

Seismic Performance of FRP-Retrofitted RC Building Using Pushover Analysis

Nabil Mochammad Yusuf*, Ari Wibowo, Retno Anggraini

Abstract— The seismic evaluation of existing bank office buildings is critical for ensuring post-earthquake operational continuity. This study investigates a 14-story RC bank building in Kendari using the Nonlinear Static Procedure (NSP) per ASCE 41-17, targeting an Immediate Occupancy (IO) performance level. The initial analysis revealed a critical contradiction: while the structure's global performance appeared to meet the IO target, this assessment was found to be misleading. A detailed, element-based analysis identified a concealed local failure where a critical beam reached the Life Safety (LS) performance level, caused by a non-trivial Positive Moment at the support. A sequential Fiber Reinforced Polymer (FRP) retrofitting strategy was then implemented. The primary contribution of this study is the demonstration of the Failure Migration phenomenon. It was shown that a naive, 'single-point' retrofit (on LS-1) did not solve the problem but merely shifted the failure mode to the next weakest element (LS-2). This sequential retrofitting procedure proved necessary to track the migrating failure, which moved non-linearly between various floors, until all migrating vulnerabilities were eliminated. This finding proves that a sequential procedure is necessary to address Force Redistribution and achieve a true IO performance. The final minor global base shear increase (0.16%) was only a secondary benefit, confirming the objective was local vulnerability elimination, not a significant increase in global stiffness.

Index Terms— Fiber Reinforced Polymer, Pushover Analysis, Reinforced Concrete, Seismic Performance.

I. INTRODUCTION

The seismic resilience of office buildings, particularly those used by financial institutions such as banks, plays a crucial role in maintaining economic stability and social functions [1]. Over time, many bank office buildings undergo functional changes, such as converting floors into storage areas. These changes alter load distribution and may reduce the structural performance against lateral seismic loads, thereby requiring a comprehensive structural evaluation [2], [3].

Among various approaches, pushover analysis is recognized as an effective method for evaluating building performance under seismic loads. It provides insights into structural capacity, identifies potential weak points, and guides decision-making for retrofitting strategies [4], [5]. On the other hand, Fiber Reinforced Polymer (FRP) has emerged as an efficient retrofitting material due to its lightweight, high strength, and corrosion resistance, making it suitable for seismic retrofitting [2], [6].

However, much of the literature focuses on global 'before-and-after' performance comparisons. A critical gap exists regarding the consequences of a naive, localized, 'single-point' retrofit. Strengthening only one "weak link" can be misleading due to Force Redistribution.

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Strengthening one element merely shifts (migrates) the failure mode to the next weakest element, a concept known as altering the "hierarchy of strengths" [7].

This study has a twofold objective. First, to evaluate the global seismic response of the structure in terms of base shear and displacement. Second, to demonstrate the effectiveness of FRP retrofitting in mitigating identified local vulnerabilities, aiming to restore failed beam elements from a Life Safety (LS) performance level back to Immediate Occupancy (IO).

The primary contribution of this study is not the global performance evaluation, which was found to be insignificant and potentially misleading. Rather, the contribution lies in an in-depth analysis of the complexity arising from this second objective. This study demonstrates that global performance assessments (e.g., IO) can conceal critical local (LS) vulnerabilities. Furthermore, it demonstrates the Failure Migration phenomenon, or Force Redistribution, where fixing a single element (LS-1) to restore it to IO merely shifts the failure mode to the next weakest element (LS-2, LS-3, etc.). This finding highlights the necessity of a sequential retrofitting procedure to identify and eliminate all migrating vulnerabilities, as the only means to achieve a comprehensively resilient IO performance.

II. METHODS

A. Data and Modeling

The subject of this study is a 14-story reinforced concrete bank office building in Kendari, Indonesia, constructed in 2017. The building underwent a functional change where one floor was converted into a storage area, necessitating a structural evaluation. The material properties adopted in the analysis were a concrete compressive strength (f'_c) of 25 MPa and a reinforcement yield strength (f_y) of 390 MPa.

The detailed geometry and member sizes for the primary beams and columns used in the model are presented in Table I. This information forms the basis for developing an accurate analytical model of the building. It also ensures that the evaluation reflects the actual structural capacity and behavior under the modified loading conditions.

