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Retrofit Proposal for a 70-Year-Old Educational Building in a High-Seismicity Zone

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Abstract **-** This article analyses the structural behaviour of an educational pavilion more than 70 years old in a highly seismic area, with damage to beams and columns attributed to the current use of the roof as a warehouse, in addition to having a high probability of collapse according to the FEMA P-154 seismic vulnerability study. Diamond extractions were carried out based on the ASCE 41-06 standard to obtain the compressive strength of the columns and beams. After the tests, linear and nonlinear analysis were performed based on the E.030 and ASCE 41-17 standards, respectively. The structural modelling confirmed non-compliance with the maximum distortion of 7/1000 allowed by the standard for the Y-axis gantry system. Therefore, RC jacketing was proposed as a reinforcement alternative according to the UNAM reinforcement guide and the IS 15988:2013 standard. The encasement of columns along the Y-axis reduced the drift by 58%, while the encasement of beams resulted in a 45% reduction. However, when implemented separately, neither of these encasements met the regulatory limit, prompting the adoption of a combined encasement solution, which achieved an 80% reduction. Whereas the pushover analysis indicated that in the event of frequent and occasional earthquakes, the reinforced structure would be fully operational, which shows the importance of developing a reinforcement plan at the superstructure level addressed in this research, which not only offers a reinforcement solution, but also serves as a reference for similar initiatives in other countries.

Keywords**:** RC jacketing, nonlinear analysis, structural reinforcement, vulnerability

1. Introduction

In recent years, global events have exposed the structural weakness caused by deterioration resulting from high-magnitude seismic events, lack of maintenance, poor construction practices, or the absence of regulations during the construction phase. Buildings with high levels of deterioration may exhibit issues such as reinforcement steel corrosion in columns and beams, or the appearance of cracks in structural elements, indicating material fatigue. These and other visible defects may be sufficient to classify the structure as unsafe. Structural deterioration poses a significant threat to the safety of occupants, as weakened structures may fail during low-magnitude seismic events, potentially resulting in human and economic losses. Furthermore, in the event of a collapse, such failures could have environmental repercussions.

Structural retrofitting encompasses a wide range of techniques tailored to the type of failure observed in structural elements, such as concrete jacketing, steel jacketing, CFRP application, plate addition, bracing installation, among others [1]. Each method offers specific advantages over others, such as cost efficiency, implementation speed, drift reduction, or increased strength. [2] presented a study on the axial strength of a reinforced concrete jacketed column, basing the calculations for jacket thickness and required steel reinforcement on the IS 15988:2013 standard. The author validated these calculations through ABAQUS modelling, reporting a 16.6% error margin for the jacketed column. [3] reinforced a two-storey frame using steel jacketing, achieving sufficient deformation capacity and a 33% increase in stiffness. Regarding corrosion issues, UHPC jacketing proved effective when applied to damaged specimens, doubling their drift capacity [4]. [5] investigated the application of steel jacketing, FRP, and CFRP on a structure damaged by corrosion, identifying specific advantages and disadvantages for each retrofitting method.

Most articles on the subject perform linear analysis, either static or dynamic, as the aim is generally to obtain results focused on local improvements rather than on the overall structure. However, this analysis is insufficient when aiming to enhance the global performance of the structure, making it necessary to conduct a pushover analysis to determine damage levels in columns and beams, as well as overall structural performance. [6] carried out a nonlinear static analysis on four reinforced concrete frame buildings using the capacity spectrum method and the displacement coefficient method. The study concluded that the capacity spectrum method is the most conservative, as it results in higher displacements compared to the displacement coefficient method. [7, 8] performed a nonlinear analysis on a school structure similar to the one examined in this paper. The authors used fiber plasticity to represent the nonlinearity of columns, beams, and walls, employing a wide column model to assign nonlinearity to the latter.

As previously mentioned, the lack of global structural analysis hinders the ability to assess the behaviour of a building under seismic events or to quantify the impact of deterioration on the structural performance of individual elements or the structure. Therefore, this study conducts a global structural analysis and proposes the retrofitting of an educational pavilion over 70 years old, deteriorated, and constructed without compliance to any code. The study considers the nonlinear behaviour of materials and subsequently verifies the effectiveness of the retrofitting through the building's capacity curve and its response to different types of seismic events.

