ASPS Conference Proceedings 1: 611-617 (2022)

Proceedings of
12th Structural Engineering Convention - An International Event (SEC 2022)

Available at https://asps-journals.com/index.php/acp

Seismic Performance Assessment of a Tall RC Building Retrofitted with Shear Walls

A.H. Tramboo [1, *] and T. Choudhury[2]

[1] ME Scholar, Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala, 147004, India
[2] Assistant Professor, Department of Civil Engineering, Thapar Institute of Engineering and Technology, Patiala, 147004, India

Paper ID - 060050

Abstract

Reinforced concrete (RC) buildings are the most commonly constructed structural forms across the world, and equally common is its susceptibility to damage during any seismic activity. Thus, there is especially a need to correctly assess and study the behaviour of RC structures, particularly the tall RC buildings. Tall RC buildings are increasingly becoming popular nowadays because they provide abundant floor area, where the land available for construction is less due to the rapid urbanisation. The understanding of the seismic performance of such buildings makes the seismic retrofitting procedures convenient and optimal to use. Seismic retrofitting measures are present in plenty for RC structures, each of which is used according to the need or the damage level incurred in the structure or its components. The present study thus focusses on a G+14 tall reinforced concrete (RC) building that was severely damaged during the 2015 Nepal Earthquake and was being retrofitted located at the Hattiban area in Nepal. As recommended in the recently developed guidelines for tall RC structures in IS 16700, the use of a 3D computer model is necessary for the seismic analysis and structural safety assessment of tall buildings. Thus, seismic performance is assessed using the structural analysis program SAP2000 for non-linear analysis of the considered structure. Further, seismic fragilities at specific damage levels are also determined.

Keywords: *Tall Structures, Reinforced Concrete, Shear Wall, Nonlinear analysis, Seismic Performance, Fragility.*

1. Introduction

The functionality and performance of a structure must be maintained both during and after the occurrence of an earthquake. In this regard, seismic retrofitting techniques are often considered as a solution for the restoration of the structural capacity, particularly in developing countries, where a lot of constructions are marred with shortcomings in terms of design. The shortcomings are either due to the lack of proper construction practice, or the disregard of standard seismic design code provisions.

The present study considers one such seismically deficient building located in Nepal that was affected during the 2015 Gorkha earthquake. The PGA of the earthquake varied in the range of 0.8g to 1.0g [1]. After a thorough screening, the building was found to be damaged locally at several structural members that needed retrofitting. The building has been retrofitted by implementing different techniques based on visual inspections. The motivation of the present study, is thus, to verify the seismic performance as well as the fragility of the original building to locate the damage at different members based on rigorous non-linear analysis as suggested for tall buildings in IS 16700 [2]. This will lead to a specific application of retrofitting technique, and estimate the efficiency of the specific strategy applied.

A study of the performance of buildings retrofitted with shear walls using nonlinear analysis is done. Seismic assessment based on standard Capacity Spectrum Method as suggested in ASCE 41 [3] is carried out for several hazard levels. This further gives a generalized idea on the specific capacity for the original and retrofitted buildings to perform under a given hazard level. Probabilistic evaluation of the buildings is done based on the exceedance probability of a specified damage level for different engineering demand parameters. The use of fragility curves and discrete probabilities, thus aid in arriving at the efficiency of the implemented seismic retrofit strategy when compared to the original capacity of the building.

Most retrofitting techniques will increase stiffness and slightly increase in mass which causes in return, a shorter period of vibration. Shortening in the period of vibration often increases the strength and ductility of the retrofitted structure [4]. Thus, a proposed retrofit scheme can be said to be successful, if it results in an increase in strength and ductility capacity of the structure, sufficiently higher than the demands imposed on it by earthquake

Seismic strengthening techniques have been in use since long, and with more and more research, structural engineers are finding out new and improved techniques. The basic requirements to fulfil during the implementation of the technique is to check its strength, stiffness, ductility, and energy dissipation capacity among others, once the retrofit technique is applied.

