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Assessing and enhancing the seismic performance of existing RCC buildings

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ABSTRACT

Due to collapse of buildings, the earthquakes in India and other countries have resulted in the death of people and damage to property. While it is impossible to prevent earthquakes, it is possible to lessen the damage they cause to existing structures by taking the necessary measures. It becomes crucial to perform an assessment first, then identify the weak members, and finally, carry out the necessary strengthening. Using pushover analysis, this study aims to evaluate the existing R.C. building in Zone-III. The analysis research shows the building's performance levels, component behaviour, failure mechanism, and hinge formation. Externally wrapped GFRP sheets were used to strengthen the existing member's deficiency.

Keywords: Zone-III buildings; Seismic Assessment; Pushover Analysis; Seismic Strengthening.

1. INTRODUCTION

Most of the Zone III buildings [1, 2] made of reinforced concrete were designed and constructed before 2002. Most the city's current structures were solely intended to support gravity loads. Many of the existing structures have brick walls but not considered in their proposal. Existing deficient structures should be improved to reduce the earthquake risks. It is necessary to assess the seismic resistance of existing structures and their deficiencies before a suitable repair or upgrade system can be created.

The majority of buildings experience considerable inelastic deformations under strong seismic activities, despite the fact that buildings are often planned for seismic resistance using elastic analysis. The actual behaviour of structures under these situations needs to be ascertained for use in contemporary performance-based design methodologies. As a result, non-linear analysis can be utilised effectively in both the design of new and existing structures [3]. The highest permitted damage condition (performance level) for a recognised seismic hazard is used to define seismic performance (earthquake ground motion). An objective for performance that takes into account damage states for different levels of ground motion is known as a dual or multiple level objective [4]. The goal of the pushover study performed by the authors in [5] was to assess the reinforced concrete building chosen for zone-IV in order to perform POA. The study found that columns and beams have evolved hinges that stand for the three phases of immediate occupancy, life safety, and collapse prevention. This article [6] uses SAP2000 to perform POA on Structures. Additionally, it assesses how these RCC buildings' seismic performance has improved as a result of different retrofitting techniques. It also talks about how these retrofitted beams' Beam-Column Joints behave. The original redundants in the details of these joints maintained, despite the noticeable improvement in performance. Two of these additions also imply that such situations would render repair useless [7, 8].

The study on the seismic evaluation of a five-story RC structure in Madinah City with various infill configurations was presented [9]. The four model systems they showed were as follows: bare frame, frame with infill from field test, frame infilled per ASCE 41, and frame infilled along with open ground story per ASCE 41. Based on the capacity and demand spectra for each model, the response modification factor (R) for the five-story RC building was calculated. The authors concluded that the existence of infill causes the R-factor to grow and satisfies the code's (SBC 301) requirement, but the bare frame does not satisfy the response modification factor

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requirement. Used pushover analysis to evaluate [10] the response reduction factor (R-factor) of 12 current irregular reinforced concrete buildings in the Kathmandu Valley. They concluded that the calculated R-factor values for the various bare frame constructions were lower than the levels recommended by the IS 1893 (2002) regulation. Worked on the reconstruction of damaged structures from the Sarpole earthquake and the seismic evaluation of such structures [11].

A three-story reinforced concrete structure in Iran was assessed for that study. The NDT was carried out after they had first gathered general information on the sample building. They came to the final conclusion, following a thorough investigation, that a building is more vulnerable when there is inadequate monitoring and when construction errors like stirrup spacing, coverings, and bending reinforcement are made. Consequently, in order to eliminate the disaster impact, it is imperative that the building be closely observed to ensure correct construction. Assessed [12] the resistance of masonry infills to both in-plane and out-of-plane seismic collapse in a hospital building. The base-isolation retrofitting technique was employed in that study to enhance the infill walls' IP (in-plane) and OOP (out-of-plane) responses. Developed [13] dissipative steel exoskeletons (DEX) for RC building seismic control. It seems convenient to consider the dissipative exoskeleton (DEX) from both an energetic and functional standpoint. Externally, three DEX configurations were used: a mixed (DEX.Mi) solution, parallel (DEX.Pa) and perpendicular (DEX.Pe) to all of the building's façades. The study's findings show that overturning moments brought on by seismic loads are responsible for the axial load in the columns, which is greater for DEX.Pa than for DEX.Pe and DEX.Mi. DEX.Pe is the most desirable option for the tensile axial load transformation to the foundation.

