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Seismic response of current RC buildings in Kathmandu Valley

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Abstract. RC buildings constitute the prevailing type of construction in earthquake-prone region like Kathmandu Valley. Most of these building constructions were based on conventional methods. In this context, the present paper studied the seismic behaviour of existing RC buildings in Kathmandu Valley. For this, four representative building structures with different design and construction, namely a building: (a) representing the non-engineered construction (RC1 and RC2) and (b) engineered construction (RC3 and RC4) has been selected for analysis. The dynamic properties of the case study building models are analyzed and the corresponding interaction with seismic action is studied by means of non-linear analyses. The structural response measures such as capacity curve, inter-storey drift and the effect of geometric non-linearities are evaluated for the two orthogonal directions. The effect of plan and vertical irregularity on the performance of the structures was studied by comparing the results of two engineered buildings. This was achieved through non-linear dynamic analysis with a synthetic earthquake subjected to X, Y and 45° loading directions. The nature of the capacity curve represents the strong impact of the P-delta effect, leading to a reduction of the global lateral stiffness and reducing the strength of the structure. The non-engineered structures experience inter-storey drift demands higher than the engineered building models. Moreover, these buildings have very low lateral resistant, lesser the stiffness and limited ductility. Finally, a seismic safety assessment is performed based on the proposed drift limits. Result indicates that most of the existing buildings in Nepal exhibit inadequate seismic performance.

Keywords: non-engineered buildings; performance evaluation; P-Delta effect; seismic vulnerability

1. Introduction

Nepal is located in the highly seismically active Himalayan region. Over the last centuries, huge earthquakes occurring in 1803, 1833, 1897, 1905, 1934 and 1950 in the Himalayan region resulted in large numbers of casualties and caused extensive damage to structures (Roger *et al.* 2001). The great Gujarat Earthquake in India in 2001 revealed the vulnerability of unplanned cities

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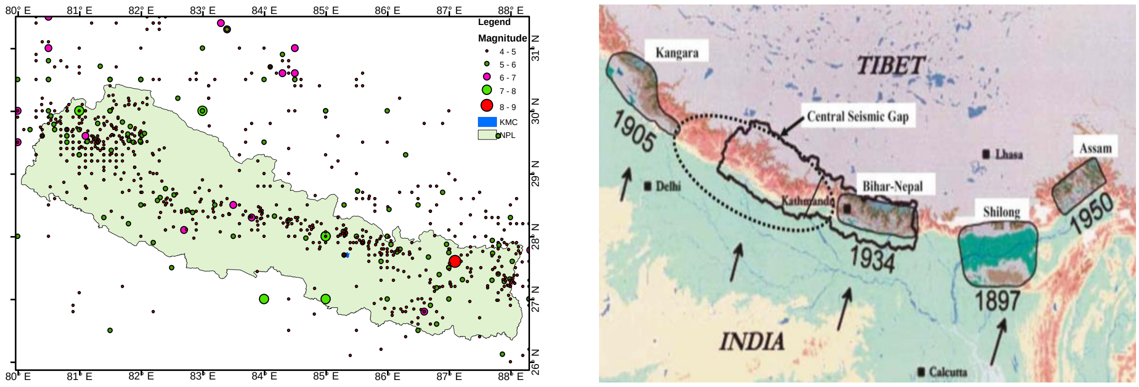


Fig. 1 (a) A catalogue for Nepal Himalaya earthquakes from 1255 to 2012 and (b) Distribution of probable rupture zones of the 1897, 1905, 1934 and 1950 earthquakes along the Himalayan arc

and villages. Nepal lies closer than Gujarat to the subduction zone where the Indian plate passes under the Himalayas, and may actually be susceptible to an even larger-scale earthquake.

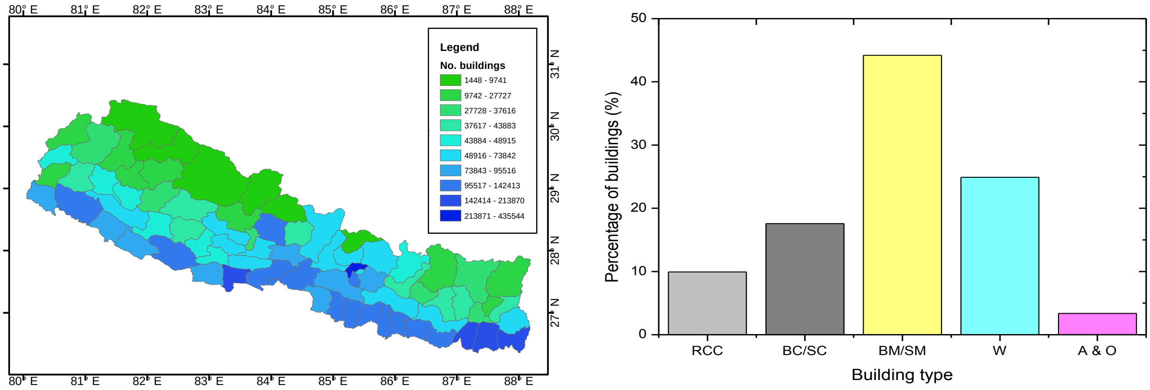
In last one century alone, over 11,000 people were killed in four major earthquakes in Nepal. In 1934, an earthquake of magnitude 8.4 killed 8,519 people and damaged over 80,000 buildings in Nepal (Rana 1935). Later, the 1988 Udayapur earthquake also resulted in heavy loss of life in the eastern region and also in the Kathmandu Valley (Thapa 1935). The location of rupture areas shows a gap along the mountain range between the location of the 1905 Kangra and 1934 Bihar-Nepal earthquakes, as shown in Fig. 1 (Yeats *et al.* 1991). It is believed that this region has not experienced such an earthquake since the last large earthquake. It is hard to estimate how much casualty and damage will be caused in Nepal if an earthquake happens today in the central seismic gap. A study of the seismic record of the region suggests that earthquakes producing a shaking of MMI-IX or more occur approximately every 75 years, while smaller earthquakes occur more frequently (see Table 1) (BCDP, 1994). Past records have shown that Nepal can expect two major earthquakes of magnitude 7.5-8 every 40 years. Thus, there is cause for great concern that the next great earthquake may occur at any time, after around 70 years of silence.

Over the last few decades, RC building construction has rapidly increased, replacing other construction materials, like adobe, stone and brick masonry, in the Kathmandu Valley as well as in other parts of the country (JICA 2002). Most RC buildings in Nepal were constructed with light reinforced frames with infill masonry panels. These buildings offered insufficient capacity, lacked ductile detailing and were poorly constructed and may have limited durability (UNDP/Nepal 1994). A past study of the seismic vulnerability of Nepal has also shown that more than 60% of these buildings in the Kathmandu Valley are unsafe and extremely vulnerable to any large impending earthquake (NSET 1999).

Earthquakes are thus a relatively frequent and disastrous natural event in Nepal, and a major earthquake is likely in near future. The earthquake disaster risk of urban areas in Nepal, especially the capital area of Kathmandu Valley, is ever increasing alarmingly due to rapid urbanization, poor construction practice, and lack of disaster preparedness. In this context, this study aims to evaluate the seismic response of the most common building stock in Nepal. In this context, the present paper studied the seismic behaviour of existing RC buildings in Kathmandu Valley. For this, four representative building structures with different design and construction, namely a building: (a) representing the non-engineered construction (RC1 and RC2 building models) and (b) engineered

Table 1 Magnitude-frequency data on earthquakes in Nepal and the surrounding region in the period of 1911-1991 (modified after BCDP 1994)

Earthquakes of magnitudes in Richter scale	5-6	6-7	7-7.5	7.5-8	>8
No. of events	41	17	10	2	1
Approximate recurrence interval (years)	2	5	8	40	81



(a) (b)

Fig. 2 District wise distribution of (a) Population and (b) Type of building structures in Nepal (NPHC 2011)

construction (RC3 and RC4 building models) has been selected for analysis. The dynamic properties of the case study building models are analyzed and the corresponding interaction with seismic action is studied by means of non-linear analyses.

2. Characteristics of Nepalese building structures

2.1 Building typologies in Nepal

In this study, the data obtained from National Population and Housing Census has been used for the general building inventory in Nepal (NPHC 2011). The information obtained from the National Census Report includes: type of foundation of house, type of outer wall and roof of the house. In 2011, Population of Nepal stands at 26,494,504 showing population growth rate of 1.35 per annum. Similarly, total number of individual households in the country is 5,423,297. Terai (southern part) constitutes 50.27% of the total population while Hill (middle part) and Mountain (northern part) constitutes 43% and 6.73% respectively (Fig. 2(a)). The distribution of the buildings in Nepal is also similar to the distribution of the population. The data obtained from NPHC indicates that mud bonded brick/stone buildings are more common in Nepal for all the geographical regions, occupying about 44.21% of buildings. The wooden buildings are more popular in rural area of Terai region which occupied as around 24.90%. Cement bounded brick/ stone and cement concrete with pillar buildings are highly popular in urban area in most of the Terai region, Kathmandu Valley and some district headquarter of mountainous region. These buildings occupy 17.57% and 9.94 % building stock in Nepal (see Fig. 2(b)). The rest of the buildings are classified as others and not stated building typologies. These buildings are generally

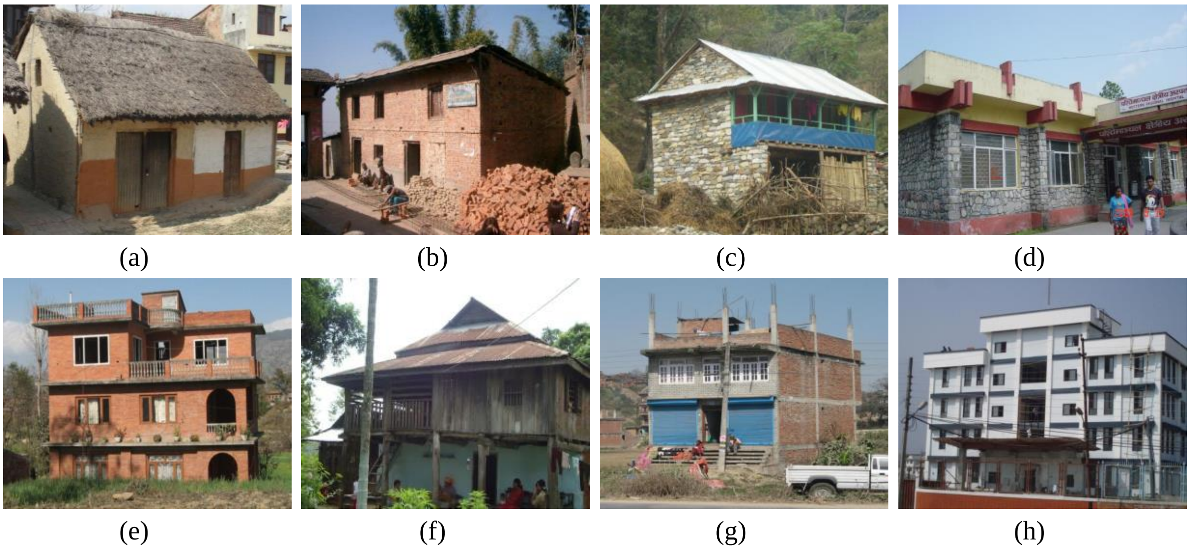


Fig. 3 Existing building typologies in Nepal: (a) Adobe, (b) Brick in mud mortart, (c) stone in mud mortar, (d) Brick in cement mortar, (e) Stone in cement mortar, (f) Wooden, (g) Non-engineered building, and (h) Engineered building

constructed with the combination two or more than two different building materials. These are the mixed buildings like stone and adobe, stone and brick in mud, brick in mud and brick in cement, wooden and brick cement mortar. The pictorial representation of each building typologies in Nepal is presented in Fig. 3. The briefly description of each building typology is discussed in the following sub-sections.

