 | 2514.35 | +566.0 |

The single-ply CFRP gain of 116.6% sits above the 22 to 74% range reported by Dingorkar and Srivastava (2016), which is expected because the present column has much weaker concrete: for a fixed confining pressure the relative gain from confinement grows as $f'_c$ falls. The jacketing gain of 566% is far above the 117 to 119% they report, but their jacket was thinner relative to the original section, so the ratio of final to initial concrete area was much smaller. Both comparisons are shown in Fig. 9.




**a** **b**

**Fig. 9** Increase in column axial capacity for RC jacketing and CFRP wrapping, the present study compared with Dingorkar and Srivastava (2016): **a** percentage increase in axial strength, **b** axial load carrying capacity

## 6.2 Storey stiffness

Storey stiffness separates the two schemes more clearly than any other quantity (Table 5). Jacketing raises the lateral stiffness of every storey by a factor of three to five. In the X direction the first storey stiffness increases from 34,018 to 156,671 kN/m, a factor of 4.6, and the third storey from 11,893 to 42,569 kN/m, a factor of 3.6. The Y direction behaves the same way. The explanation is geometric: the lateral stiffness of a column varies with the second moment of area, and enlarging a square section from 230 to 430 mm increases that quantity by $(430/230)^4$, about 12 times, for the sixteen retrofitted columns. Since only sixteen of the columns were jacketed and only in the lower two storeys, the storey stiffness gain is smaller than the member level gain and largest where the jacketed columns are concentrated.

In the CFRP model, storey stiffness stays within about 1% of the as-built values at every level, which is the only outcome the model can produce: the section dimensions and the reinforcement are unchanged and the concrete modulus was retained. Confinement raises strength without raising stiffness, so any global response quantity governed by stiffness stays where it was. Much of the rest of the comparison follows from that, as the three models placed side by side under EQX in Fig. 10 make clear.

**Table 5** Storey stiffness (kN/m) at the ultimate limit state

| Storey | EQX before | EQX jacketing | EQX CFRP | EQY before | EQY jacketing | EQY CFRP |
|---|---|---|---|---|---|---|
| 3rd | 11,893 | 42,569 | 11,914 | 11,984 | 44,950 | 12,010 |
| 2nd | 24,151 | 82,636 | 24,289 | 23,906 | 88,798 | 24,047 |

| Storey | EQX before | EQX jacketing | EQX CFRP | EQY before | EQY jacketing | EQY CFRP |
|---|---|---|---|---|---|---|
| 1st | 34,018 | 156,671 | 34,053 | 31,669 | 161,528 | 32,014 |

