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Seismic Vulnerability Assessment of Plan Irregularity Reinforced Concrete Framed Building

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Abstract

The study presents the seismic vulnerability assessment of plan irregular reinforced concrete framed building. At PEC, there are a total of 5 blocks among which Block-B, established in 2000 AD was selected for detailed assessment due to its structural irregularities. The assessment started with visual inspection using FEMA guidelines, which revealed undersized members, poor concrete quality, irregular plan layout, torsional irregularity, and visible deterioration. Non-destructive tests (Ultrasonic Pulse Velocity and Rebound Hammer) confirmed good internal concrete integrity but weak surface strength. To understand the building's seismic performance more clearly, a structural model was developed in ETABS v22.5 following NBC 105:2020. The analysis showed that 130 members failed seismic checks, including torsional irregularities, storey drift, and displacement limits. Based on these findings, a hybrid retrofitting plan was proposed combining concrete jacketing for columns and beams, steel jacketing for selected beams, and the addition of reinforced concrete shear walls. Post-retrofitting analysis showed clear improvements in displacement, drift, and torsional behavior bringing the building's performance closer to national seismic safety standards. From an economic point of view, retrofitting of the building is far more affordable than demolishing and constructing a new one. In other words, retrofitting required only 15.6% of the replacement cost making it the practical choice. Overall hybrid retrofitting stands out as both technically effective and economically practical.

Keywords: Seismic Vulnerability, Non-Destructive Test, Hybrid Retrofitting

1. Introduction

Nepal, located at the convergence of the Indian and Eurasian tectonic plates, is among the most seismically active regions globally. Nepal ranks as the 11th most vulnerable country globally to earthquakes, as reported by UNICEF(UNICEF Nepal, 2017). Pokhara falls under Seismic Zone V according to NBC 105:2020, indicating very high earthquake risk. Its proximity to the Main Central Thrust Fault and soft layered soil conditions further amplify ground shaking, increasing seismic vulnerability(Baruwal, Chhetri and Chaulagain, 2020).

Educational buildings play a significant role in disaster recovery. In addition to their role as classrooms, they often serve as shelters and emergency hubs. For this reason, NBC 105:2020 grants them an Importance Factor of 1.25 to increase seismic design loads and ensure functionality before and after earthquakes.

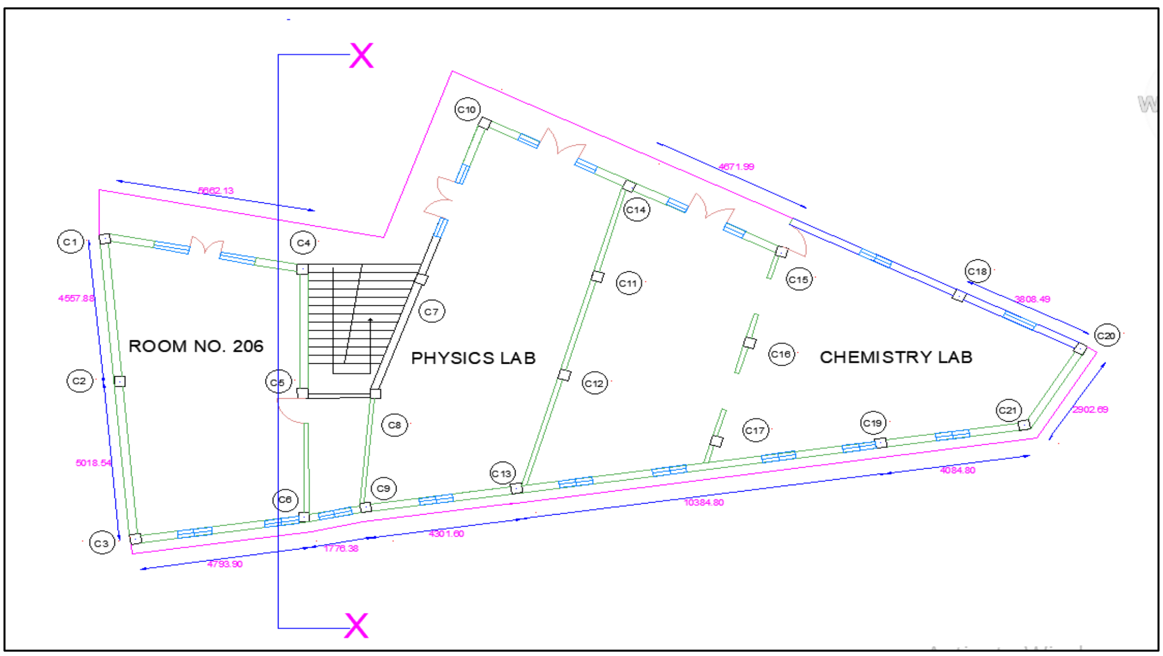


Figure 1: Architectural drawing of first floor

The Diploma Building (Block B) of the Pokhara Engineering College (PEC) is a six storey reinforced concrete (RC) frame structure with hollow block infill walls built in 2000 AD. Since the structure was built before the implementation modern seismic codes, it has serious plan irregularities in the form of re-entrant corners and an asymmetric column layout which would induce torsional effects and stress concentrations during earthquakes. Over time, material deterioration reduces the structure's capacity to resist lateral loads, raising the risk of severe damage if an earthquake occurs.

This paper presents the seismic vulnerability assessment of plan-irregular RC framed building in regional centers like Pokhara. Block B was evaluated based on visual inspection, (NDT), and structural modeling in ETABS v22.5. The findings highlight critical structural deficiencies and recommend hybrid retrofitting measures to ensure the building remains safe, stable, and functional during future earthquakes.