2.1.1 Adobe Buildings (A)

Adobe buildings are more popular in rural community in Nepal. Due to the poor economic condition, peoples built their house using natural building materials which is made from sand, clay, water and some kind of organic materials (sticks, straw, and/or manure). The wooden frames are usually for proper shape. These buildings are also constructed in sun-dried bricks (earthen) with mud mortar for the construction of structural walls. The wall thickness is usually more than 350 mm.

2.1.2 Brick/stone in mud mortar buildings (BM/SM)

These are the low strength masonry buildings. The brick in mud mortar buildings are made of fired bricks in mud mortar where as stone in mud mortar buildings are constructed using dressed or undressed stones with mud mortar. These types of buildings generally have flexible floors and roof.

2.1.3 Brick/stone in cement mortar (BC/SC)

In the advancement of the cement in Nepal, brick/ stone buildings with mud mortar is replaced by the cement mortar. The brick in cement buildings are constructed with fired bricks in cement or lime mortar. For stone in cement mortar buildings, dressed or undressed stones are used with cement mortar.

2.1.4 Wooden buildings (W)

These buildings are more popular near the forest area in Nepal (mostly in Terai region). In these buildings, tree trunks are used for wooden pillar where as a dressed piece of wood is usually used for columns. The walls of these buildings are constructed with wooden planks or bamboo net cement/mud mortar plaster.

2.1.5 Reinforced concrete buildings (RC)

The RC buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs. Lateral force resisted by concrete moment frames that develop their stiffness through monolithic beam column connections.

2.2 Classification of RC buildings in Nepal

Reinforced concrete (RC) building construction in Nepal has begun from late 1970s. In the last 3-4 decades, RC building construction rapidly increased, replacing other construction materials and solutions like adobe, stone and brick masonry in Kathmandu Valley as well as in other parts of the country. The RC buildings consist of a frame assembly of cast-in-place concrete beams and columns. Floor and roof framing consists of cast-in-place concrete slabs. Lateral force resisted by concrete moment frames that develop their stiffness through monolithic beam column connections. The four variation of the typical moment resistant frame in Nepal are presented as: (i) the first type corresponding to moment resisting frame design represent the current construction practices in Nepal (called CCP structure); (ii) the second design type is based on Nepal building code based on Mandatory Rules of Thumb (called NBC design structure); (iii) the third type of structure is the modified version of the Nepal building code (called as NBC+ structure) and the last type of RC frame represent the moment resisting frames which is designed based on Indian standard code with seismic provisions, namely seismic design with ductile detailing (called Well Designed Structure, WDS). Due to lack of adequate provisions for seismic design on RC building structures in Nepal Building Code (NBC), well designed structure (WDS) was designed by Indian standard codes. Most of the CCP buildings were based on non-engineered construction where as remaining building types are engineered buildings. Engineered buildings are designed and supervised by the engineers. These buildings are designed on the basis of some standard guidelines. Some of the newly constructed reinforced concrete buildings in Nepal are of this type. Where as, non-engineered buildings are not structurally designed and supervised by engineer during construction. This category also includes the buildings that have architectural drawings prepared by engineers. In the following sections, the particular characteristics of each building are described.

2.2.1 Current construction practices (CCP)

These are buildings with reinforced concrete frames and unreinforced brick masonry infill in cement mortar. The thickness of the infill walls is 230 mm or 115 mm and the column size is predominantly 230 mm×230 mm. The prevalent practice in most urban areas of Nepal for the construction of residential and commercial complexes generally falls under this category. These buildings are not structurally designed and their construction is not supervised by engineers. This category also includes buildings that have architectural drawings prepared by engineers.

2.2.2 Nepal building code (NBC)

The NBC structure is designed with the Mandatory Rules of Thumb (MRT) (NBC 201 1994).

MRT provides some ready-to-use provisions in terms of dimensions and details for structural and non-structural elements for up to three storeys with room sizes of no more than 4.5 m×3.0 m in RC framed, ordinary residential buildings commonly built by owner-builders in Nepal (NBC 205 1994). In 2003 this document became mandatory in Nepal. Thus, the NBC structure was designed according to these simplified rules.

2.2.3 Modified Nepal building code (NBC+)

In 2010, the Department of Urban Development and Building Construction published additional recommendations for the construction of Earthquake Safer Buildings in Nepal with the assistance of UNDP (UNDP 2010). This document is an improvement on the NBC, and specifies that the minimum sizes of columns for up to three storeys with room sizes of no more than 4.5 m×3.0 m should be 300 mm×300 mm or 75 mm more than the width of the beam. There should be a minimum of 4 and 8 nos. of 16 mm dia. reinforcement bars in columns located in the outer faces and centre of the building structure. The detailing of the beam is the same as specified in the NBC document.

2.2.4 Well designed structure (WDS)

The WDS building structure was designed based on the Indian code, considering seismic design with ductile detailing to the building located in seismic zone V and medium soil. Due to the low height, and regular plan and elevation, seismic analysis is performed using the seismic coefficient method (IS 1893 2002). The effect of the finite size of joint widths (e.g., rigid offsets at member ends) is not considered in the analysis. However, the effect of shear deformation is considered. The detailed design of the beams and column sections according to IS 13920 (1993) recommendations have been carried out.

3. Statistical analyses of RC buildings in Nepal

In this section, general overview of existing Nepalese RC building is presented. For this, the detailed information has been collected from previous studies, private practitioners, design offices, public institutions, and a field survey in different localities of Nepal (Chaulagain *et al.* 2010, Chaulagain *et al.* 2012, JICA 2002, NSET 1999). The statistical information includes: number of storey, age of building, size and detailing of RC elements (beams and columns), inter-storey height, numbers of bays and dimensions, years of construction, quality of concrete and plinth area of the building. The random sampling of 300 drawings and design specifications from different district headquarter is collected. From the 300 drawings, only 200 were used for the statistical analysis. In fact, the National Census data only have the limited information namely construction type, building use, types of foundations, types of walls and types of roofing. The distribution of RC buildings in Nepal is presented in Fig. 4a. The number of sampling data and the corresponding location is presented in Fig. 4(b). In this study, nearly 50% of the surveyed data was taken from Kathmandu Valley. It is mainly due to the fact that number of RC buildings in Kathmandu Valley is nearly equal to the remaining country.

3.1 General statistical analysis

Over the past half century, building construction trends and practices has been extremely

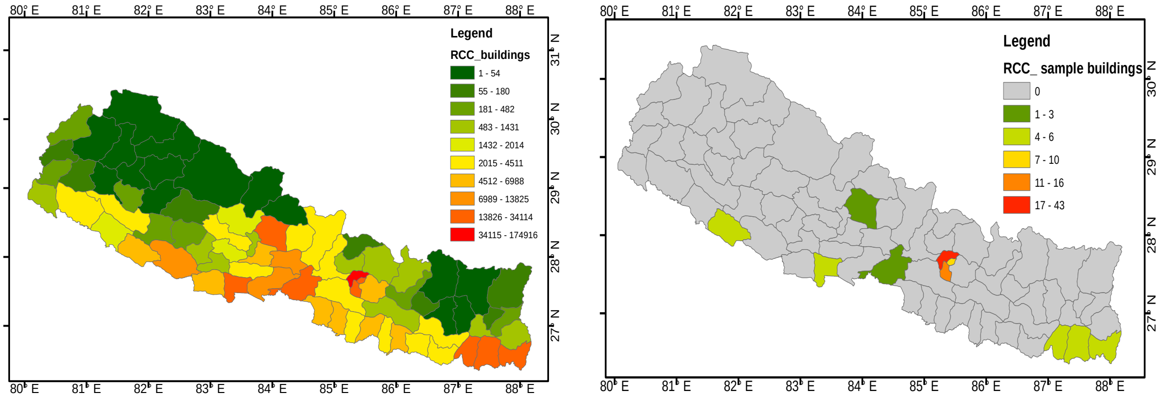


Fig. 4 (a) District wise distribution of RC buildings and, (b) Sampling location for studied building structures

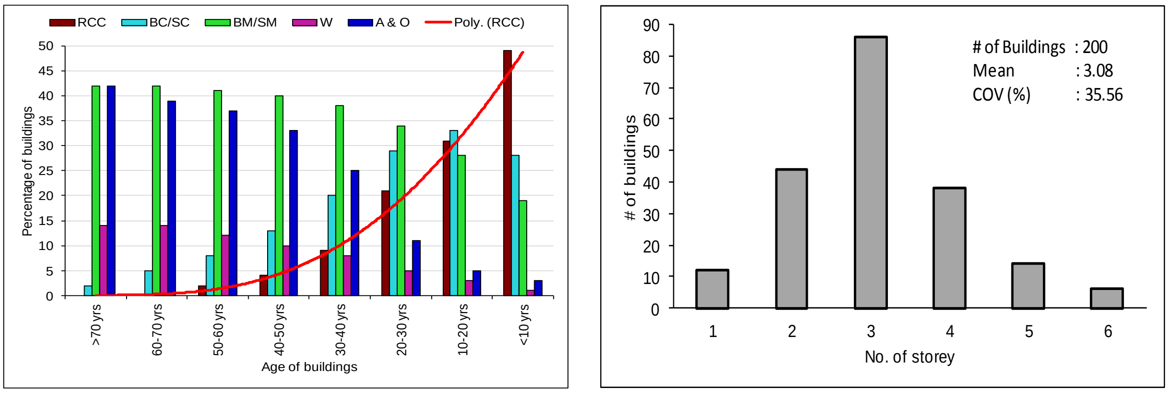


Fig. 5 Distribution of (a) age of RC buildings and (b) number of storey of RC buildings

changed. Since the last decade or so, RC framed structures has become highly popular replacing other construction materials like adobe, brick/stone with mud, and brick/stone with cement mortar buildings (see Fig. 5(a)). The current construction practices of the buildings in the urban areas of Nepal use light reinforced concrete frames with infill walls. Most of the residential buildings are 1 to 6 stories; the majority of them are of three storeys (see Fig. 5(b)). There is an increase in the prevalence of frame-structures nowadays, but unfortunately, many of them are constructed without the input from qualified engineers, making them potentially highly vulnerable to earthquakes (see Fig. 6(a)). The overwhelming majority of such buildings are of by ‘owner-builder’, construction of buildings with informal building process (see Fig. 6(b)). The “owner-builder” makes his own decisions, supported by advice from friends, neighbors, well-wishers and infrequently by professionals and small builders on personal basis.