Examined the seismic susceptibility of two RC buildings that are still standing in Egypt [14]. The two case studies—the old and the new school buildings—were chosen for the purpose of the seismic evaluation. The study's findings suggested that an old school building would be more susceptible to earthquake damage. Most residential buildings were only intended to withstand gravity loads in areas with strong seismic activity, therefore pushover analysis is constantly required to assess the structures' seismic performance. By highlighting the weak points, the pushover analysis may encourage engineers to take the lead on rehabilitation projects [15, 16].

An essential component that needs to be evaluated is the reinforced concrete structure's failure pattern. Numerous studies have employed various approaches to investigate the failure pattern of RC frames. The most practical and recently established technique for determining the damage to reinforced concrete structures is called the "material strain limit approach." This method will assist in obtaining data about the real damage to the materials of reinforced concrete structures [17–19]. The significance of reliability analysis for reinforced concrete structures stems from the fact that performance-based earthquake engineering takes uncertainties into account in both modeling and design [20, 21].

2. MODELING AND ANALYSIS OF EXISTING RCC BUILDING – PUSH OVER ANALYSIS USING SAP 2000

A G + 2 reinforced concrete frame structure's response to earthquake forces at the zone III level was being studied. According to analysis's findings, the building's performance levels, component behaviour, and failure mechanism were all displayed. Moreover, it showed the order in which hinges form.

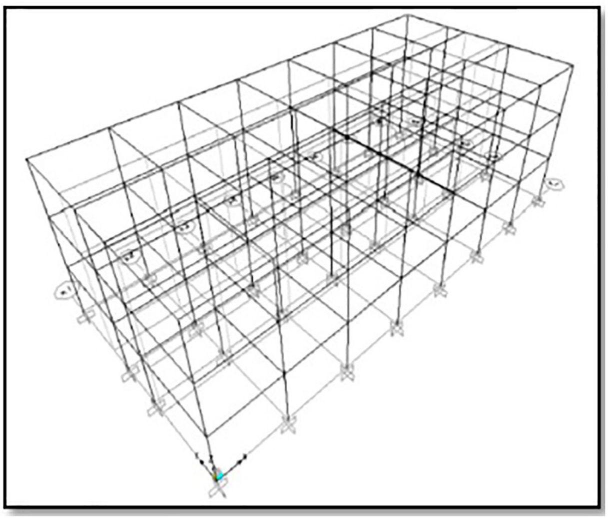

Figure 1: View of the existing structure in three dimensions.

The study led to the identification of the elements that require retrofitting. The existing building's three-dimensional view, which was utilized in the current analysis, is shown in Figure 1. The structure, which is real and representative of many others in this region, is in Coimbatore. It was constructed with Fe415 steel and M15 concrete. The existing building strength of concrete was assessed as 16.5 N/mm^2 using Rebound Hammer test and strength of steel was assessed as Fe420 N/mm^2 using tensile strength of steel [9–11]. The pushover analysis made use of these values. The 8.79-meter overall height of the G + 2 existing hostel structure that was chosen. The structure measures 25.41 metres long and 11.95 metres wide. The typical slab thickness is 140 mm, with a floor-to-floor height of 2.93 m.

3. PUSHOVER ANALYSIS RESULTS

3.1. Pushover curve

These curves Figure 2 and Figure 3, represent the overall stiffness and ductility behaviour of the frame. According to a pushover analysis, bare frames had a base shear of 1300 kN at a displacement of 180 mm in the abscissa [12–15]. Pushover study results showed a base shear of 2950 kN at a displacement of 30 mm in the ordinate. The base shear in the ordinate was greater than the base shear in the abscissa due to the placement of the columns in the frames.

