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Comprehensive Seismic Performance Assessment of Low Rise RC Buildings by Numerical Modelling

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ABSTRACT

The collision of the tectonic plate between Indian and Eurasian plates results the large number of earthquake in countries like Nepal. The geology of the country is different as there is variation in latitude and longitude, which results in different ground motion during earthquake. Due to the variation in ground motion there is variation on earthquake intensity which changes the performance of the low rise buildings. In the present study 10 different irregular low rise buildings is considered for the modelling. Linear static analysis is performed to check time period, displacement, drift and storey shear of models. Later the building models are checked and analyzed using nonlinear pushover analysis. Hinges were introduced on the beam and columns as per FEMA 356 and ATC 40, thus making the building models as nonlinear models. For execution of nonlinear analysis both push in X and push in Y direction load is applied in controlled displacement mode. After execution of nonlinear pushover analysis different color of hinges were formed which forms the basis of study. The maximum displacement, max storey drift and storey shear were calculated on both X and Y direction. Peak ground acceleration of Gorkha Earthquake, EI Centro Earthquake and Kobe Earthquake are use for time history analysis. Different twelve ground motions are used for analysis to obtain results of max displacement, base shear and max storey drift. The results concluded that model having irregular plan with different structure size is more seismically strong. Similarly building models have less value of design value than demand value during nonlinear dynamic analysis, thus demanding the need to rectify and strengthen building models.

Keywords— *RC Building, Ground Motion, Earthquake, Peak Ground Accelerogram*

1. INTRODUCTION

The tectonic plate movement have resulted in the generation of large number of earthquake and also forms the Himalayan mountain ranges. The variation in ground motion causes the variation on earthquake intensity which will change the performance of the low rise buildings. During the study low rise regular and irregular building structure as per different pattern of building construction is considered. Irregular building are of two types' plane and vertical irregularity. Based on the pattern of construction of building on numbers of building is considered as sample for the study. Earthquakes can neither be prevented nor predicted precisely, but the large-scale destruction can be minimized by employing seismic-resistant measures in buildings. Reinforced concrete building construction increased drastically over the last few decades in the major urban centers in countries like Nepal to meet the rapidly increasing settlement of the region. Reinforced concrete construction commenced around four decades ago as an alternative to traditional unreinforced masonry (URM) buildings that lack structural integrity and ductility.

Nonlinear analysis methods are best applied when either geometric or material nonlinearity is considered during structural modeling and analysis. Nonlinear analysis is of two types: dynamics and statics. Nonlinear static analysis means pushover analysis, and dynamics nonlinear analysis means time history analysis. Non-linear dynamic time-history analyses conducted as part of a performance-based seismic design approach often require that the ground motion records are scaled to a specified level of seismic intensity. If only elastic material behavior is considered, linear analysis methods should suffice, though P- Delta formulation may still be applied. Linear and nonlinear methods may be static or dynamic. Nonlinear time-history analysis is a central ingredient of the collapse assessment, the accuracy of which depends on how faithfully the model captures the strength and stiffness degradation that can lead to structural collapse. Seismic performance of building of Nepal after earthquake and have concluded that RC buildings that were not properly designed to resist the seismic forces and suffered extensive damage, including partial or complete collapse, mostly due to vertical irregularities in their construction that caused stiffness differences and subsequent soft-story mechanisms, as is often associated with no engineered (Dumaru et al.,2018). Nonlinear response history analysis for validating proposed design of new or performance assessment of existing structures, during their study the seismic demands are determined by nonlinear RHAs of the structure excited by several ground motion acceleration records and test results based on three-dimensional computer models of idealized 5, 10, 15 and 20-story reinforced concrete structures demonstrate that the proposed method is not only viable, but also capable of controlling discrepancies in estimates of engineering demand parameters (EDPs) such as peak roof displacement (Reyes et al.,2019).

