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Seismic assessment and retrofitting of existing RC buildings in Kathmandu

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Abstract

The effectiveness of reinforced concrete (RC) column jacketing for improving the seismic performance of existing RC building structures were studied. Four three storey buildings with different structural configuration and detailing were selected for seismic assessment and retrofitting purpose. The response of structures (original and retrofitted) was evaluated in terms of capacity curve and inter-storey drift. The case studies also intend to verify the effect of P-delta effects and bi-axial response of columns under non-linear time history analysis. 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. Finally, a seismic safety assessment is performed based on the drift limit proposed by FEMA-356. The assessment of original building structures indicates that they may exhibit inadequate seismic performance. However, RC column jacketing highly improves seismic performance of all the structures and results maximum drift demand within the drift limit proposed by FEMA-356.

Keywords: Seismic retrofitting; Non-linear analysis; Existing RC structures; Performance evaluation; P-delta effect

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1 Introduction

The trend of the construction of reinforced concrete (RC) buildings has been rapidly increased in earthquake prone regions because of their seismic performance (Watanabe et al., 1998, Naeim et al., 2000). In past earthquake, many structural deficiencies were reported such as low-quality concrete, poor confinement of the end regions of columns and beams, weak column–strong beam behaviour, short column behaviour, inadequate splice lengths and improper hooks of the stirrups (Kaplan, 2004, Yakut et al., 2005). In fact existing structures have: the design and construction defects, modifications in codes of practice and standards, environmental effects, changes in their usage and loading conditions and the need to increase the number of floors. Therefore the majority of structures do not possess adequate lateral strength and stiffness during past earthquake in worldwide.

It is, therefore, of primary importance for strengthening of the existing building structures in order to prevent the resulting loss of lives and property. These structurally deficient buildings should be retrofitted to withstand earthquake forces in compliance with modern design codes. Seismic strengthening of the structures is mainly based on two strengthening schemes. Seismic performance of the existing structures is improved by limiting the lateral drifts or improving the lateral drift capacity of the building (Moehle 2000, Higashi et al., 1984). The former scheme is implemented by the addition of new structural elements which improve the strength and stiffness of the system. Alternatively, local strengthening of the low-ductility structural elements, which deteriorate the stability of the structure, may be an economical solution for buildings with limited number of deficient elements. However, local strengthening methods create a negligible improvement in base shear capacity and lateral stiffness of the system. For most of the cases, element based techniques may not be sufficient by themselves to adequately enhance the earthquake safety of strengthened structures.

This study aims to evaluate the seismic performance of the most common building stock in Nepal. Four three storey buildings with different configuration and detailing were selected for seismic evaluation and retrofitting the structures in Kathmandu valley. Structural behaviour of both original and retrofitted structures was studied through non-linear dynamic analysis with synthetic earthquake ground motion records. Finally, a vulnerability assessment of the structures was evaluated based on drift limit proposed by FEMA recommendations (FEMA-356, 2000).

2 Case study buildings

The four three-storey reinforced concrete moment resisting frame structures were selected for the analysis. Two buildings were engineered designed buildings with irregular (B1) and regular configuration (B2), constructed in northern part of the Kathmandu valley. Inter storey height of these buildings were 2.85 m in all storey. Building B1 has trapezoidal plan area measuring 70.8 m^2 which has two and four moment resisting frame in X and Y direction respectively. B2 building structure have plan area 9.6 m x 7.9 m (75.84 m^2), measured from the column centre lines. Four identical moment-resisting frames, in X- and Y-directions, act as the lateral load resisting system. Similarly, building B3 and B4 were nearly rectangular and square plan configuration built in southern part of Kathmandu valley. Both the buildings have 3 m inter storey height in all storeys. In B3 model, the lateral load resisting elements in X-direction consist of three moment resisting

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frame, and Y-direction the frame are four. The building dimensions in plan were 10.5 m x 8 m (84 m^2). In contrast, B4 building has 9 m x 9 m (81 m^2) with three moment resisting system for both in X- and Y-direction. At the 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. The plan, tridimensional model and RC cross section detailing of all the case study buildings are presented in Fig. 1 to 4.
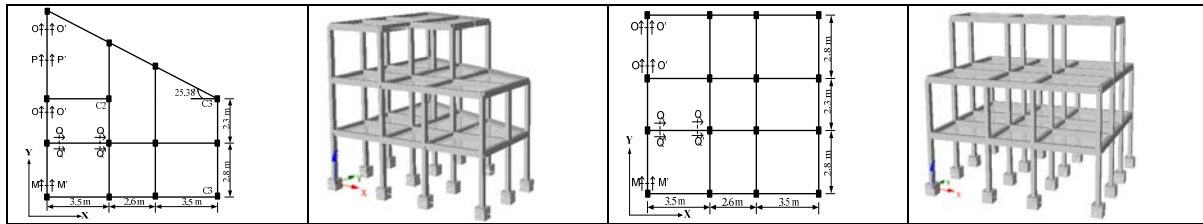