*Corresponding author. Tel: +917889372283; E-mail address: :ahtramboo@gmail.com

Proceedings of the 12th Structural Engineering Convention (SEC 2022), NCDMM, MNIT Jaipur, India | 19-22 December, 2022

doi:10.38208/acp.v1.556

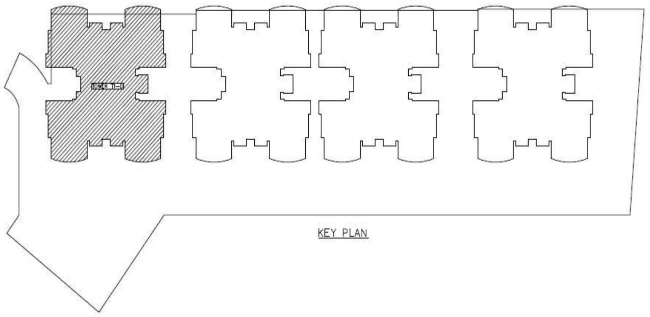


Fig. 1. Key Plan of the Building Assembly (Source: Vintech Consultants)

2. Details of the Considered Building

The RC building considered for the present study is one of the residential building towers of "City Space Apartment Homes" located in Hattiban, Nepal. The key plan area of the buildings is shown in Figure 1. All the buildings in the area are the same, and Tower 1-B is selected for the study. The case study building is a 15-story RC frame building with an overall height of 43.42 m. The first story is 2.12 m high from the ground level which comprises of the basement of the building. The next story is of 2.95 m height from the stilt level. Rest all the stories comprise the same story heights. The building is irregular in the plan. The basement plan is rectangular with dimensions of 37.635 m long by 26.05 m wide. All the upper stories have floor area that spans over a length of 26 m and width 21.13 m.

2.1 Material and Geometric Details

Normal weight concrete having a specified compressive strength of 25 MPa was used for the design purposes in beams and slabs and for columns a concrete compressive strength of 30 MPa was used. For reinforcing, HYSD bars of grade Fe 415 was used as main bars and Fe 250 grade was used as confining reinforcement. The beams at the different story levels of the building have different section sizes. Mostly, for the primary beams, a section size of 450 × 750 is used, other sizes of beams are 450 × 600, 300 × 600, 300 × 450, 230 × 450 and 150 × 450. The section details of the beams at the first-floor level are shown in Table 1. A typical design and detail of a beam are also shown in Figure 2. The column section sizes also vary, and are mostly rectangular, except one circular column with a diameter of 600 mm. Other rectangular column sizes are of 600 × 300, 750 × 300, 1185 × 300, etc. Table 2 summarizes the reinforcement in the columns at specific floor levels.

The slabs in the building are of 125 mm thickness, while many of the slabs in the basement are of 150 mm thickness. Corridors and lobbies at different floor levels have a thickness of 140 mm. Staircase slab has a thickness of 125 mm. All these slabs were designed for gravity loads using design methodology adopted from IS 456 [5]. In the design of beams and columns, the minimum criteria for transverse reinforcement laid down by codal provisions in IS: 456 [5] and IS: 13920 [6] were followed.

Table-1. Reinforcement details in beams at the 1st-floor level

Floor Level	Section		Reinforcement Details		
	Width (mm)	Depth (mm)	No. Of bars	Bar Size	Stirrups
1st	450	750	Top 5 Bottom 5	25	10mm 4 legged @125c/c
1st	300	450	Top 3 Bottom 3	20	8 mm 2 legged @ 100 mm c/c
1st	150	450	Top 2 Bottom 2	16	8 mm 2 legged @ 100 mm c/c
1st	230	450	Top 3 Bottom 3	25	8 mm 2 legged @ 100mm c/c

Table-2. Reinforcement details in columns at various locations

Columns	Story	Dimension (mm)	No. of bars	Bar size	Tie bar size
Exterior	Foundation to stilt	750 × 300	18	25 & 20	8 mm @ 100
Interior	Foundation to stilt	1185 × 300	20	25 & 20	8 mm @ 100
Interior	Foundation to stilt	750 × 450	20	25	8 mm @ 100