2. Materials and methods

2.1. Characteristics of the structure

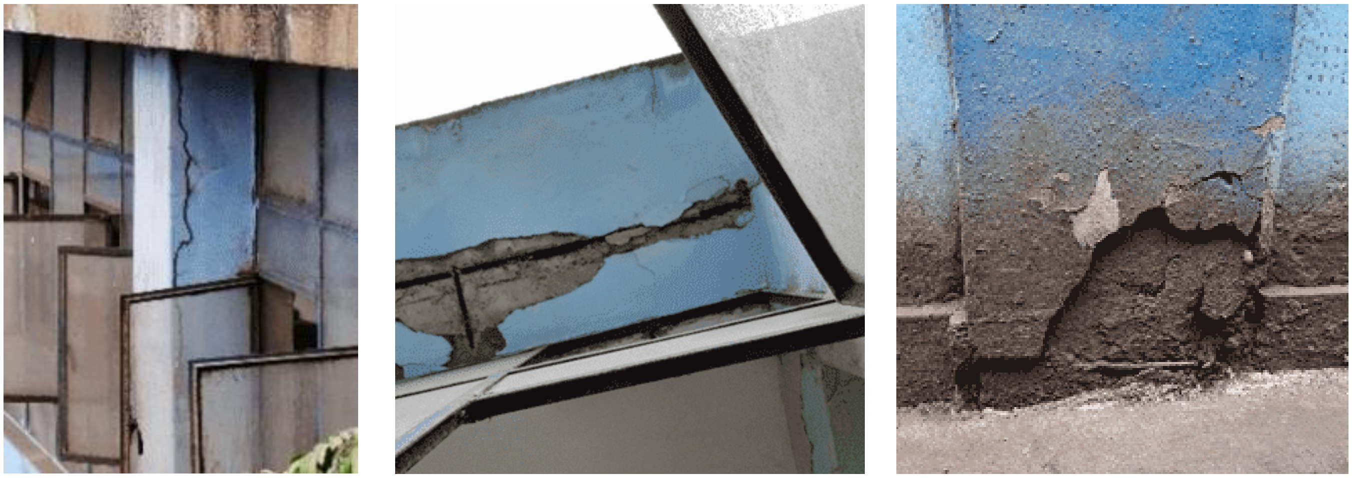
In this study, a building over 70 years old was analyzed, and therefore, the NTP (Peruvian Technical Standard) E.030 and E.060 were not used in its construction. Consequently, the structural design and analysis of the building were carried out, and diamond cores were extracted in accordance with the provisions of NTP 339.059, E.060, and ASCE 41-06. Additionally, a structural survey of the building was conducted since the original construction plans are not available due to its age. For this purpose, a masonry system was considered in the X direction and a frame system in the Y direction. Likewise, based on field visits, columns with dimensions of 30x35 cm, 35x35 cm, and 30x50 cm were identified, as well as deep beams in the X-axis and flat beams in the Y-axis.

2.2. Seismic vulnerability

The seismic vulnerability of the school pavilion was assessed by the standards of the Rapid Visual Screening manual FEMA P-154 (2015). The study area exhibits ground acceleration values ranging from 0.44g to 0.42g; therefore, the level of seismicity and format considered is classified as "moderate". Furthermore, a type C1 building was proposed, as it is the most similar to the analyzed structure. For the SL1 level of moderate seismicity, it yielded a final score of SL1= 0.7 < β=2. The structure shows a high probability of collapse during a seismic event; that is, it exhibits seismic vulnerability because it failed to meet or approach the β=2.0 cutoff parameter.

2.3. Structural pathologies and failures

To identify the pathologies and failures of the structure, the photographic recording tool has been used. This tool enables the visual documentation of the building and helps identify potential construction issues, such as cracking in columns and beams, as well as detachment of transverse steel. These problems, as shown in Fig. 1(a), Fig. 1(b), and Fig. 1(c), respectively, could be caused by adhesion failure and corrosion.