Fig. 1– Plan and tridimensional model of B1 and B2 structures
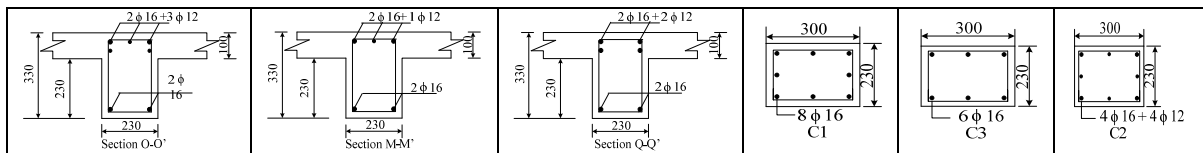

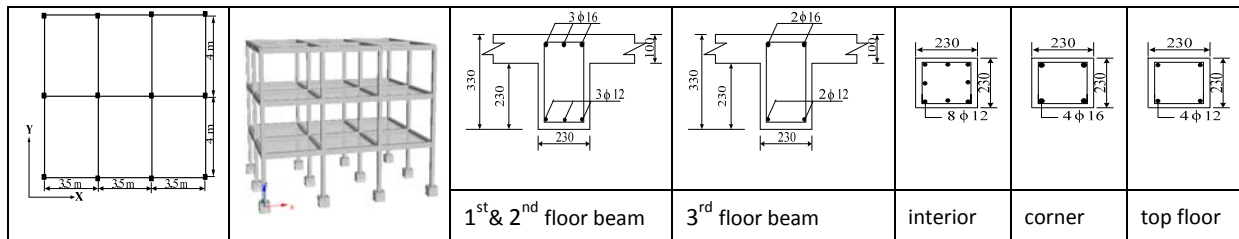Fig. 2 – RC cross-section detailing of B1 and B2 structures

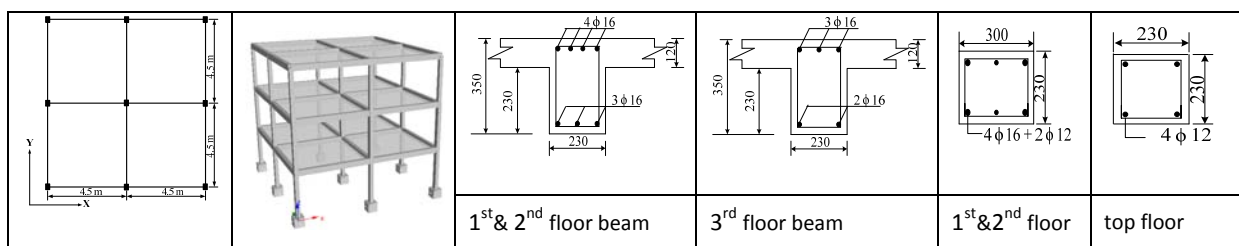Fig. 3 – Plan, tridimensional model and cross-section detailing of B3 structure

Fig. 4 – Plan, tridimensional model and cross-section detailing of B4 structure

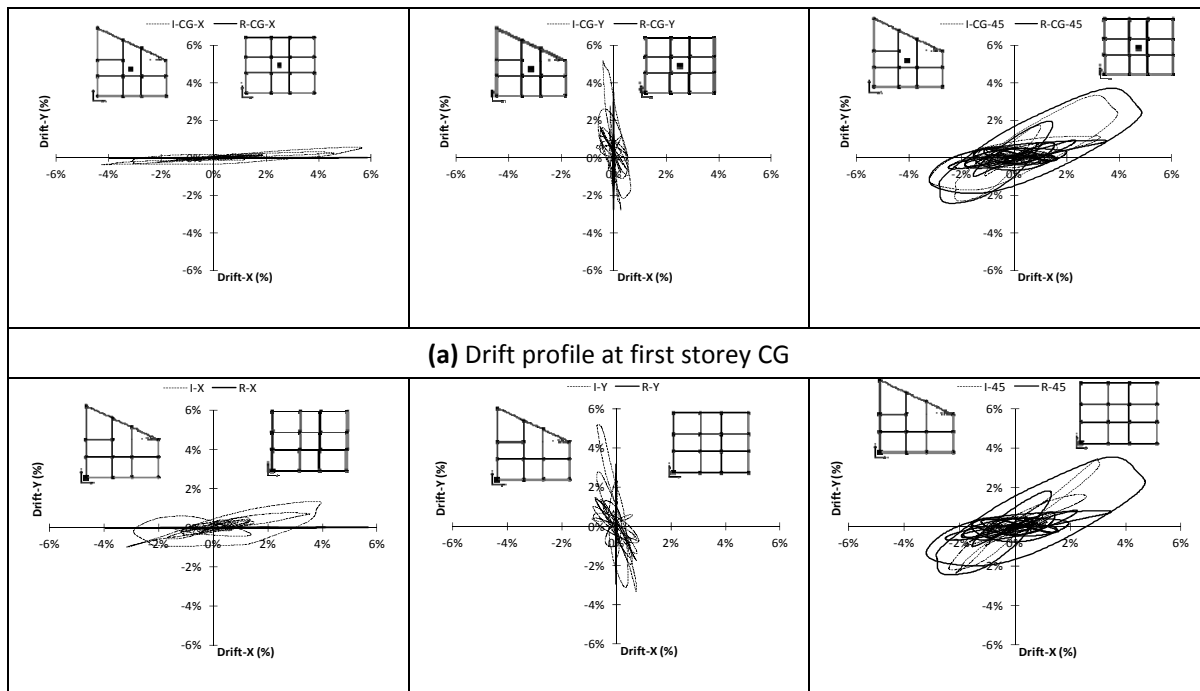*Note: All dimensions are in mm unless stated otherwise.*
All the exterior columns are C1 unless stated otherwise.

