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ORIGINAL PAPER

Seismic vulnerability assessment of reinforced concrete school building in Nepal

Narayan Ghimire[1] · **Hemchandra Chaulagain**[2]

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Abstract

The school building built with reinforced concrete in Nepal confront high seismic risk during past seismic events. The vast extent structural damage and loss of human life's and property was due to the poor enforcement of the standards, lack of ductile detailing and poor construction materials and practices in Nepal. The effect of earthquake excitation on buildings was ignored after the latest earthquake in 2015. Most of the school structures were used in full-fledged without performing any seismic evaluation. Seeing these situations, especially in Nepal, a vulnerability assessment of such school structures is essential. The seismic risk owing to future earthquake can be minimize by proper vulnerability assessment of the school structures. To this end, the purpose of the study is to evaluate seismic performance of school structure through both vulnerability and fragility assessment. The main research issues are explored through analytical method. For this, structural response parameters are analytically studied through linear and non-linear analyses by finite element program-based software. In dynamic analysis, numerical building models were subjected to three synthetic earthquakes Gorkha, El-Centro and Kobe. The fragility function of case study building structure was plotted with the probability of failure at every 0.1 g interval of PGA, which provided technical base for seismic vulnerability assessment of school buildings. The result indicates that the selected school building is found to be vulnerable compared with the standard international guidelines. The results of this study are more useful for different governmental authorities, emergency response organization who are directly involved for proper planning and implementation.

Keywords Seismic vulnerability · School building · Fragility function · Vulnerability curve · Non-linear analysis

Introduction

Nepal is centrally located in a seismically active Hind-Kush Himalaya region which has a long past history of devastating earthquake. The major seismic events have been occurred in the years of 1255, 1408, 1505, 1803, 1810, 1833, 1934, 1988 and 2015 AD (Pandey et al. 1995). The geological location of Indian and Tibetan tectonic plates results to cause large earthquakes in the entire Himalayan region (Khattri 1987). Similarly, as indicated in Fig. 1, Nepal and adjoining Himalayan arc has experienced large historical earthquakes: some of them are 1897 Shillong earthquake ($8M_w$), 1905 Kangara earthquake ($7.8M_s$), 1934 Bihar-Nepal earthquake ($8M_w$) and 1950 Assam earthquake ($8.6M_w$) (Gupta & Gahalaut 2014). These all evidences are the indicator for the possibility of future earthquakes in the entire Himalayan region.

Among the recorded past history of major earthquake in Nepal, the 1934 Bihar-Nepal earthquake magnitude around 8 (Mw), with maximum intensity of X (MMI), stocked eastern half of Nepal and killed more than 8,500 people and heavy damaged buildings (Rana 1934). In 1988, the Udayapur earthquake in Nepal of magnitude 6.6 (Mw) damaged more than 60,000 buildings and approximately 721 people died and injured thousands of people (Thapa 1988). The latest 2015 Gorkha earthquake caused around 9,000 casualties, 20,000 injuries and destroyed 500,000 buildings structures in Nepal (Chaulagain 2018). Past studies had shown that damaged building and consequently loss of life and economic property due to earthquake are related

✉ Hemchandra Chaulagain
hchaulagain@gmail.com

[1] Master of Science in Structural Engineering, School of Engineering, Pokhara University, Pokhara-30, Kaski, Nepal

[2] School of Engineering, Pokhara University, Pokhara-30, Kaski, Nepal

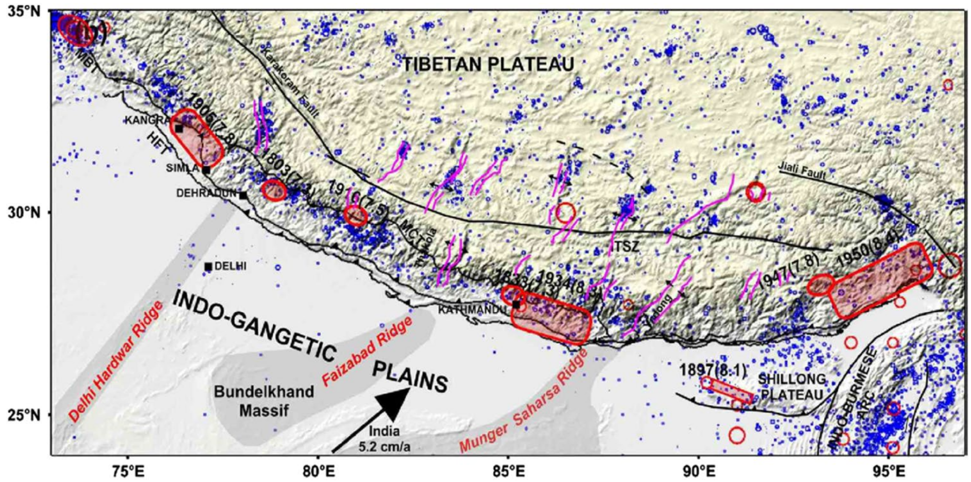

Fig. 1 Major tectonic features and seismicity of the Himalayan arc (Gupta & Gahalaut 2014)

with uncontrolled building construction and urbanization in developing countries (Ahmad et al. 2014).

In 2009, Parajuli studied and highlighted the past earthquakes and location of geological faults in Nepal (see Fig. 2). The earthquakes and location of faults in and around the entire Pokhara valley highlighted the possibility of future earthquake in the region.

In previous studies, researchers found out the damages of school buildings and casualties in their research (Gautam et al. 2020; Miyamoto et al. 2011). Augenti et al. (2004) highlighted the damage and collapse of 300 schools building in Molise located in southern Italy zone. Miyamoto et al. (2011) discussed the different level of damages of school building structures in developing countries. These all evidences clearly picture the possible damage scenario during earthquakes and conditions of school building in developing countries. The construction practices of school building in Nepal have shown improper design guidelines, large spacing stirrups in beam and column, insufficient lap-splice length and structure deficiencies, among other (Gautam et al. 2020). RC structure if constructed not properly can result in structure failure, loss of human life's and economic property losses. On the other side, poor performance of seismic design buildings is most excited if the constructed buildings possess construction deficiencies (Ahmad et al. 2018; Rizwan et al. 2020; Manfredi et al. 2014). Furthermore, depending on how and when buildings were built, designed and furnished, buildings may have feebleness that makes them more vulnerable during earthquakes.