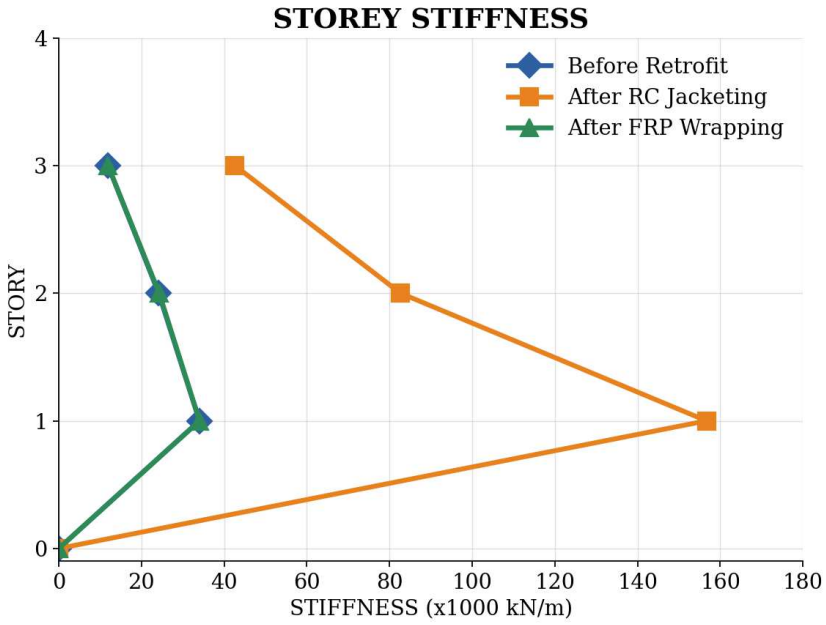
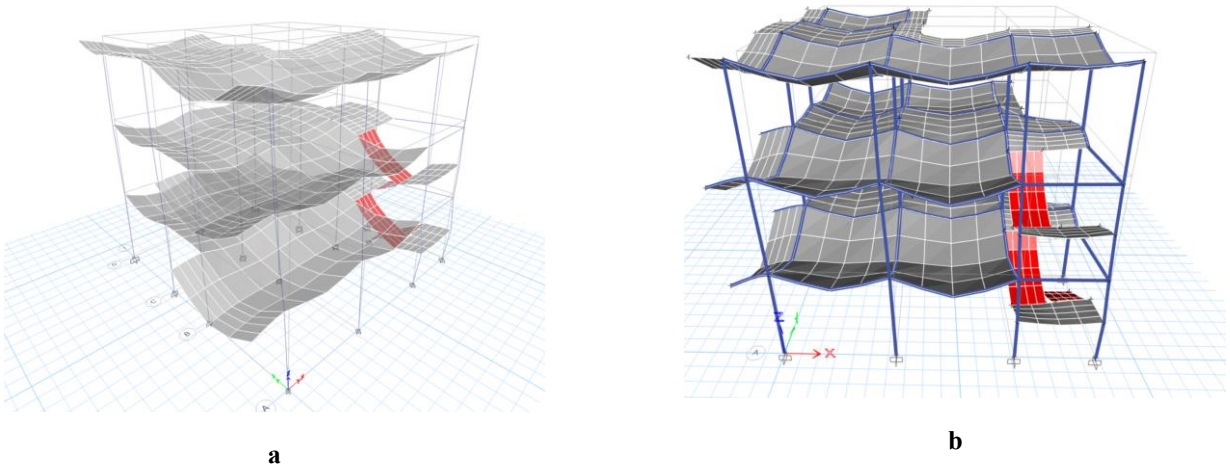



**Fig. 10** Comparison of storey stiffness under EQX at the ultimate limit state for the three models

### 6.3 Displacement and storey drift

The stiffness result propagates directly into the displacement and drift response. Under EQX at the ultimate limit state the maximum roof displacement of the as-built model is 156.1 mm. After jacketing it falls by roughly 75%. In the CFRP model it is 153.9 mm, a reduction of 1.4%, which is within the numerical noise of the analysis and is consistent with the 1% change in storey stiffness. The deformed shapes in Fig. 11 show the same thing qualitatively: the jacketed frame deforms far less under the same load, and Fig. 12 compares the storey displacement profiles of the three models.