TABLE I. DESIGNED MEMBER SIZES

Element Type	Name	Size (mm)	Location (Floor)
Column	K1	900 x 900	1 – 4
	K2	800 x 800	5 – 7
	K3	700 x 700	8 – 10
	K4	600 x 600	11 – 14
Beam	Main Beam	350 x 700	All Floors

A three-dimensional (3D) structural model was developed using ETABS software to represent the actual condition of the building. The 3D model of the structure is shown in Fig. 1.

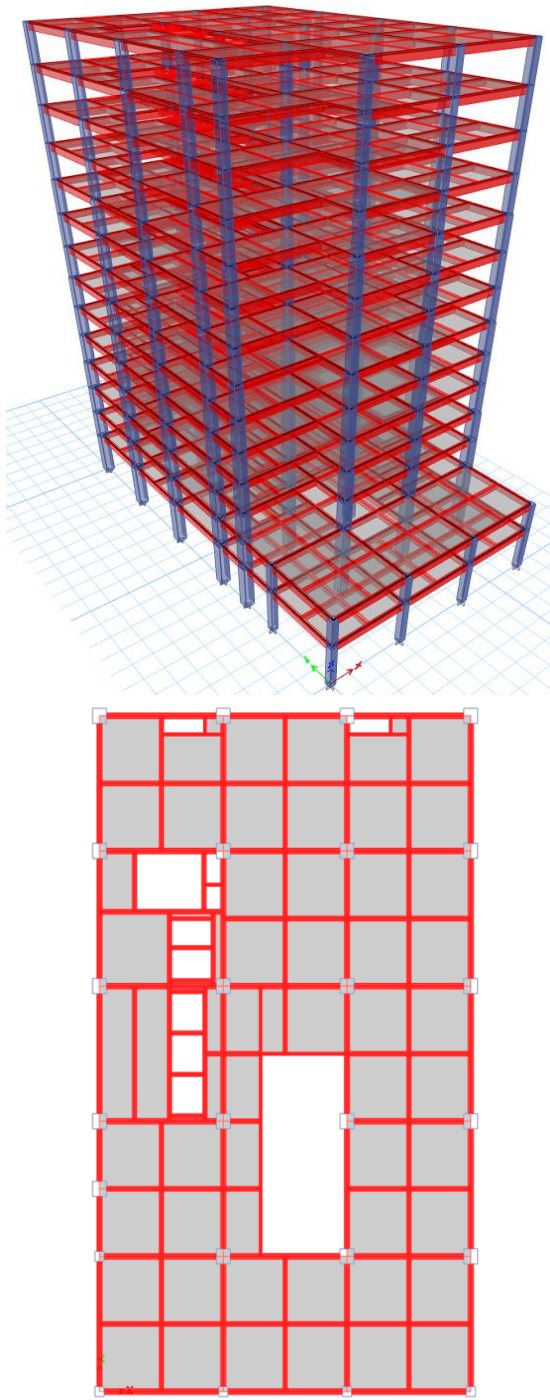


Fig. 1. Finite element model and structural grid

B. Modeling Assumptions and Validation

This 3D model adopts the semi-rigid diaphragm assumption for all floor slabs. Unlike the *rigid diaphragm* assumption, which considers the slab infinitely stiff, the *semi-rigid* approach models the slab as a *shell* element with realistic in-plane stiffness. This choice ensures that the lateral seismic force distribution accounts for the actual in-plane deformation of the slab. Comparative studies have confirmed that the *semi-rigid* approach yields a more accurate representation of the actual structural behavior, especially in buildings with irregularities [8].

Furthermore, the stiffness and strength contributions from non-structural elements, such as masonry infill walls, were disregarded in the analysis. This "bare frame" approach is a common scenario in seismic assessment [9]

and represents a deliberate conservative technical choice. The primary reason is to avoid reliance on elements whose behavior is highly complex and brittle under cyclic seismic loading [10]. Moreover, this omission aims to ensure the safety of the primary frame and prevent unforeseen local failures, such as the "short-column effect", which has been experimentally proven to arise from partial frame-infill interaction [11].

C. Non-Linear Static Procedure

The seismic performance was evaluated using the Non-Linear Static Procedure (NSP), or pushover analysis, in accordance with ASCE 41-17. This method was selected (over Non-Linear Dynamic Procedure) as it is highly effective for the study's primary objective: identifying local vulnerabilities and tracking the migrating failure mechanisms [7] during a design optimization process [[12].