Based on the above contextual situations, this study explores the seismic performance status of existing school building in Nepal. To this end, the school building in Pokhara University in Nepal was considered as a case study. Numerical building block models were prepared in finite element software program ETABS (ETABS 2016) and models were examined along various analyses such as nonlinear static and linear dynamic analyses. In linear dynamic analysis, numerical building models were subjected to three synthetic earthquakes Gorkha, El-Centro and Kobe. Based on these analyses, fragility curve was constructed with the probability of failure at every 0.1 g interval of PGA. This will consequently help decision makers to assess the seismic

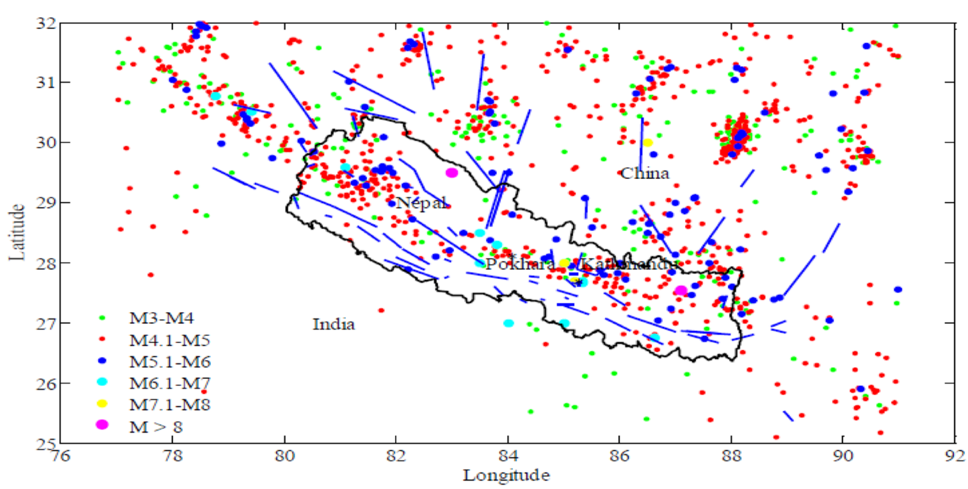

Fig. 2 Historical earthquakes and faults in and around Nepal (Parajuli 2009)

performance condition of structures under investigation during its life period and earthquake loss estimation.

Necessity of vulnerability assessment in Nepalese structure

Historically school buildings in Nepal were built with adobe followed by stone masonry, brick masonry and composite masonry. In some regions, buildings with timber frame were more popular. After the increasing maturity of education system as well as construction industries in Nepal, RC building construction has started from late 1970s. In the recent years, structures made with adobe, brick masonry, stone masonry, timber frame structures and composite masonry are significantly replaced by reinforced concrete construction (Chaulagain et al. 2014). These were basically owner-built construction. The structures with owner-built construction were constructed without following any standard guidelines leads to insufficient strength, lack of ductile detailing and poorly constructed. All these types of constructions will highly vulnerable in future earthquakes. Considering the facts, Nepal government realized and approved the first building code in 1994 (NBC: 1994). The implementation of Nepal building code is not strictly enforce due to lack of trained technical skilled human resources (Chaulagain et al. 2014). As a result, majority of the buildings were built without strictly following engineering guidelines. These situations lead to increase the demand for vulnerability assessment of school structures in Nepal.

Seismic vulnerability assessment

The seismic vulnerability of building structure can be described as a lack of ability of existing buildings like monumental, school and historical buildings etc. sufficiently resist earthquake force during ground shaking. The need of vulnerability assessment is to compute performance levels of structures in earthquake scenarios. The degree of structural performance level mainly depends on the geometric functions: plan configuration, length breath ratio, age etc. and structural characteristic: stiffness variation, mass variation, strength, elastic property and ductility etc. Researchers have proposed and implemented the several vulnerability assessment methods for different structural types (Calvi et al. 2006; Lagomarsino and Giovinazzi 2006). Empirical method based on identifying the damage patterns of building from the past earthquake data based on the post-earthquake surveys. Similarly, in the expert opinion methods, the judgement of the experts of the relevant field and their capable to relate probability of damage of buildings with earthquake intensity (Kostov et al. 2007). This method is used in absent of damage data in post-earthquake. The hybrid method is based on combine of expert opinion or post-earthquake data with analytically derive the damage levels from a mathematical model of building topology (Calvi et al. 2006). Analytical methods follow the concept of detailed analysis of existing building based on assessment of displacement capacity of building related to various damage states and displacement demand (Akkar et al. 2005). From above discussion, it can be concluded that the vulnerability of building has been evaluated by fragility curves. Fragility curves are useful tool to estimate the overall risk and loss estimation of civil infrastructure from probable earthquakes and to predict impact of future earthquakes on an economic sector.

Modelling approach

The structural model for numerical analysis was created and analyzed using the ETABS software (ETABS 2016). The software provided four different analyses, such as model analysis, non-linear static analysis and linear and no-linear dynamic analysis. The purpose of these analyses used to identify the behaviors of the spatial frame of blocks by providing dynamic and static loading.

For performing non-linear analysis in this study, the non-linear stress strain curve of concrete for confined and unconfined model based on Mander et al. (1988). Similarly, a simple stress strain curve consisting controlling parameters were computed for modelling the steel reinforcement. The beam and column elements were modelled as elastic elements with plastic hinges at ends of members. The plastic hinges represent the concentrated behavior of the structure member during numerical analysis. During modelling, the moment curvature characteristics of the plastic hinges were determined with non-linear constitutive laws for steel and concrete. The finite element software was based on lumped plasticity approach. Default hinges characteristics used for concrete sections was based on FEMA-356 (2000) and ATC-40 (1996) criteria. Flexural default hinge (M3) was assigned each ends of the beams member and column interacting (P-M2-M3) coupled frame hinges type of hinge property were assigned both lower and upper ends of member.

Non-linear static analysis

The use of non-linear static analysis came in practice in 1970; but potential of the pushover analysis comes in popularity in last two decade due to its simplicity and validity. Pushover analysis especially used to estimate strength as well as drift capacity value of existing building structure (Astriana et al. 2017). It can be proceeding into several standard seismic guidelines and design codes in the last some years (ATC 40 1996; FEMA-356 2000).

The pushover analysis are normally categorized as force and displacement controlled approach. In force-controlled method, total lateral load is applied in incrementally and each incremental load, stiffness matrix of structure has to be changed when the structure passes from elastic to inelastic state. In displacement-controlled method, the building top storey displacement is incremented in such a way that required horizontal force applied on building laterally proportional to fundamental horizontal mode of the building in that direction of lateral loading (Datta 2010). In this study, displacement control method was used for pushover analysis. In this approach, the analysis could be carryout up to the desired level of the building displacement (Wilson 2002). During analysis, building models was subjected to both gravity loading and monotonic displacement lateral load. In the analysis, load in the building structure is continuously increasing from elastic to non-elastic behavior of structure until target displacement is achieved. The target displacement is a maximum top storey displacement of building structure that was identified by standard guidelines under the selected earthquake ground motion. The following procedures were adopted during analysis:

- Create 3-D building models of study RC buildings
- Apply dead and live load on a numerical model
- Define and assign of plastic hinge properties on beam column elements, assuming 10% relative distance
- Apply the displacement on the top storey of structure whose value is large than those associated with target displacement
- Developing plastic hinge progressing sequence in different steps of the loading
- Develop tables of roof displacement verse base shear or pushover curve.

Time history analysis

The study of seismic response of structural behaviors under the dynamic loading of representative past earthquake data is known as time history analysis. The earthquakes of Gorkha, Kobe and El-Centro are used for excitation of structures during dynamic analysis. In the analysis, structural performance is also observed in linear time history analysis. In fact, the linear time history analysis computes the solution of dynamic equilibrium equations at stipulated time using the material properties and applied load on the structure. (Wilson 2002). The preliminary aim of the linear time history analysis in this study was to compute the displacement demand of structure response in terms of top storey displacement (Maskey 2012).