3.2 Detailed statistical evaluation of the structure

The probabilistic distributions are defined as the most representative of both normal, log-normal, gauss and exponential distributions. All the parameters have been examined in terms of the number of data, number of buildings, mean values, coefficient of variation, the best fit-

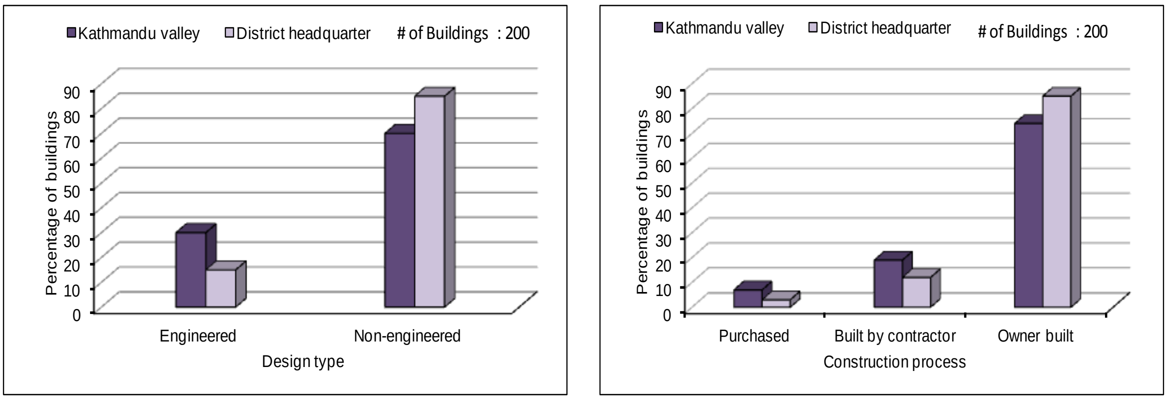


Fig. 6 Distribution of building (a) construction process (b) building design process

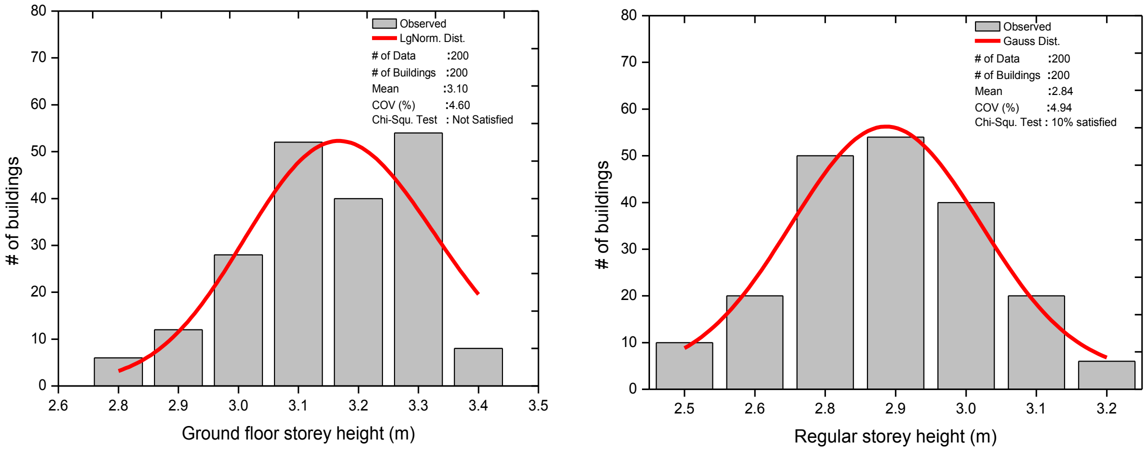


Fig. 7 Distribution of (a) ground floor (b) regular storey height for RC buildings

distribution type, goodness-of-fit test (chi-square test) results and lower and upper bound of the data used to calculate the distribution. The X^2 is a function of the difference between the observed and expected frequencies, should be less than one of the X^2 percent point function for significance levels of 10%, 5% or 1%.

3.2.1 Storey properties

Storey properties have been defined as height and area based. In this study, inter-storey height is analyzed considering the ground floor heights and regular storey height in order to represent the frequency of occurrence of soft-storey. The floor area of sample building structures also has been investigated. The histogram showing the frequency of different values of floor area in different RC building structure is presented in Fig. 9(b). The evaluation of 200 sample building has lead to a mean 94.75 m^2 with variation of 36.30%. The suggested distribution is a log-normal distribution between 50 and 200 m^2 with a 10% satisfaction of the X^2 test.

The storey height has been investigated in terms of ground floor and regular-storey height. The distribution of frequency of each parameter is presented in Fig. 7(a). Regarding the statistical evaluation of 200 sample buildings in the data set, the ground floor height distribution is found to

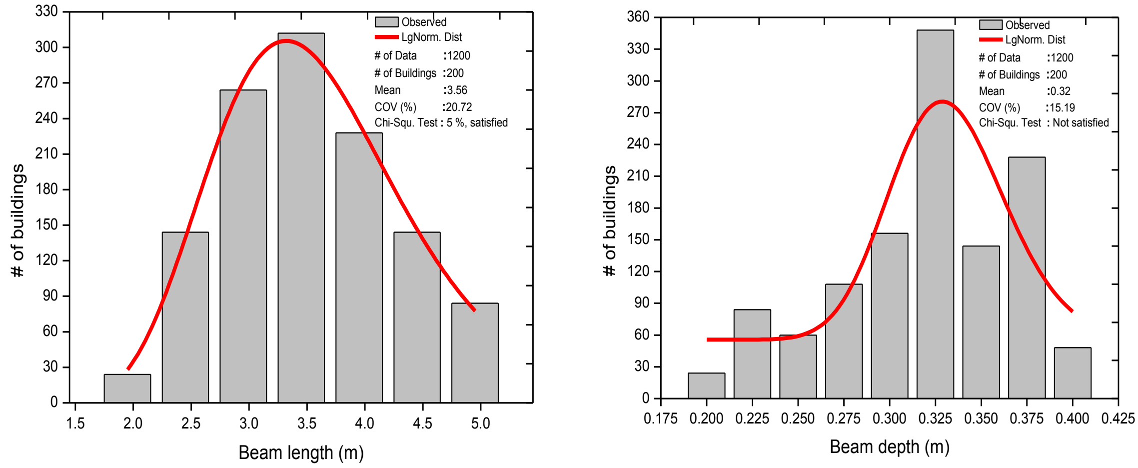


Fig. 8 Distribution of (a) beam length and (b) beam depth of RC buildings

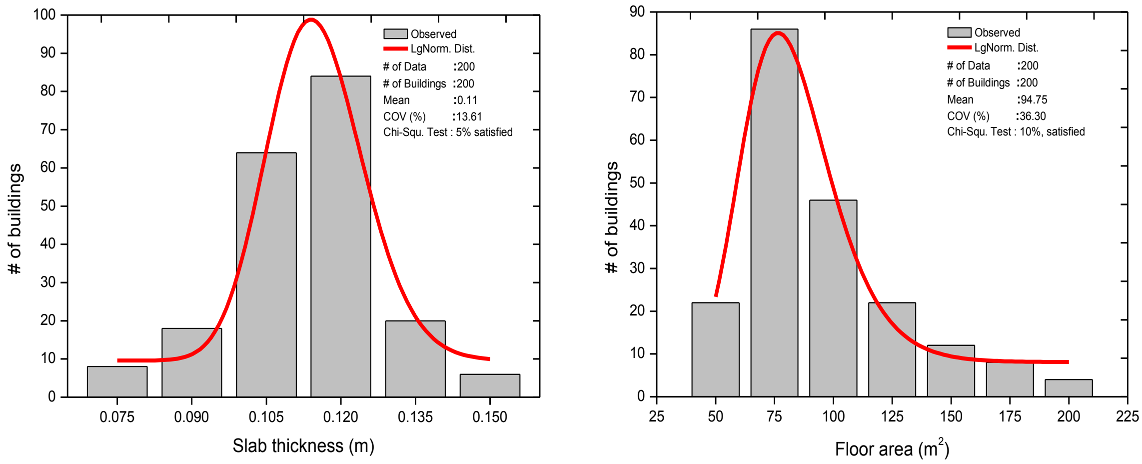


Fig. 9 Distribution of (a) slab thickness and (b) floor area for RC buildings

have a log-normal distribution with a mean value of 3.10 m and a coefficient of variation of 8%. The distribution should ideally apply between 2.60 and 3.50 m. However, the X^2 test which is applied to investigate the goodness-of-fit could not satisfied for any of the satisfaction levels considered herein (i.e., 10%, 5%, and 1%). In contrast, regular storey height is found to have mean and coefficient of variance is limited to 2.84 m and 4.94% respectively. The suggested distribution is a gauss distribution between 2.5 and 3.2 m with a 10% satisfaction of the X^2 test (see Fig. 7(b)).

3.2.2 Structural elements

The structural parameters of RC buildings which have been studied herein include width and depth of column, beam length, beam depth and slab thickness. For the statistical analysis of column section, the smaller dimension is considered as width. The column section has been defined by considering the frame with main structural resistance in each principal direction of the buildings. The beam has also been investigated in terms of their length and depth values. Herein, 1200 beams have been studied from 200 different buildings and the beam length distribution has been found to be a log-normal distribution with a mean length of 3.56 m and a coefficient of variation of 20.72%. The X^2 test has been satisfied with a 5% satisfaction (see Fig. 8(a)). For beam depth, the distribution has been found to be a log-normal distribution with a mean depth of 0.32 m and a coefficient of variance of 15.19% (see Fig. 8(b)). The mean and coefficient of variation of slab thickness of Nepalese RC building is 0.11 m and 13.61% respectively. The suggested distribution is log-normal, with upper and lower bound data of 7.5 and 15 cm respectively and the result of the X^2 test is 5% satisfaction (see Fig. 9(a)).

4. Description of the study building structures

As described in the aforementioned section 3, authors have collected the detailed building information from previous studies, private practitioners, design offices, public institutions, and a field survey in different localities in Kathmandu Valley. The information collected during field surveys includes plinth area, size and detailing of RC elements (beams and columns), inter-storey height, number of bays and span lengths, structures' age, quality of concrete, and type of steel.

Based on the results from the statistical analysis of reinforced concrete building structures as discussed in section 3, four existing reinforced concrete buildings in different localities in Kathmandu Valley is selected for case study. The reinforced concrete buildings in Nepal can be divided in to two groups namely a) non-engineered and b) engineered. Also considering this fact, two buildings from each category is selected in the present study. These entire building configurations are typical of seismically active regions like Kathmandu Valley, where the vast majority of dwellings are RC buildings, with similar characteristics (Chaulagain *et al.* 2013). The first type of study buildings are representative of non-engineered construction, namely: (a) RC1 and (b) RC2, and second type of buildings are based on engineered RC-MRF constructions, denoted as: (c) RC3 and (d) RC4.