3.2. Spectrum of capacity and building performance

Figure 4 depicts the existing building performance point, which was reached at a shear level of 1020 kN and an abscissa displacement of 15 mm. The building acquired immediate occupancy at this performance stage, as seen in Figure 5. The performance point was reached, as shown in Figure 6, at a shear level of 1,335 kN and a displacement of 4 mm in the ordinate [16, 17]. As illustrated in Figure 7, the building attained this performance

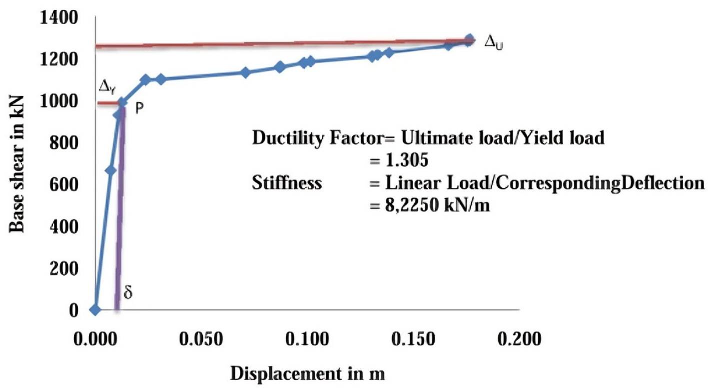

Figure 2: PO curve in abscissa.

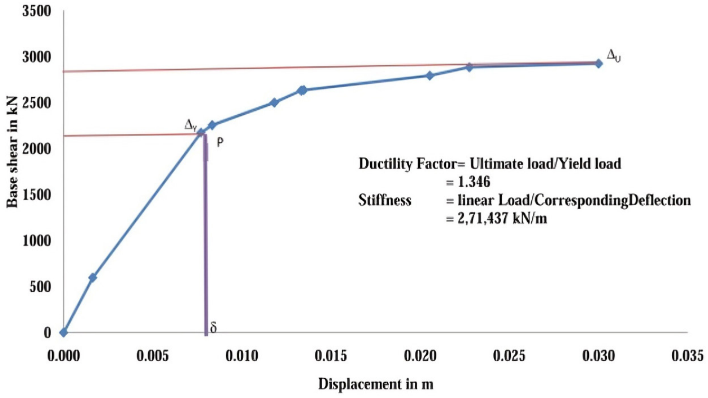

Figure 3: PO curve in ordinate.

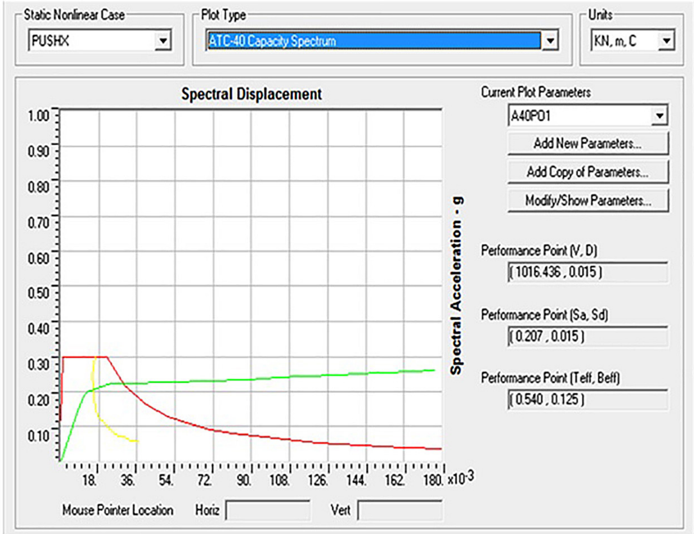

Figure 4: Spectrum of capacity in abscissa.

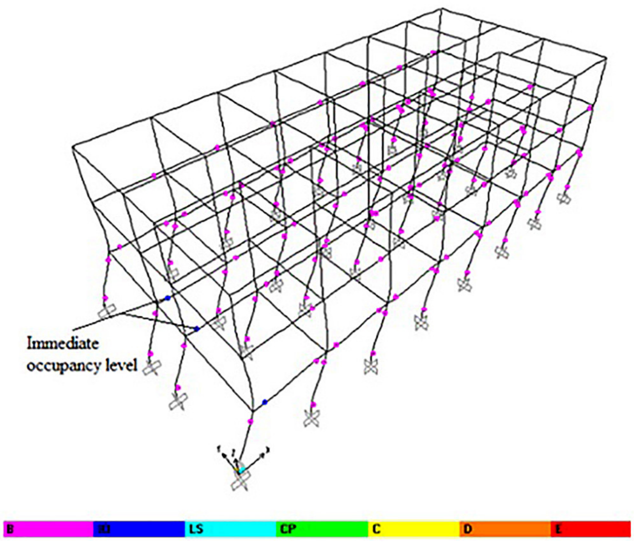

Figure 5: Formation of hinges in abscissa.