Several key non-linear modeling parameters were defined for the analysis. Boundary conditions for all columns at the base level were modeled as Fixed Supports. The seismic mass source was accurately defined per SNI 1726:2019, including 100% of Dead Load (DL) and 25% of Live Load (LL). A 5% viscous damping ratio ($\beta=0.05$) was assumed to construct the elastic Demand Spectrum, as recommended by ASCE 41-17 for RC structures.

The pushover analysis was performed in ETABS. Lateral loads were applied incrementally, distributed proportionally to the fundamental mode shape of the structure. The capacity curve (base shear vs. roof displacement) was then generated. Non-linear plastic hinges were assigned: M3 (flexural) hinges for beams and P-M2-M3 (axial-flexural) hinges for columns, per ASCE 41-17 properties. The progression of these hinges (IO, LS, CP) was used to identify local failures.

Finally, the Displacement Coefficient Method (DCM) from ASCE 41-17 was adopted to determine the performance point. This point represents the intersection of the capacity curve and the demand spectrum constructed based on the criteria of SNI 1726:2019 for the Kendari site.

D. Seismic Load

Seismic loads were determined in accordance with SNI 1726:2019, considering the seismic hazard parameters for Kendari. Based on its function as a 14-story office building, the structure is classified as Risk Category II, with a corresponding importance factor of $I_e = 1.0$.

The seismic hazard parameters adopted include the short-period spectral acceleration (S_s) and the 1-second spectral acceleration (S_1). These values were adjusted using the site coefficients (F_a and F_v) according to the soil classification to obtain the design spectral accelerations (S_{DS} and S_{D1}), which are summarized in Table II.

TABLE II. DESIGN SPECTRAL RESPONSE PARAMETERS

Parameter	225-Year	975-Year
Spectral acceleration at base rock (0.2-s) (S_s)	0.300	0.500
Spectral acceleration at base rock (1-s) (S_1)	0.100	0.150
Short-period coefficient (0.2-s) (F_a)	2.260	1.700
Short-period coefficient (1-s) (F_v)	4.200	3.750
Spectral acceleration for short period (S_{DS})	0.678	0.850
Spectral acceleration for 1-s period (S_{D1})	0.420	0.562
Control period (T_0)	0.123	0.132
Control period (T_1)	0.619	0.661

Based on these parameters, the Design Response Spectrum (Demand Spectrum) was developed, as shown in Fig. 2. Two seismic hazard levels were considered to represent different performance objectives: a 225-year return period, corresponding to the Life Safety (LS)

performance level, and a 975-year return period, corresponding to the Collapse Prevention (CP) performance level. This demand spectrum was then used in the nonlinear pushover analysis to determine the performance point of the structure.

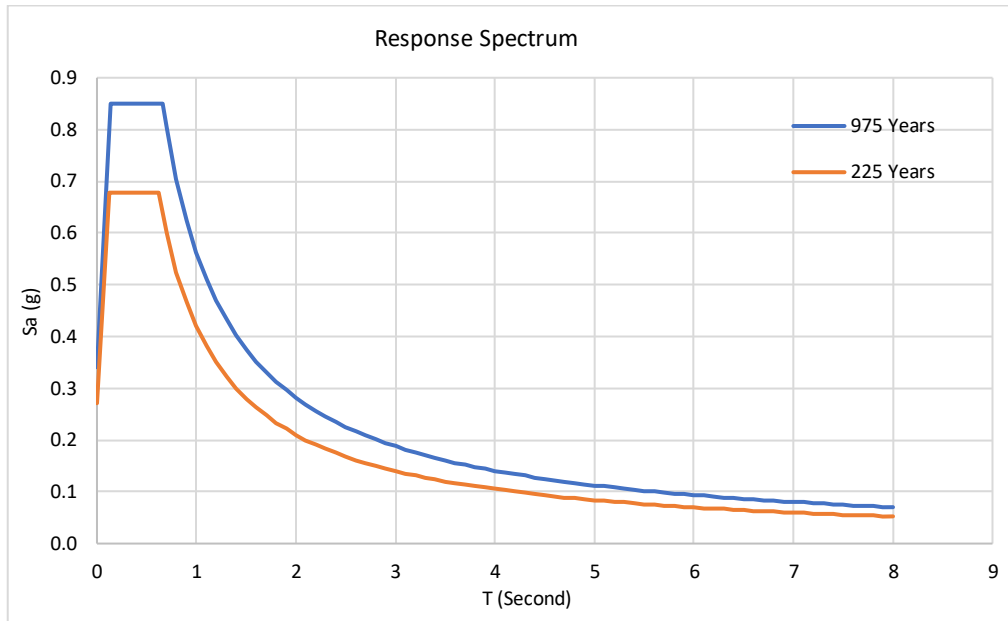


Fig. 2. Design response spectrum

E. FRP Retrofitting Method

The strengthening material selected was a high-strength, unidirectional Carbon Fiber Reinforced Polymer (CFRP) sheet, SikaWrap®-600 C WV. The flexural strengthening design followed the guidelines of ACI 440.2R-17. Material properties are presented in Table III. Based on ACI 440 design calculations, it was determined that one ply of laminate was required.