The first two buildings are non-engineered RC-MRF structure with square (RC1) and rectangular plan configuration (RC2), built in southern part of Kathmandu Valley. All these types of buildings have 3 m inter storey height in all story's. RC1 model has 4 rooms per story where as the room number is limited to 6 in RC2 model. RC1 building model having 9 m×9 m with moment resisting system. In first and second storey, the dimensions of the sections of all the columns are 23×30 cm^2, of all the beams are 23×35 cm^2 and at the top storey such dimensions are respectively 23×23 cm^2 and 23×35 cm^2. Likewise, in RC2 building model, the lateral load resisting elements in *X*- direction consist of three moment resisting frame, and in *Y*-direction the frame are four. The building dimension in plan is 10.5 m×8 m.

Similarly, remaining two case study buildings are engineered RC-MRF structure with regular (RC3) and irregular plan configuration (RC4), recently constructed in the northern part of the Kathmandu Valley was considered. Inter storey height of these buildings are 2.85 m in all storey. Building model RC3 having plan area 9.6 m×7.9 m (75.84 m^2), measured from the column centre lines. Four identical moments resisting frames in *X* and *Y*-directions acts lateral load resisting elements. Building model RC4 has trapezoidal plan area measuring 70.8 m^2 which has three and four moment resisting frame in *X* and *Y* direction respectively. Plan, tridimensional model, and cross sectional detailing of the entire building models has been summarized in Figs. 10-13.

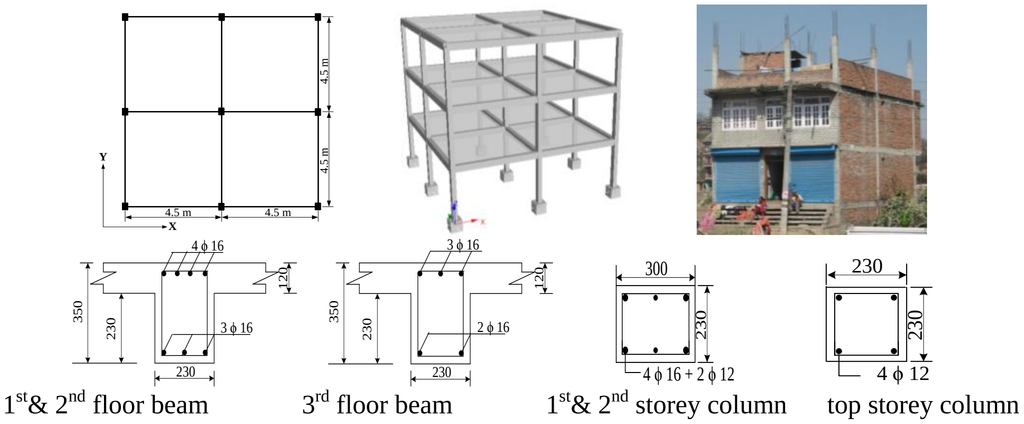


Fig. 10 Plan, tridimensional model, and cross-section detailing of building model RC1

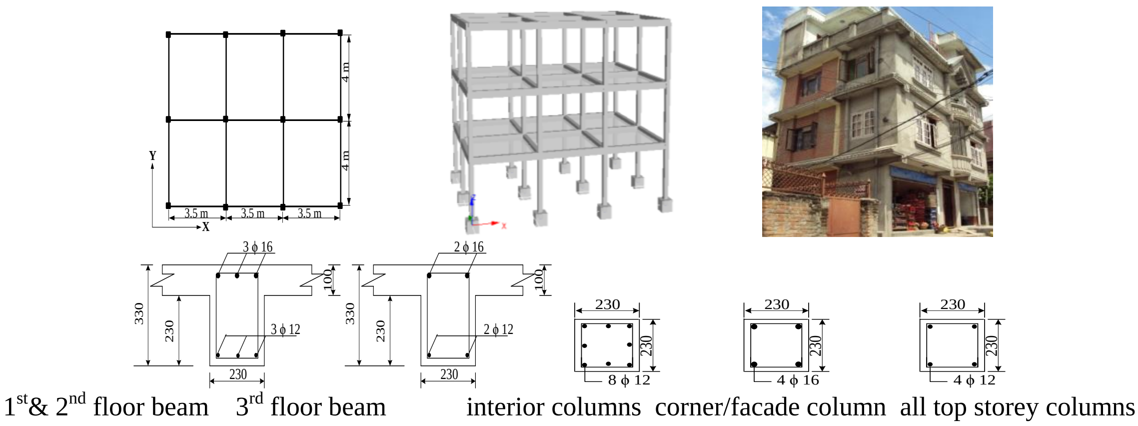


Fig. 11 Plan, tridimensional model, and cross-section detailing of building model RC2

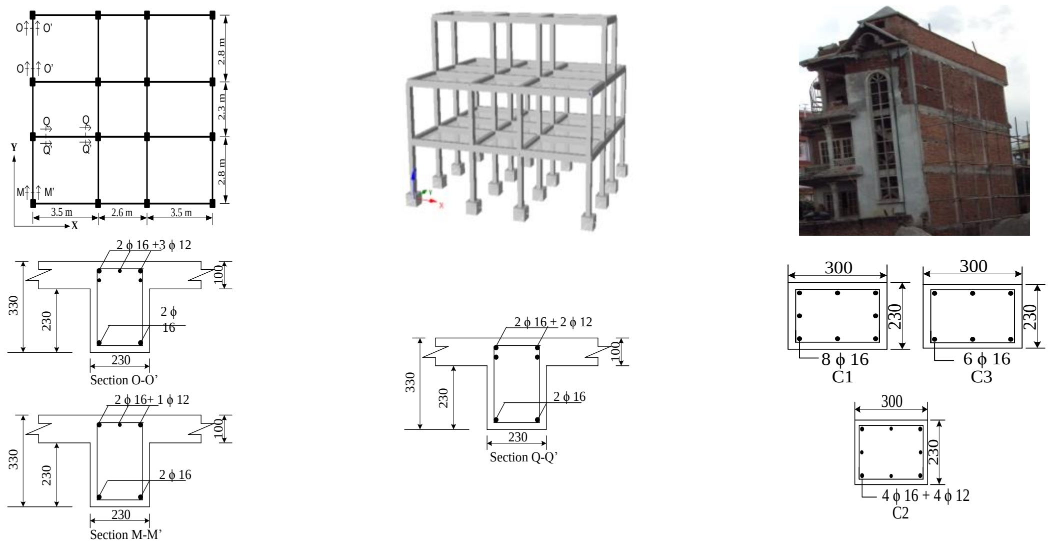


Fig. 12 Plan, tridimensional model, and cross-section detailing of building model RC3

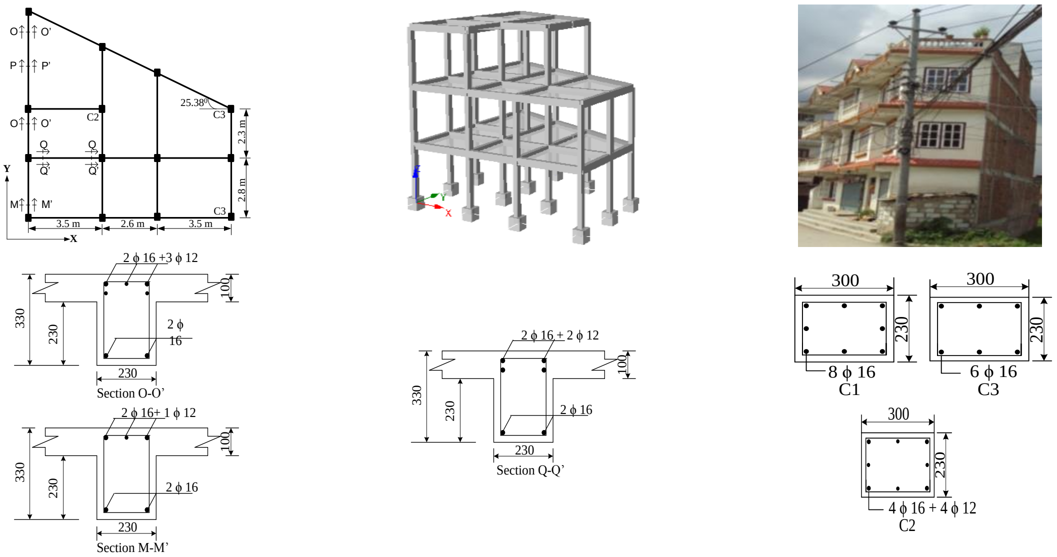


Fig. 13 Plan, tridimensional model, and cross-section detailing of building model RC4

Note: All dimensions are in mm unless stated otherwise

: All the interior, interior, façade and corner columns of building models B1 and B2 are C1, C2, and C3 respectively unless stated otherwise.

Table 2 Properties of materials used in this research

Materials	Characteristics
Reinforcing steel yield strength, f_y	415 MPa
Concrete compressive strength, f'_c	20 MPa
Brick on peripheral beams	230 mm thick
Brick wall on internal beams	115 mm thick
Density of brick masonry including plaster	20 kN/m^3
Density of reinforced concrete	25 kN/m^3

Table 3 Loading for numerical analysis of structure

Loading characteristics	Loading
Live load on roof	1.5 kN/m^2
Live load on floors	2 kN/m^2
Roof and floor finishing	1 kN/m^2

5. Non-linear building modelling

In order to assess the seismic capacity of the four case study building structures presented, numerical simulation have been performed through adaptive pushover and non-linear dynamic analysis. It provides the most accurate method for evaluating the inelastic seismic response of structures.

The computer program SeismoStruct (2006) was used to produce a lumped plasticity model. A three-dimensional model of each structure was created to undertake the non-linear analysis. In the analyses performed in this paper, half of the larger dimension of the cross-section was considered as the plastic hinge length, with fibre discretization at the section level. The consideration of non-linear material behaviour in the prediction of the RC columns' response requires accurate modelling of the uniaxial material stress-strain cyclic response.

Concrete modelling is based on the Madas uniaxial model (Mandas *et al*. 1992), which follows the constitutive law proposed by Mander *et al*. (1988). The cyclic rules included in the model for the confined and unconfined concrete were proposed by Martinez-Rueda (1997), Elnashai (1993). The confinement effects provided by the transverse reinforcement were considered through the rules proposed by Mander *et al*. (1988), whereby constant confining pressure is assumed throughout the entire stress-strain range, traduced by the increase in the peak value of the compression strength and the stiffness of the unloading branch.

The uniaxial model proposed by Menegotto and Pinto (1973), coupled with the isotropic hardening rules proposed by Filippou *et al.* (1983), was adopted for the steel reinforcement representation in these analyses. This steel model does not represent the yielding plateau characteristic of the mild steel virgin curve. The model takes into account the Bauschinger effect, which is relevant for the representation of the columns' stiffness degradation under cyclic loading. The effect of confinement due to shear reinforcement in the analysis is considered for both engineered and non-engineered buildings.The model adopted in the analyses performed in this study is represented in Figs. 10-13.