**a** **b**

**Fig. 11** Deformed shape of the frame along section X-X: **a** before retrofitting, **b** after RC jacketing

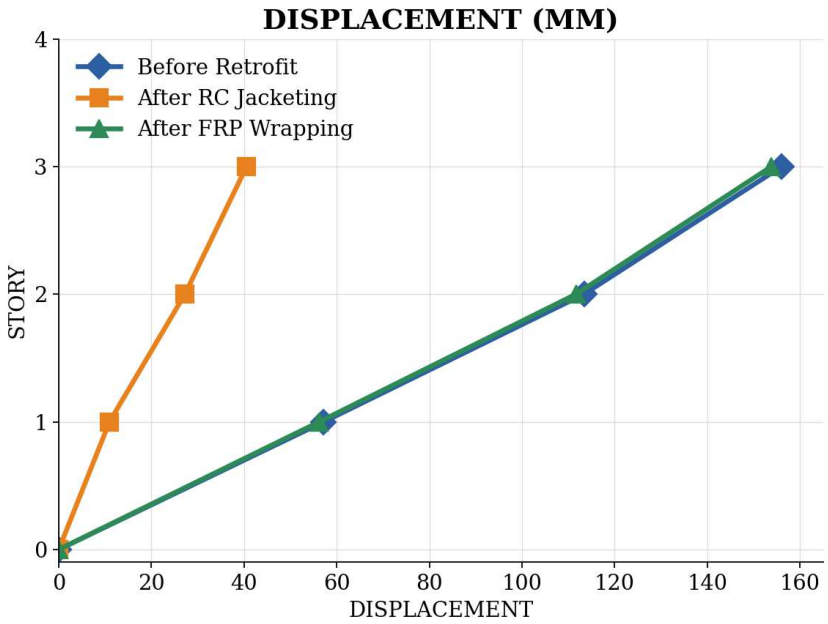



**Fig. 12** Maximum storey displacement under EQX at the ultimate limit state for the three models

Tables 6 and 7 give the storey drift ratios in the two directions. All three models satisfy the ultimate limit state drift limit of 0.025 given in NBC 105, but the margin differs greatly. In the as-built structure the first storey drift under EQY reaches 0.02407, about 4% below the limit, which leaves no allowance for the uncertainty in the material strength that the rebound survey itself carries. Jacketing brings the same value to 0.00397. CFRP leaves it at 0.02304, still within 4% of the as-built value.

**Table 6** Storey drift ratio under EQX at the ultimate limit state, against a limit of 0.025

| Storey | Before | RC jacketing | CFRP wrapping | Check |
|---|---|---|---|---|
| 3 | 0.01474 | 0.00459 | 0.01458 | OK |
| 2 | 0.01940 | 0.00563 | 0.01914 | OK |
| 1 | 0.01975 | 0.00378 | 0.01942 | OK |

**Table 7** Storey drift ratio under EQY at the ultimate limit state, against a limit of 0.025

| Storey | Before | RC jacketing | CFRP wrapping | Check |
|---|---|---|---|---|
| 3 | 0.01254 | 0.00406 | 0.01252 | OK |
| 2 | 0.02375 | 0.00586 | 0.02332 | OK |
| 1 | 0.02407 | 0.00397 | 0.02304 | OK |

The serviceability drifts follow the same pattern, with a larger relative improvement after jacketing. The first storey drift under EQX at the serviceability limit state falls from 0.00381 to 0.00068, a reduction of 82%. That matters for a residential building, because serviceability drift governs cracking in infill walls and finishes, which is the damage most owners see after a moderate earthquake. The ultimate limit state drift profiles of the three models are compared in Fig. 13.

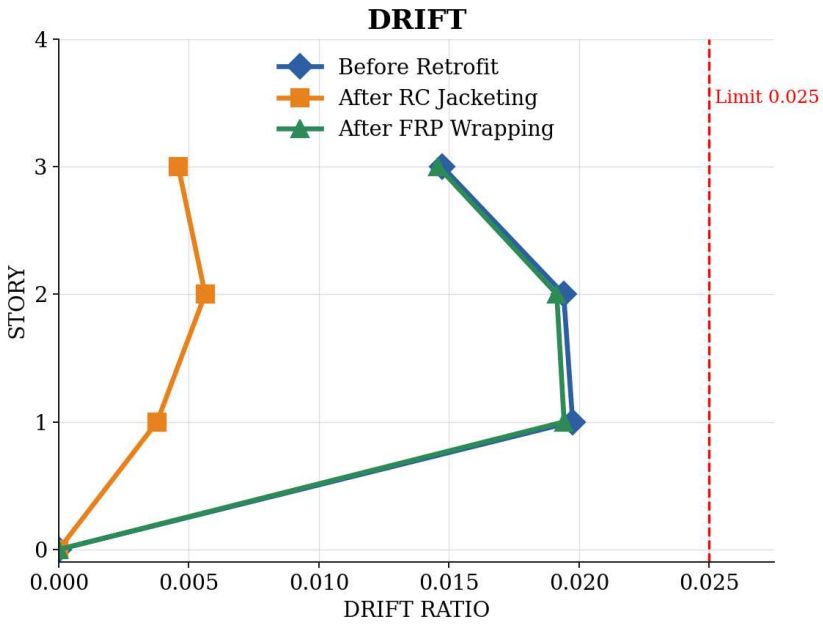



**Fig. 13** Comparison of storey drift ratio under EQX at the ultimate limit state for the three models

### 6.4 Torsional irregularity

NBC 105 treats a structure as torsionally irregular when the ratio of the maximum to the minimum storey displacement exceeds 1.5. The as-built building fails this check in every case examined, with ratios from 1.726 to 3.129, the worst values occurring under EQY where the eccentricity between the centre of mass and the centre of rigidity is largest (Table 8). Twelve checks fail in total across the storeys, directions and limit states considered.