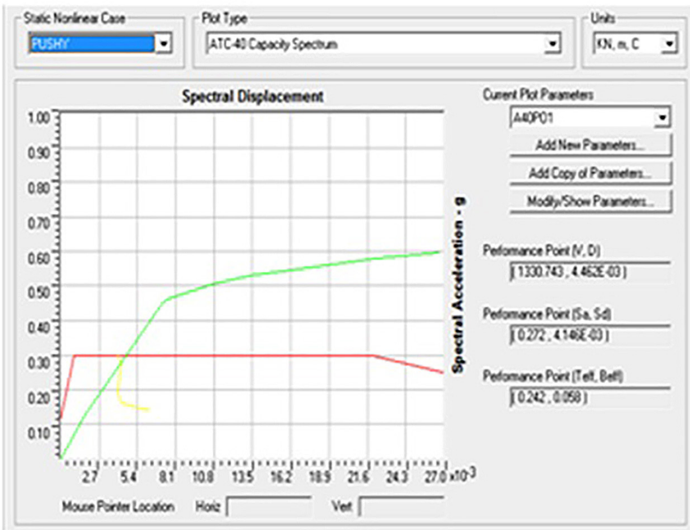

Figure 6: Spectrum of capacity in ordinate.

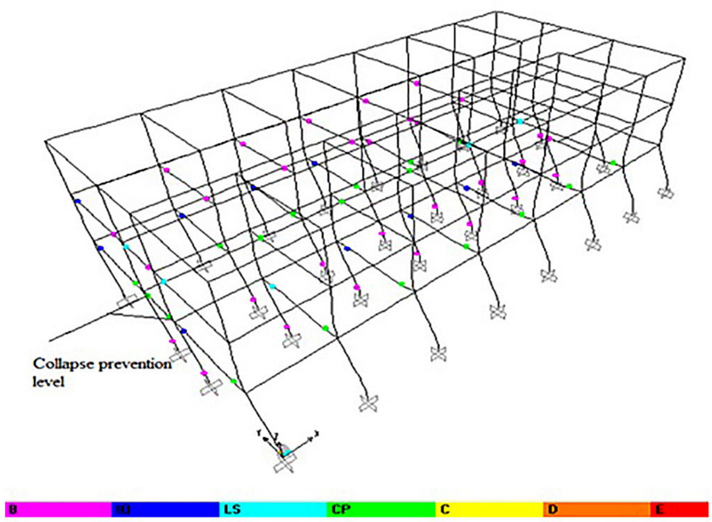

Figure 7: Formation of hinges in ordinate.

level of collapse prevention. As a result, it was found that the structure is more essential in the ordinate than the abscissa, as was predicted.

3.3. Formation of plastic hinges

When there is a force of 665 kN and a displacement of 7 mm, the first hinge develops in the abscissa. Moment and rotation indices for the yielding point were 1.5767 kNm and 0.0017 radians, respectively. Moment and rotation values were 1.6 kNm and 0.00255 radians for point instantaneous occupancy, respectively [18–20]. The moment and rotation values were 1.9344 kNm and 0.0356 radians, respectively, for preventing point collapse.

When there is a load of 665 kN and a displacement of 7 mm, the initial hinge development happens in the ordinate. For point yielding, the moment and rotation values were 1.6 kNm and 0.0017 radians, respectively [21]. For point instantaneous occupancy, the moment and rotation values were 1.65 kNm and 0.0026 radians, respectively. For preventing point collapse, the moment and rotation values were 1.9344 kNm and 0.0356 radians, respectively [22–24]. According to the assessing the existing building findings, weak beams caused existing buildings to collapse first in the ordinate. It has been determined that earthquakes in Zone-III lead to collapse in the ordinate first. As a result, [25] one of the strengthening techniques needs to be applied on weak beams in the ordinate before an earthquake.