Synthetic accelerograms

In the study, the demand parameters are identified with standard earthquake time history records. The dynamic analysis was performed by fulfilling the requirement of the appropriate set of acceleration (Fahjan 2008). Real accelerograms were the more advantageous to use, since they were sincere records of ground motion from earthquakes (Chaulagain et al. 2017). But in context of Nepal, real accelerograms records were not available sufficient for time history analysis. Hence, the minimum three earthquake time history data: Gorkha, El-Centro and Kobe was employed for reliable estimate of structural response (ASCE 7–16 2017; Eurocode-8 2004). The earthquake accelerogram had compatible with given region design response spectrum. The given damping ratio, each accelerograms corresponding with single response spectrum and each response spectrum correspond an infinite value of accelerograms (Lagaros et al. 2013). The earthquake artificial accelerograms generated in 1976 based on given region code define response spectrum (Gasparini and Vanmarke 1976). The three-time history data used in this study is presented given Table 1. Similarly, the plots of time history data is shown in shown in Fig. 3.

Damage state criteria

The structural behavior can be evaluated by damage thresholds. These are normally called limit states which the threshold between various damage level of the building structures. Researchers proposed damage states based on drift ratio and yield displacement and ultimate displacement of structures. Dumova-Javanoska (2000) was proposed five grade of damage states none, minor, moderate, severe and collapse based on damage index for their fragility curve. Akkar et al. (2005) proposed three damage states: light damage, moderate damage and severe damage in terms of global parameter such as maximum drift ratio. Barbat et al. (2008) proposed four damage state thresholds namely: slight damage, moderate damage, severe damage and complete damage based on spectral yielding and ultimate displacement. Jiang et al. (2012) used four damage levels based on maximum IS- drift ratio and global damage index for their fragility curves that described performance

Table 1 Peak ground acceleration used for the dynamic analysis

Name of earthquake	Peak ground acceleration (PGA)
Kobe earthquake	0.379 g
El Centro earthquake	0.365 g
Gorkha earthquake	0.4 g

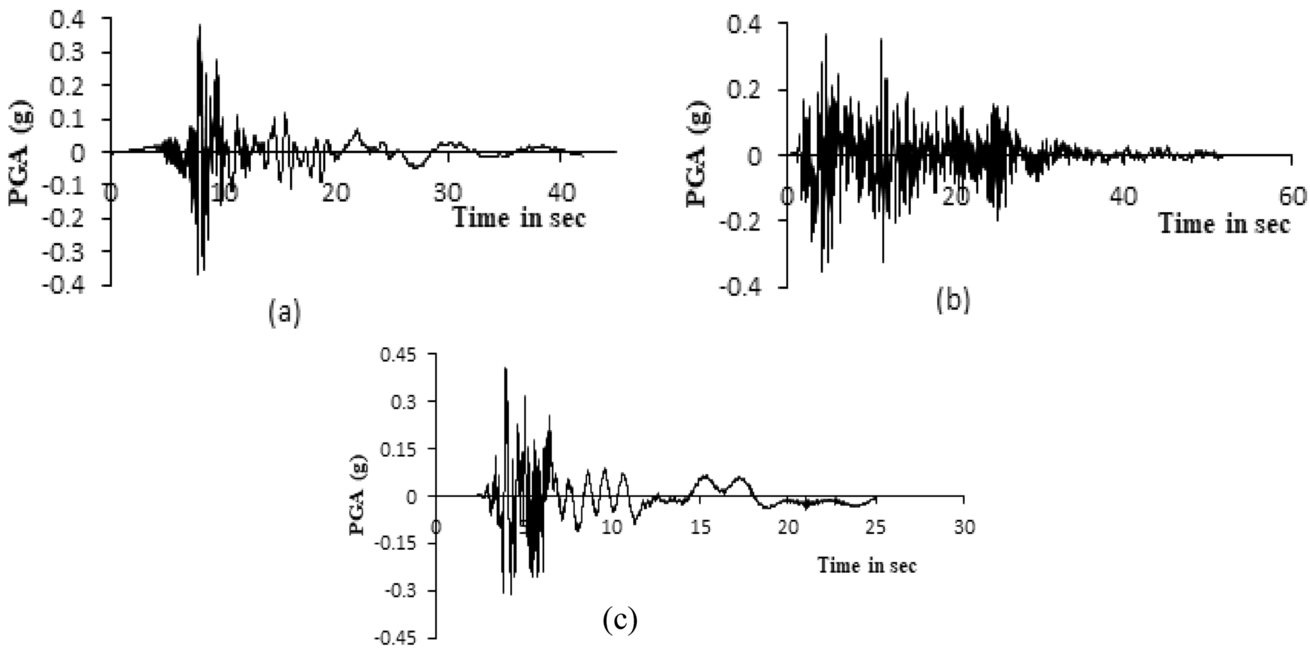

Fig. 3 Time history graph: **a** Kobe earthquake, **b** El-Centro earthquake and **c** Gorkha earthquake

level of RC buildings. Kricil and Polat (2006) proposed yielding and collapse limit states for describing performance levels of structure. The four damage states namely: slight damage, moderate damage, extensive damage and complete collapse is used in HAZUS for fragility curves. Similarly, Rosetto and Elnashai (2005) proposed seven damage states: none, slight, light, moderate, extensive, partial collapse and collapse to check the performance level of building structures. Ahmad (2019) developed four damage limit states: slight, moderate, extensive and incipient damage for RC building structure types in Himalayan region that used to construct fragility curves. Lagomarsino and Giovinazzi (2006) proposed four damage states: slight damage, moderate damage, extensive damage and complete damage in terms of yield (dy) and ultimate (du) displacement for their fragility curve.

- Slight damage = 0.7dy
- Moderate damage = 1.5 dy
- Extensive damage = 0.5 (dy + du)
- Complete damage = du

Based on the review of limit states proposed by different researchers, it is found that most commonly adopted limit states are: slightly damage, moderate damage, extensive damage and complete damage. So, in this paper, Lagomarsino and Giovinazzi (2006) proposed four limit state was used for the construction of fragility curve that used to describe performance level of study building.

Case study

Building model

In this study, one of the school buildings of Pokhara University Nepal (School of Health and Allied Science, SHAS) was considered as a study building. The whole structure consists of three blocks built in 2004 AD. These building represents the majority of school buildings in Pokhara region. The building consists of three parts: left portion before the expansion joint is called block A, mid portion between two expansion joints is called block B and right portion after expansion joint is called block C (see Figs. 4, 5, 6).