2. Materials and Methods

2.1 Qualitative assessment

Qualitative assessment is performed to determine whether the building in existing condition meets the seismic performance capability(DUDBC, 2011). It was carried out by on-site inspections, reviewing available drawings and interviewing to gather information. The checklist was prepared as per FEMA 310 to identify vulnerabilities factors(FEMA, 1998). This assessment serves as a qualitative measure to identify seismic weaknesses in a building before conducting a detailed evaluation(DUDBC, 2011).

2.2 Quantitative assessment

It is the second phase in seismic vulnerability assessment which follows a quantitative approach after the qualitative analysis. It includes a detailed seismic evaluation to identify deficiencies more accurately and identify appropriate retrofitting measures.

2.2.1 Non-Destructive Test

Non-destructive tests were performed to evaluate the in-situ strength and quality of concrete without damaging the structure. The Rebound Hammer test was conducted to determine the surface hardness and compressive strength of concrete. For each test point, six sets of readings were taken and average rebound number was calculated after discarding outliers(Bureau of Indian Standards, 1978). As per IS 516 (Part 5/Sec 4), average rebound number was used to classify concrete quality. The correlation graph was used to determine the corresponding compressive strengths(Bureau of Indian Standards, 2020).

Indirect method was used to test UPV by placing the transducers on opposite faces of the same surface of the concrete member. At each test point, nine readings were taken and the average pulse velocity was calculated. A velocity criteria for concrete quality grading specified in IS 516 (Part 5/Sec 1) was used to interpret the result of the tests(Bureau of Indian Standards, 2018).

2.2.2 Structural modeling and analysis

The building was modeled in ETABS v22.5 to evaluate its seismic performance under earthquake loading. The analysis was carried out using the Equivalent Static Method (ESM) where lateral seismic forces were applied to the structure based on parameters such as the seismic zone factor, importance factor, response reduction factor, and the building's seismic weight and non-parallel load combination was used to realistically capture multi-directional forces which is especially important in case of irregular buildings. Then the building response was checked against the provisions of NBC 105:2020 including torsional irregularity, storey drift, soft storey condition, and overall storey displacements(DUDBC, 2020). These evaluations were carried out to ensure that the building's seismic response meets the requirements of NBC 105:2020.

2.3 Hybrid Retrofitting

To enhance seismic performance of the building and also for compliance with present code requirements, a hybrid retrofitting approach was adopted combining concrete jacketing for columns and beams, steel jacketing for selected beams, and the addition of reinforced concrete shear wall. This combination was selected to address both local member deficiencies and global structural irregularities identified during analysis. These retrofitting measures were modeled in ETABS by updating member properties and by introducing new lateral load-resisting components. The structure was then reanalyzed under the same seismic load combinations to evaluate improvements in its performance.

2.4 Economic Feasibility

The economic feasibility of the proposed retrofitting was evaluated by estimating costs based on present market rates for construction materials and skilled labor. The tentative retrofitting cost was calculated and compared with the plinth area cost of constructing a new building. Retrofitting was considered viable where the estimated expenditure remained below 25% of the replacement cost of the structure(Nateghi and Shahbazian, 1992).

3. Result and Discussion

3.1 Qualitative Evaluation

The qualitative evaluation revealed that the existing building does not meet the desired seismic performance capability. A comparison with the requirements of NBC 105:2020 highlighted that columns and beams were undersized and M15 concrete was used.

Table 1: Key structural parameters vs NBC 105:2020

Aspect	Observed	NBC105:2020 Requirement	Compliance
Column size	230×230mm	≥ 300 × 300 mm	NC
Beam size	230×230mm	≥ 230 × 300 mm	NC
Concrete grade	M15	≥ M20	NC

The FEMA 310 checklist further identified seismic vulnerabilities including irregular plan layout, discontinuous load paths, lack of redundancy, torsional irregularity and material deterioration.

Table 2: Identification of Vulnerability Factors as per FEMA 310.

Vulnerability Factor	Compliance	Remarks
Plan Regularity	NC	Irregular layout
Load Path Continuity	NC	Discontinuity in transfer of seismic force to foundation.
Redundancy	NC	Lack of proper redundancy.
Weak storey	C	No weak storey observed.
Short column	C	No short column was observed.
Adjacent building	NC	Gap < 4% of shorter building.
Torsion	NC	The eccentricity of the building isn’t within the limit.
Material Deterioration	NC	Visible deterioration of concrete and rusting of reinforcement observed.

Note; C= Compliance and NC= Non-Compliance

Also misalignment of parapet walls, looseness of window frames and ceiling fixtures were some of the non-structural hazards that were observed which are prone to falling during seismic events. Quick checks was done to assess seismic capacity of columns under shear and axial loads to supplement the visual observations.

Storey-wise Shear Stress in RC columns

Storey-wise shear stress in column was calculated using IITK-GSDMA Guidelines for Seismic Evaluation and Strengthening of buildings 6.5.1(Indian Institute of Technology Kanpur, 2005).

Table 3: Calculation of shear stress in RC column

Storey	**nc**	**n_{f1}**	**n_{f2}**	**Ac (m^2)**	**Storey Shear**	**Shear Stress (Mpa)**		**DCR in**		**Remarks**
						τcol1	**τcol2**	**X-direction**	**Y-direction**	
Roof	6	2	3	0.56	160.66	0.43	0.58	0.61	0.81	DCR<1; So safe in shear
5	19	5	7	1.77	1085.47	0.83	0.97	1.18	1.37	DCR>1; So unsafe in shear
4	19	5	7	1.77	2029.28	1.56	1.82	2.20	2.56	
3	19	5	7	1.77	2726.46	2.10	2.45	2.95	3.44	
2	19	5	7	1.77	3195.08	2.46	2.87	3.46	4.04	
1	19	5	7	1.77	7839.72	6.03	7.03	8.49	9.90	

Since the DCR in the top storey is less than 1, this storey is safe but the DCR of lower 5 storey is greater than 1. So, these storeys are overstressed and maybe liable to failure during an earthquake.