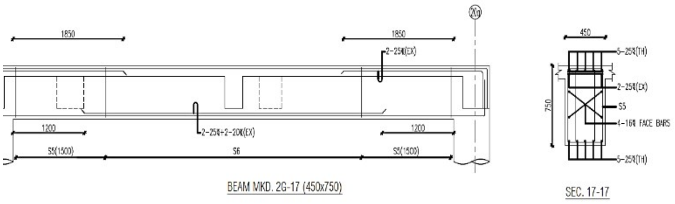


Fig. 2. Typical beam reinforcement details in the considered buildings

2.2 Damage Reconnaissance

The considered building was subjected to the 2015 Gorkha earthquake and was found to be heavily damaged at certain locations due to the seismic activity. Although the building performed well during the earthquake with no loss of life and property, several visible cracks on the main structural members led to a need for the strengthening of the structure. At the site, several observations on damage were recorded based on visual inspection. The masonry walls were cracked at the lower stories up to about 7th storey of the tower, both at outer and inner locations. Cracks were observed at many locations in beams, but the coupling beams in the lift lobby area were subject to major cracks. Some minor cracks were seen in columns, but they were mostly spalling of the cover. Hairline cracks were observed in slabs at beam-slab junctions. Cantilever slabs performed well and no major cracks were observed. Foundation settlement is not observed at any location.

3. Nonlinear Numerical Model of Original and Retrofitted Building

The fundamental equation of motion used to determine the dynamic response for the structural models is shown in Eq. (1). The [K] matrix is updated stepwise to get the nonlinear response.

$$[M]\{\ddot{u}\} + [C]\{\dot{u}\} + [K]\{u\} = -[M]\,\ddot{u}_g \qquad (1)$$

Where,
$[M]$ = Mass matrix
$\{\ddot{u}\}$ = Acceleration vector
$[C]$ = Viscous damping matrix
$\{\dot{u}\}$ = Velocity vector
$[K]$ = Structural stiffness matrix
$\{u\}$ = Displacement vector
$\ddot{u}_g$ = Ground acceleration

The RC building is assumed to be fixed at the base and soil-structure interaction effects are not considered in the study. Columns and beams of the building are modelled using two-noded frame elements with three degrees of freedom at each node. According to available building data, different grades of concrete and reinforcing bars are used in various structural components as shown in Table 3. The modulus of elasticity (E_c) of the RC members of the building is calculated using IS 456 [5] as shown in Eq. (2).

$$E_c = 5000\sqrt{f_{ck}} \quad \text{(MPa)} \qquad (2)$$

where f_{ck} is the characteristic cube strength of concrete in MPa.

Table 3 – Material data for various structural components

Structural Element	*Concrete Grade*	*Rebar Grade*	
RC Beams	M-25	Main Bars Confinement	Fe 415, 20mm Fe 415, 12 mm
RC Column	M-30	Main Bars Confinement	Fe 415, 25mm Fe 415, 12 mm
RC Slabs	M-25	All bars of Fe-415	
RC Walls	M-30	All bars of Fe-415	

For frame elements (beams and columns) modelled as auto sections, lumped nonlinearity is defined using auto hinges for which the limiting values are based on the table 10-7 and table 10-8 of ASCE 41 [3] for concrete beams and columns, respectively. The plastic hinge length (lp) is decided based on concept prescribed by Pauley and Priestley [7] and the hinges are assumed to form at a distance equal to half the average plastic hinge length (lp) from their rigid ends. It can be calculated by the Eq. (3) [7], where, L is the length of the member in m (taken at the point of contra flexure from the end), d_b is the diameter of longitudinal reinforcement in m, and f_y is yield stress of longitudinal reinforcement in MPa.