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3 Results and discussion

3.1 Biaxial response of reinforced concrete column

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 (Rodrigues et al., 2013).

During earthquakes, the columns of buildings are regularly subjected to cyclic biaxial flexure with varying axial force. 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 (Rodrigues et al., 2012). Since earthquake excitation is, in general, multi-dimensional, biaxial structural interaction must, therefore, be considered. 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. Moreover, for RC members subjected to biaxial bending an increase in the degradation of strength and stiffness has been observed (Rodrigues et al., 2012).
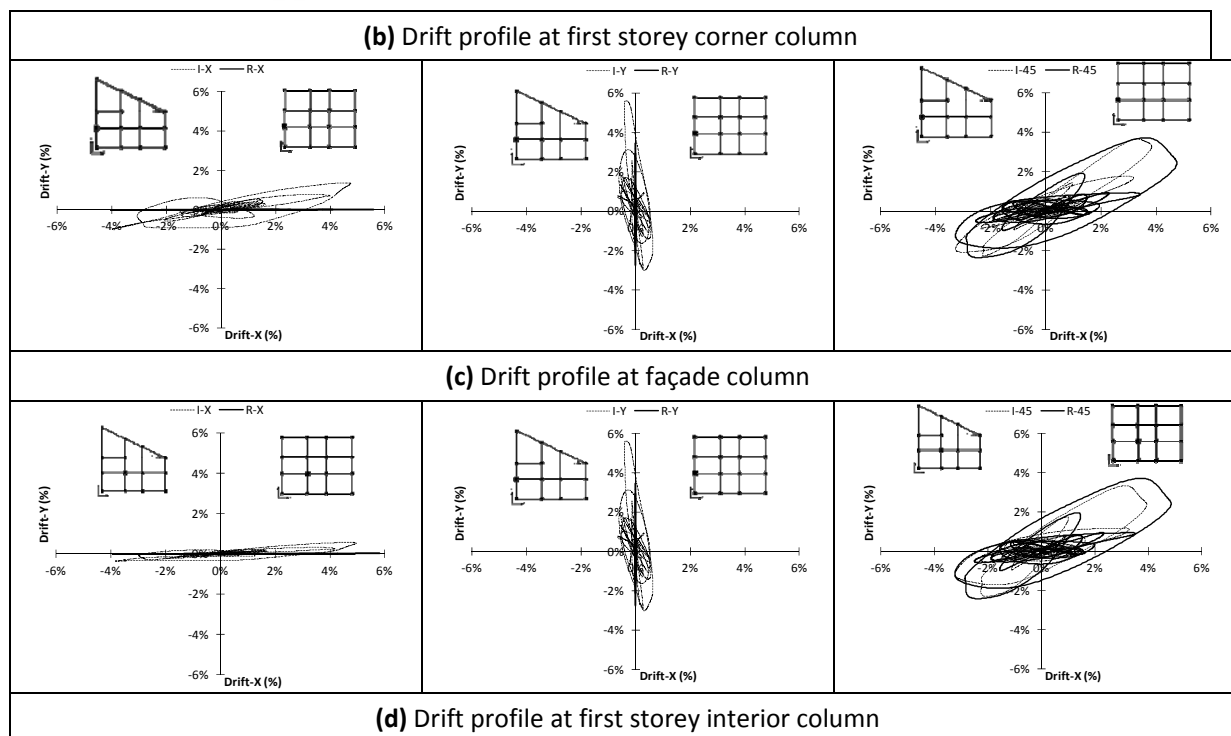

(a) Drift profile at first storey CG

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(b) Drift profile at first storey corner column

(c) Drift profile at façade column

(d) Drift profile at first storey interior column

Fig. 5 – Drift profile in first storey for B1 (l) and B2 (R) structures in X, Y and 45° of loading