TABLE III. MECHANICAL PROPERTIES OF CFRP MATERIAL (SIKAWRAP®-600 C WV)

Property	Value	Unit
Type	CFRP Wrap	–
Thickness per ply	0.331	mm
Tensile strength	4000	MPa
Ultimate strain	1.56	%
Elastic modulus	230,000	MPa

The technical justification for the retrofit strategy was based on the local vulnerability findings (detailed in Section III.A). The existing analysis showed that all column elements already met the Immediate Occupancy (IO) performance level, with none reaching LS. Therefore, the retrofit strategy was focused solely on the beam elements, where local failures were identified. The existing analysis showed the LS beam failure occurred at the Support (Relative Distance = 0.1). The ETABS output explicitly recorded this failure at a Positive Moment (+488 kNm). By definition, a Positive Moment induces tension forces at the bottom face of the beam. Therefore, the FRP laminate was applied to the bottom face of the beam in the support region, as detailed in Fig. 3.

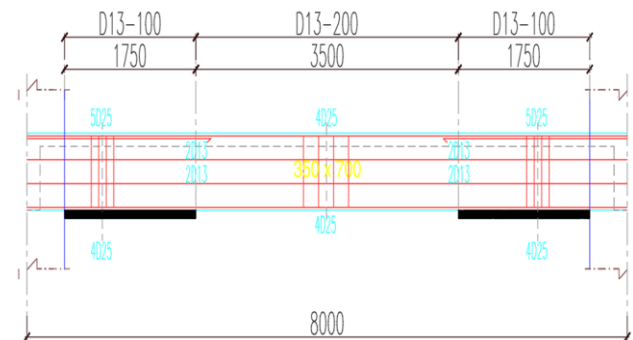


Fig. 3. Detail of the FRP flexural strengthening configuration applied to the beam's bottom face at the support region to resist tension forces caused by Positive Moment.

The effect of this strengthening was then modeled in ETABS by manually modifying the plastic hinge properties. The calculated flexural capacities (including the new Yield Moment) for the retrofitted beam were used to define a new backbone curve for the hinge, replacing the program-generated default properties.

A sequential retrofitting procedure (detailed in Section III.B) was applied instead of a naive, 'single-point' retrofit. The technical justification for this approach is based on the well-established scientific fact that externally bonded (EB) flexural strengthening significantly increases Strength but does not appreciably increase elemental Stiffness (EI) [13]. Because stiffness remained largely unchanged, the local increase in strength (Stage 1) forced the failure to migrate to the next weakest element (Force Redistribution), a concept known as altering the "hierarchy of strengths" [7]. Therefore, this stepwise 'chase-and-eliminate' strategy was necessary to identify and remove all migrating local vulnerabilities.

III. RESULTS AND DISCUSSION

A. Seismic Performance of the Existing Building.

This study evaluates an office building (Risk Category II) against an enhanced performance objective. While the minimum code requirement for this category is Life Safety (LS), this study adopts a stricter target of Immediate Occupancy (IO) to ensure post-earthquake operational continuity.

A nonlinear pushover analysis was performed to generate the capacity curve of the existing structure, shown in Fig. 4. This curve plots the Base Shear versus Roof Displacement, indicating the structure's capacity from its elastic to inelastic phase. The Performance Points (PP) for the two hazard levels were then determined from this curve.

A summary of these performance points is presented in Table IV. As the table shows, the global roof drift ratio analysis for both the 225-year and 975-year return periods indicated an Immediate Occupancy (IO) performance level.