Axial Stress Check

Axial stresses due to overturning forces were calculated according to FEMA 310, Clause 3.5.3.6(FEMA, 1998).

Table 4: Axial stresses due to overturning forces

Direction	**Base shear × Load factor (KN)**	**F_o (KN)**	**σ (MPa)**	**σ_{all} (MPa)**	**DCR**	**Status**
X	11759.58	1001.35	18.93	3.75	5.05	Unsafe
Y	11759.58	1798.25	33.99	3.75	9.07	Unsafe

Moreover axial stresses in both X- and Y-directions of the column exceed the permissible limit of 3.75 MPa. Demand Capacity Ratios of 5.05 and 9.07 further confirm severe overstress. Hence, the column is unsafe in axial stress.

Taken them all together, the NBC comparison, FEMA 310 checklist, and quick checks all show that the building does not meet seismic performance capability. When linked to EMS-98, the structure corresponds to Damage Grade 2, which refers to moderate damage characterized by visible cracks and impairment of non-structural elements.

3.2 Quantitative Evaluation

3.2.1 Data Interpretation for Non-Destructive Tests

The Ultrasonic and Rebound Hammer tests were carried out as per the provisions of IS 516. The results have been summarized below.

Table 5: Ultrasonic Pulse Velocity (UPV) test result

Section	PV (km/s)	Quality	CS (MPa)
Slab	5.93	Excellent	37.45

Table 6: Rebound hammer test result

Section	R-value	Quality	CS (MPa)
Column	31	Good layer	22
Slab	24	Fair	17

The UPV test on the slab revealed a velocity of 5.93 km/s which according to IS 516 (Part 5/ Sec 1), describes excellent concrete quality with a strength of 37.45 MPa. On the other hand, the Rebound Hammer test gave very low rebound numbers for column and slab that is 31 and 24 respectively. These corresponds fair to good layer *quality* concrete with strengths of 22 MPa and 17 MPa respectively according to IS 516 (Part 5/ Sec 4).This difference in results implies good internal integrity but weak surface strength.

3.2.2 Structural modeling and analysis

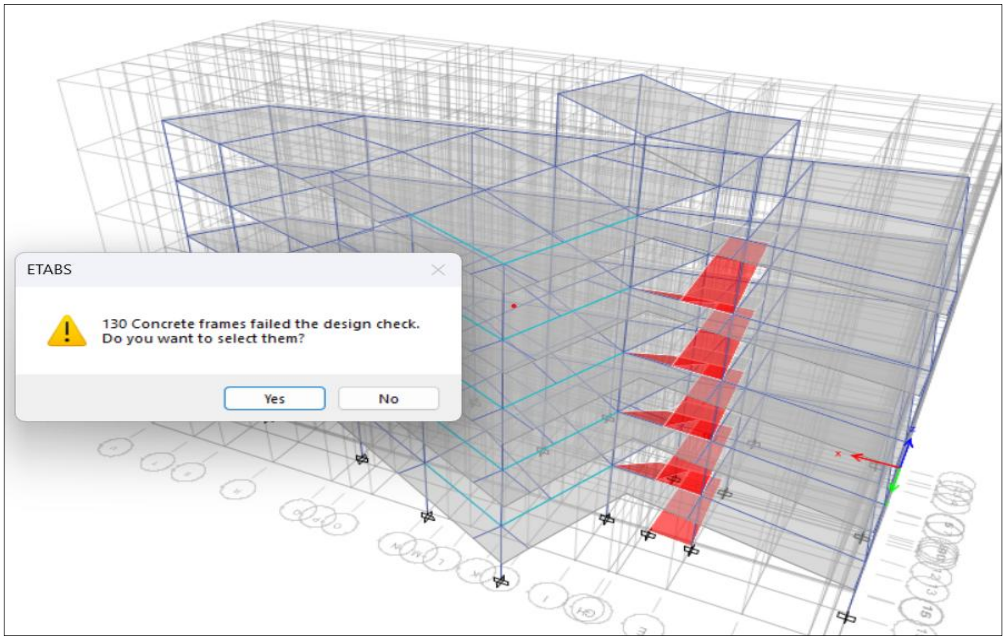


Figure 2: Verifying if all members passed the design check

A. Soft storey condition

As shown in Tables 7-8, soft storey checks revealed no critical weakness in either direction. The stiffness ratios remained above the minimum requirement of 0.7 across all storeys, except for minor irregularity observed at the roof level, which is not structurally significant.