(a) (b) (c)

Fig. 1: Structure failures.

2.4. Structural element properties

Given that the study involves a building over 70 years old, there is insufficient information regarding the structural characteristics of the building's elements, such as the compressive strength of the concrete (f'c), the distribution of reinforcement steel, and the dimensions of the structural elements. Consequently, measurements of the structural elements were taken, and core sampling tests were performed by NTP 339.059 to determine a representative f'c for both columns and beams, and the results are presented in Table 1. Furthermore, reinforcement steel scanning was carried out on the key structural elements to standardize the distribution of the steel. Table 2 shows the input data considered for both the old and new elements of the structure.

Table 1: Compressive Strength of Diamond Witnesses.

Test N°	Identification	Area (cm2)	Ratio (Height/Diameter)	Strength Correction Factor	Maximum Load (kg)	Compressive Strength (kg/cm2)
1	Floor2/C3 Axis 3A	38.48	2	1	5812.3	151.0
2	Floor2/C4 Axis 4A	38.48	2	1	2997.9	77.9
3	Floor2/Beam Axis 4A	38.48	2	1	4242.0	110.2
4	Floor1/C8 Axis 8A	38.48	2	1	5057.7	131.4
5	Floor1/C11	38.48	1	0.87	3636.3	82.2
6	Floor1/Beam Axis 7A	38.48	2	1	3619.9	94.1

Table 2: Mechanical properties of materials.

Material	Mechanical properties	Old elements (kg/cm2)	New elements (kg/cm2)
Concrete	Compressive strength for beams (f'c)	100	250
	Compressive strength for columns (f'c)	110	250
	Elastic Modulus for Beams (Ec)	150000	237171
	Elastic Modulus for Columns (Ec)	157321	237171
Reinforcing Steel	Yield Strength (fy)	4200	4200
	Ultimate Strength (fu)	7000	7000
	Elastic Modulus (Es)	2000000	2000000
Masonry	Compressive Strength (f'm)	40	40
	Elastic Modulus (Em)	20000	20000

With all the collected information, a proper structural survey of the school pavilion can be conducted, enabling precise and reliable modeling and analysis of the structure.

2.5. Design of the jacket

Since the analysed structure has exceeded the maximum distortion limits established by the Peruvian E.030 standard, retrofitting was carried out on the elements along the Y-axis (frames) to increase their stiffness. This, in turn, aimed to reduce displacements along that axis and consequently minimise the drifts of the structure.

2.5.1 Columns

For the column jacketing design, the formula presented in the NTP E.060 under the chapter “Bending and Axial Load” was used and adapted. Various criteria were considered to analyse the jacket thickness, but the hypothesis selected assumed that the jacket bears 100% of the axial load without relying on the existing steel reinforcement as shown in Eq. (1) below. For the example, a 35x35 cm column was selected, resulting in a jacket thickness of 15 cm, which produced a new section of 65x65 cm, as shown in Fig. 3(a).

$$P_u = 0.80 \cdot \phi \cdot (0.85 \cdot f'_{cc} \cdot A_{jacketing}) \tag{1}$$

where P_u is the amplified axial force, ϕ is the strength reduction factor, f'_{cc} is the jacketing compressive strength and $A_{jacketing}$ is the jacketing area of the section.