Building block A, B and C measures 14.1 m × 14.8 m, 24.6 m × 16.8 m and 14.1 m × 14.8 m in x and y direction respectively. Block A and C have the same plinth area 208.68 m^2 and block B has a plinth area 413.28 m^2. All the building blocks have storey height of 3.3 m and slab thickness 150 mm. All the buildings consist of isolated footing in foundation. Block A comprises five moment resisting frame in x and four moment resisting frame in y directions. Similarly, block B consist of six number of moment resisting frame in x direction and four moment resisting frame in y directions while in middle portion, one additional structural frame was inserted. Likewise, in building block C models have five moment resisting frame in x and four moment resisting frame in y directions. The plan and 3-D of the study building blocks structures are presented in Figs. 4, 5, 6. Similarly, Table 2 characterized the beam and

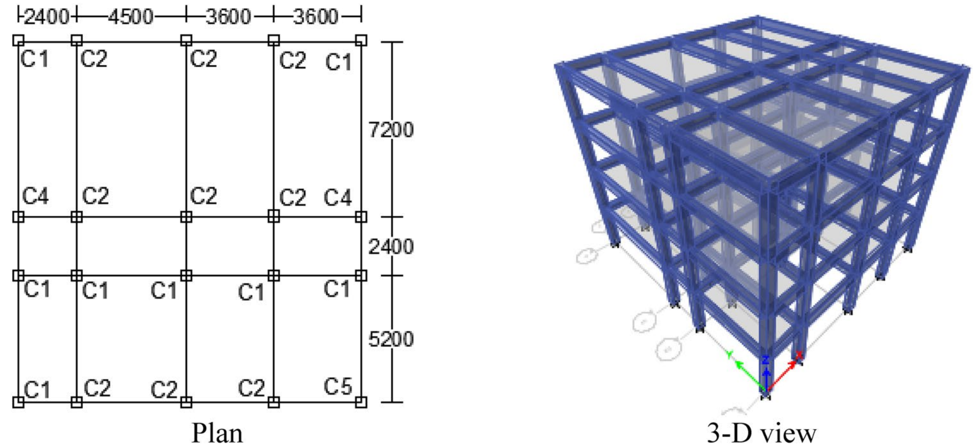

Fig. 4 Plan and 3-D view of block A building model

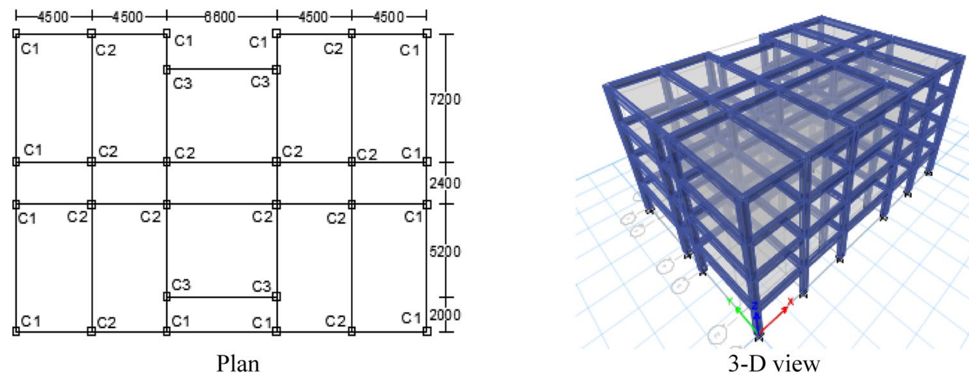

Fig. 5 Plan and 3-D view of block B building model

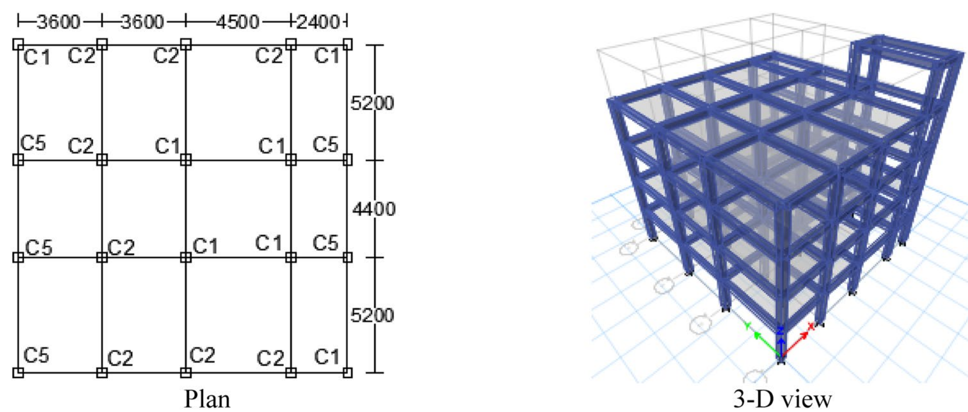

Fig. 6 Plan and 3-D view of block C building model

column configuration of study buildings. The floorwise layout of beam and column is presented in Fig. 7.

During modelling, weight of infill wall was considered but stiffness of the masonry wall was ignored. Generally, masonry infill in bare frame structure used to separate the spacing of structure. But, masonry infill increased seismic resistance of the bare frame structure and avoided flexural damage in beam and column and shears damage in join panel region due to it shown behaviors of diagonal compression strut, and it was due to damaged characteristic of masonry infill that provided energy dissipation through masonry sliding over multiple cracks (Ahmad et al. 2019; Chaulagain et al. 2016).

Material properties

To identify response of the structure, several material properties should be adopted. The concrete material properties and properties of steel reinforcement were taken from experimental testing and blue print of existing drawing. The yield strength value of steel reinforcement for all the building blocks was considered as 415 MPa. The value of concrete

Table 2 Building section properties

Beam			Column			Slab
Section	Size (mm)	Rebar	Section	Size (mm)	Rebar	
B1	300×500	3 Ø25 3 Ø20	C1- (G.F & 1st floor)	400×450	4 Ø20 4 Ø16	150 mm thickness
B2	300×500	4 Ø20 3 Ø20	C2- (G.F & 1st floor)	400×450	4 Ø20 8 Ø16	
B3	300×500	5 Ø20 3 Ø20	C3- (G.F & 1st floor)	400×450	4 Ø25 4 Ø20	
B4	300×500	3 Ø20 3 Ø20	C4- (G.F & 1st floor)	400×450	8 Ø 20	
B5	300×350	5 Ø20 4 Ø20	C5- (G.F & 1st floor)	400×450	8 Ø16	
B6	300×350	4 Ø20 3 Ø20	C1- (2nd & 3rd floor)	400×450	8 Ø16	
B7	300×350	5 Ø20 3 Ø20	C2- (2nd & 3rd floor)	400×450	12 Ø16	
B8	300×350	6 Ø20 3 Ø20	C3- (2nd & 3rd floor)	400×450	4 Ø25 4 Ø16	
B9	300×600	5 Ø20 3 Ø20	C4- (2nd & 3rd floor)	400×450	4 Ø20 4 Ø16	
B10	300×600	6 Ø20 3 Ø20	C5-(2nd & 3rd floor)	400×450	4 Ø16 4 Ø12	

properties such as: elastic modulus, shear modulus, weight density and Poisson's ratio are considered as 19,364.92 N/mm^2, 8068.72 N/mm^2, 25 kN/m^3 and 0.2 respectively.

The building has a 230 mm thick external and 115 mm thick internal wall respectively. The density of wall material taken as 20 kN/m^3. Similarly, dead load was calculated as per drawing specified and live load was taken from IS 875 part 2 specified by Indian standard code (IS 875: 1987). The live load in this study is considered as 4 kN/m^2. Similarly, live load in class room, store, floor finish bath room taken as 3 kN/m^2, 5 kN/m^2, 2 kN/m^2 and 1.5 kN/m^2 respectively.