The realistic response of existing structures is studied through the biaxial response of an RC column with nine real earthquake records of different magnitudes. The behaviour of the structure in terms of storey drift in the corner, façade and interior columns at the first storey is plotted in the X, Y and 45^0 directions of loading. Both B2 and B1 building structures have maximum drift in the interior column for the X loading direction (5.84% and 4.97%). However, the values for the façade column in the Y direction are 3.49% and 5.61%. In B2 structure, when the earthquake loading is in the Y direction, the resulting drift in X is negligible and vice versa. By contrast, in B1 structure there is a significant effect which is more pronounced in the X direction (up to 1.38%). Moreover, the effect in the 45° loading condition is very marked in both the X (4.85%) and Y (3.94%) directions. In this loading, the corner and façade columns have higher values in the X and Y directions respectively. The maximum drift profiles of the centre of gravity of the structures behave similarly to the interior columns. The results of the drift profiles at the centre of gravity, corner, façade and interior columns are presented in Fig. 5.
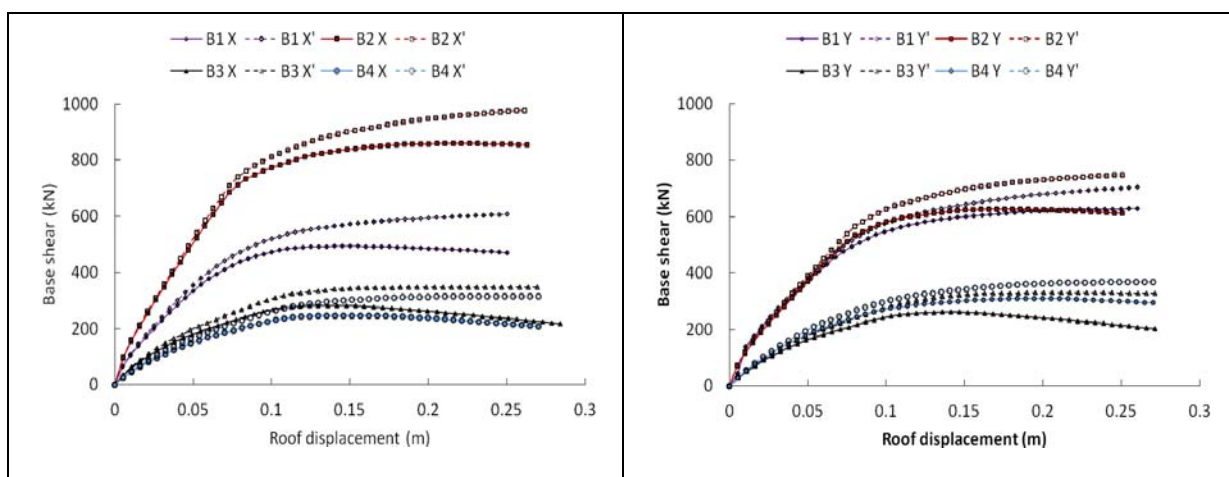
3.2 P-delta effects

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). Recently, Black (2011) presented a study in which the P-delta effect in moment-resisting frames is quantified through two stability coefficients. Black derived the expression for the modal-elastic stability coefficient from analytical considerations and the modal inelastic stability coefficient through regression analysis. Many of these investigations have focused on the

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displacement amplifications that result from the second-order actions, using both single- (SDOF) and multi-degree-of-freedom (MDOF) systems. Such displacement-based studies have become of more interest in recent years as a result of the shift towards performance-based design and assessment procedures. Moreover, different researchers have set out to assess the global P-delta effect in frame structures by means of equivalent single-degree-of-freedom (ESDOF) systems (Takizawa, 1980). The P-delta effect may contributes to a loss of lateral resistance, residual deformations and dynamic instability of the structure (Adam et al., 2011).

Meanwhile, in structural analysis, if the storey drifts remain rather uniformly distributed over the height, regardless of the extent of inelastic deformation, a global assessment of the P-delta effect by means of ESDOF systems is not difficult. Thereby, it is assumed that P-delta is primarily governed by the fundamental mode. However, if a partial mechanism develops, the global P-delta effect will be greatly affected by the change of the deflected shape, and it will be amplified in those storeys in which the drift becomes large (Bernal, 1998).

In a global pushover curve, the base shear is plotted against characteristic deformation parameters. In general, this deformation parameter is the lateral displacement of the roof. It is assumed that the shape of the global pushover curve reflects the global or local mechanism involved when the structure approaches dynamic instability during a strong motion earthquake (Mander et al., 1988; Martinez-Rueda et al., 1997).

Fig. 6 shows the global pushover curves of the case study buildings and the corresponding IS-drift, representing the response of structures with and without consideration of the P-delta effect. The capacity curve indicates that the analysis results without considering the P-delta effect have improved shear strength capacity. The average increases in the X and Y directions of loading are 20% and 16% respectively. The increment is higher in non-engineered structures (B3 and B4). 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.

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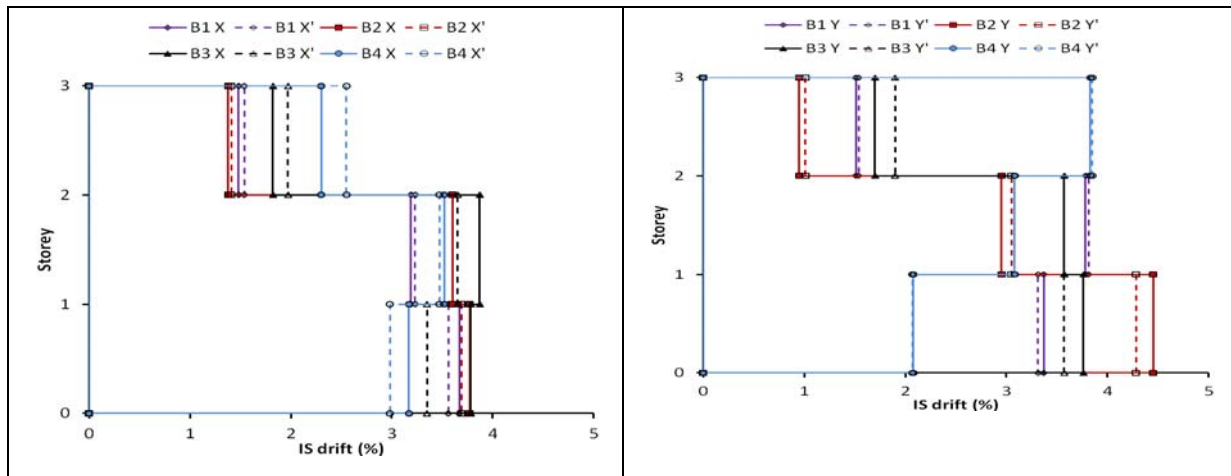*João Pessoa-PB (Brasil), 2 a 5 de junho de 2013*