$$lp = 0.08L + 0.022 d_b f_y \text{ (in m)} \qquad (3)$$

For frame elements modelled using section designer in SAP2000 [8], fibre hinges are assigned. The length and location of the fibre hinges are the same as in the case of an auto hinge. The elastic behaviour of the frame element is determined by the frame section (and hence material properties) assigned to the element. An idealized moment rotation (M-θ) curve (Figure 3) showing the controlling points in the curve according to FEMA 356 [9]. The five points A, B, C, D, and E are used to define the backbone for hinge rotation behaviour of RC members. The portions of the hinge moment-curvature curve from A to B represents the linear behaviour of the member. After hinge yields at point B, plastic deformation is determined by the curve in the region B-C-D-E with all plastic deformation measured relative to B. The elastic slope of a hinge is given by the elastic stiffness of the element over the assumed length of the hinge. The strain-hardening slope is the slope of line BC. The points – IO (Immediate Occupancy), LS (Life Safety), and CP (Collapse Prevention) are used to define the acceptance criteria of the hinge.

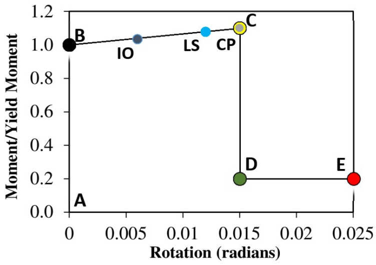


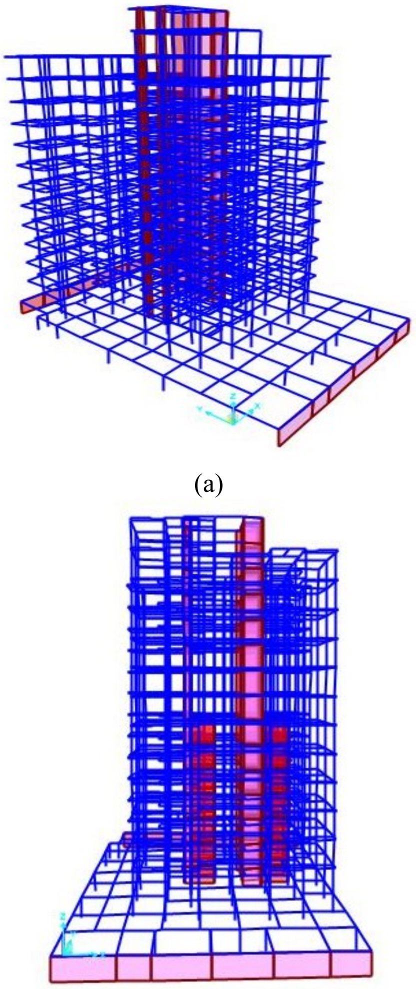
Fig. 3. Idealized moment-rotation curve for flexural members [9]

2.1 Shear Wall Modelling

Shear wall is modelled using area elements similar to a slab section, however, they differ in terms of their load-deformation behaviour. A slab is a structural element that helps in gravity load transfer only from slab to beams. But the shear wall is not only a gravity load resisting element, but it also forms a lateral load resisting component of the building and plays a vital role in developing its seismic resistance. Hence, when the building is, in general, to be studied globally under its nonlinear behaviour, such components need to behave non-linearly i.e. beyond the elastic phase of its behaviour. Thus, nonlinear layered shell elements are used in SAP2000 for modelling the shear walls. The multi-layer approach for shell element has proven to be a suitable model that can capture the coupled in-plane or out-of-plane bending, as well as the in-plane direct shear and coupled bending or shear behaviour of RC shear walls. The building models, both original and retrofitted are shown in Figures 4 and 5.

2.2 Modal Analyses

The modal analysis determines the undamped free-vibration mode shapes and frequencies of the system based on Eigenvectors. These natural modes provide an excellent insight into the behaviour of the structure. Ritz vector analysis seeks to find modes that are excited by a particular loading. Ritz vectors can provide a better basis than eigenvectors when used for response-spectrum or time-history analyses that are based on modal superposition [8]. In the present study, Eigenvector has been considered based on the scope of the study, and the mass participation ratios are observed. Table 4 shows the modal analysis results in both the orthogonal directions – X and Y of the building.