Table 7: Soft storey check for ULS along X-direction (EQX)

Storey	ki	0.7*ki	0.7*(ki+1)	Check if ki>0.7*(ki+1)
6	0	0	0.7	FALSE
5	18871.93	13210.35	13211.05	TRUE
4	27144.51	19001.16	19001.86	TRUE
3	32148.89	22504.22	22504.92	TRUE
2	37321.56	26125.09	26125.79	TRUE
1	55440.43	38808.3	38809	TRUE

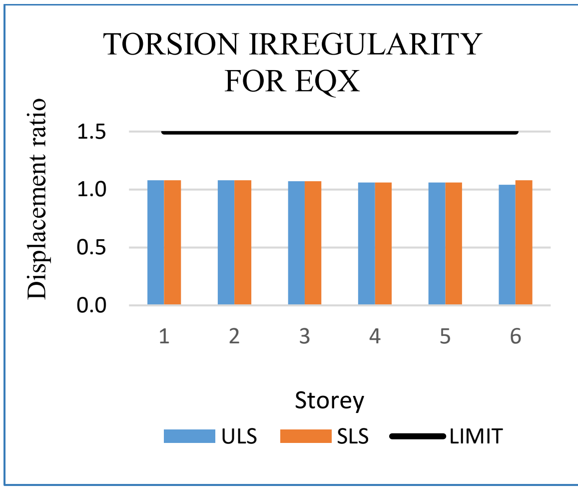
Table 8: Soft storey check for ULS along Y-direction (EQY)

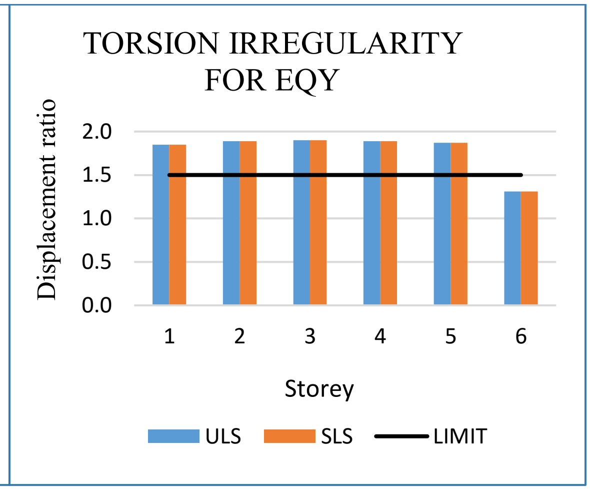
Storey	ki	0.7*ki	0.7*(ki+1)	Check if ki>0.7*(ki+1)
6	0	0	0.7	FALSE

5	16292.41	11404.68	11405.38	TRUE
4	18063.78	12644.64	12645.34	TRUE
3	18665.85	13066.1	13066.8	TRUE
2	19501.22	13650.85	13651.55	TRUE
1	27663.47	19364.43	19365.13	TRUE

B. Torsion Irregularity

As shown in the Figure 3, the building exhibits torsion irregularity in the Y-direction with displacement ratios exceeding 1.5 between storey 1 to 5 under both ULS and SLS. While in the X-direction the displacement ratio remain within the permissible limit, indicating no torsion irregularity.



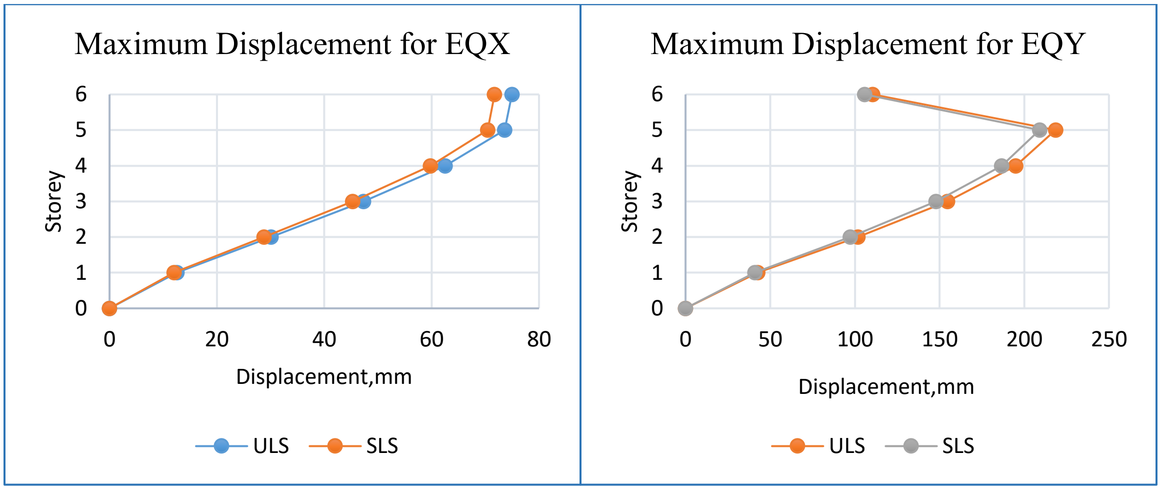


(a) (b)

Figure 3: Torsion irregularity for ULS and SLS in: (a) X-direction; (b) Y-direction.

C. Storey Displacement

As show in the Figure 4, the maximum displacement in X-direction was 74.96 mm under ULS and 71.72 mm under SLS which are both within the permissible limits of 112.5 mm and 108 mm. In Y-direction, the maximum displacement was 218.49 mm and 209.06 mm exceeding the permissible limits.



(a) (b)

Figure 4: Maximum storey displacement for both ULS and SLS in: (a) X-direction; (b) Y-direction

D. Storey Drift

As shown in the Figure (5-6), the maximum storey drift was 0.00567 under ULS and 0.00549 under SLS in X-direction, both within the permissible limits of 0.00625 and 0.006 respectively. In the Y-direction the maximum drift was 0.0197 under ULS and 0.0188 under SLS both exceeding the permissible limits.