Fig. 6 – 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

3.3 Performance evaluation

The moment carrying capacity of existing beam and column members and expected moment of critical member of all the study buildings are presented are presented in Table 1. The comparison of demand capacity ratio of shear stress, axial stress, shear capacity, flexural capacity of column, beam column capacity ratio of all the study buildings are presented in Table 2 (IS 1893: 2002, IS 13920: 1993). In most of the case, demand capacity ratio (DCR) of all the study building does not satisfied. It implies that all the structures have lower seismic capacity.

Table 1 - Moment carrying capacity of existing beam and column

Study building structures	Moment capacity of existing structures (kN·m)		Expected moment of study building structures :IS 1893-2002 (kN·m)	
	Beam ($\sum M_b$)	Column ($\sum M_c$)	Beam ($\sum M_b$)	Column ($\sum M_c$)
B1	142.01	144.9	153.27	195.78
B2	142.01	144.9	143.03	179.32
B3	138.10	48.50	239.50	290.42
B4	193.45	52.00	285.28	342.26

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Table 2 - Seismic performance evaluation of existing beam and column elements

S. N	Check	Demand capacity ratio (DCR)				Remarks
		B1	B2	B3	B4	
1	Shear stress in RC frame column	1.15	1.1	2.15	2.05	Check not satisfied
2	Axial stress to gravity load	>2	>2	>2	>2	Check not satisfied
3	Axial stress in moment frame at base	0.29	0.24	0.34	0.22	Check satisfied
4	Shear capacity of column	0.32	0.31	0.54	0.77	Check satisfied
5	Moment of resistance of beam in compression	1.14	1.02	1.31	1.26	Check not satisfied
6	Moment of resistance of beam in tension	1.44	1.35	3.10	1.98	Check not satisfied
7	Column flexural capacity	1.35	1.24	5.99	6.58	Check not satisfied
8	Column beam capacity ratio	-	-	-	-	Check not satisfied

3.4 RC column Jacketing

Jacketing is effective in correcting specific deficiencies in strength and/or deformation capacity, to improve longitudinal reinforcement development or lap-splices, identified during the detailed evaluation (Rodrigues et. al, 1991). Previous studies indicated that jacketing was effective to rehabilitate the existing structure, thus improving the strength, stiffness and energy dissipation characteristics of existing structures (Fardis 1998, Varum 2003, Alcocer, 1992). For retrofitting purpose, seismic design of all the columns was based on Indian standard code of practices. The additional cross section and reinforcement detailing of study building structures are presented in Fig. 7. For B1, B2 and B3 building structures 8-16 mm dia. additional reinforcement bar is required whereas the value is nearly double in B4 (8-16 mm dia.). The numerical analyses of all the structures were again done using the RC column jacketing. The influence of jacketing on performance of structures will discussed in the following sections.
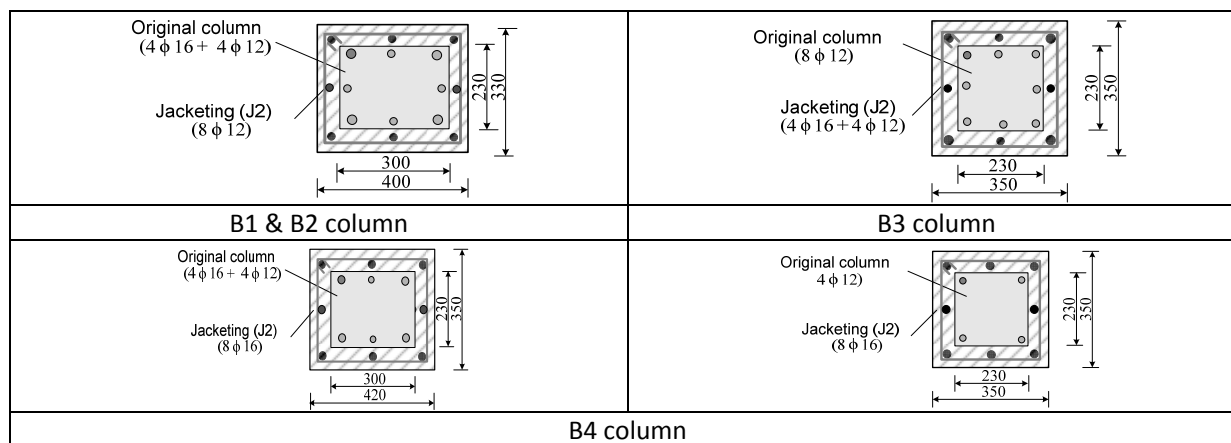