Results and discussion

All the outcomes of this study is analytically discussed in this section. First section presents the results of pushover analysis. In middle section, the vulnerability statues of school buildings are discussed. In last section, the results from fragility analysis are presented.

Capacity curve and inter-story drift

The capacity curve represents the resistance behavior of structure during elastic to inelastic range. It is normally represented by horizontal base shear and roof displacement. In fact, it estimates for the representative top storey nodes for both in X-direction and Y-direction of loading. Figure 8 represents the results of pushover analysis for studied building models for each loading direction. In the present study elasto perfectly plastic method is used for idealizing the pushover curve to determine yield displacement and ultimate displacement of the building structures (Park 1988). Similarly, the inter-storey drift represents the response indicator of structures. In this study, the base shear force of block B building was found higher than block A and C both in x and y direction of loading. The rate of change of IS drift in all the building models in both direction of loading was found to be regular and consistent in all the storey level. It was also found that ground storey had higher IS drift than other storey level.

Vulnerability assessment of structure

In vulnerability assessment, the standard accepted performance evaluation methods were adopted. The different study explores the same concept but differ to indicate the performance level. This study is based on the limit states proposed by international guidelines (ATC-40 & FEMA 356).

As shown in Table 3, FEMA-356 suggested the maximum drift limit for various performance levels for RC building structures. The performing of non-linear dynamic analysis was used to determine the maximum drift demands value for the proposed vulnerability curve with different peak ground acceleration value. Table 4 reflect that basic performance indicators for the buildings as per FEMA-356 is presented. The results of vulnerability curves of three studied building blocks were plotted in Fig. 9. The curves were observed that all the three building blocks have similar behaviors and damage level at different ground motion values. The results

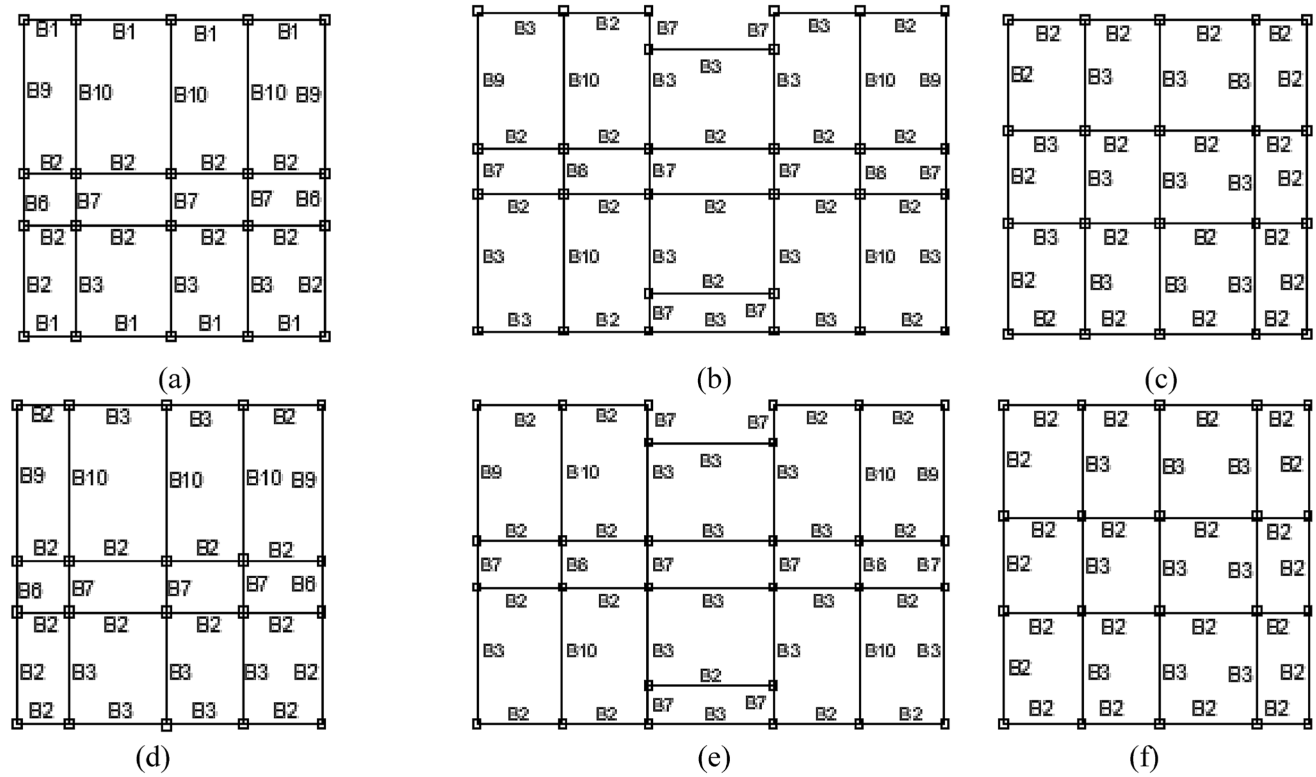

Fig. 7 Floor beam layout **a** Block A: layout of 1st floor beam, **b** Block B: layout of 1st floor beam, **c** Block C: layout of 1st floor beam, **d** Block A: layout of 2nd, 3rd and 4th floors beam, **e** Block B: layout of 2nd, 3rd and 4th floors beam and **f** Block C: layout of 2nd, 3rd and 4th floors beam

indicate that the buildings blocks have near collapse performance at 0.40 g PGA in 475 years return period (see Table 5). However, based on FEMA-356 drift limit, all the structures must be within the extensive damage or life safety performance level at 475 years return periods of earthquake. It is seen that all the building models do not meet the standards to withstand in future earthquakes.

Fragility analysis

Fragility curves are obtained from fragility analysis. For this analysis, fragility curves are drawn with probability of failure (Pf) and increasing demand displacement (Sd) value, obtain from best fitted log-normal distribution function of the equation that defined by median (Sc) and standard deviation parameters (β). The probability of failure in the structure is mainly derived using the following relationship:

$$\mathrm{P}(f) = \varnothing\left[\frac{ln(Sd/Sc)}{\beta}\right]$$

where, $\phi()$ be the standard cumulative normal distribution function, Sd and Sc be the demand displacement and medium of the damage state. The demand displacement in the structure was obtained by performing dynamic analysis (linear) of three different earthquakes inputs (Kobe, El Centro and Gorkha earthquakes). The medium damage states were determined with yield displacement and ultimate displacement proposed by Lagomarsino and Giovinazzi (2006). Table 6 indicates that summary of demand and capacity of all three building models. β be the log standard deviation that represents total uncertainty. Its value is considered as an equivalent to 0.64 (HAZUS-MH-MR4 2003). Fragility curves are constructed for four damage state with three different synthetic earthquake data as shown in Figs. 10, 11, 12. The curves represent the cumulative probability of failure from 0 to 1 corresponding to peak ground acceleration from 0 to 1 g. The probability of failure of building for three earthquakes in four different damage grades are summarised in Tables 7, 8 and 9.

Figures 10, 11, 12 value in the Pokhara region for 475 years return period was identified as 0.40 g (Parajuli, 2009). So, the buildings probability of failure at different damage state at different earthquake scenarios was observed at 0.4 g PGA.