2.5.2 Beams

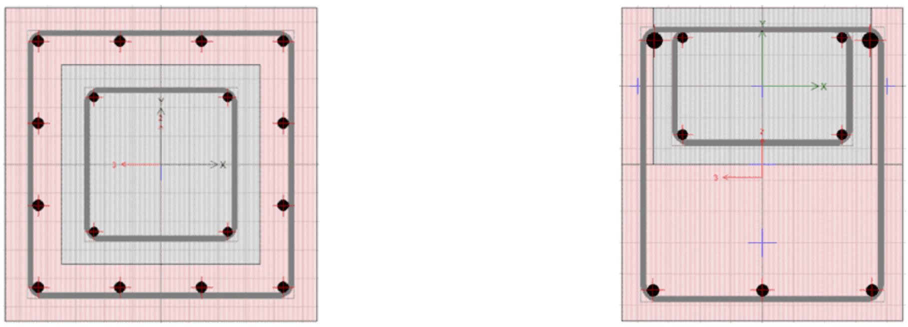
For the beam design, conventional formulas from reinforced concrete design textbooks were used and adapted. The modified formulas, as shown in Eqs. (2) - (4) below, considered the presence of the existing concrete but excluded the existing steel reinforcement, as corrosion of the longitudinal steel was observed in the beams of the analysed pavilion. For the example, a 35x25 cm beam was selected, with a jacket thickness of 5 cm on the sides and 25 cm on the beam’s depth, resulting in a new section of 45x50 cm. For the longitudinal reinforcement, 2 bars of 1” were proposed for the negative steel and 3 bars of 3/4" for the positive steel, as shown in Fig. 2(b).

$$a = d - \sqrt{d^2 - \frac{2 \cdot M_u}{\phi \cdot 0.85 \cdot (f'_c \cdot b + f'_{cc} \cdot 2e)}} \tag{2}$$

$$a = \frac{A_s \cdot f_y}{0.85 \cdot (f'_c \cdot b + f'_{cc} \cdot 2e)} \tag{3}$$

$$\rho_b = \frac{0.85 \cdot (f_c' \cdot b + f_{cc}' \cdot 2e) \cdot \beta_1}{f_y \cdot (b + 2e)} \cdot \frac{6000}{f_y + 6000} \tag{4}$$

where a is the distance from the extreme compression fiber to the centroid of the tensile longitudinal reinforcement, d is the depth of the equivalent rectangular stress block, M_u is the amplified moment at the section, f_c' is the concrete compressive strength, e is lateral jacketing thickness, b is the width of the compression face of the element, A_s is the area of tensile longitudinal reinforcement, f_y is the specified yield strength of the reinforcement, ρ_b is the balanced steel ratio, and β_1 is the factor relating the depth of the equivalent rectangular compression stress block to the neutral axis depth.



(a) (b)

Fig. 2: Jacketed elements.

3. Results and Discussion

Once the jacketing of all required elements was completed, the structure was evaluated using a modal dynamic analysis and a nonlinear static analysis to verify the drifts and structural performance of the analysed pavilion, respectively.

3.1 Linear dynamic analysis

The jacketed elements were modelled using ETABS software, and a comparison was made between the drifts, periods, and base shears of the original and retrofitted structure.

3.1.1 Story drifts

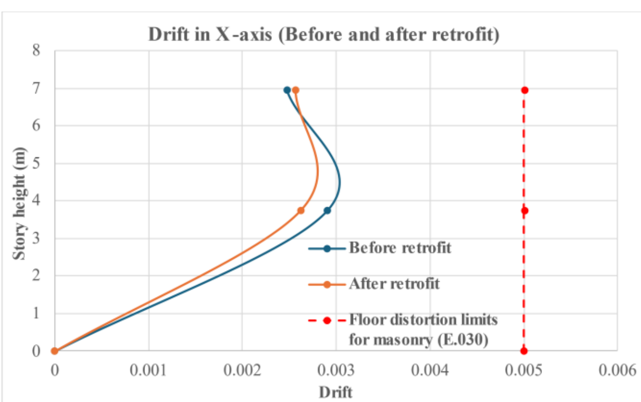


Fig. 3: X-drifts for the unreinforced and reinforced pavilion.

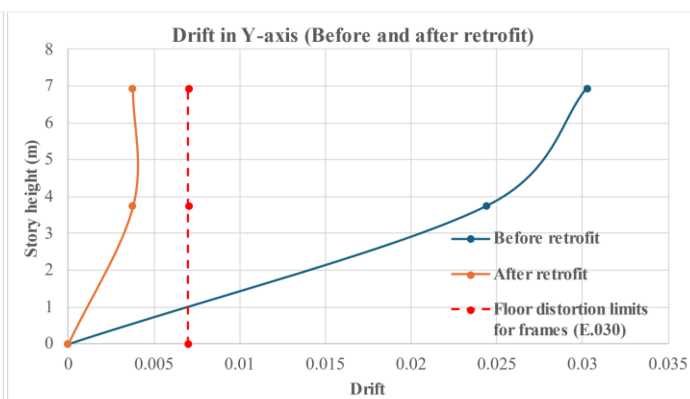


Fig. 4: Y-drifts for unreinforced and reinforced pavilion.