(a)

(b)

Fig. 4 – 3D model of the (a) original building, and (b) Retrofitted Building using shear walls in SAP2000 [8]

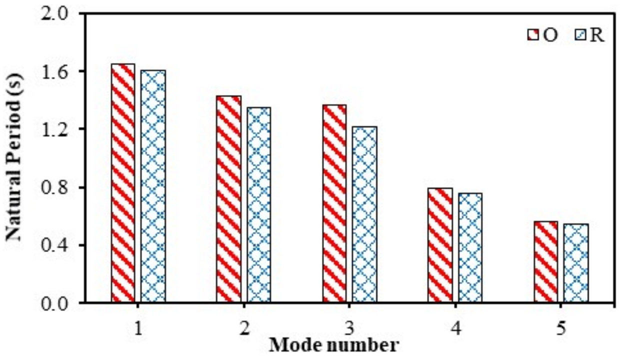


Fig. 5 – Comparison of Natural period (in seconds) for the original (O) and retrofitted (R) buildings.

Table 4 – Natural time periods and modal participation of the original (O) and retrofitted (R) buildings

Mode	Natural Period (sec)		Mass Participation Ratios (%) along			
	O	R	O		R	
			X	Y	X	Y
1	1.65	1.61	00.45	16.23	00.96	13.65
2	1.43	1.35	66.00	01.28	03.30	52.48
3	1.37	1.22	00.81	50.91	57.17	01.59
4	0.79	0.76	00.04	00.04	00.88	00.01
5	0.56	0.55	00.13	02.16	00.20	02.19

The modal participation ratios indicate that once the building is subjected to the vibration of a particular time period, these mass percentages get involved in the reaction. The values indicate the predominance of masses in a particular direction. This information is necessary in further defining the push over cases in a particular direction. The modal participating ratio defines the predominance of a mode shape in a particular direction. A comparative plot of the natural period of the buildings is shown in Figure 5. A marginal decrease is observed in the natural period with the introduction of shear walls.

4. Nonlinear Seismic Performance Assessment

Nonlinear static (pushover) analyses of the considered building are carried out and maximum displacement at the roof level is monitored. The pushover (PO) curves obtained in both the two orthogonal directions (X and Y) upon analysis are shown in Figures 6 and 7. These push over curves along the two directions indicate the building behaviour as it is subjected to pushover analysis. The variations in the slope of the curve indicate the changes that are taking place along the building, the drop on graphs implies the strength decrease, which in turn, indicates the failure of certain members, like the beams or columns in the building. For locating the damaged members in the building, individual nonlinear hinges in RC members are checked. The introduction of the shear walls as retrofitting technique improved the capacity of the building more in the X direction as compared to Y direction as observed from the pushover curves.

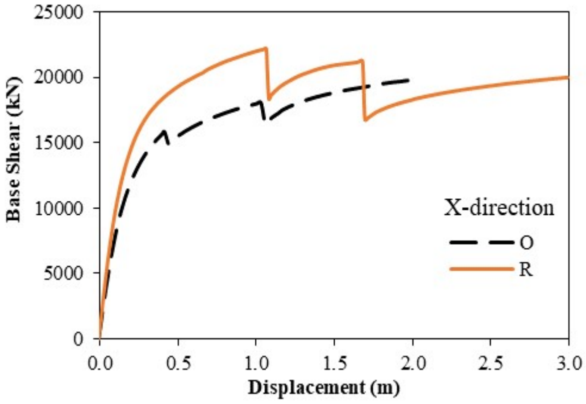


Fig 6 – Comparison of pushover curves for the original (O) and retrofitted (R) building in X-direction

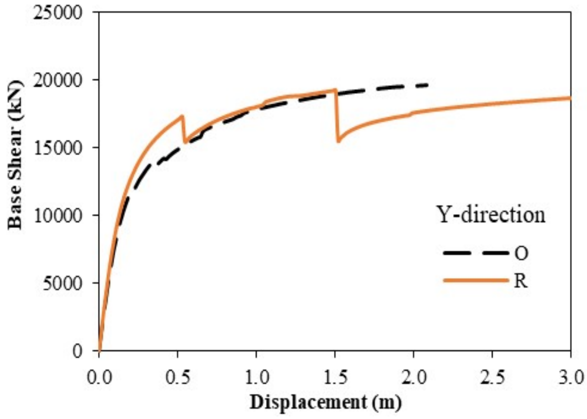


Fig 7 – Comparison of pushover curves for the original (O) and retrofitted (R) building in Y-direction