Fig. 7 – Detailing of RC column jacketing

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The results from non-linear dynamic analysis of existing and retrofitted structures in terms of capacity curve and corresponding inter-storey drift is presented in Fig. 8 - 9. The analysis of the results indicates that RC column jacketing significantly improves the performance of existing structures. The maximum increment in base shear is about 160% appeared in B3 buildings (from 323 kN to 841 kN) whereas minimum increment of 36% in B1 building (from 799 kN to 1087 kN). The maximum decrease in inter-storey drift is 2.57% in B4 building (from 5.75% to 3.18%). The average reduction of IS drift in engineered buildings (B1 & B2) was 1.75% whereas the value is only 1.55% for non-engineered buildings (B3 & B4) in Kathmandu valley. The average increment of shear strength of engineered buildings was only approximately 50% whereas the value is more than double in non-engineered buildings.
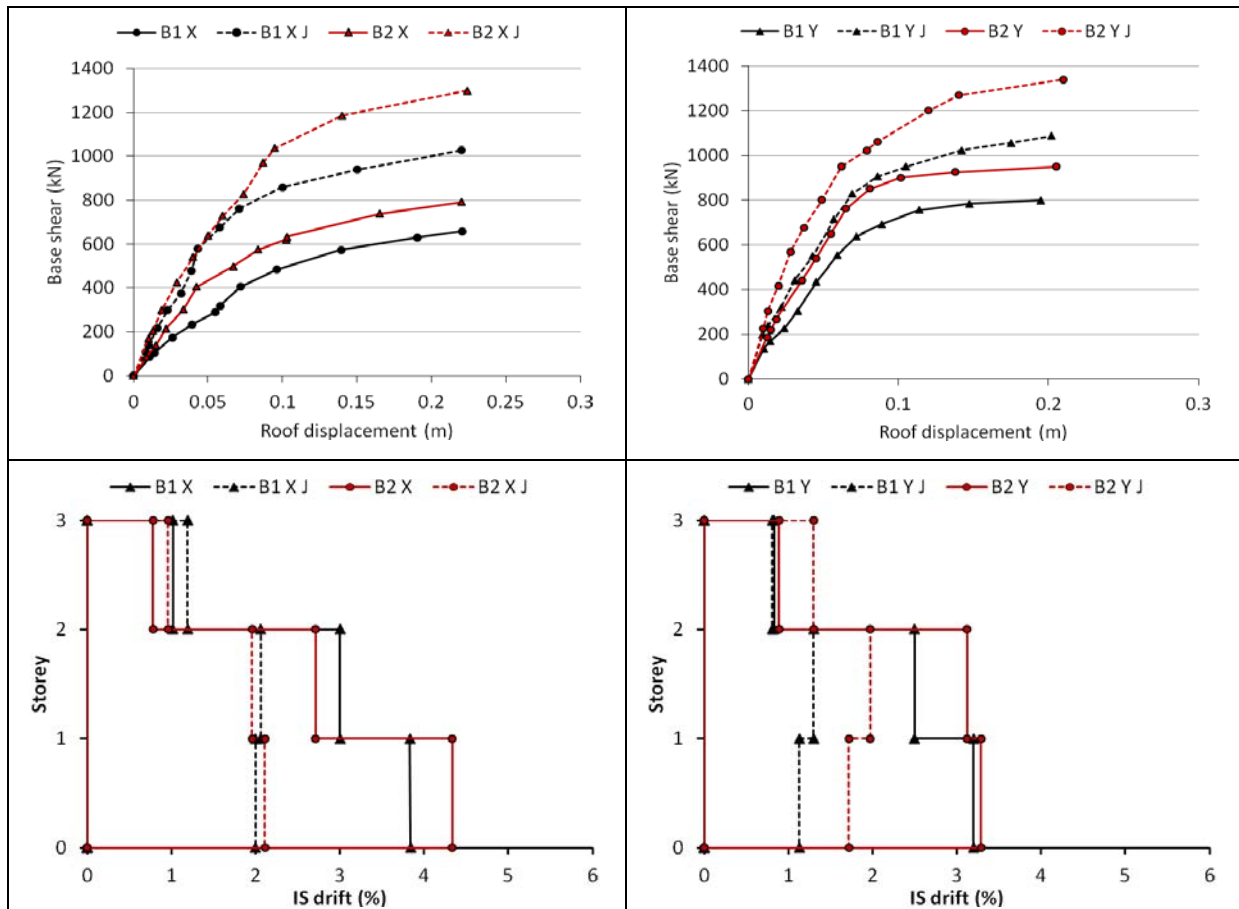

Fig. 8 – Capacity curve and corresponding IS drift of building model B1 and B2 with and without retrofitting for longitudinal (X) and transverse (Y) directions of loading