4. SEISMIC STRENGTHENING OF EXISTING RCC BUILDINGS

In order to conduct experimental study, RCC beams with a section of 0.2 m × 0.25 m and a length of 2 m were chosen. The strengthening combinations for flexure beams that are listed below were combined with GFRP matting [26–28]. Externally, they were wrapped in two layers: one at the tension bottom face and the two vertical sides (GB1), and the other at those locations (GB2).

5. EXPERIMENTAL RESULTS

5.1. Compare load and deflection

Each beam's mid-span deflection was compared to that of the corresponding control beam, B1a. Also, load deflection behaviour of beams that had the same reinforcement wrapped around them was compared [29–31].

It was demonstrated that the flexure deficient beams behaved better than their matched control beams when bonded with GFRP sheets. We look at the graphs that illustrate how wrapping beams' mid-span deflection compares to the appropriate control beams [32–34]. The use of GFRP sheet affects how long fractures take to spread. Flexure failure is the way that control beam B1a was intended to fail [35]. Up to 42 kN was tested on the beam B1a, and the resulting deflection was 20 mm. Steel began to yield with a force of 37 kN, and deflection was 8 mm. Figure 8, displays the compare load and deflection curve that was created using this load and deflection data and Figure 9, failure Pattern of Beam B1a.

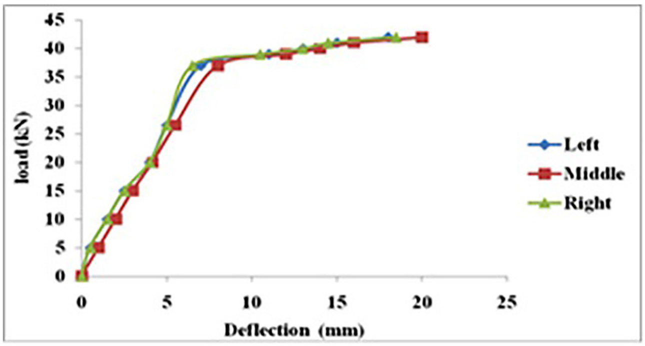

Figure 8: Curve of load versus deflection for beam B1a.

Figure 9: Pattern of beam B1a failure.

In order to strengthen beam GB1, a single layer of GFRP sheet was wrapped across the bottom face of tension and the two vertical sides [36]. The beam's maximum load carrying capability was 67 kN, and deflection was 29 mm. Figures 10 and 11 depict the failure pattern of Beam GB1 and indicate the maximum load to be 57 kN at a deflection of 9.5 mm [37].

Figure 12 shows that the linear load of GB1 was 54% more than that of B1a. The use of GFRP wrapping caused the beam GB1 to fracture earlier than the corresponding control beam B1a due to higher stresses [38–40]. According to research, adopting GFRP wrapping for flexure strengthening is more efficient. The maximum improvement in GB1's ultimate strength over B1a was 60%.

A double layer of GFRP sheet was wrapped around the bottom face of tension and the two vertical sides of beam GB2 to enhance it. The highest load the beam could sustain was 92 kN, which caused a 38 mm deflection. According to Figures 12 and 13, the yield load was 78 kN, and there was an 11 mm deflection as shown pattern of Beam GB2 Failure. According to Figure 12, GB2 had a linear load that was 111% higher than B1a's and 36% higher than GB1s. Initial cracks developed in the beam GB2 at higher stresses than in the GFRP-wrapped beams B1a and GB1 [41]. When compared to B1a and GB1, the greatest percentage increases in ultimate strength were 119% and 37%, respectively. According to Figures 10 and 12, every reinforced beam that had mid-span deflections had values higher than those of the control beam at respective failure loads [42]. The ductility of the beams grew together with the amount of reinforcing materials, raising the mid-span deflection values. As the stiffness of the beam increases, the deflection at yield level rises. This is because adding more strengthening material causes the steel to yield more slowly than it would otherwise [43]. For beams GB1 and GB2, which were reinforced with GFRP laminates, the deflections at yield level were 9.5 mm and 11 mm, respectively. When compared to control beam B1a's 8 mm deflection, these results showed a higher deflection [44, 45]. At ultimate level, deflections of 30 mm and 39 mm, respectively, for GB1 and GB2, were also noted. These numbers showed greater deflection as compared to the control beam, which was a 20 mm B1a beam. The beam's strength will therefore rise as the amount of laminates provided to it increases. The development of the diagonal cracks is prevented by the presence of the GFRP laminate beam. This initial crack was caused by a sizable variation in the load. For beams that had been retrofitted with GFRP wrapped beams, the load versus deflection behaviour was better.