The fragility curves in building model A, the probability of failure of building get slight damage will be quite high, approximately 98%. However, the damage level of moderate,

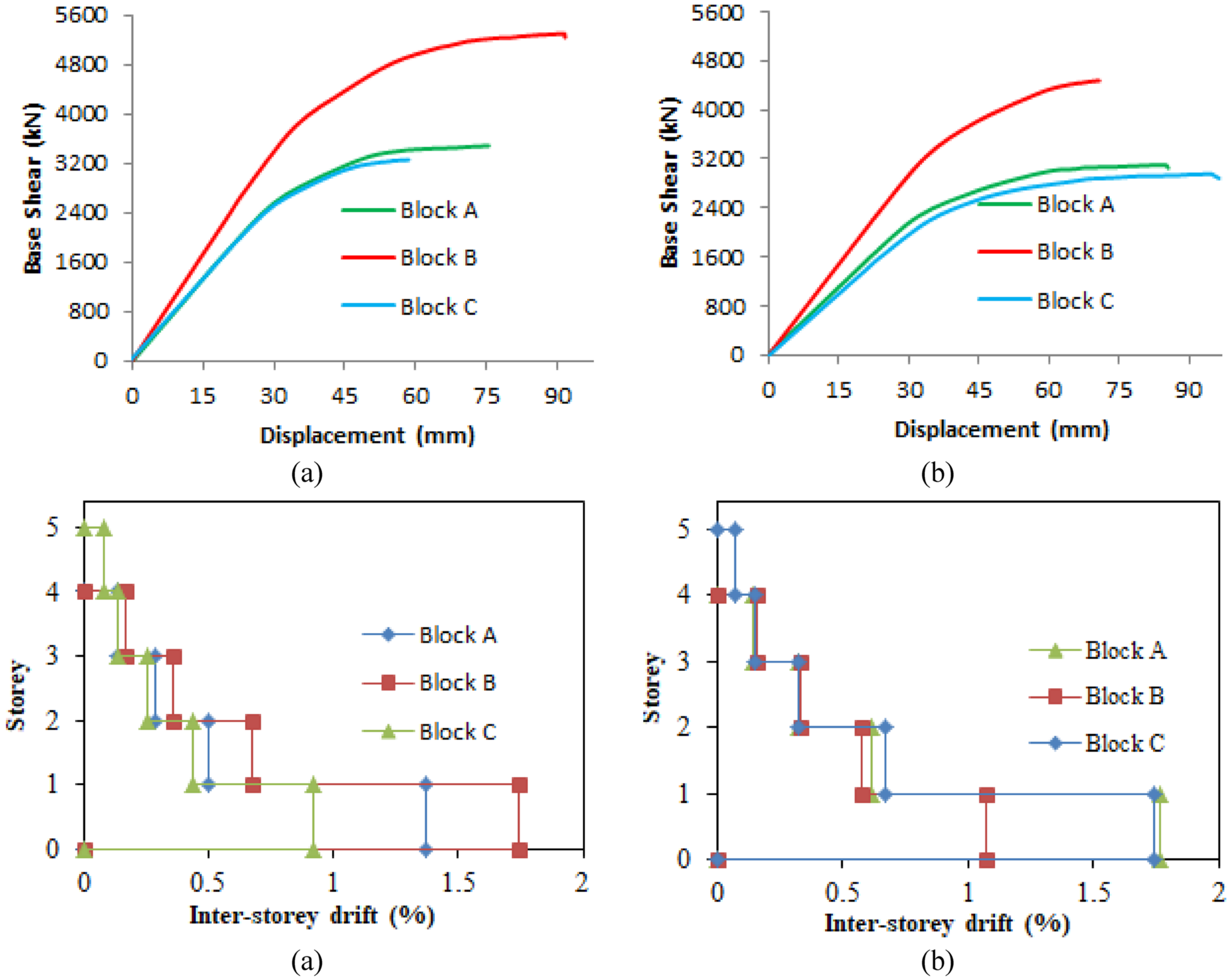

Fig. 8 Pushover curve and corresponding IS drift of Block A, B and C with **a** X-direction and **b** Y-direction

Table 3 Performance level and corresponding maximum drift limits of RC buildings

Performance level	FEMA-356
Slight damage	0.2
Moderate damage	0.5
Extensive damage	1.5
Near collapse	2.5

extensive and complete damage was estimated as 85.6%, 83% and 67.6% respectively. Nearly similar results obtain figures b and c because of same building but different earthquake data. Similarly, the fragility curves in building model B shows that the slight damage of the building is higher as compare to other damage levels with same earthquake

Table 4 Basic performance objectives for buildings according to FEMA-356, 2000

		Fully operational	Operational	Life safety	Near collapse
Engineering Design levels	Frequent(43-yrp)				
	Occasional (98-yrp)		X		
	Rare (475-yrp)			X	
	Very rare (975 yrp)				X

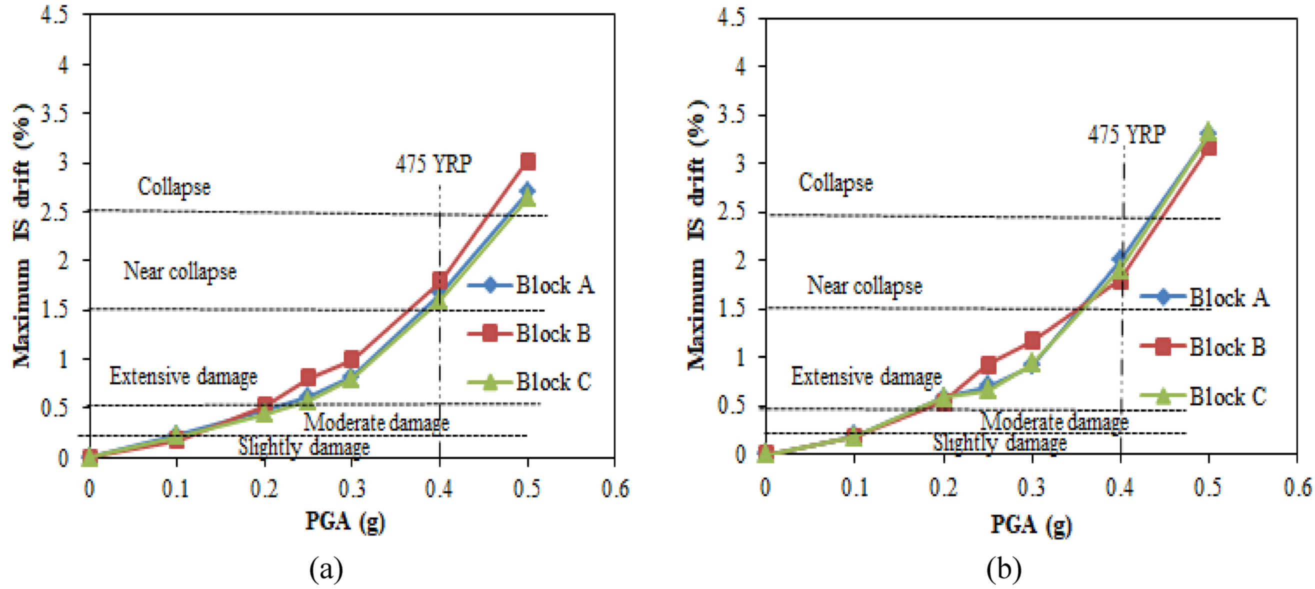

Fig. 9 Vulnerability curves of maximum IS drift for Block A, B and C with **a** X- direction and **b** Y-direction