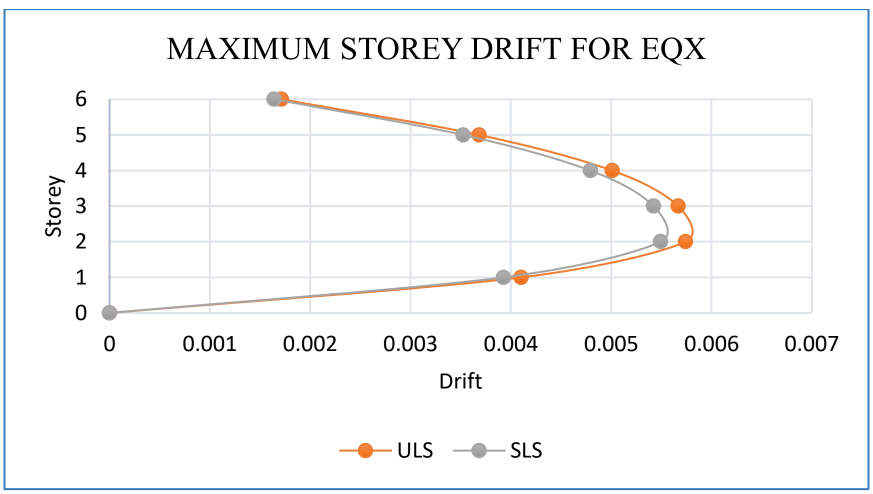


Figure 5: Maximum storey drift for both ULS and SLS in X-direction

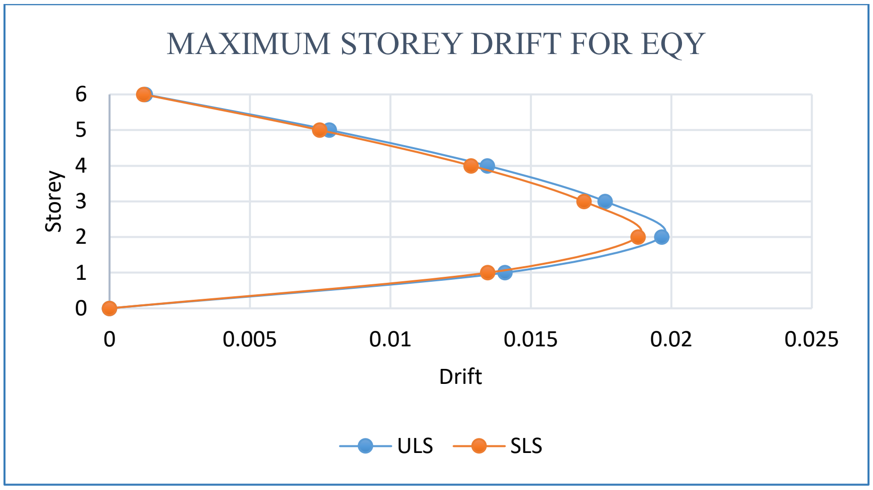


Figure 6: Maximum storey drift for both ULS and SLS in Y-direction

Overall analysis confirms that the building fails to meet seismic performance criteria of NBC 105:2020. Structural analysis shows torsional irregularity in the y-direction, excessive storey displacement and drift values exceeding permissible limits. These failures clearly indicate that the building is vulnerable and requires retrofitting measures to enhance safety and ensure code compliance.

3.3 Hybrid Retrofitting

Since the building was facing several seismic deficiencies at the same time like torsion irregularity, excessive storey drift and displacement we adopted a hybrid retrofitting approach combining concrete jacketing of columns and beams, steel jacketing of beams and addition of shear wall. Concrete jacketing of columns was chosen to increase axial load capacity, ductility and to improve confinement as recommended by IS 15988:2013. Beam showed inadequate flexural and shear resistance so both concrete and steel jacketing was chosen. Furthermore to improve the overall lateral stability of the building and control excessive storey drift reinforced concrete shear wall were added at strategic location. This hybrid approach improves both the strength of individual members and enhance the building's overall seismic performance.

Concrete jacketing of columns was carried out in accordance with IS 15988:2013. As per the code, the minimum thickness of the jacket should be 100 mm and the grade of jacket concrete should be at least 5 MPa higher than that of the existing concrete(Bureau of Indian Standard, 2013).

Table 9: Column Retrofitting Details as per is 15988:2013

Stage	Column specification

Before retrofitting	230×230 mm (M15)
After retrofitting	430×430 mm (M20); 16-12 mm Ø bars
	500×550 mm (M25); 12-16 mm Ø+6-12mm Ø bars
	750×530 mm (M25); 10-20 mm Ø+6-16mm Ø bars