TABLE IV. TARGET DISPLACEMENT OF THE STRUCTURE

Period	Direction	Displacement	Roof drift	Structural Performance Level	Shear	Step
		(mm)	Ratio (%)		(kN)	
225 Year	X	678.136	1.17%	IO	9263,751	3
	Y	633.125	1.09%	IO	10113,233	3
975 Year	X	902.271	1.56%	IO	11566,968	4
	Y	843.519	1.45%	IO	12428,232	4

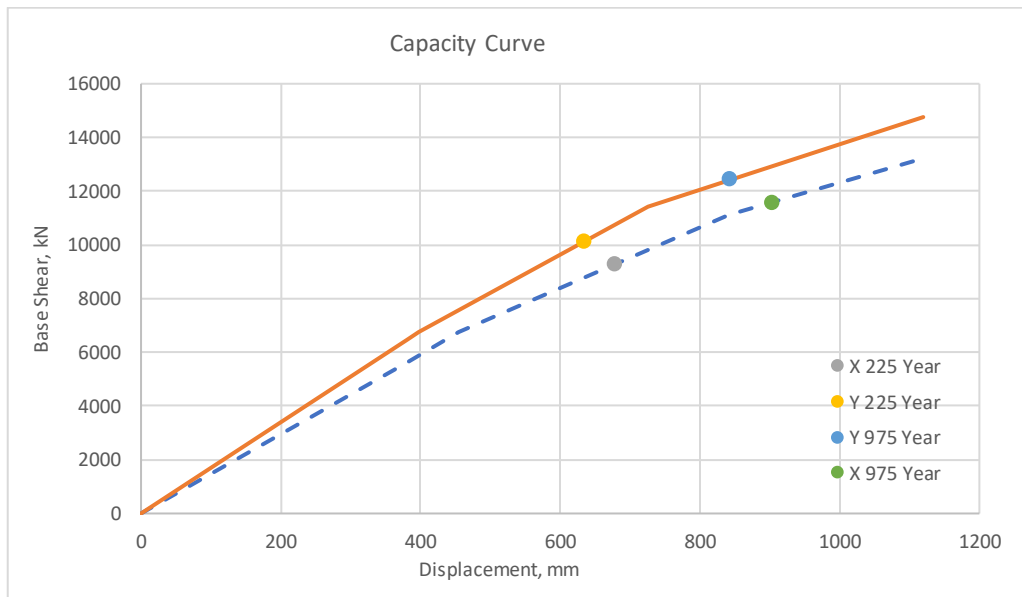


Fig. 4. Capacity curve of the structure before FRP strengthening

B. Plastic Hinge Mechanism

The plastic hinge mechanism analysis was conducted to track inelastic deformation. The evaluation point was selected at Step 4 of the pushover curve, corresponding to the Performance Point (PP) where the structure exceeded its elastic limit.

In the existing condition, Fig. 4a shows that in the X-direction, one critical beam element (LS-1) on Floor 3 reached the Life Safety (LS) state (light blue dot). A non-trivial finding revealed this LS failure was caused by a Positive Moment (+488 kNm). After the Stage 1 retrofit (on beam LS-1, Floor 3), the pushover analysis was repeated. The results in Fig. 4b showed improved capacity in that beam (now IO/green); however, a new LS condition

Based on these global results, the structure appears to meet the enhanced IO target. However, this global performance assessment based solely on drift (as shown in Fig. 3 and Table III) was found to be misleading and concealed a critical local vulnerability.

Despite the structure appearing globally safe (IO), a detailed analysis of the plastic hinge mechanisms (detailed in Section III.B) revealed a significant deficiency. One local failure was identified in a critical beam element in the X-direction. This beam failed prematurely, reaching the Life Safety (LS) performance level.

Any local failure at the LS level, even in a single element, automatically disqualifies the structure from meeting the stringent IO performance target. This "weak link" became the structural justification and primary target for the FRP retrofitting strategy. Therefore, the goal of the retrofit was not to improve the global IO performance, but rather to eliminate this critical local (LS) vulnerability.

appeared at a different beam location (LS-2) on Floor 4. This finding demonstrates the phenomenon of Force Redistribution or Failure Migration. As explained in Section II.E, because FRP adds Strength (not Stiffness) [13], strengthening one "weak link" (LS-1) merely shifted the inelastic ductility demand to the next weakest element [7].

This sequential retrofitting procedure continued in a non-linear, non-intuitive sequence, proving the necessity of the sequential analysis over a naive 'single-point' retrofit. Retrofitting LS-2 (Floor 4), the results in Fig. 4c show the failure migrated again, causing a new LS-3 (light blue dot) on Floor 2. Following this finding, the retrofit was applied to LS-3 (Floor 2). The subsequent pushover

analysis, shown in Fig. 4d, revealed the failure migrated once more, now causing LS-4 (light blue dot) on Floor 1.