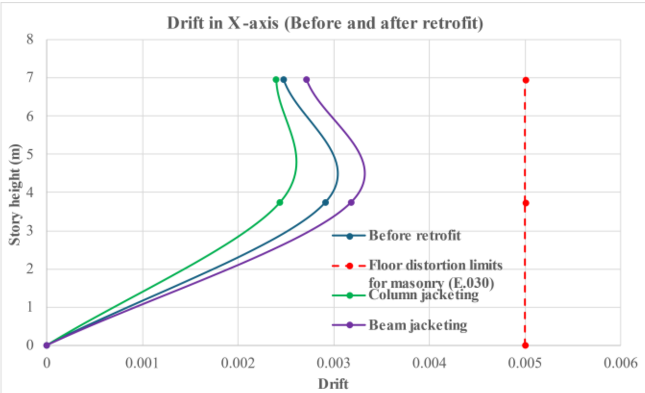


Fig. 5: X-direction drifts: columns vs. beams jacketing.

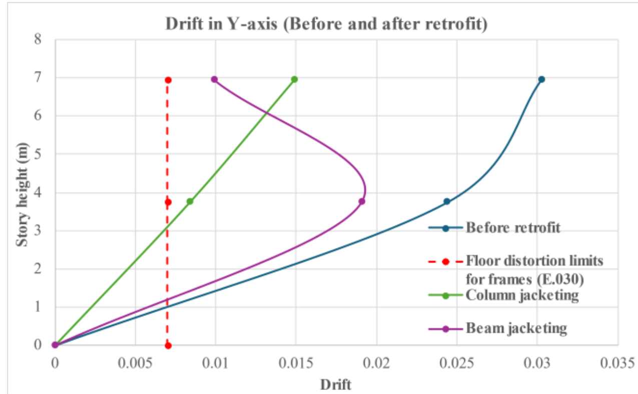


Fig. 6: Y-direction drifts: columns vs. beams jacketing.

Fig. 3 indicates that the storey drifts of the existing building along the masonry axis (X-axis) were within the maximum distortion limit imposed by the E.030 standard. However, along the frame axis (Y-axis), the distortion exceeded the designated limit of 7/1000 by a factor of four, as observed in Fig. 4. After jacketing the beams and columns, the drifts along the Y-axis were reduced by 80% for both floors. Fig. (5) – (6) demonstrates that when only the columns were jacketed, the drift reduction was 66% for the first floor and 51% for the second floor. Conversely, when only the beams were jacketed, the drift decreased by 22% for the first floor and 67% for the second floor.

3.1.2 Periods of the structure

From the linear dynamic analysis, Fig. 7 show a fundamental period of 0.76 seconds was obtained for the two-storey unreinforced structure, attributed to the low stiffness of the structural elements along the frame axis. After retrofitting the structure, the fundamental period decreased to 0.25 seconds due to the increased stiffness in both beams and columns along the analysed axis.

3.1.3 Base shear

Regarding the base shears, as shown in Fig.8, these increased after the retrofitting was implemented in the structure. This is because the increase in the cross-sectional area of each jacketed elements also resulted in a weight increase, which contributed to the variation in base shears.

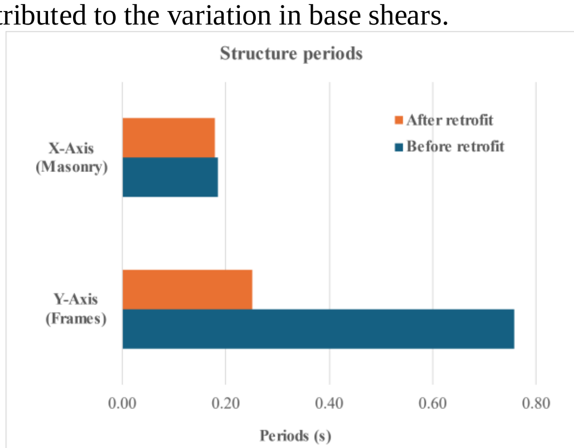


Fig. 7: Period comparison: X vs. Y axes.