Many codes and guidelines (e.g., Eurocode8 2005, ATC-40 1996, FEMA-356 2000) recommend the use of nonlinear static methodologies to evaluate structural behavior under seismic action. In order to improve the efficiency of pushover analysis, different nonlinear static procedures have been proposed in the literature. In conventional procedures, the shape of the load distribution is constant during the analysis. Such techniques are not able to take into account progressive structural stiffness degradation, change of modal characteristics and period elongation of the structure for increasing values of external action. These drawbacks spurred the recent proposal of the so-called Adaptive Pushover methods (e.g., Reinhorn 1997, Bracci *et al.* 1997, Gupta and Kunnath 2000). Adaptive pushover is employed in the estimation of the horizontal capacity of a structure, taking full account of the effect that the deformation of the structure and the frequency content of input motion have on its dynamic response characteristic (Antoniou and Pinhoh 2006). The lateral load distribution is not kept constant but rather continuously updated during the analysis, according to the modal shapes and participation factors divided by eigen-values analysis carried out at each analysis step (Ghobaraha *et al.* 2006, Kazem *et al.* 2012). The results from adaptive pushover are close to the ones obtained with dynamic time history analysis.

In the present study, nonlinear analysis of the building structures is performed with adaptive pushover and dynamic time history analysis. For adaptive pushover analysis, response spectrum provided in Indian seismic code is used (IS 1893 (part 1): 2002, 2002). It is due to the fact that, Nepal building code does not possess sufficient data required for standard design consideration. Currently, most of the engineered buildings in Nepal have been designed based on Indian seismic code. Earthquake ground motion histories are important for dynamic analyses of the structures. Though, many earthquakes have been reported in the history of Nepal, no accelerations have been recorded. Due to the lack of actual time history data in Nepal, dynamic time history analysis was performed with synthetic time history data. For this, three different artificially generated time history records in Nepal with increasing peak ground acceleration (PGA) values ranges from 0.07g to 0.51g has been used (Parajuli 2009). During inelastic time history analyses, the scaling of time

Table 4 Seismic risk scenarios for various return periods (Parajuli 2009)

Return period (years)	Peak ground acceleration (m/s^2)
98	0.07g
475	0.40g
975	0.51g

Table 5 Natural frequencies (hz) of structures

Mode	Natural frequency/directions			
	RC1	RC2	RC3	RC4
1st mode	0.99(X)	1.02(X)	1.59(X)	1.45(X)
2nd mode	1.15(Y)	1.05(Y)	1.98(Y)	1.79(Y)
3nd mode	2.62(Θ)	1.11(Θ)	2.02(Θ)	2.01(Θ)

history data has been employed for the intermediate values.

The series of three artificially generated earthquake input motion for a medium/high seismic risk scenario for various return periods are adopted for the seismic vulnerability assessment of the building in Nepal. Artificially generated PGA for various return periods in Kathmandu Valley is presented in Table 4.

6. Results and discussion

In this section the results of numerical analysis of current reinforced concrete buildings in Kathmandu Valley is discussed. The results from non-linear analyses of all the case study buildings with different response measures such as natural frequencies, capacity curves, inter-storey drift, tangent stiffness, strength, deformation, energy dissipation and the effect of geometric non-linearity (P-Delta effect), are evaluated for the two orthogonal directions. In the last section; the effect of irregularity on response of column is presented. It is achieved through the two case study building structures with irregular and regular configuration. The detail analyses and interpolation of the results are discussed in each sub-section.

6.1 Natural frequencies

The dynamic characteristics directly affect the response of the considered structures. The elastic structural frequencies from eigen-value analysis are in first three modes are tabulated in Table 5. In most of the cases, engineered structures (model RC3 and RC4) have higher frequencies than non-engineered (model RC1 and RC2) building models. From Table 5, it can be seen that the higher increment of frequencies in the structure is as a result of better structural configuration and detailing. In fact, engineered building attracts higher forces due to the increase of stiffness, which results in a reduction in the natural period of the structures.

6.2 Capacity curves and maximum inter-storey drift profile

In this section, the results are analysed in terms of capacity curves and the maximum drift

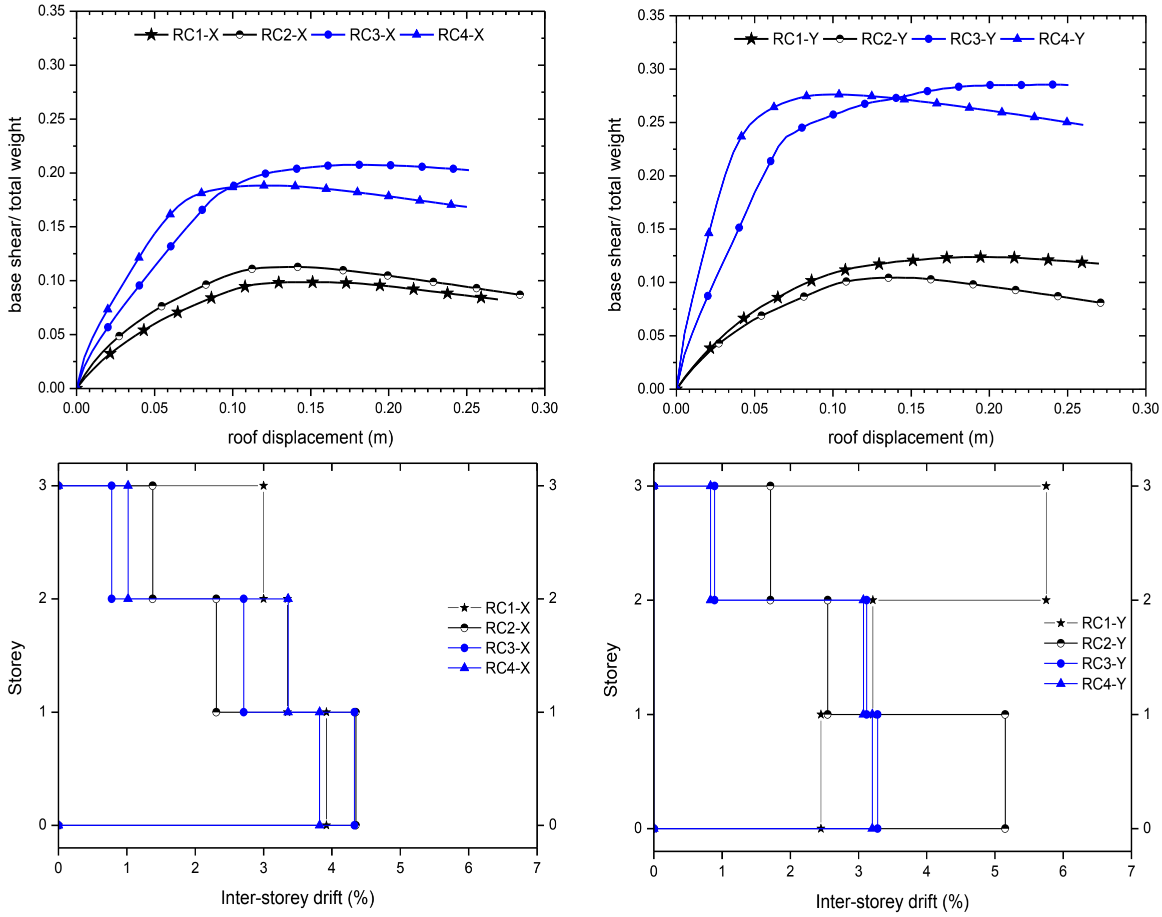


Fig. 14 Capacity curves and corresponding IS drift of NRCB1, NRCB2, NRCB3 and NRCB4 building structures with (a) longitudinal (*X*) and (b) transverse (*Y*) directions of loading

profiles for each building and the direction of analysis. Capacity curves, representing the resistance of the structure when deforming into the inelastic range, come in the form of top displacement versus base shear plot. Similarly, inter-storey drift (IS drift) is an important parameters as they are closely related to the damage that can be sustained by a loading in the recent trends of performance based engineering. Fig. 14 presents the results of the adaptive pushover analysis for each building and for each loading direction. Based on the results, the main conclusions are summarized as follows:

• The shear strength capacity and tangent stiffness of engineered buildings (RC3 and RC4) are nearly two times the value obtained with the non-engineered structures (RC1 and RC2).

• Engineered structure presents better performance in terms of strength, tangent stiffness and deformation capacity as compared with non-engineered structures. In particular RC1 building model present a soft storey mechanism in the third storey, due to the reduction of the column-section between the second and third storey, which is considered non-adequate for earthquake prone area like Kathmandu Valley.

• RC1 and RC2 structures have maximum IS drift profile, minimum shear capacity and low

Table 6 Tangent stiffness, maximum strength and corresponding deformation of the structure

Standard	Direction of loading	Tangent stiffness (kN m)	Max. strength (kN)	Roof displacement for max. strength(m)
RC1	X	4297.78	281.87	0.141
	Y	3854.43	261.37	0.141
RC2	X	3578.92	246.68	0.140
	Y	4190.55	309.57	0.199
RC3	X	6930.37	493.29	0.150
	Y	7169.05	628.95	0.260
RC4	X	9854.21	858.91	0.210
	Y	7515.82	626.85	0.175

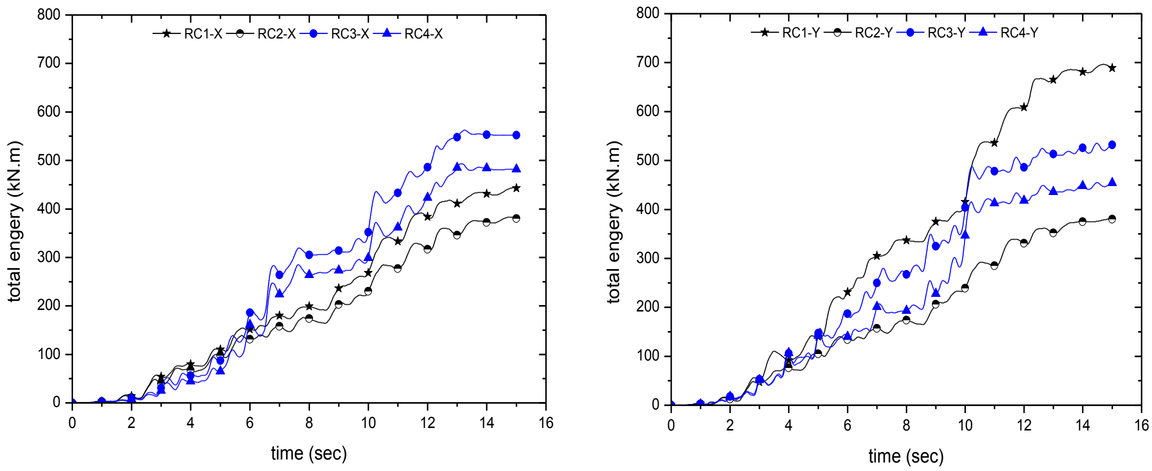


Fig. 15 Total energy dissipation profiles for existing building structures in Nepal

stiffness as compared with RC3 and RC4 structures.

• In engineered building structures, the rate of change of IS drift is quite regular and consistent in all the floor levels. While, there is highly irregular and inconsistent IS drift profiles in non-engineered structures.

6.3 Stiffness, strength and deformation of the study buildings

In order to evaluate the behaviour of the building structures under study, and for the same loading conditions, different parameters were quantified and reported in Table 6, namely the tangent stiffness, maximum strength and corresponding roof displacement. The maximum strength and tangent stiffness of the engineered buildings (RC3 and RC4) have nearly two times than that of non-engineered building structures (RC1 and RC2).