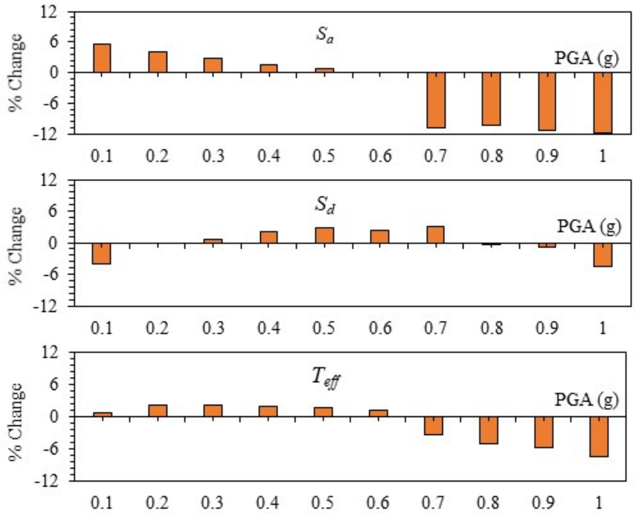


Fig. 8 – Change in performance points (PP) – Sa, Sd, Teff for the retrofitted building with respect to the original.

2.3 Performance Point Evaluation using Capacity Spectrum Method

For a systematic study of the seismic performance of the buildings, capacity spectrum method (CSM) is adopted. The capacity spectrum method gives the performance points (PP) of a structure in terms of various engineering demand parameters (EDPs) – base shear (V), roof displacement (D), spectral displacement (S_d), spectral acceleration (S_a) [11]. To compare this capacity of the buildings with the demand imposed by an earthquake, seismic hazard in terms of response spectra is used. The response spectrum as specified in the Indian seismic code IS 1893 [10] are considered for assessing the seismic performance of the considered buildings. The response spectrum is effectively reduced for hysteretic effects through standard procedures mentioned in ATC 40 [11] and ASCE 41 [3]. The EDPs obtained for different levels of peak ground acceleration (PGA) represented by response spectra are shown in Figure 8. A value of 0% indicates no effect of the retrofit strategy applied. It is observed that up to a PGA of about 0.6g, the change in the demand imposed on the building in terms of S_a and S_d is marginal. A reduction of about 10% in the S_a demand is observed at PGA greater than 0.6g. Thus, the spectral acceleration demand can be effectively reduced in the building at higher levels of PGA (i.e., greater than 0.6g). In reducing the spectral displacement (S_d) demand, the retrofit strategy becomes active even at higher PGA (greater than 0.8g). Also, a significant reduction in the effective time period is observed after about 0.6g. Thus, clearly from Figure 8, the effectiveness of the retrofitting scheme applied depends on the PGA of the earthquake. In simple words, the use of shear walls is much effective when the building is subjected to an earthquake of higher PGA because it's around that force where the original building is in need of extra resistance, which in here is made possible through the use of shear walls.

5. Seismic Fragility Analysis

Seismic fragility analysis is the key component for seismic vulnerability and risk assessment methodologies [12]. The fragility function can be defined as a mathematical function that expresses the probability that some undesirable event occurs as a function of some measure of environmental excitation. Mostly, certain damage states (ds) are specified for which the probability of exceedance is determined given a hazard, represented as P[ds ≥ ds|EDP]. Discrete probabilities for a given damage state can also be estimated that indicates relative probability occurrences of the damage states [13]. The present study considers specific levels of damage states based on the amount of post-yield stiffness degradation. Accordingly, five different damage states (LS-1 to LS-5) has been considered. For each damage state, fragility curves are shown in Figure 9 (a) and Figure 9 (b) for original and retrofitted building respectively

A comparison of the discrete fragilities for a given level of uncertainty for two different PGA values – 0.4g and 0.8g is shown in Figure 10. The effect of change in the hazard level can be observed on the discrete probabilities of damage. At lower PGA, the discrete probability is high for lower damage grades (dg), and this probability gradually increases with an increase in damage grade. For a higher PGA, the discrete probability of lower damage grade is very high (about 50%). However, this probability decreases as we move on to higher damage grades. Thus, the building will incur minor or lower damages even at higher PGA values.