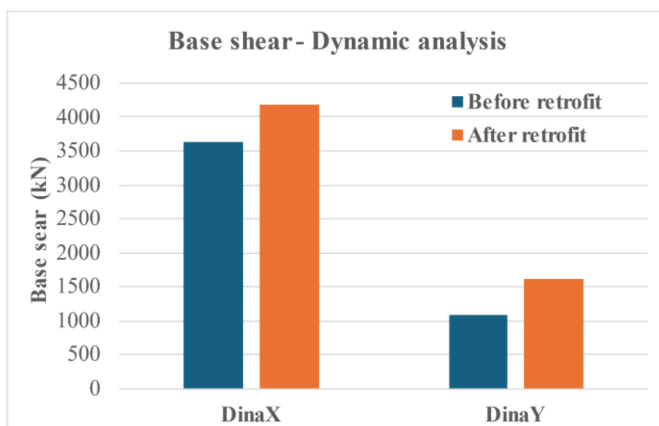


Fig. 8: Shear forces: unreinforced vs. reinforced pavilion.

3.2 Nonlinear static analysis

The capacity curve obtained from the analysed structure in ETABS software enabled, through the ASCE 41-13 standard, the determination of the structure's performance point for each type of earthquake and the identification of its location within the capacity curve-whether in operational, immediate occupancy, life safety, or collapse prevention zones, as shown in Fig. 9.

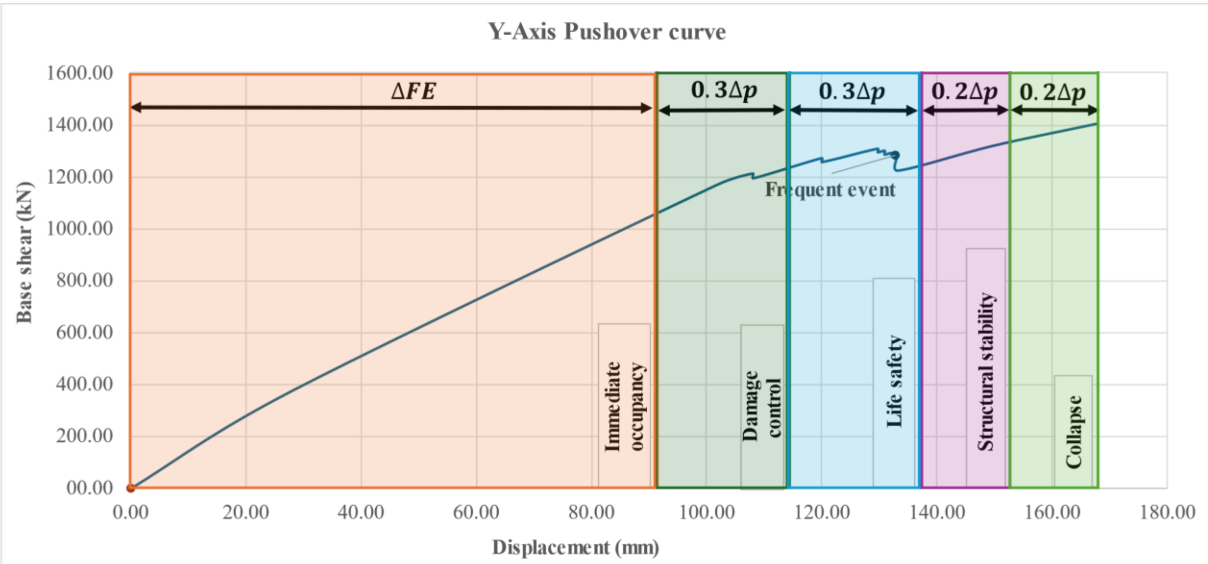


Fig. 9: Unreinforced pavilion performance.