After jacketing every ratio falls below 1.5, the largest remaining value being 1.435. Added stiffness alone does not account for this. The sixteen deficient columns sit in the first two storeys and are distributed asymmetrically over the plan, so enlarging them alters the relative stiffness of the vertical elements and pulls the centre of rigidity towards the centre of mass. The smaller eccentricity is what removes the irregularity. A uniform stiffness increase would have cut the absolute displacements without changing their ratio.

The CFRP model fails every check, with ratios up to 2.865. Wrapping does not alter the relative stiffness of any column, so the centre of rigidity does not move and the torsional eccentricity is exactly what it was. In two cases the ratio comes out marginally higher than in the as-built model, which is within the numerical tolerance of the comparison rather than a real deterioration. The CFRP retrofit therefore passes every member check while leaving the building irregular under the code, which is the practical difference between member level and system level strengthening. The three models are compared under EQX at the ultimate limit state in Fig. 14.

**Table 8** Torsional irregularity ratio, taken as the ratio of maximum to minimum storey displacement, against a limit of 1.5

| Storey | Load case | Before | RC jacketing | CFRP wrapping |
|---|---|---|---|---|
| Storey 3 | EQX ULS | 1.726 | 1.196 | 1.746 |
| Storey 2 | EQX ULS | 2.102 | 1.220 | 2.145 |
| Storey 1 | EQX ULS | 2.515 | 1.253 | 2.625 |
| Storey 3 | EQY ULS | 2.088 | 1.289 | 2.034 |
| Storey 2 | EQY ULS | 3.129 | 1.435 | 2.204 |
| Storey 1 | EQY ULS | 3.008 | 1.382 | 2.865 |

Values above 1.5 exceed the limit of NBC 105. All as-built and CFRP values fail the check; all jacketed values pass.

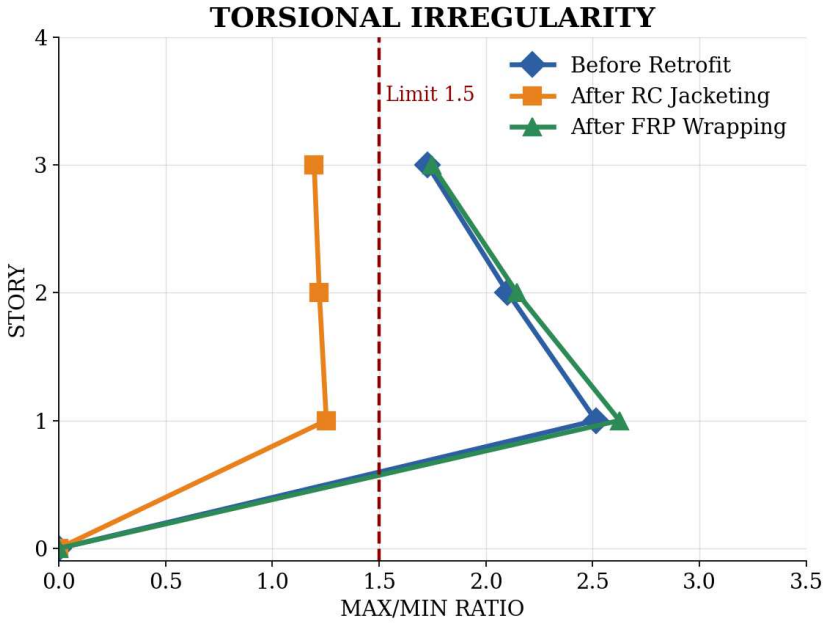
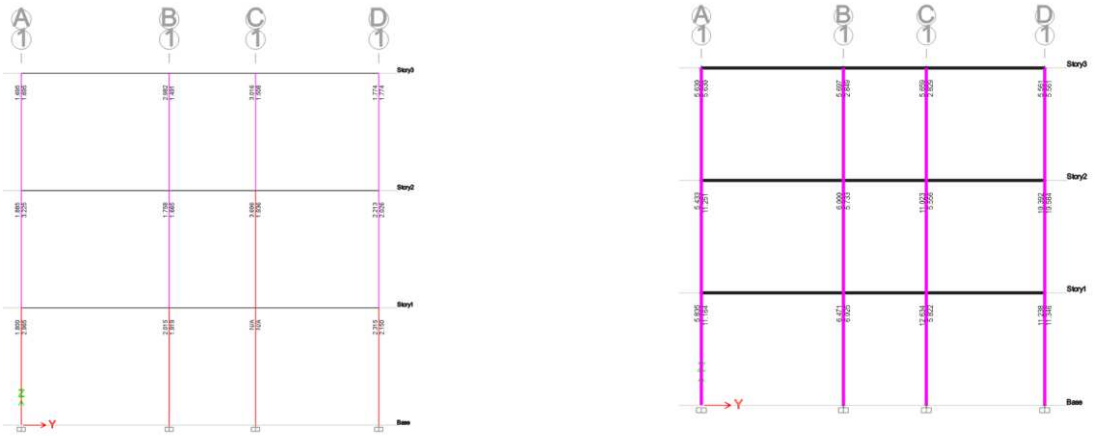