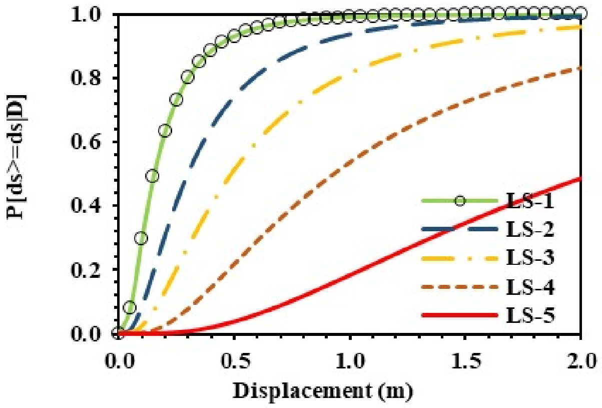


Fig. 9 (a) –Fragility curves for the original building

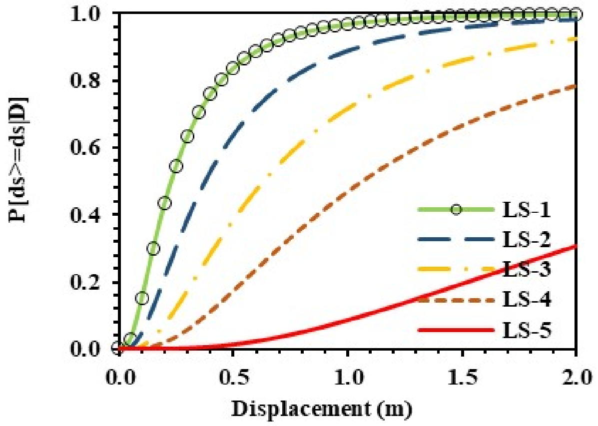


Fig. 9 (b) –Fragility curves for the retrofitted building

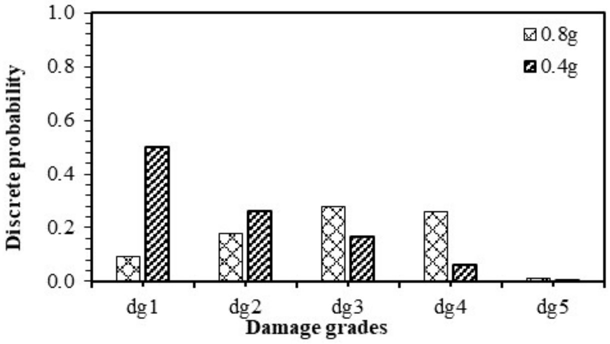


Fig. 10 – Comparison of discrete fragilities at a PGA of 0.4g and 0.8g for the retrofitted building

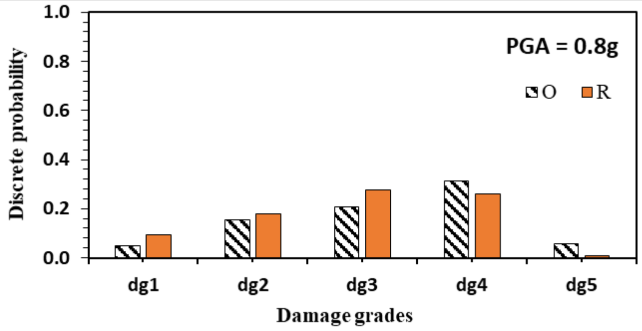


Fig. 11 – Comparison of discrete fragilities at a PGA of 0.8g for the original and retrofitted buildings

A comparison of the discrete damage probabilities of the original and retrofitted building is also shown in Figure 11. A PGA of 0.8g is chosen for the comparison as the building was recorded to experience a PGA of 0.8g to 1g during the Gorkha earthquake [1]. The original building has nearly 40% chance of higher damage grades at the considered PGA level. The building with the shear walls has less probability of damage at higher damage grades. Thus, it can be seen that the original building suffered damage when the earthquake with PGA ranging in between 0.8g to 1.0g hit the building. Moreover, the adopted retrofit strategy is effective for the building at higher PGA values.