Table 5 Seismic risk for Pokhara Valley (Parajuli 2009)

Return period (years)	Peak ground acceleration ($\mathbf{m/s^2}$)
475	0.4 g

Table 6 Summary of demand and capacity of model

Name	Yield displacement (mm)	Ultimate displacement (mm)	Maximum demand for earthquakes (mm)		
			Gorkha	El Centro	Kobe
Block A	38.7	85.5	114.5	105.7	107.9
Block B	42.0	92.0	108.6	122.2	117.3
Block C	41.0	96.4	120.9	111.4	113.9

intensity. The damage state of Gorkha earthquake, it had slight, moderate, extensive and complete damage is 97.9%, 80.2%, 77.4% and 60.2% respectively. In block C building model; slight, moderate, extensive and complete damage state was estimated 98.8%, 85.5%, 81.1% and 63.8% for Gorkha earthquake respectively. The results highlighted that at lower PGA value the slope of fragility curve was larger and vice versa. Therefore, probability of failure of damage states increased higher when small change of PGA value at lower level PGA and less increased probability of failure of damage state when small change of PGA value at higher level PGA. From the aforementioned discussion, it can be concluded that the structural performance mainly depends on the probability of failure of each damage state. The higher

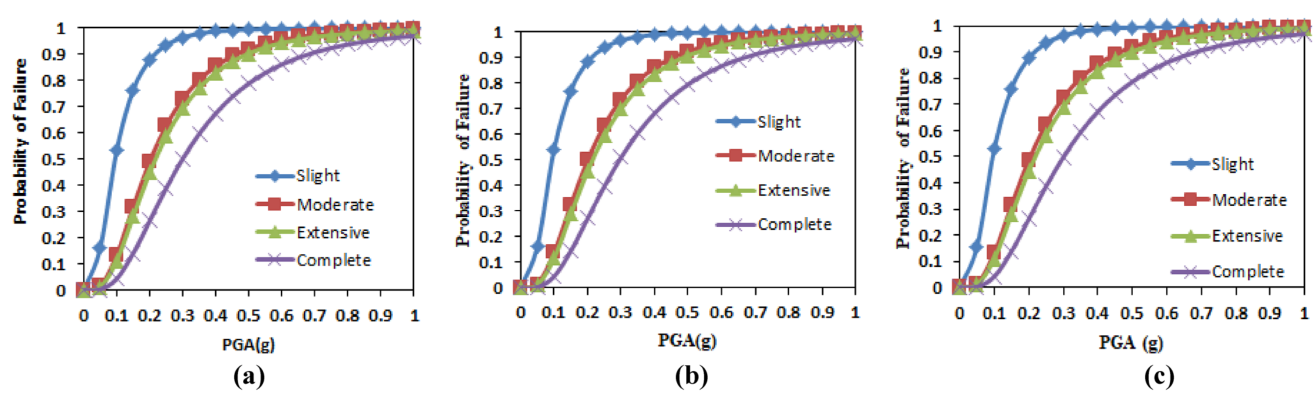

Fig. 10 Fragility curves of block A building: **a** Gorkha earthquake, **b** El- Centro earthquake, **c** Kobe earthquake

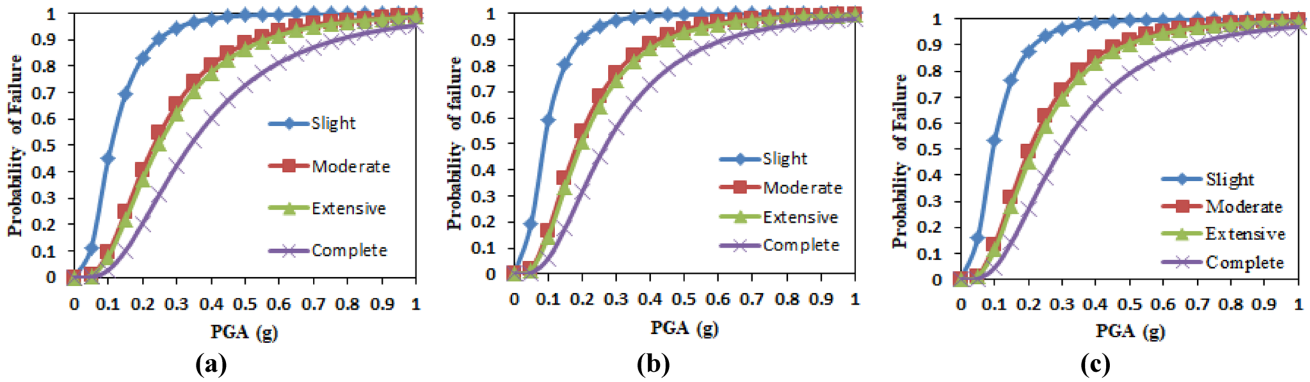

Fig. 11 Fragility curves of block B building: **a** Gorkha earthquake, **b** El- Centro earthquake, **c** Kobe earthquake

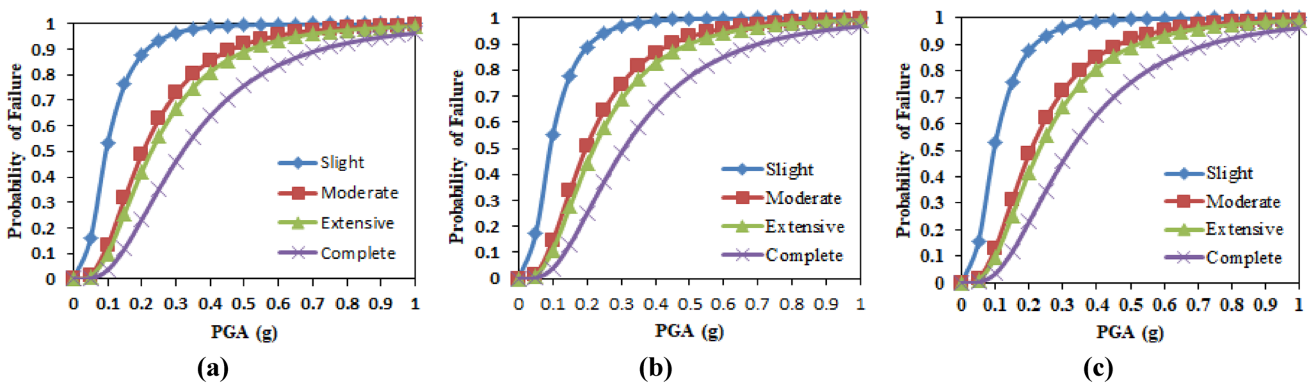

Fig. 12 Fragility curves of block C building: **a** Gorkha earthquake, **b** El- Centro earthquake, **c** Kobe earthquake

Table 7 Probability of failure of buildings at Gorkha earthquake

Buildings	Slight (%)	Moderate (%)	Extensive (%)	Complete (%)
Block A	98.0	85.6	83.0	67.6
Block B	97.9	80.2	77.4	60.2
Block C	98.8	85.5	81.1	63.8

Table 8 Probability of failure of buildings at El-Centro earthquake

Buildings	Slight (%)	Moderate (%)	Extensive (%)	Complete (%)
Block A	98.8	86	83.5	68.2
Block B	99.0	88.5	86.5	72.9
Block C	98.9	86.6	82.6	65.6

Table 9 Probability of failure of buildings at Kobe earthquake

Buildings	Slight (%)	Moderate (%)	Extensive (%)	Complete (%)
Block A	98.8	85.4	82.8	67.3
Block B	98.8	85.4	83.1	67.9
Block C	98.7	85.2	80.9	63.5

probability of failure reflects the low performance level in the structure.