**Fig. 14** Comparison of torsional irregularity ratio under EQX at the ultimate limit state for the three models

## 6.5 Strong column weak beam hierarchy

NBC 105 requires the sum of the column flexural capacities at a joint to be at least 1.2 times the sum of the beam capacities, so that inelastic action develops in the beams rather than in the columns. Several joints of the as-built frame do not meet this requirement, which is a direct consequence of using 230 × 230 mm columns together with 230 × 350 mm beams: the beam is the deeper member, and with the same weak concrete in both it can easily be the stronger one in flexure.

Jacketing restores the hierarchy at every joint. The column moment capacity rises with the enlarged section and the new Fe500 cage while the beams are untouched, so the ratio moves decisively in favour of the columns. In the CFRP model the check is unchanged and the same joints still fail. Confinement raises the compressive strength of the core, but the flexural capacity of a lightly reinforced column is governed by the tension steel and the lever arm rather than by concrete strength, so the moment capacity barely moves. Every joint that failed before continues to fail. The check along section X-X before and after jacketing is compared in Fig. 15.




a b

**Fig. 15** Strong column weak beam check along section X-X: **a** before retrofitting, **b** after RC jacketing

## 6.6 Floor mass and vertical regularity

Jacketing sixteen columns adds concrete to the lower storeys, so the vertical distribution of mass has to be checked. NBC 105 defines a mass irregularity where the mass of one storey differs from that of an adjacent storey by more than 50%. Table 9 and Fig. 16 show that the first storey mass increases from 106,753 to 121,957 kg, about 14%, and the second from 102,276 to 115,878 kg, about 13%. No mass irregularity is created, and since the stiffness increase is largest in the same storeys, no soft storey appears either. The CFRP model carries the as-built masses, since the fabric and adhesive were neglected, which is a real advantage where the existing foundations are marginal.

**Table 9** Floor mass (kg) for the three models

| Floor | Before | RC jacketing | CFRP wrapping | Check |
|---|---|---|---|---|
| 3 | 54,922 | 61,446 | 54,922 | OK |
| 2 | 102,276 | 115,878 | 102,276 | OK |
| 1 | 106,753 | 121,957 | 106,753 | OK |