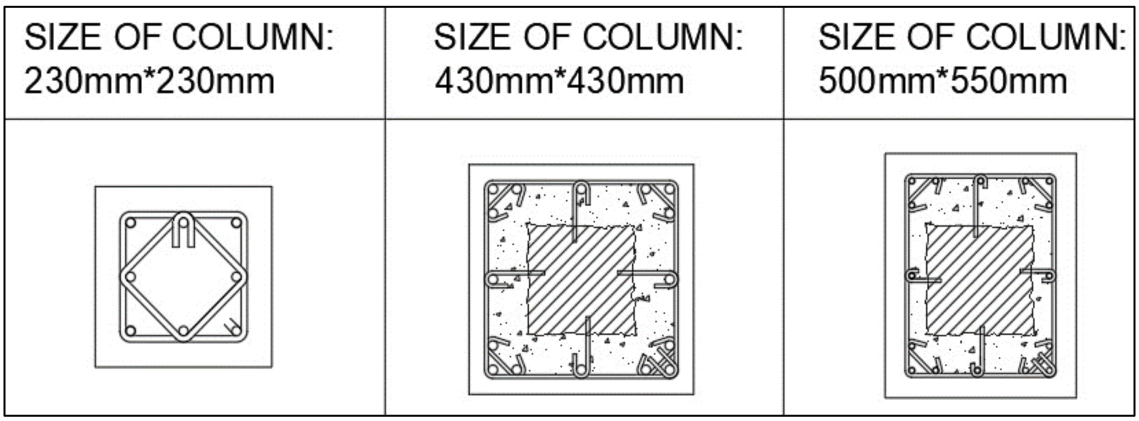
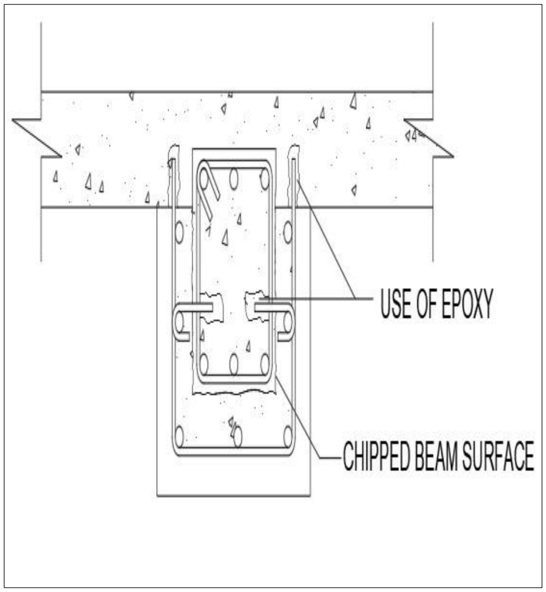


Figure7: Column cross-sections before and after retrofitting

Beam jacketing was designed in accordance with IS 15988:2013. After concrete jacketing, the beam section was enlarged to 375 × 375 mm reinforced with 3-16 mm bars and 4-12 mm bars tied with 8mm stirrups at 100 mm spacing using M20 concrete. For steel jacketing, five different sizes of angles (ISA 70×70×10, ISA 65×65×8, ISA 45×45×6, ISA 45×45×5, ISA 45×45×4) were proposed.



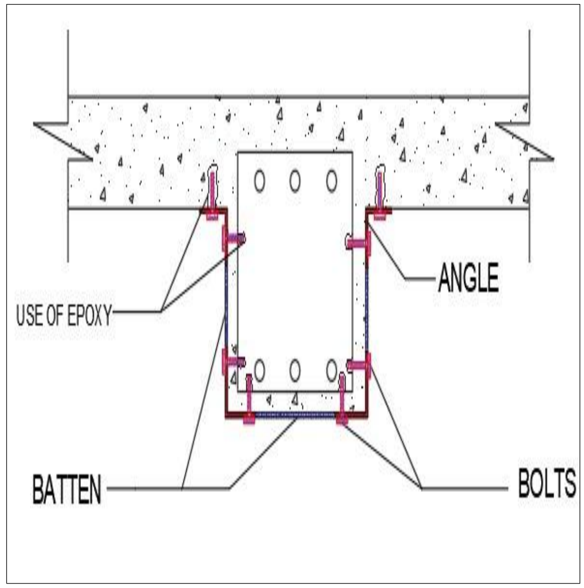


Figure 8: Typical cross-section of beam after RC jacketing and Steel jacketing

Shear walls were introduced at three locations, each 300 mm thick and extending up to 15 m in height, with lengths of 1.2021m, 1.0259m using M20 concrete. At the building corners shear walls of 300 mm thickness were arranged along both the x-axis and the y-axis.