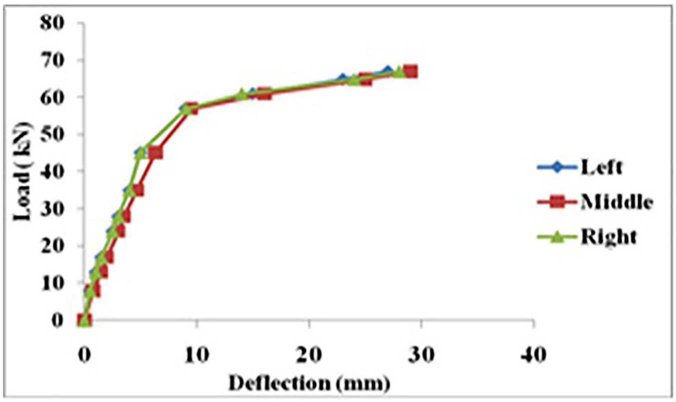

Figure 10: Curve of load versus deflection for beam GB1.

Figure 11: Pattern of beam GB1 failure.

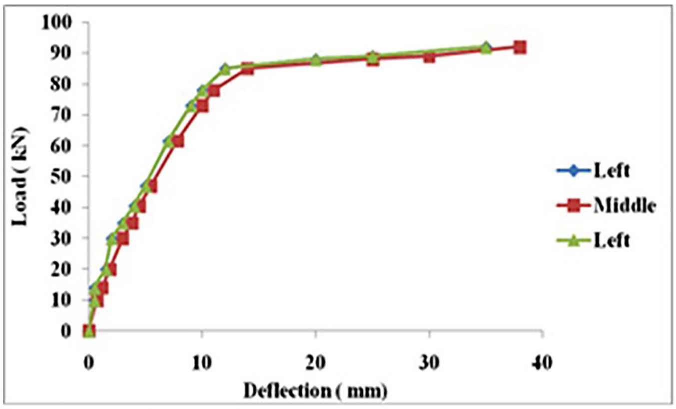

Figure 12: Curve of load versus deflection for beam GB2.

Figure 13: Pattern of beam GB2 failure.

6. THE COMPOSITE BEAMS DUCTILITY

For the strengthened beams GB1 and GB2, the percentage of displacement ductility index was 23 percentage and 39 percentage, respectively. These beams displayed higher displacement ductility index values as compared to control beam B1a. The displacement ductility index for GB2 was 13% higher than GB1's. The energy ductility index values for the strengthened beams GB1 and GB2 were 3.5 percentage and 4.5 percentage, respectively. When compared to control beam B1a, these beams showed greater values. GB2 has an energy ductility index that was 27% higher than GB1's. The improved ductility in strengthened beams displayed higher values in both cases when compared to control beams. As a result of the GFRP strengthening, all strengthened beams demonstrated sufficient ductility. They are resilient to the effects of Zone-III earthquakes. Hence, it may be said that GFRP wrapping enhances a building's seismic protection.

7. CONCLUSIONS

Pushover analysis is a static nonlinear analysis that reveals the structure's nonlinear behaviour, which is its actual behaviour. Since that the shear capacity of the beam was lower in the ordinate than the abscissa at the initial hinge development level, it is determined that the existing building frame is seismically unsafe. As a result, one of the strengthening techniques needs to be applied on weak beams in the ordinate before an earthquake. All the wrapped beams exhibited the higher value of load and deflection. The strength of the beam grows when more laminates are added to it, which is how it gains strength. In both cases, the strengthened beams' increased ductility displayed higher values as compared to the control beams. Due to GFRP strengthening, all reinforced beams displayed appropriate ductility. They are especially crucial for using composite elements to enhance existing concrete structures so they can withstand seismic effects. In order to modify buildings to resist the forces of Zone-III earthquakes, this was very valuable.

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