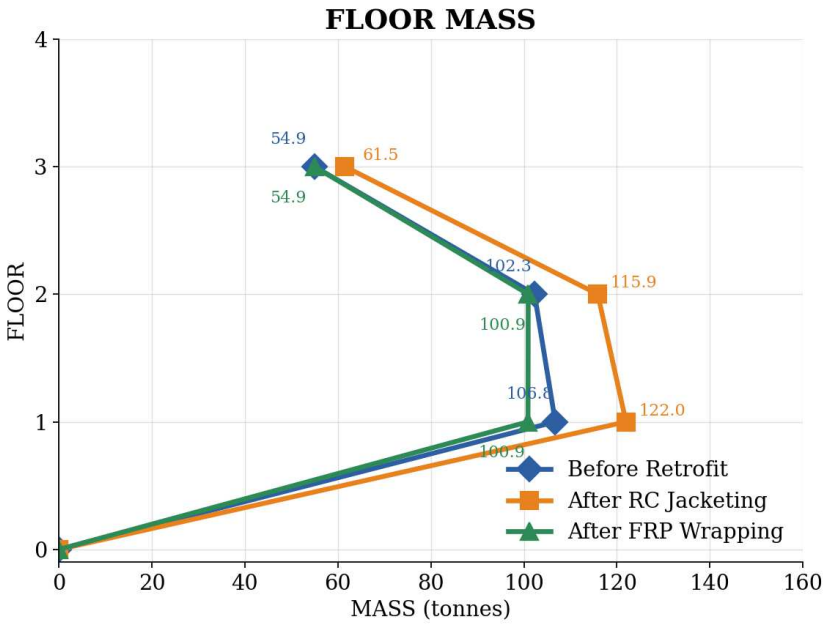



**Fig. 16** Comparison of floor mass for the three models

## 7 Cost comparison

Both schemes were priced on the same basis so that the comparison is like for like. Quantities were taken off per column from the adopted retrofit details and scaled to the sixteen deficient columns, twelve in the first storey and four in the second. Rates combine material and labour in the format of the district rate analyses published by the Department of Urban Development and Building Construction, at indicative mid-2026 Kathmandu market prices (Table 10). The M25 jacket concrete rate was built up from cement, sand and aggregate with a premium for small pours. Contingency of 5%, overhead and profit of 12% and VAT of 13% were applied identically to both options.

**Table 10** Indicative unit rates used in the cost estimate for 2026

| Item | Rate (NPR) |
|---|---|
| M25 jacket concrete | 17,660 per m³ |
| Reinforcement steel (Fe500) | 110 per kg |
| CFRP fabric (1 ply) | 2,800 per m² |
| Structural epoxy | 1,250 per kg |
| Formwork (jacket) | 850 per m² |
| Skilled labour | 1,400 per day |
| OPC cement (53 grade) | 850 per bag |
| Sand / 20 mm aggregate | 4,200 / 4,000 per m³ |
| Contingency / overhead and profit / VAT | 5% / 12% / 13% |

Rates are taken from the DUDBC district rate analyses for 2025 together with Nepalese supplier price lists and manufacturer data for 2026. One US dollar is taken as NPR 151.

The jacketing estimate (Table 11) comes to NPR 823,923 including VAT for the sixteen columns, about NPR 51,495 per column or USD 5456 for the scheme. Reinforcement accounts for roughly 35% of the

direct cost, with concrete and formwork forming most of the remainder. Each column requires about 0.38 $m^3$ of M25 concrete and 105 kg of Fe500 steel.

**Table 11** Cost estimate for RC column jacketing of 16 columns

| Work head | Amount (NPR) |
|---|---|
| Jacket concrete (M25) | 108,163 |
| Reinforcement (Fe500) | 217,857 |
| Formwork | 67,837 |
| Surface preparation | 19,210 |
| Epoxy bonding agent | 27,747 |
| Welding to existing bars | 16,000 |
| Foundation dowels (12 columns) | 43,200 |
| Mobilization and preliminaries | 120,000 |
| Direct cost | 620,013 |
| With contingency 5% and overhead and profit 12% | 729,135 |
| Total including VAT 13% | 823,923 |

The CFRP estimate (Table 12) comes to NPR 437,169, about NPR 27,323 per column or USD 2895 for the scheme. Fabric and epoxy make up roughly 65% of the direct cost. There is no formwork, no curing period and no reinforcement, and each column needs about 3.41 $m^2$ of fabric with 3.1 kg of epoxy.

**Table 12** Cost estimate for CFRP wrapping of 16 columns

| Work head | Amount (NPR) |
|---|---|
| CFRP fabric (1 ply, 4 faces) | 152,916 |
| Epoxy primer and saturant | 62,060 |
| Surface preparation and corner rounding | 28,800 |
| Application (specialist labour) | 35,200 |
| Mobilization and preliminaries | 50,000 |
| Direct cost | 328,976 |
| With contingency 5% and overhead and profit 12% | 386,876 |
| Total including VAT 13% | 437,169 |

CFRP is therefore about 47% cheaper than jacketing for this building, a saving of roughly NPR 387,000 over sixteen columns, and jacketing costs about 1.9 times as much (Fig. 17). Construction time also differs, at roughly 30 days for jacketing against 10 days for wrapping, with correspondingly different disruption to the occupants. Table 13 sets the cost against the structural outcome.