6. Conclusion

The seismic capacity of the considered high-rise RC building is assessed through rigorous nonlinear analysis, both before and after applying the shear walls as a seismic retrofitting strategy. The effectiveness of the specific retrofitting technique, assessed through the capacity spectrum method is found to be highly dependent on the specific earthquake measure. The seismic strengthening of the building by the addition of shear walls was found to be efficient as compared to the performance of the original building in terms of both spectral acceleration and displacement as the engineering demand parameter.

From the seismic fragility analysis, it can be stated that the probability of reaching or exceeding a particular damage state is dependent on the hazard level. Overall, the probability increases as the level of hazard increases. However, speaking of discrete probability, high probability of lower damage grades is expected at lower hazard level. Whereas, high probability of higher damage grades is expected at greater hazard levels. The shear walls as a retrofit strategy are found to become active and participate in the nonlinear behaviour of building at a higher hazard level for the considered RC building.

7. Acknowledgement

The authors acknowledge the Vintech Consultants, New Delhi, India for providing the data related to the case study building.

Disclosures

Free Access to this article is sponsored by SARL ALPHA CRISTO INDUSTRIAL.

8. References

[1] Earthquake of Gorkha (Nepal), (2020, April 29), Retrieved from https://www.edac.biz/en/news/earthquake-reports-mostly-german/earthquake-of-gorkha-nepal

[2] Indian Standard, IS 16700 [2017], "Criteria for Structural Safety of Tall Concrete Buildings". Bureau of Indian Standards, New Delhi, India.

[3] ASCE-41 [2017], "Seismic Evaluation and Retrofit of Existing Buildings". Reston, VA: American Society of Civil Engineers.

[4] Oliveto, G., Mmarletta, M. (2005)." Seismic Retrofitting of Reinforced Concrete Buildings using Traditional and Innovative Techniques." ISET Journal of Earthquake Technology, Paper no. 454, Vol 42 , No. 2-3, pp. 21-46

[5] Indian Standard, IS 456 [2000], "Plain Reinforced Concrete Code of Practice." Bureau of Indian Standards, New Delhi, India.

[6] Indian Standard, IS 13920 [1993], "Ductile detailing of reinforced concrete structures subjected to seismic forces - Code of practice". Bureau of Indian Standards, New Delhi, India.

[7] Paulay T., and Priestley M. J. N. (1992). "Seismic design of concrete and masonry structures". *John Wiley and Sons*, New York.

[8] SAP2000, [2015] Structural Analysis Program, "Integrated Software for Structural Analysis and Design," Computers and Structures Inc CSI., Berkeley, California.

[9] FEMA-356 [2000], Federal Emergency Management Agency, Pre standard and Commentary for Seismic Rehabilitation of Buildings, Washington DC.

[10] Indian Standard IS 1893, Part 1, [2016], "Criteria For Earthquake Resistant Design Of Structures: Part 1 General Provisions And Buildings, 6th Revision". Bureau of Indian Standards, Fifth revision, New Delhi, India.

[11] ATC-40, [1996]. "Seismic evaluation and retrofit of concrete buildings". Applied Technology Council. Redwood City, California, USA.

[12] Kaushik, Hemant B., & Choudhury, T., [2015] "Vulnerability Analysis of Buildings for Seismic Risk Assessment: A Review. Evaluation and Enhancement of Seismic Capacity of Typical Assam-Type Housing View project.", *The Bridge and Structural Engineer*, 45 ,63-76

[13] Earthquake Model-Technical Manual, (2013). Hazus-MH 2.1. Washington DC: Department of Homeland Security, Federal Emergency Management Agency, Mitigation Division.