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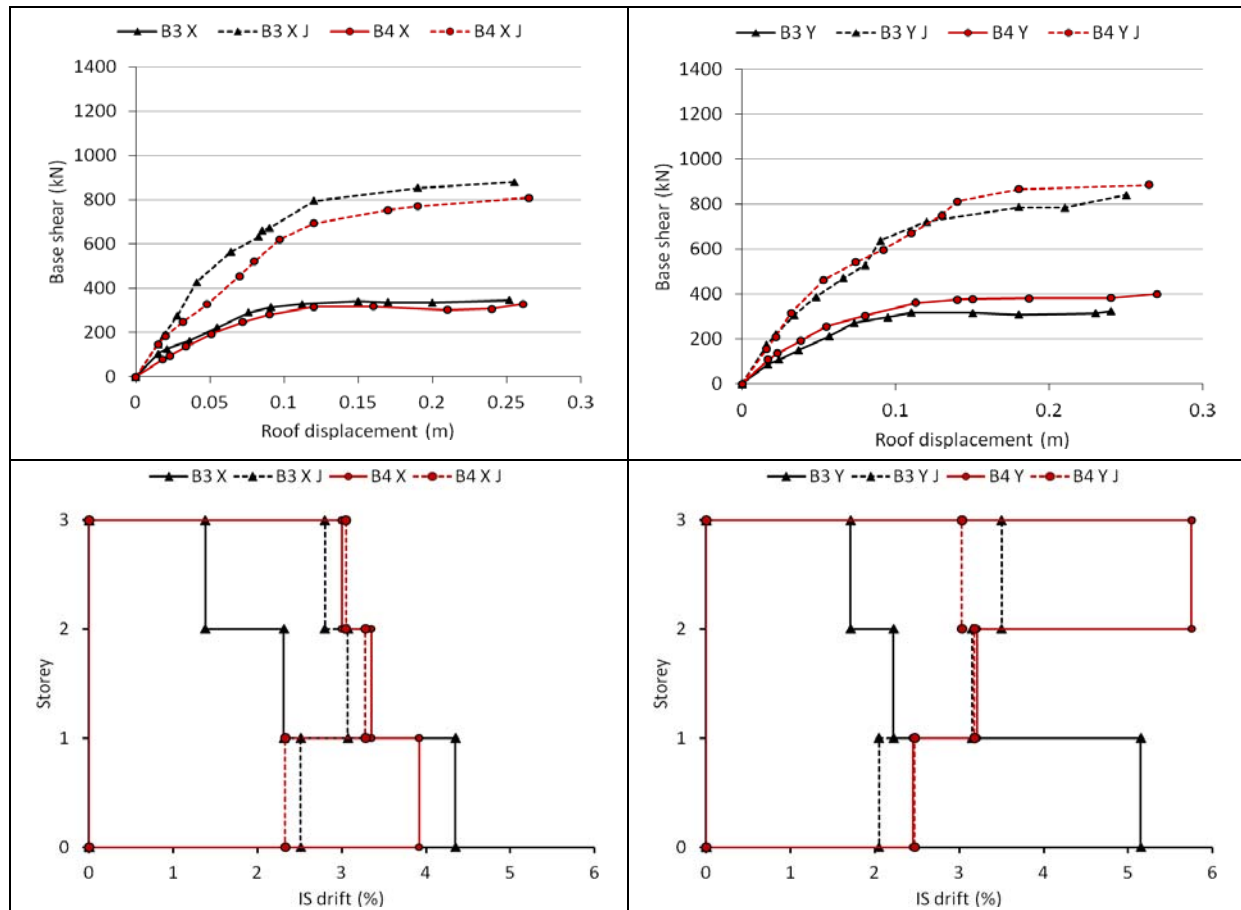*João Pessoa-PB (Brasil), 2 a 5 de junho de 2013*

Fig. 9 – Capacity curve and corresponding IS drift of building model B3 and B4 with and without retrofitting for longitudinal (X) and transverse (Y) directions of loading

3.5 Assessment of seismic safety of the building

The seismic vulnerability of the buildings under study was assessed with original and retrofitted structures. Results from non-linear dynamic analysis for each direction were compared in terms of the maximum drift demands and the basic performance objectives proposed by FEMA-356 (Tables 3 and 4). These proposed values have been used by several authors to assess the safety of RC buildings, and they point to acceptable performance of the structures. Various documents point to the same concepts with small differences in the values, however, and even though this cannot be the most appropriate for Nepal's local buildings characteristics, it provides a good estimate of the building safety (FEMA 2000; Chaulagain et al., 2013; SEAOC, 1995).

Table 3 - Basic Performance Objectives for buildings according to FEMA-356

		Fully operational	Operational	Life safety	Near collapse
Earthquake Design level	Frequent (43-yrp)				
	Occasional (72-yrp)		X		
	Rare (475-yrp)			X	
	Very rare (970-2000-yrp)				X