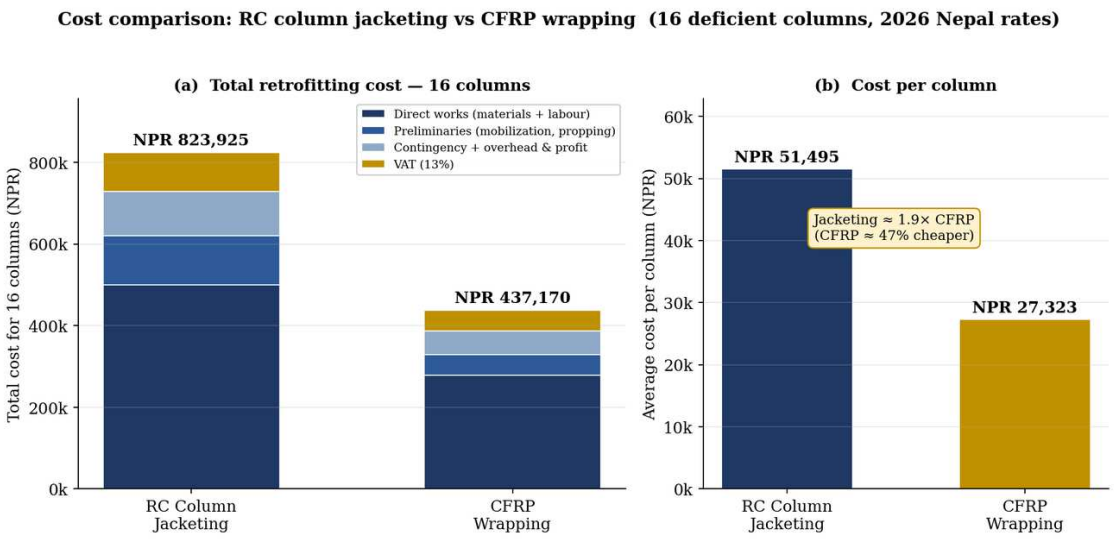



**Fig. 17** Cost comparison of RC column jacketing and CFRP wrapping: **a** total retrofitting cost, **b** cost per column

**Table 13** Cost against performance for the two retrofit schemes

| Criterion | RC column jacketing | CFRP wrapping |
|---|---|---|
| Cost, 16 columns (NPR) | 823,923 | 437,169 (about 47% lower) |
| Axial capacity of reference column | +566.0% | +116.6% |
| Storey stiffness | Three to five times higher | Within about 1% of as-built |
| Maximum storey displacement | About 75% lower | 1.4% lower |
| Torsional irregularity | Eliminated, all ratios below 1.5 | Not corrected, up to 2.865 |
| Strong column weak beam | Restored at all joints | Unchanged |
| Added mass and section size | About 14% floor mass, +200 mm per side | Negligible mass, no size change |
| Construction time | About 30 days | About 10 days |
| Occupant disruption | High | Low |

The two schemes answer different questions. CFRP is cheaper and faster, leaves the geometry and the mass of the structure alone, and clears the member level deficiency completely, but it cannot change how lateral force is distributed through the building. The governing problem here is a torsionally irregular frame with excessive drift rather than a shortage of column capacity on its own, so the scheme that helps is the one that redistributes stiffness. Jacketing costs 1.9 times more and is the only one of the two that brings the building into compliance with NBC 105 on every check performed. CFRP would be the right choice on a building whose global response already meets the code, or where added mass, loss of floor area or the need to keep the building occupied rules out enlarging the columns.

## 8 Conclusions

An existing G+3 RC residential building in Lalitpur was assessed in the field, modelled in ETABS and retrofitted numerically by two independent schemes, RC column jacketing to IS 15988:2013 and single-ply

CFRP confinement to ACI 440.2R-08. The following conclusions are drawn from the linear elastic analyses performed.