6.4 Energy dissipation

In this section, the total cumulative energy dissipation of existing RC building in Nepal is

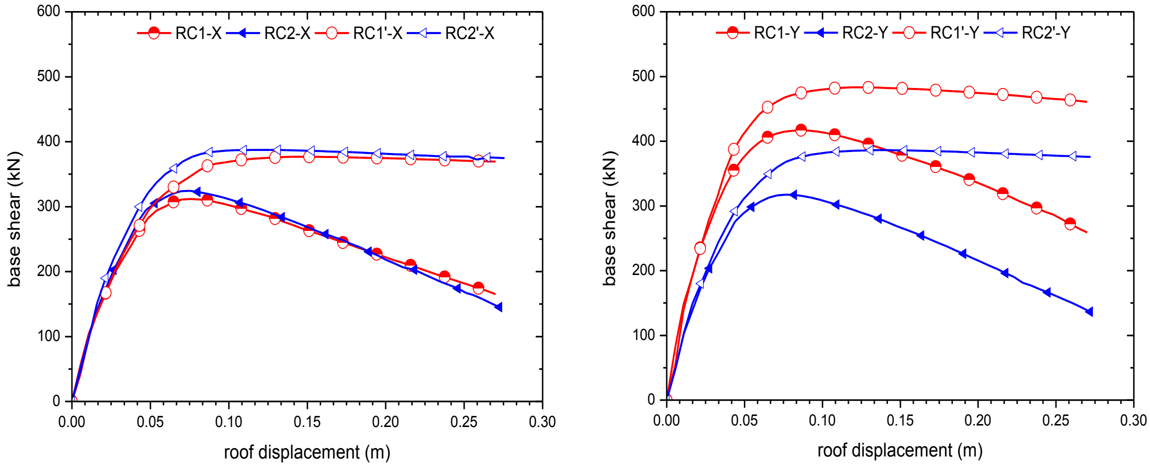


Fig. 16 The capacity curve and corresponding IS drift of the studied building structures with and without considering the *P*-Delta effect for longitudinal (*X*) and transverse (*Y*) directions of loading

discussed. In most of the loading conditions, the evolution of energy dissipation of existing non-engineered structures has lower range compared to engineered one. In fact, for proper seismic behaviour of structure, the input energy to the structure due to earthquake needs to be dissipated, depending on the expected performance of the structure. However, the area enclosed in hysteretic loops of non-engineered structure is smaller than that of engineered one. Furthermore, the results from the numerical analyses also show that engineered building structures have good energy dissipation potential in addition to increased stiffness and strength of the structures. Fig. 15 plots the evolution of the total cumulative energy dissipation (TCED) in the existing building structures.

6.5 P-Delta effect

The *P*-Delta effect, also known as geometric non-linearity, involves the equilibrium and compatibility relationships of a structural system loaded about its deflected configuration. The P-Delta effect and its influence on structural response has been the subject of significant research in recent decades. Researchers have studied the global *P*-delta effect on the performance of structures analytically, numerically, and experimentally (Bernal 1997, Bernal 1998, Macrae 1994, Vian *et al.* 2003).

The comparison of the results of two analyses with and without *P*-Delta will illustrate the magnitude of the *P*-Delta effects. An engineered building usually has well-conditioned level with higher stiffness/weight ratios. For such structures, *P*-Delta effects are usually not very significant. The changes in displacements and member forces are less. However, if the weight of the structure is high in proportion to the lateral stiffness of the structure, the contributions from the P-Delta effects are highly amplified and, under certain circumstances, can change the displacements and member forces by 20 percent or more. Excessive *P*-Delta effects will eventually introduce singularities into the solution, indicating physical structure instability. Such behavior is clearly indicative of a poorly designed structure that is in need of additional stiffness. In the present study, an analysis of four RC building was conducted with and without *P*-Delta effects. Figs. 16 and 17 show the global pushover curves of the case study buildings, representing the response of

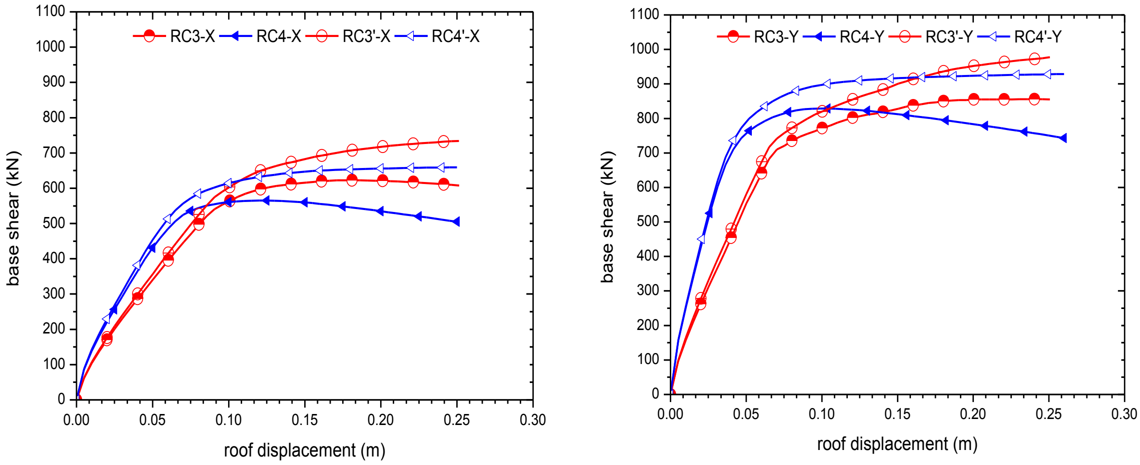


Fig. 17 The capacity curve and corresponding IS drift of the studied building structures with and without considering the *P*-Delta effect for longitudinal (*X*) and transverse (*Y*) directions of loading

structures with and without considering the P-delta effect. The capacity curve indicates that the analysis results without considering the *P*-Delta effect have improved shear strength capacity. The increment is higher in non-engineered structures (RC1 and RC2). The nature of the capacity curve shows the strong impact of the *P*-delta effect, leading to a reduction of the global lateral stiffness and reducing the strength of the structure.

6.6 Vulnerability assessment of the structures

The vulnerability condition is directly related to the accepted performance of the structure. Different documents promote the same concepts but differ in detail and specify different performance levels (SEAOC 1995). In ATC 40 (1996) and FEMA-273 (1996), four limit states are defined based on global behavior (inter-story drift) as well as element deformation (plastic hinge rotation). Rossetto and Elnashai (2003) used five limit states for derivation of vulnerability curves based on observational data while Chryssanthopoulos *et al.* (2000) used only two limit states. In the latter studies, the global limit states are independent of the specific response of the structure.

The selection of the appropriate drift associated with different levels of damage for the design is significant in terms of economy safety of the structures. The identification of drift levels associated with different states of damage remains one of the unsolved issues in the development of performance objectives. However, it is accepted that drift levels associated with specific damage categories may vary considerably with the structural system and construction materials. For rigorous analysis, it is necessary to define limit states for each individual structure. However, more research is needed, particularly in the development of realistic and quantitative estimates of drift-damage relationships. It is due to the fact that performance levels are associated with earthquake hazard and design levels. For a precise analysis, it is necessary to define limit states levels for each individual structure because displacement capacity maybe affected by different factors such as level of gravity force, local strains, and intended plastic hinge mechanism.

In this study, authors have proposed the limit states value for RC building structures in Nepal. Four limit states are defined which are termed as slight damage (fully operational), moderate damage (operational), extensive damage (life safety) and collapse. In this study, the local damage of individual structural element, such as beam, column, or beam–column joint, is not accounted for. Instead, the limit states are defined in terms of simple global parameters. Only inter-story drift is used as a global measure of damage.

For the estimation of damage level of buildings, an adaptive pushover curve was derived for each bare frame structures. For each damage state of criteria capacity curve, inter-storey drift, and global drift of each prototype building structures was plotted. For this, the structure with different design and construction practices in Nepal was used (Chaulagain *et al.* 2013). The criteria for drift limits were categories as:

• Slight damage: the global drift when 50% of the maximum base shear capacity is achieved

• Moderate damage: global drift when 75% of the maximum base shear capacity is achieved

• Extensive damage: global drift when the maximum base shear capacity is achieved

• Collapse: global drift when the base shear capacity decreases by 20% or 75% of the ultimate global drift taken from the pushover curve, whichever is achieved first.

In this study, four drift limits which are termed as slight damage, moderate damage, extensive damage, and collapse prevention are considered for the vulnerability assessment of the building structures. The seismic vulnerability of the buildings was assessed with and without considering the P-Delta effect. Results from non-linear dynamic analysis for each direction of loading were compared in terms of the maximum drift demands and the basic performance objectives proposed in Table 7. The similar thresholds for the global drift limits have been used by various authors (Papaila 2011, Silva 2013, Bilgin 2013). The results of FEMA-356 (2000), Ghobarah (2004) and proposed drift limits are presented in Table 7. The values in Table indicates the maximum drift values for various performance levels, slight damage, moderate damage, extensive damage and near collapse for non-engineered and engineered buildings are 0.30, 0.70, 1.50, 2.50, and 0.50, 1.0, 2.15 and 3.50 respectively. The basic performance objectives proposed by FEMA-356 is presented in Table 8. All the building structures have been studied through dynamic time history analysis with Nepalese ground acceleration value with increasing intensity (see Table 4). Due to the lack of sufficient time history data, the intermediate time history data has been employed with scaling the existing time history data. The seismic vulnerability curves of all the case study buildings plotted with the maximum inter-storey drift corresponding to peak ground acceleration. The vulnerability curves for non-engineered (RC1 and RC2 building models) and engineered (RC3 and RC4 building models) building structures in Nepal has been presented in Figs. 18 and 19.

The structural characteristic of the buildings varied to represent a large class of contemporary RC buildings in Nepal. Comparing the maximum storey drift demands with the limit states, it is observed that RC1 and RC2 building structures have higher drift demand. However, the limiting drift is only 2.5% for non-engineered and 3.5% for engineered buildings for the 'near collapse' performance level. In fact, non-engineered structures have drift value higher than the standard one. From figures, it can be seen that:

• The existing non-engineered buildings exhibit high vulnerability, i.e. the buildings have very low lateral resistant and limited ductility. The non-engineered building structures only satisfied the ‘operational ‘performance level at design intensity.