Post-retrofitting analysis

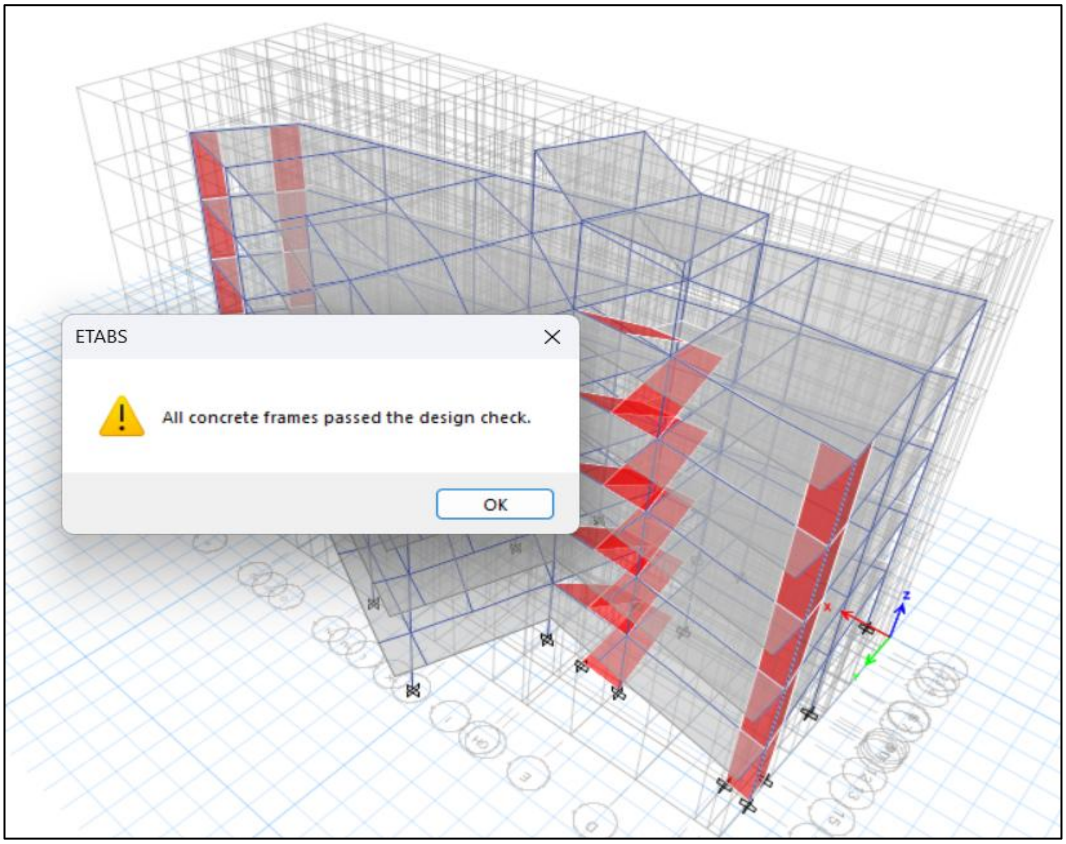


Figure 9: Verifying if all members passed the design checks after retrofitting

A. Soft storey condition

As show in the table (10-11) soft storey condition remained similar to pre-retrofitting case.

Table 10: Soft storey check for ULS along X-direction (EQX) after retrofitting

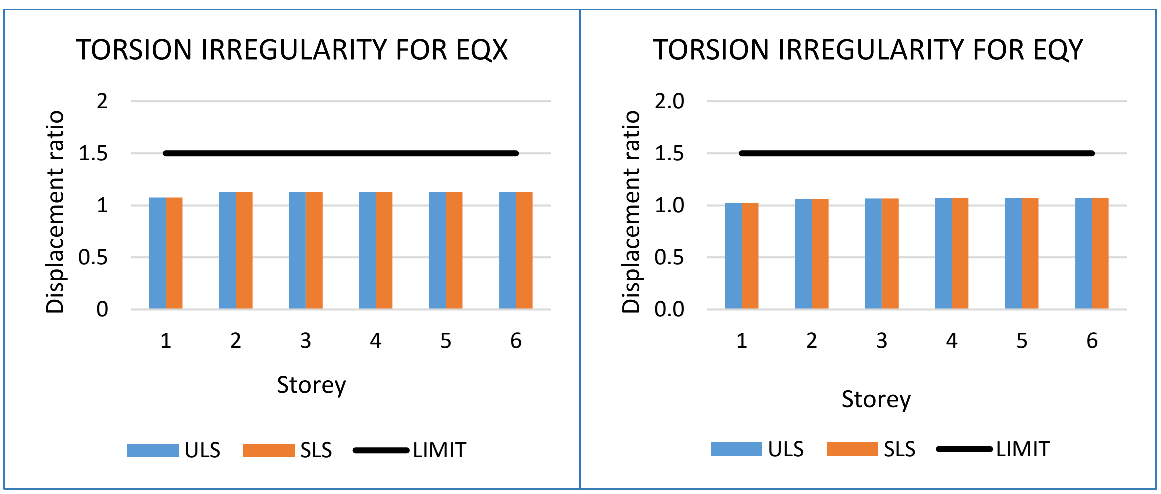
Storey	Stiffness	0.7*ki	0.7*(ki+1)	Check if ki>0.7*(ki+1)
6	0	0	0.7	FALSE
5	45792.22	32054.55	32055.25	TRUE
4	97209.11	68046.38	68047.08	TRUE
3	131303	91912.09	91912.79	TRUE
2	165903.4	116132.4	116133.1	TRUE
1	339327.3	237529.1	237529.8	TRUE

Table 11: Soft storey check for ULS along Y-direction (EQY) after retrofitting

Storey	Stiffness	0.7*ki	0.7*(ki+1)	Check if ki>0.7*(ki+1)
6	0	0	0.7	FALSE
5	41225.21	28857.65	28858.35	TRUE
4	93383.43	65368.4	65369.1	TRUE
3	135481.4	94836.97	94837.67	TRUE
2	202069.3	141448.5	141449.2	TRUE
1	433925.3	303747.7	303748.4	TRUE

B. Torsion irregularity

As shown in the Figure 10, the torsion irregularity in the Y-direction was resolved with ratios below 1.5, while the X-direction remained within the permissible limits.

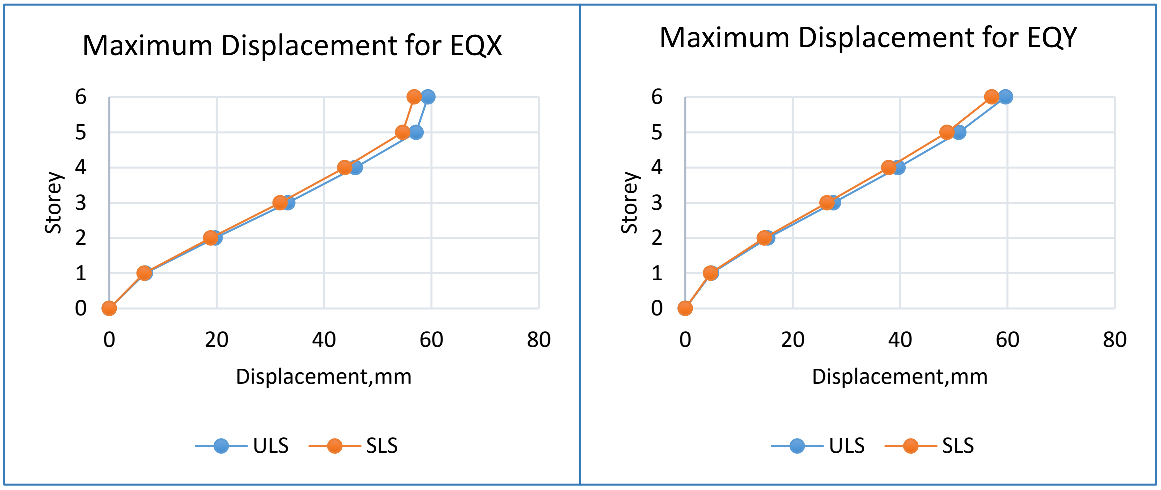


(a) (b)

Figure 10: Torsion irregularity for ULS and SLS after retrofitting in: (a) X-direction; (b) Y-direction

C. Storey Displacement

As shown in the Figure 11, the building no longer shows storey displacement in the Y-direction and the X-direction remains within the limits.