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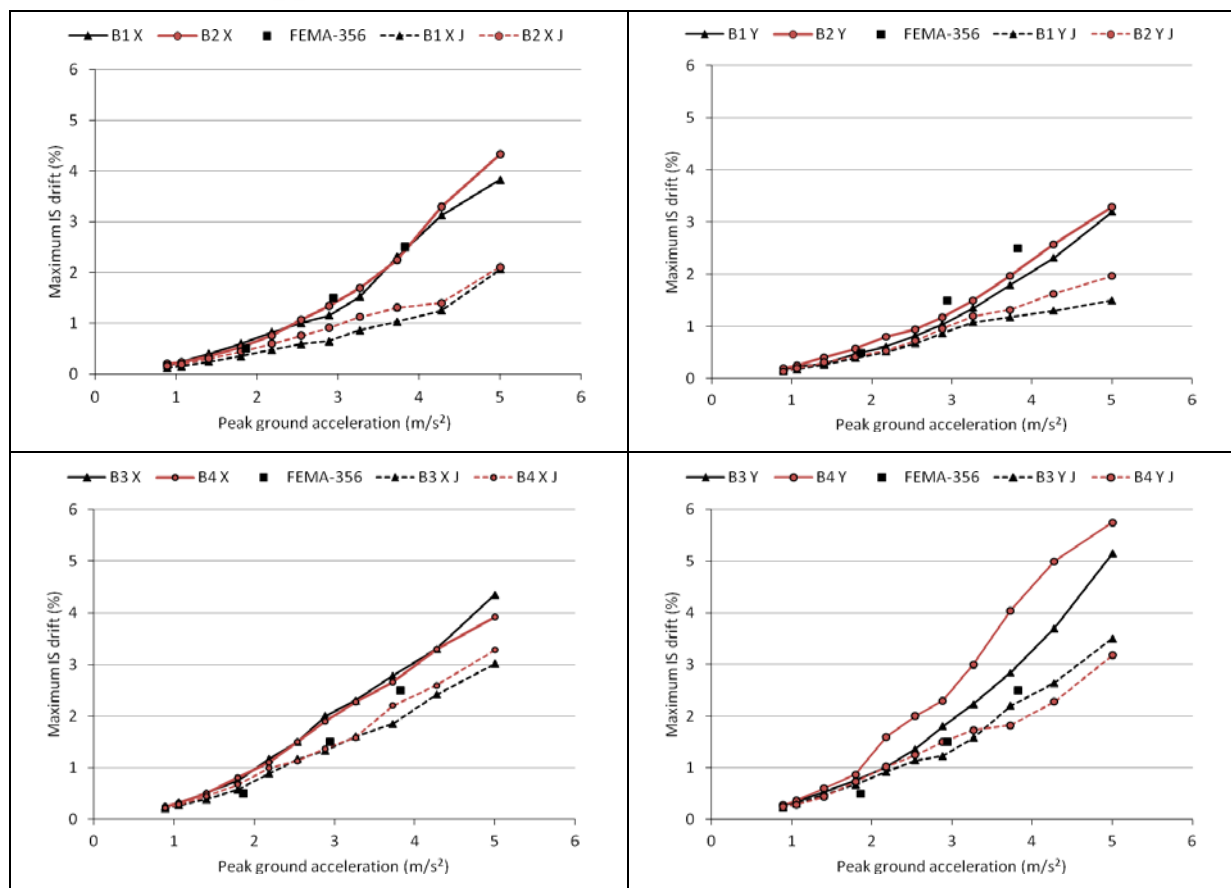João Pessoa-PB (Brasil), 2 a 5 de junho de 2013

Fig. 10 – Vulnerability curves of the maximum IS drift for study building structures with and without retrofitting for longitudinal (X) and transverse (Y) directions of loading

Table 4 - Storey drift limits according to FEMA-356

	Performance level			
	Fully operational	Operational	Life safe	Near collapse
Drift limit	0.2%	0.5%	1.5%	2.5%

Fig. 10 presents the vulnerability curves of the structural models including the drift limits proposed by the FEMA-356 recommendations. The structural characteristics of the buildings were 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 all the building structures have higher drift demand. The limiting drift is only 2.5% for the performance level 'near collapse'. In fact, all the study building structures have drift values greater than the FEMA-356 guidelines.

The result indicates that in many cases the construction of both conventional (B3 and B4) and modern buildings (B1 and B2) in Nepal may be unsafe for the 'life safety' performance level. However, engineered buildings perform better than non-engineered buildings, although still not sufficiently well to withstand future earthquakes without retrofitting measures. RC column

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jacketing significantly improves the performance level of the building structures. After retrofitting, all the study buildings were quite safe for future earthquake.

4 Concluding remarks

The paper presents the seismic performance evaluation and retrofitting of existing RC building structures in Nepal. The effect of plan irregularity on the performance of the structures was studied by comparing the results of the B1 and B2 buildings. The dynamic properties of the B1, B2, B3 and B4 building models are analysed and the corresponding interaction with seismic action is studied by means of non-linear analyses; different response measures such as capacity curves, inter-storey drift and the effect of geometric non-linearities are evaluated for the two orthogonal directions. The results of the analyses show that the original buildings exhibit inadequate seismic performance in all cases. In contrast, RC jacketing highly improves seismic performance existing structures. The main conclusions from the analysis can be summarised as follows:

- The effect of axial load variation is greatly influenced by the stiffness of the structure. This is very apparent in the third storey columns (façade and interior) in the Y direction. Due to the vertical irregularity, there is degradation of stiffness in the Y direction in the B2 structure, so that it is very evident in the third storey column in the Y direction of loading.

- Analyses indicate that a biaxial response is very apparent in the B1 structure. In fact, in the symmetric structure (B2) the biaxiality of the bending action is due to the instantaneous presence of the two horizontal components of the seismic excitation, whereas in the asymmetric structure (B1) such an orthogonal loading condition is due also to the lateral-torsional coupling.

- The failure mechanism of RC columns is highly dependent on the load path, ductility and energy dissipation capacity of the columns. In the case of existing buildings, the structural strength to resist seismic action is often not sufficient. 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.

- Original engineered (B1 & B2) and non-engineered (B3 & B4) building structures do not satisfied only the FEMA-356 performance level. This implies that both conventional and modern building construction in Nepal is highly unsafe in the 'life safety' performance level. Moreover, RC jacketing significantly improves performance of structure, and results that all the retrofitted building structures are safe in 'life safety' performance level (FEMA-356).

- 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 structures.

Acknowledgements

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