• The engineered buildings have the better performance. According with the obtained results, these buildings are safe for the aforementioned performance criteria/level. These are the similar results obtained in the Algiers buildings. In Algiers, the structural behaviour of the buildings reflects the construction phase. Buildings designed with pre-code (very poor structural behavior before 1955), buildings designed with low code (poor structural behavior, between 1955-1981),

Table 7 Performance levels and corresponding maximum drift limits

Performance Level	FEMA-356	Ghobarah (2004)		Proposed drift limits	
	RC buildings	Non-ductile MRF	Ductile MRF	Non-engineered buildings	Engineered buildings
Slight damage (fully operational)	0.20	0.20	0.40	0.30	0.50
Moderate damage (operational)	0.50	<0.50	<1.0	0.70	1.0
Extensive damage (life safety)	1.50	0.80	1.80	1.50	2.15
Near collapse	2.50	>1.0	>3.0	2.50	3.50

Table 8 Basic performance objectives for buildings according to FEMA-356, 2000

		Fully operational	Operational	Life safety	Near collapse
Earthquake Design level	Frequent (43-YRP)				
	Occasional (98-YRP)		X		
	Rare (475- YRP)			X	
	Very rare (975 YRP)				X

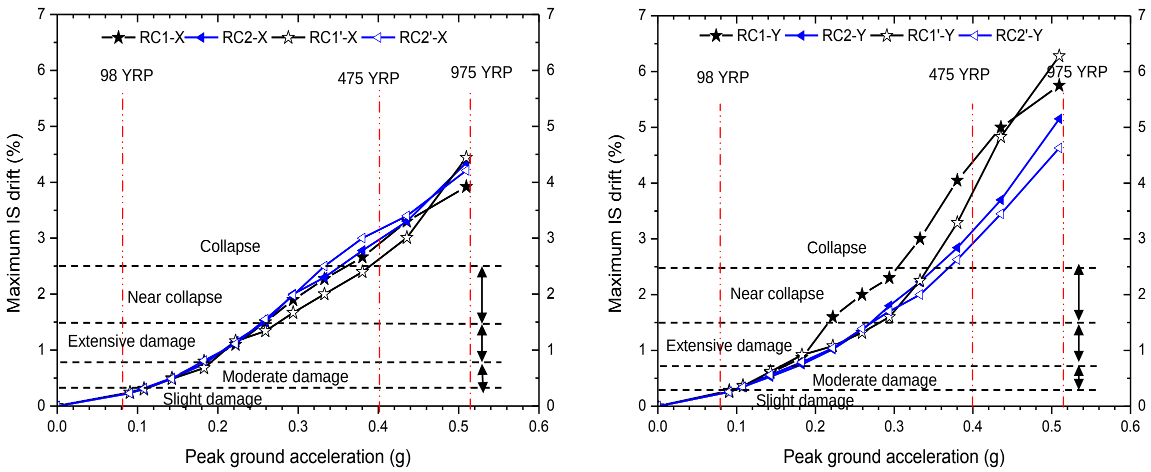


Fig. 18 Vulnerability curves of the maximum IS drift for RC1 and RC2 structures with and without *P*-delta effect for longitudinal (*X*) and transverse (*Y*) directions of loading

buildings designed with medium code (moderate structural behavior, between 1981-1999), and buildings designed with high code (good structural behavior, after 1999) (Mehani *et al.* 2013). In fact, the performance of building structure mainly depends on material properties, concrete strength and steel yield stress (Maria *et al.* 2011). Moreover, the effect of geometrical non-linearity of the structure is clearly seen in the vulnerability curve. In figures it can be also seen that the vulnerability curves without P-Delta effect have the lower range in all the analyses models. In fact, the *P*-Delta effect changes the deflected shape, which amplified the storey drift of the structures.

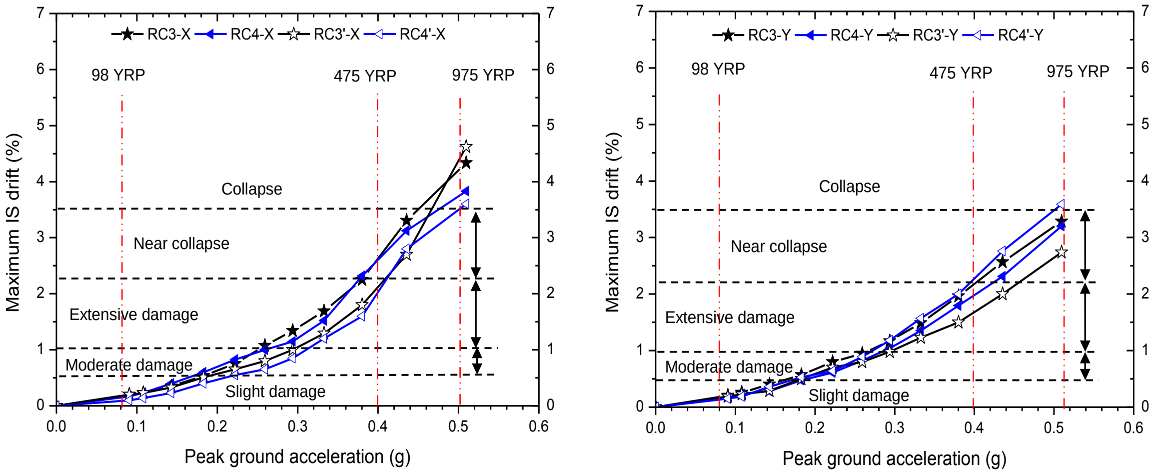


Fig. 19 Vulnerability curves of the maximum IS drift for RC3 and RC4 structures with and without P-delta effect for longitudinal (X) and transverse (Y) directions of loading

Note: RC1, RC2, RC3 and RC4, and RC1', RC2', RC3' and RC4' represent the vulnerability curves of the case study buildings with and without considering the P-Δ effect respectively.

6.7 Effect of irregularity on response of structure

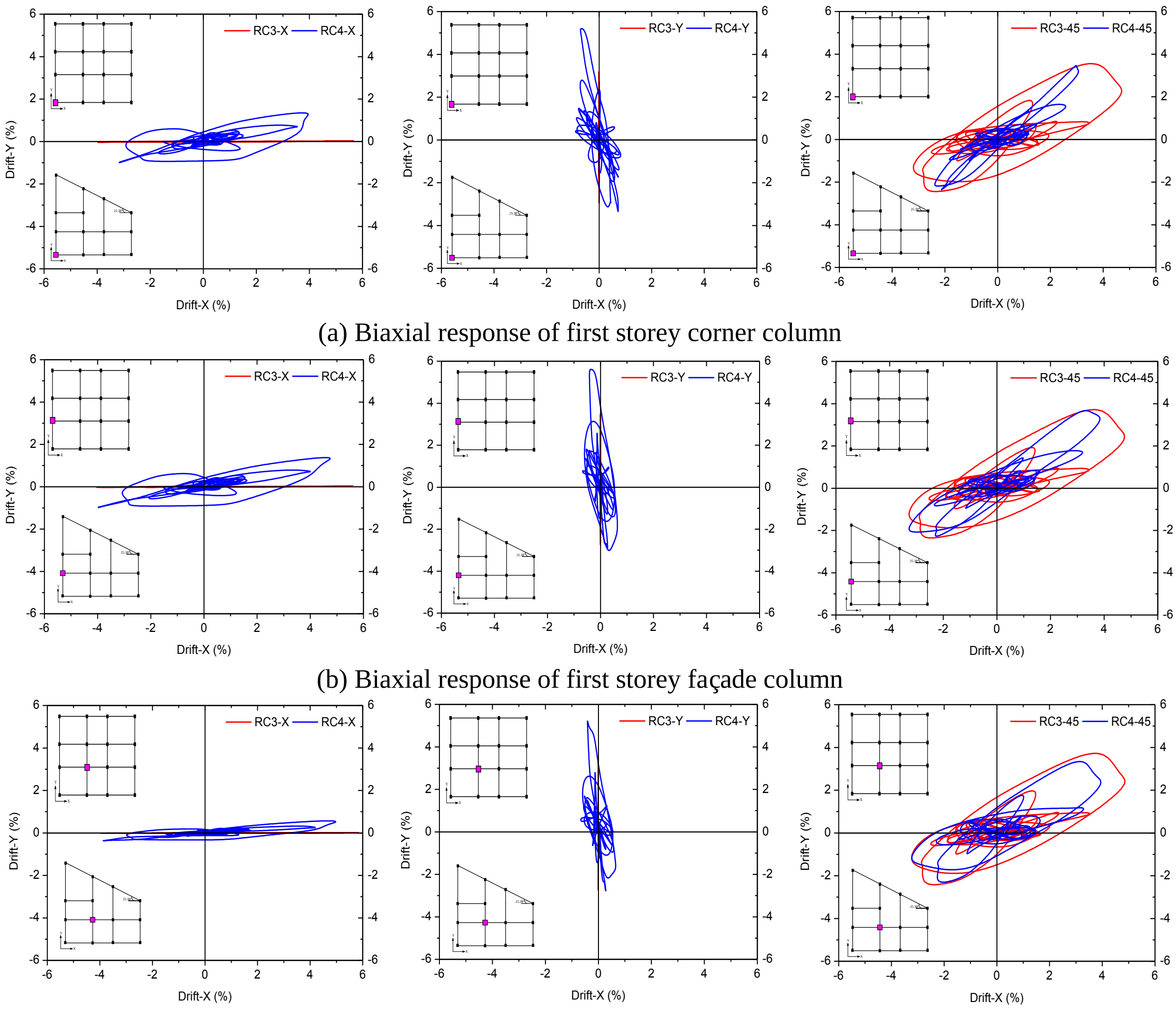
6.7.1 Biaxial response of reinforced concrete columns

The behaviour of the RC elements subjected to axial loading in conjunction with cyclic biaxial bending is accepted as a very important research issue for building structures in earthquake-prone regions. There are still a number of unresolved problems with the adequate modelling of RC buildings under general earthquake loading. One of the main issues is related to the fact that buildings are three-dimensional structures and in several cases it is impossible to simplify the 3-D models into two-dimensional ones without considerable loss of accuracy (Dundar and Tokgoz 2012, Rodrigues *et al.* 2013). A structural member subjected to biaxial flexure suffers greater damage than with one-dimensional loading (Takizawa *et al.* 1976). In fact, the biaxiality of the cyclic moments tends to reduce the capacity of the columns because of the biaxial interaction effect (Rodrigues *et al.* 2012). The results of the drift profiles at the centre of corner, façade and interior columns are presented in Fig. 20.

In this context, the biaxial response of existing RC column is studied for the structures with regular and irregular plan configurations. For this, the dynamic time history analysis has been performed with synthetic earthquake in Nepal. The biaxial response of corner, façade and interior columns at the first storey level is plotted and analysed, considering the earthquake loading in the *X*, *Y* and 45^0 directions. As expected, the biaxial response is more important in façade and corner columns, and specially in the irregular building, even in the case where the action is unidirectional (*X* or *Y*) the earthquake induces an important drift demand in the opposite direction in the irregular building RC4 (around 25% whan compared with the demand in the load direction). From the hysteretic behaviour of all the studied columns, it is clearly seen that columns of irregular buildings have torsional oscillation. In symmetric structures, the biaxiality of the bending action is due to the instantaneous presence of the two horizontal components of the seismic excitation, whereas in asymmetric structures such an orthogonal loading condition is due also to the lateral-torsional coupling. In many situations, biaxial structural interaction and torsional oscillation may arise, namely as a result of structural irregularity, affecting the structural response. However, even for structures with regular and symmetric configurations and uniform mass distributions in the building plan, planar models cannot obtain an accurate enough response. Since earthquake excitation is, in general, multi-dimensional, biaxial structural interaction must, therefore, be considered.