(a) (b)

Figure 11: Maximum displacement for ULS and SLS after retrofitting in: (a) X-direction; (b) Y-direction

D. Storey Drift

As shown in the Figure (12-13) the building no longer shows storey drift in Y-direction and storey drift in X-direction has also been improved.

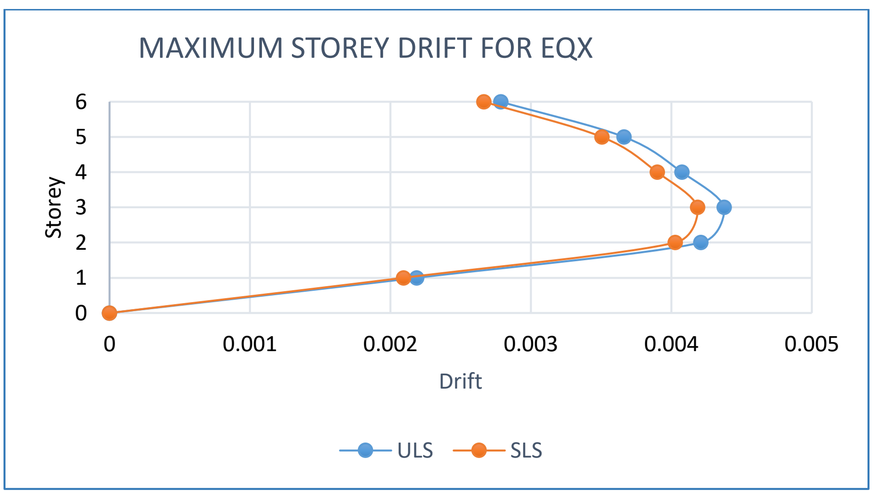


Figure 12: Maximum storey drift for ULS and SLS after retrofitting in X-direction

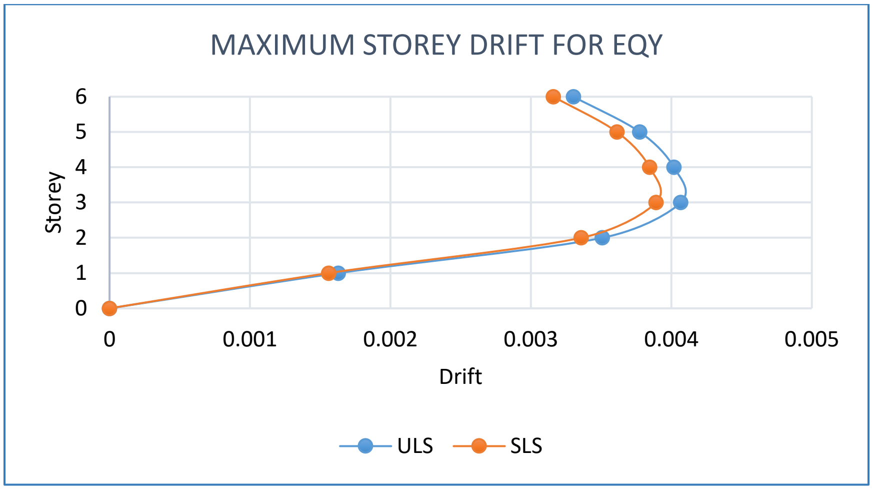


Figure 13: Maximum storey drift for ULS and SLS after retrofitting in Y-direction

Post retrofitting analysis confirms that the building meets the seismic performance criteria of NBC 105:2020. These improvements demonstrate that the retrofitting measures effectively enhanced safety and ensured compliance with code requirements.

3.4 Economic Feasibility

From an economic perspective, the tentative total retrofitting cost was estimated at NPR 80,94,000 based on present market rates for materials and skilled labor. In comparison, the plinth area cost of constructing a new building was approximately NPR 5,20,00,000. This indicates that the retrofitting expenditure amounted to nearly 15.6% of the replacement cost, which remains below the 25% threshold suggested by(Nateghi and Shahbazian, 1992).

4. Conclusion

The combined evaluation revealed that the existing six-storey RC framed building was highly vulnerable to seismic forces, failing both qualitative and quantitative evaluation. Pre-retrofitting analysis revealed torsion irregularity, excessive displacement and drift, while NDT revealed weak surface strength.

The proposed hybrid retrofitting were effective in addressing those deficiencies. Post-retrofitting analysis showed compliance with NBC 105:2020 eliminating torsion irregularity and controlling excessive displacement and drift.

From an economic point of view, retrofitting of the building is far more affordable than demolishing and constructing a new one. The retrofitting cost came out to be NPR 80.94 lakh while a new building would have cost around NPR 5.2 crore. In other words, retrofitting required only about 15.6% of the replacement cost, making it the practical choice. Hence, hybrid retrofitting stands out as both technically effective and economically practical.

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