Figure 13 represents a combined fragility curves of school buildings for three earthquakes. The fragility curve of slightly damage grade of block A building have similar behaviors to all three earthquakes (Gorkha, El-centro and Kobe). Similarly, the fragility function of adopted damage grade (moderate damage, extensive damage and complete collapse) are also same for considered earthquakes. In combined fragility curves of blocks B and C, the behaviors of each damage states of three earthquakes have found as nearly same patterns. It is because of the application of different earthquakes excitation in the same building. This indicates that fragility curve of building depends on properties and configuration of buildings but not co-related much more with earthquakes data. The results have the similar trends associated with Ozer & Erberik (2008) and Gaudio et al (2015).

From the analysis, it is also highlighted that accuracy of structural performance in terms of fragility function depends on the more accelerograms data used in time history analysis. The fragility function defined in this study is based on function of demand displacement. The demand displacement

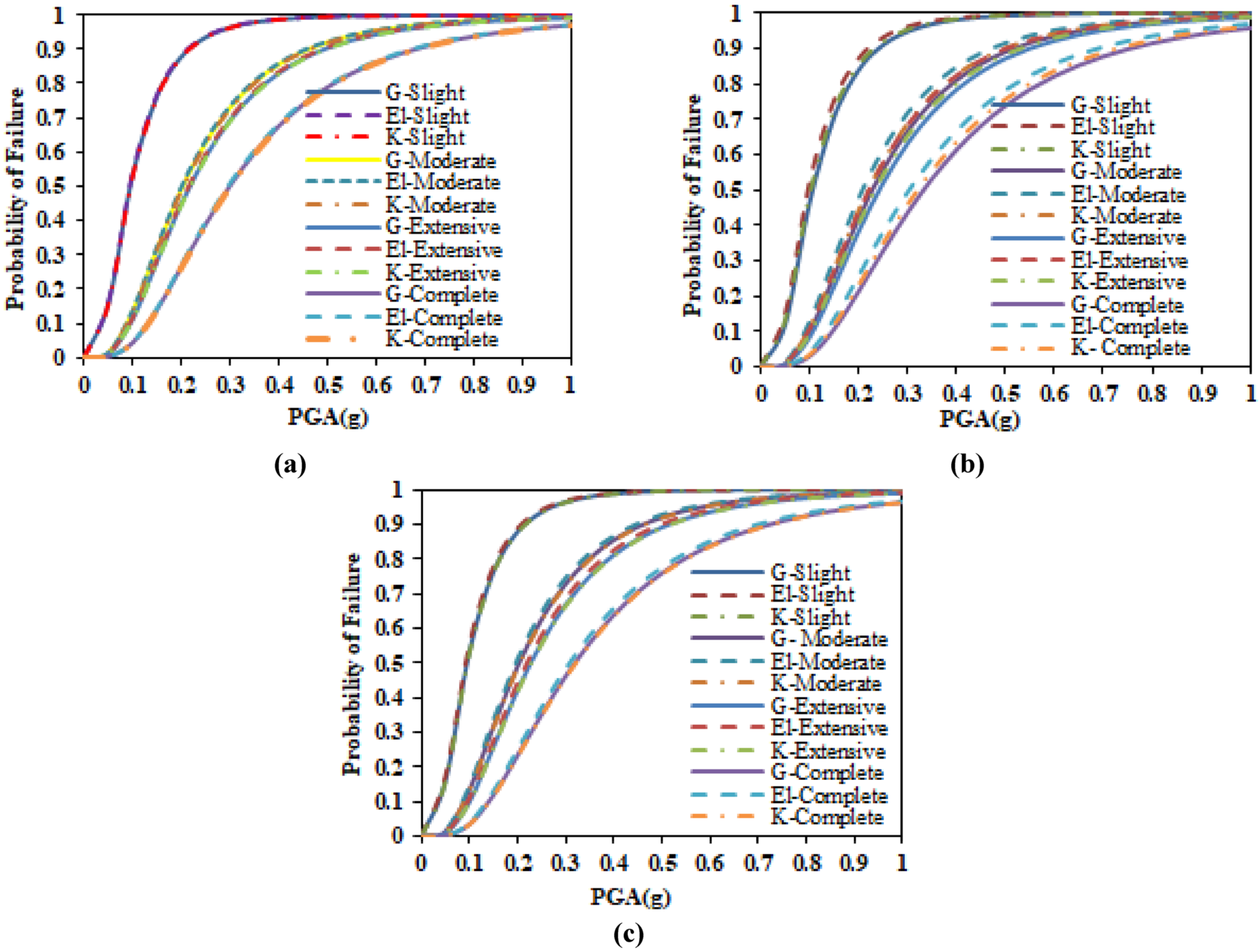

Fig. 13 A combined fragility curve of **a** block A, **b** block B and **c** block C for three earthquakes

(Sd) value in fragility function obtained from time history analysis with earthquake accelerograms inputs. Therefore, more selection of earthquake accelerograms gives more reliable result of structure response (ASCE 7–16 2017).

Conclusions

This study explored the status of seismic vulnerability of existing RC school building in Nepal. This is achieved through the case study of school building of Pokhara University. The structural performance in various damage grades namely: slight damage, moderate damage, extensive damage, and complete collapse was studied analytically. For this, the numerical analysis was done with finite element software ETABS. For this, non-linear static and linear dynamic analyses of building blocks were performed by finite element based software program ETABS. In dynamic analysis, building models were subjected to the different synthetic earthquakes. In the present study, the fragility function is derived with plotting the probability of failure at every 0.10 g interval of peak ground acceleration. The main conclusions for the analysis are summarized by following:

The rate of change of inter-storey drift of all the building models have regular and consistent in all floor levels during both direction of loading. It was also found that ground storey level has maximum IS-drift. The IS-drift was significantly reduced in the upper stories level.

The results indicate that the fragility curves of buildings in different earthquakes mostly depends on configuration and properties of building structures. However, the earthquake time history has minimal effect on its performance.

The fragility curve was flatter at lower PGA and stiff at higher PGA in all the damage grades. The vulnerability curve of all buildings model was found as nearly collapse performance level at PGA 0.4 g (475 year of return period) in both direction of loading.

The selected school building was found to be vulnerable compared with the standard international guidelines.

Recommendation

The results of this study are useful for government authorities, emergency response organizations who require the status of seismic risk for proper planning to upgrade building code and emergency response preparedness plans.

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Compliance with ethical standards

Conflicts of interest On behalf of all authors the corresponding author states that there is no conflict of interest.

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