The structural performance of the unreinforced structure indicates that for a frequent earthquake, the performance point falls within the life safety zone, while for the other earthquake types, the performance point is at the limit-in other words, within the collapse zone, as demonstrate in Fig.9. Therefore, it becomes imperative to reinforce the structure in the direction experiencing the most significant damage, which corresponds entirely to the frame axis (Y-axis).

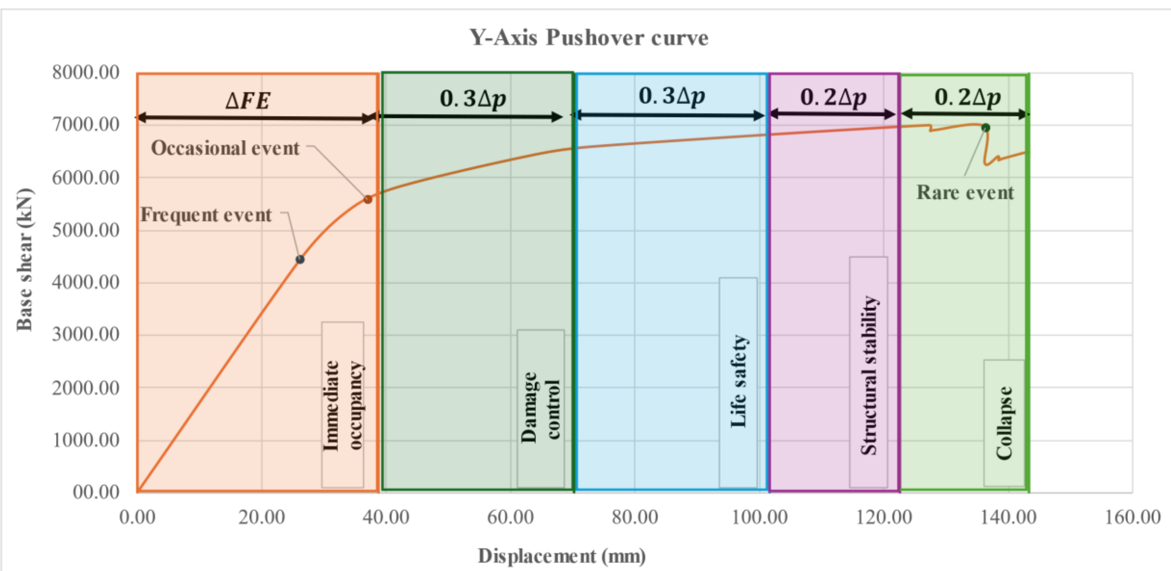


Fig. 10: Reinforced pavilion performance.

As shown in Fig. 10, for frequent and occasional earthquakes, the structure falls within the operational zone. For a rare earthquake, the structure is in the collapse zone. Lastly, for a very rare earthquake, no performance point was found, indicating that the structure cannot withstand such an event. These results suggest that while the structure demonstrates good performance against moderate to high-magnitude earthquakes, it will collapse during a very rare earthquake.

4. Conclusions

The linear dynamic analysis of the unreinforced structure revealed that the drifts along the Y-axis significantly exceeded the regulatory limits, highlighting insufficient structural stiffness. However, after jacketing the beams and columns, the drifts decreased by 80%, demonstrating the effectiveness of the retrofitting and a significant increase in structural stability, particularly along the Y-axis.

Moreover, the nonlinear analysis shows that in the absence of retrofitting, the structure exhibited critical plastic hinges and beam failures on the first floor, reaching the collapse point during a seismic event. However, with the retrofitting, structural performance improved considerably, as the presence of plastic hinges in the final step achieved the maximum capacity limit, with minor and controlled damage compared to the unreinforced pavilion.

Finally, although the jacketing increased load capacity and reduced deformations, it also caused an increase in structural weight, leading to a 50% leading to a 50% rise in dynamic shear along the Y-axis. Despite this, the retrofitting allowed the building to reach its maximum capacity without collapsing during frequent and occasional earthquakes, thereby ensuring a more efficient and safer seismic response.

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