(1) Rebound hammer testing gave a mean in-situ compressive strength of about 14.9 MPa, corresponding to M15 concrete. The low material strength, combined with 230 × 230 mm columns and non-ductile detailing, is the origin of the deficiencies found in the analysis.

(2) The as-built structure had sixteen columns failing the axial-flexure check, twelve in the first storey and four in the second, torsional irregularity ratios of up to 3.13 against a limit of 1.5, and several joints failing the strong column weak beam requirement.

(3) RC jacketing from 230 × 230 mm to 430 × 430 mm using M25 concrete and an Fe500 cage cleared every axial-flexure failure. The axial capacity of the reference column rose from 377.54 to 2514.35 kN, an increase of 566%, through the combined effect of a 3.5 times larger concrete area of higher grade and a much larger area of higher strength reinforcement.

(4) Jacketing raised storey stiffness by three to five times, reduced the maximum storey displacement by about 75%, reduced the first storey serviceability drift under EQX from 0.00381 to 0.00068, and brought all torsional irregularity ratios below 1.5 with a maximum of 1.435. The strong column weak beam hierarchy was restored, and neither a mass irregularity nor a soft storey was introduced, the largest floor mass increase being about 14%.

(5) Single-ply CFRP confinement raised the confined concrete strength to 32.98 MPa and the axial capacity of the same column to 817.75 kN, an increase of 116.6%, which is sufficient to clear the member level deficiency. Because the section size and the concrete modulus are unchanged, storey stiffness remained within about 1% of the as-built value, the roof displacement fell by only 1.4%, drift ratios stayed within 4% of as-built, and every torsional irregularity check continued to fail. The strong column weak beam check was also unchanged.

(6) At indicative 2026 Kathmandu rates the CFRP scheme cost NPR 437,169 against NPR 823,923 for jacketing, about 47% less, with an estimated construction time of 10 days against 30 days.

For this building the analysis points one way. RC column jacketing is the scheme that makes the structure compliant, because the governing deficiencies are the torsional irregularity and the drift, and only a change in the stiffness distribution corrects them. That carries a consequence beyond this case: a retrofit can satisfy every member level check and still leave a building irregular under the code, so member checks alone are not enough to choose a technique on. CFRP confinement remains the right choice where the global response is already acceptable and where added mass, loss of floor area or occupant disruption decides the matter, and it costs roughly half as much.

### 8.1 Limitations and further work

The study is limited to linear elastic analysis of a single building, so the ductility and energy dissipation benefits of confinement are not captured; a nonlinear static or dynamic analysis would be needed to quantify them and would probably narrow the gap between the two schemes. Foundation design was outside the scope and supports were modelled as fixed, yet jacketing increases the axial force transferred to the footings, so a foundation adequacy check should precede any implementation. Infill walls were not modelled as struts, which is conservative for stiffness but may misrepresent the torsional response of a building with irregular wall layout. The cost figures are indicative market rates for one location and one year and should be re-established for any specific project.

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**Author contributions** Conceptualization: Kapil Banstola, Umesh Jung Thapa; Methodology: Kapil Banstola, Umesh Jung Thapa; Formal analysis and investigation: Kapil Banstola; Writing, original draft preparation: Kapil Banstola; Writing, review and editing: Umesh Jung Thapa; Supervision: Umesh Jung Thapa. Both authors read and approved the final manuscript.

**Data availability** The datasets generated and analysed during the current study, comprising the ETABS models, the rebound hammer records and the quantity and rate take-offs, are available from the corresponding author on reasonable request.

**Ethics approval** Not applicable. This study is an engineering assessment of an existing building. It did not involve research on human participants, their data or biological material, and it did not involve animals. The building owner acted only as a source of factual information on the construction history of the structure. No personal, health or opinion data were collected, so review by an ethics committee was not sought.

**Consent to participate** Verbal informed consent was obtained from the building owner before the site visits. The purpose of the study was explained to the owner beforehand. The owner agreed to access to the building, to non-destructive testing of its members and to the use of the information provided in this study.

**Consent to publish** Not applicable. The manuscript contains no identifiable personal data or images of individuals.

**Clinical trial number** Not applicable.