At the final stage (Stage 4), after retrofitting LS-4 (Floor 1), Fig. 4e shows that no structural elements reached

the LS condition. All plastic hinges (green dots) now exhibited Immediate Occupancy (IO) performance. This proves that the sequential procedure was necessary to eliminate all migrating weaknesses.

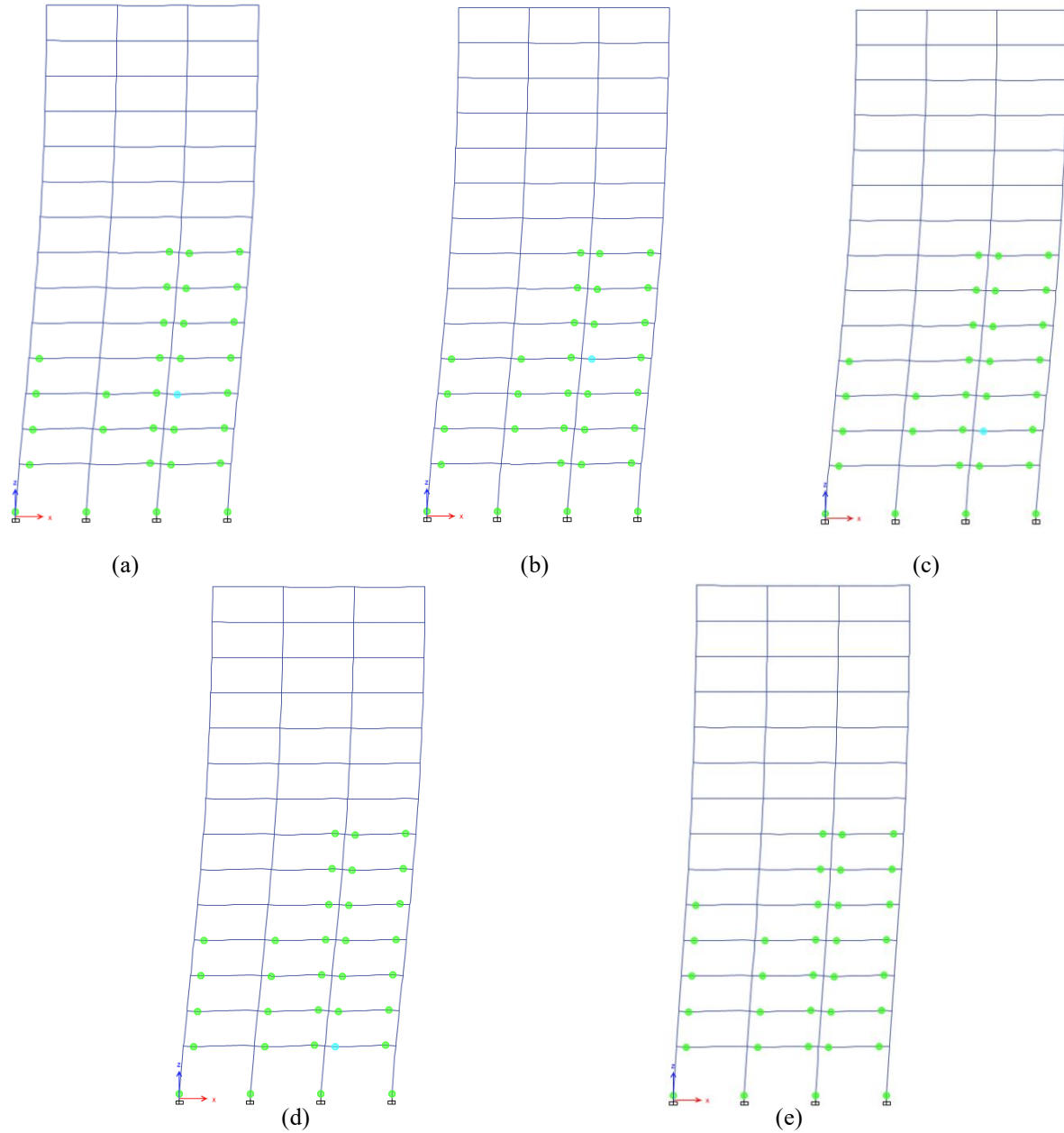


Fig. 5. Evolution of plastic hinges and failure migration sequence: (a) Existing (LS at Fl. 3); (b) Stage 1 (Migration to Fl. 4); (c) Stage 2 (Migration to Fl. 2); (d) Stage 3 (Migration to Fl. 1); (e) Final Stage (All elements achieve IO).