6.7.2 Maximum variation of axial load

The vibration characteristics of columns are influenced by their axial loads. The axial load ratio of the column has dramatic on the drift performance of lightly reinforced columns, particularly the significantly lower drift capacities that are available in compression dominated columns (Wibowo *et al.* 2014). Moragaspitiya *et al.* 2014 quantify axial deformation of columns in a structural

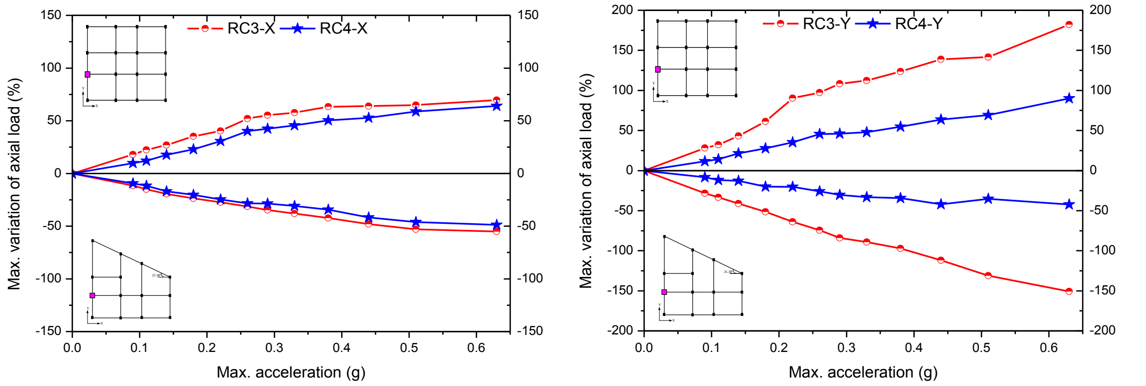


(a) Biaxial response of first storey corner column

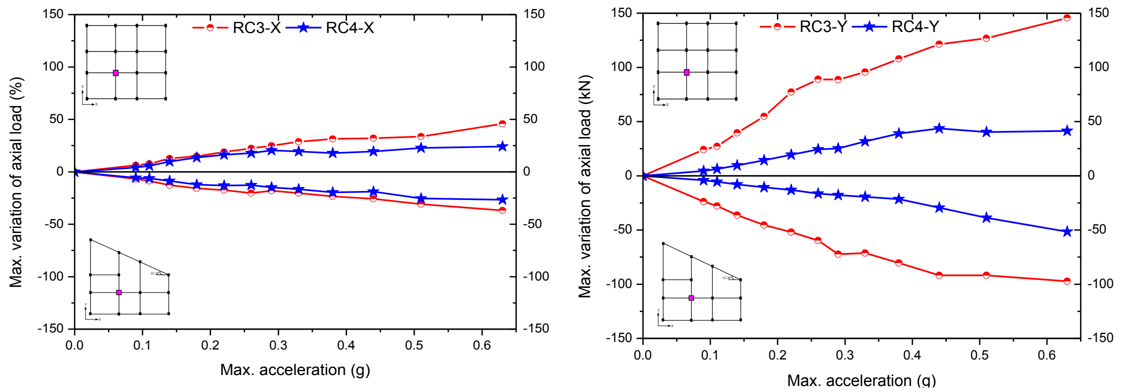
(b) Biaxial response of first storey façade column

(c) Biaxial response of first storey interior column

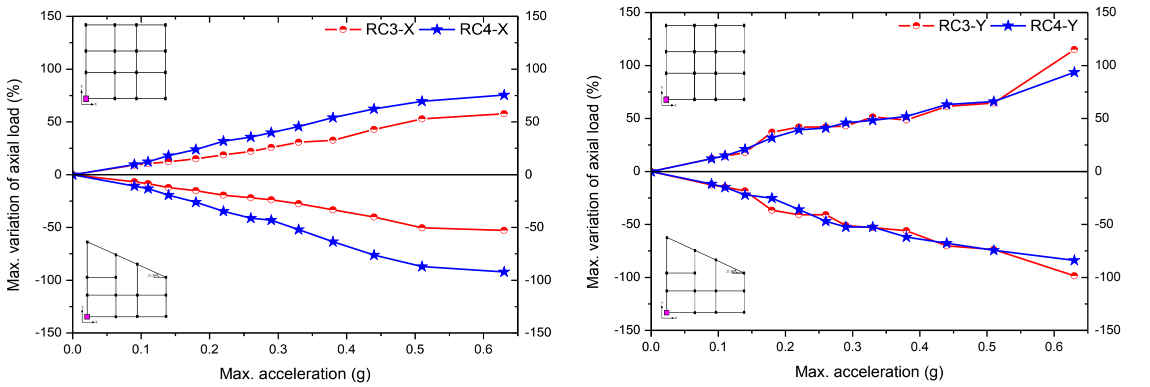
Fig. 20 Biaxial response of RC3 and RC4 building models in *X*, *Y* and 45° direction of loading condition



(a) Variation of axial load in third storey facade column



(b) Variation of axial load in third storey facade column



(c) Variation of axial load in second storey corner column

Fig. 21 Maximum variation of axial load for RC3 and RC4 building models in *X* and *Y* direction of loadings

system using its vibration characteristics, incorporating the influence of load tributary areas, boundary conditions and load mitigation among the columns. In the present study, the maximum variation of axial load in the column was studied through non-linear dynamic analysis with synthetic earthquake in Nepal. For this, the performance of interior, façade and corner columns of regular (RC3) and irregular (RC4) structures are studied. For this time history data with increasing peak ground acceleration has been employed. The results from the numerical analysis are presented in Fig. 21.

The results indicate that the maximum axial load variation of a corner column having a regular configuration (RC3) is 65.32% in the *X* and 115% in the *Y* direction of the loading condition. This limit is 92.10% in the *X* and 98.43% in the *Y* direction for the irregular configuration (RC4). For façade columns, the RC3 and RC4 structures have values of 69.69% and 64.09% in the *X* and 182.25% and 90.02% in the *Y* direction. Similarly, for interior columns, the RC3 structure has 47.54% in the *X* and 97.30% in the *Y* direction, whereas the values are 32.72% in the *X* and 51.62% in the *Y* for the RC4 structure. The maximum variation of axial load in columns up to the second storey is consistent for both structures. In the RC3 structure, the axial load variation in the façade and interior columns is sharply apparent between the second and third storeys (up to182.25%) in the *Y* direction. Moreover, the result indicates that the central column has a small variation of axial load, at around 25%. As expected, the maximum variation of axial load is in the corner column. It indicates that axial forces can alter the failure mode of the columns. Ghassemieh *et al.* 2014 observed that the presence of axial force even in a small value can change the behaviour of the columns significantly. Analysis results for the corner, façade and interior columns can be summarized as follows:

• In the corner columns, the RC4 structure has a higher axial load variation in the *X* direction than RC3. The difference is negligible in the *Y* direction. This is due to the fact that the RC3 has greater stiffness in the *X* direction, compared to the RC4 structure.

• In the façade and interior columns, the overall variation is very small and the difference is negligible in the two structures in both the *X* and *Y* directions at the first and second storeys. However, due to the effect of less stiffness in the third storey, the RC3 structure has a very high axial load variation on this floor in the *Y* direction of loading.

7. Conclusions

RC buildings constitute the prevailing type of construction in earthquake-prone region like Kathmandu Valley. Most of these building constructions were based on conventional methods. In this context, the present paper studied the seismic behaviour of existing RC buildings in Kathmandu Valley. For this, four representative building structures with different design and construction, namely a building: (a) representing the non-engineered construction (RC1 and RC2) and (b) engineered construction (RC3 and RC4) has been selected for analysis. The dynamic properties of the case study building models are analyzed and the corresponding interaction with seismic action is studied by means of non-linear analyses. The structural response measures such as capacity curve, inter-storey drift and the effect of geometric non-linearities are evaluated for the two orthogonal directions. The effect of plan and vertical irregularity on the performance of the structures was studied by comparing the results of two engineered buildings. This was achieved through non-linear dynamic analysis with a synthetic earthquake subjected to *X*, *Y* and 45° loading directions. The nature of the capacity curve represents the strong impact of the P-delta effect, leading to a reduction of the global lateral stiffness and reducing the strength of the structure. The non-engineered structures experience inter-storey drift demands higher than the engineered building models. Moreover, these buildings have very low lateral resistant, lesser the stiffness and limited ductility. Finally, a seismic safety assessment is performed based on the standard drift limits. Result indicates that most of the existing buildings in Nepal exhibit inadequate seismic performance. The additional conclusions from the analysis can be summarised as follows:

• As expected, engineered structures present higher strength, tangent stiffness and lower deformation when compared with non-engineered structures. The shear strength capacity and tangent stiffness of engineered buildings (RC3 and RC4) are nearly two times the value obtained with the non-engineered structures (RC1 and RC2).

• Drift values in RC1 and RC2 types are quite higher than in the RC3 and RC4 structures. In engineered building structures, the rate of change of inter-storey drift profile is quite regular and consistent in all the storeys. While, there is highly irregular and inconsistent inter-storey drift profiles in non-engineered structures. In particular RC1 building model present a soft storey mechanism in the third storey, due to the reduction of the column-section between the second and third storey. By this fact and due to the low rise of the buildings this procedure should be considered non-adequate for earthquake prone areas like Kathmandu Valley.

• Base on the present study different limit states value for RC building structures in Nepal, and can be now applied for a large scale study with more examples regarding the proper seismic risk analysis of Nepal.

• From the analysis result it can be seen that the existing non-engineered buildings in Nepal exhibit high vulnerability, with limited ductility. The non-engineered building structures only satisfied the ‘operational’ performance level at design intensity. In the present study the RC buildings that represents the conventional constructions methods can be considered unsafe. By this fact it is highlighted that a large study regarding the analysis of different typologies of this type of construction need to be performed and also the analysis of possible and feasible retrofitting solutions, in order to reduce the seismic vulnerability in future earthquakes. The studied engineered buildings presents a better performance.

• The effect of axial load variation is greatly influenced by the stiffness of the structure. This is apparent in the third storey columns (façade and interior) in the *Y* direction. It is due to the structural discontinuity shown in Fig. 12.

•The biaxial behaviours of columns show that the effect of seismic action is highly sensitive in non-symmetrical structures (RC4), and even for a unidirectional action in one direction can induce a demand around 25% in the opposite direction. Result indicates that a biaxial response is very clear in the RC4 structure. The failure mechanism of RC columns is highly dependent on the load path, ductility capacity, and energy dissipation of the columns. Moreover, from the analysis results it is clear that any realistic representation of the behaviour of RC structures (mostly irregular) should include a three-dimensional aspect.

There are still a number of unsolved problems associated with the modeling and safety assessment of non-engineered RC building under seismic loading. The preliminary results of the analysis showed that in a major earthquake, the buildings may suffer heavy damage when compared with engineered buildings, in particular in the case where structural irregularities are present. Many questions can be arise regarding the modeling of this typo of buildings and if the models can adequately reproduce the main characteristics of the element’s response, such as the strength and stiffness degradation, the changes in terms of ductility, and energy dissipation capacity.

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