C. Structural Performance Level after FRP Retrofitting

To verify the final effectiveness of the sequential FRP retrofitting strategy, the post-retrofit (Stage 4) structural performance was evaluated based on the ASCE 41-17 Nonlinear Static Procedure (NSP). The global analysis results are summarized in Table V and Fig. 6. Plastic hinge mechanism analysis was conducted to track inelastic deformation. The evaluation point was selected at Step 4 of the pushover curve, corresponding to the Performance Point (PP) where the structure exceeded its elastic limit.

Table V presents the final performance point after retrofitting a total of four critical beams (distributed across Floors 1, 2, 3, and 4). The results show that the Structural Performance Level for all directions (X and Y) at both the 225-year and 975-year hazard levels has successfully

achieved Immediate Occupancy (IO). This confirms that the primary objective of the study—eliminating all migrating local vulnerabilities (LS) without radically altering the structural response—was met.

Fig. 6 plots the final (post-retrofit) capacity curve against the existing condition. A minor increase in the global capacity of the X-direction (orange curve) is observable compared to the existing curve (dashed line). The data in Table V confirms this: the X-direction base shear (at the 975-Year event) increased slightly from 11,566.85 kN to 11,585.71 kN. This minor increase is consistent with the research argument that FRP retrofitting focused on enhancing local element strength and ductility, rather than adding excessive global stiffness.

Furthermore, the data shows a marginal increase in roof

displacement from 902.271 mm (Existing) to 902.857 mm (Final). This displacement increase confirms that the FRP intervention did not drastically alter the global stiffness but instead successfully preserved the system's deformation capacity (ductility) post-retrofitting. This global capacity enhancement serves as a secondary benefit, resulting from the elimination of "weak links" (LS elements) on Floors 1

through 4. By strengthening these beams, premature failure was prevented, allowing the structure to redistribute lateral forces (Force Redistribution) more effectively to other, stronger elements. In contrast, the Y-direction response (green/red dots on Fig. 6) remained stable and unchanged, as no retrofitting intervention was required along that axis.

TABLE V. TARGET DISPLACEMENT OF THE STRUCTURE AFTER FRP STRENGTHENING

Period	Direction	Displacement	Roof drift	Structural Performance Level	Shear	Step
		(mm)	Ratio (%)		(kN)	
225 Year	X	678.310	1.17%	IO	9289.520	3
	Y	633.125	1.09%	IO	10113.233	3
975 Year	X	902.857	1.56%	IO	11585.709	4
	Y	843.519	1.45%	IO	12428.232	4

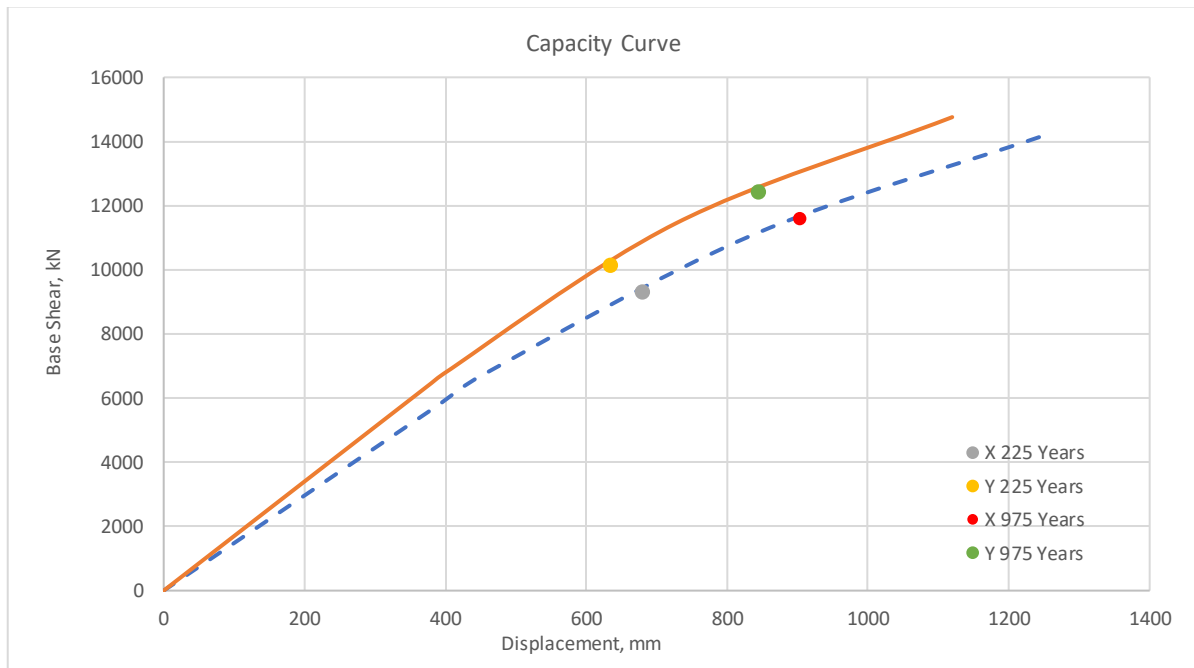


Fig. 6. Capacity curve of the structure After FRP strengthening

D. Comparative Analysis and Interpretation of Failure Mechanism

The findings of this study provide critical insights that contrast with conventional seismic assessment approaches. While previous studies, such as those by Li *et al.* [2] and Güneş & Tümer [6], prioritized the maximization of global base shear capacity as the primary success indicator, this study demonstrates that relying solely on such global metrics can be misleading. Our analysis reveals that a structure can satisfy global Immediate Occupancy (IO) criteria while simultaneously concealing critical local vulnerabilities ("hidden failures") that jeopardize stability.

The empirical observation of the Failure Migration phenomenon—where failure shifted non-linearly from Floor 3 to 4, then to 2, and finally to 1—validates the "hierarchy of strengths" concept described by Pampanin *et al.* [7]. This confirms that a localized retrofit triggers Force Redistribution, shifting the demand to the next weakest element. Furthermore, the final results show only a marginal increase in global base shear (0.16%) and a slight increase in roof displacement. This behavior is consistent with the composite mechanics principles established by

Bakis *et al.* [13], which state that FRP retrofitting significantly enhances element strength and ductility without substantially altering the structural stiffness. Therefore, the sequential procedure proves effective not by stiffening the building, but by selectively eliminating brittle failure modes to ensure a stable ductile response.

IV. CONCLUSION

This study underscores a critical limitation of conventional seismic assessment methods that rely solely on global performance metrics. In contrast to previous approaches such as those by Li *et al.* [2] and Güneş & Tümer [6], which prioritized global base shear enhancement as the primary success indicator, our results demonstrate that such global indicators can be misleading. Although the structure appeared to meet the Immediate Occupancy (IO) criteria globally in the initial condition [Table III], a detailed element-based analysis successfully detected a concealed local failure (LS) on Floor 3, triggered by a non-trivial mechanism (Positive Moment at the Support). This finding confirms that post-earthquake structural stability is determined more by the elimination of local vulnerabilities than by a mere increase in total

capacity.

The primary contribution of this study is the empirical demonstration of the Failure Migration phenomenon. The retrofitting process revealed that failure migrated in a non-linear and unpredictable sequence: from Floor 3 to 4, then down to 2, and finally to 1. This finding confirms the "hierarchy of strengths" concept described by Pampanin et al. [7]. It proves that a naive 'single-point' retrofit did not solve the problem but instead triggered Force Redistribution to the next weakest elements. This mechanism is consistent with the mechanical principle that FRP retrofitting enhances element strength and ductility without significantly increasing global stiffness, as established by Bakis et al. [13].

Therefore, a sequential (stepwise) retrofitting procedure proved absolutely necessary to track and eliminate all migrating vulnerabilities. The observed final global base shear increase of 0.16% serves merely as a secondary benefit. Furthermore, the data shows a marginal increase in roof displacement from 902.271 mm (Existing) to 902.857 mm (Final). This displacement increase confirms that the FRP intervention did not drastically alter the global stiffness but instead successfully preserved the system's deformation capacity (ductility) post-retrofitting. The authors acknowledge the limitations of this study, which utilized a conservative "bare frame" model [9] and the Non-Linear Static Procedure (NSP). While NSP proved effective for tracking failure mechanisms in this context [12], future research is recommended to employ Non-Linear Dynamic Procedure (Time History Analysis) to validate the structural response under actual dynamic seismic loads.

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