Seismic analysis on mass and stiffness variation of models and found that with increase in the column stiffness the axial forces in column and base shear of the building increase and top storey displacement is more in building where there is more mass on the top storey resulting in increase of lateral forces (Tiwari et al.,2020). Seismic performance is evaluated with regard to global strength, stiffness, energy dissipation, inter-storey drift, and total deflection of the structure and their results show that masonry infill increases the global strength and stiffness of the structures; it decreases the inter-storey drift and hence the total displacement of the structure (Chaulagain et al.,2016). A seismic hazard zone and a set of near source three component accelerogram were used and scale as per code, and series of nonlinear time history is conducted and roof displacement, acceleration and base shear force and formation trend of plastic hinges were calculated and concluded that for some earthquake building performance exceed LS PL and even in some cases they reach collapse level (Mahmood et al., 2017)

Ground motion has short period components in the period range of T < 0.5 s as well as long-period components in the period range of 4–6 s and this characteristic is unique with respect to recorded ground motions in other parts of the world and with respect to the design response spectra in the Kathmandu region. Effects of this particular composition of frequency contents are discussed within the context of both elastic and inelastic response spectra. Common ground motion parameters for this earthquake have been investigated to compare the destructiveness of this earthquake to other historical earthquakes (Whitney et.al. 2015).

2. NUMERICAL MODELLING USING ETABS

In the present study different 10 low rise reinforced concrete moment resisting frame building models is considered, RCC low rise building is considered which is of different floor plan and shape. The top floor have less number of columns compared to the other floors. The plan of low rise building is shown in Fig. from 1 to 9. Fig. 10 shows the typical 3D of the building. Different building configuration is used to compare the seismic behavior among different configuration of building. On assigning different load and load combination there is variation on plan and configuration in buildings.

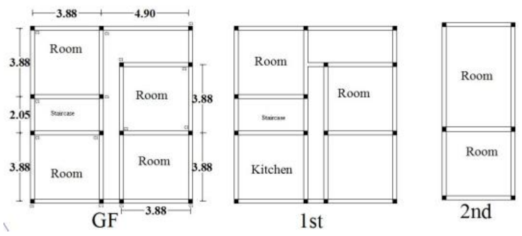

Fig. 1: Plan of the model 1building

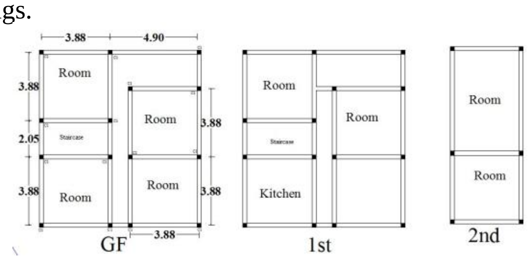

Fig. 2: Plan of the model 2 building

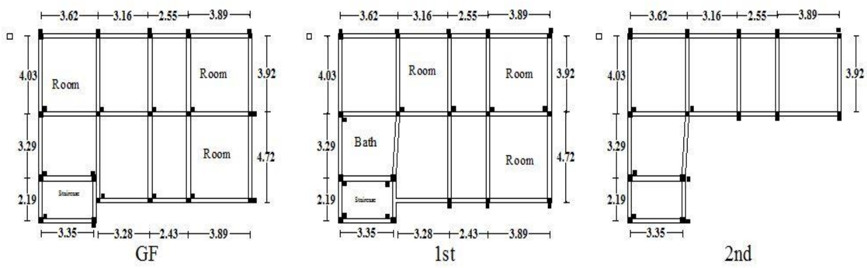

Fig. 3: Plan of the model 3 building

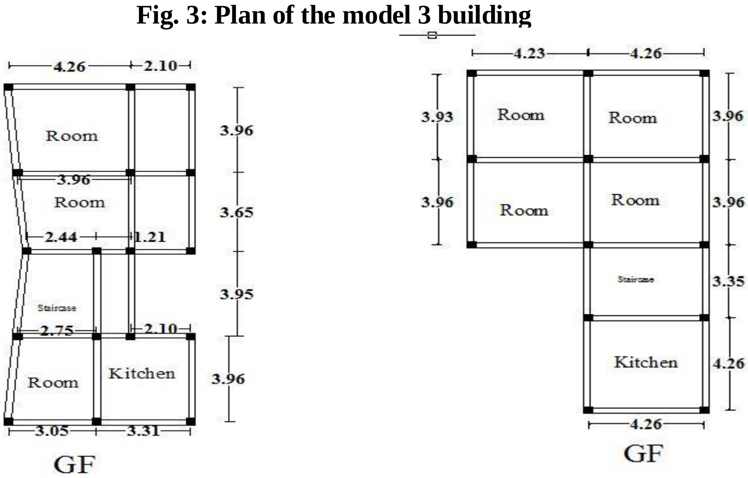

Fig. 4: Plan of the model 4 & 5 building

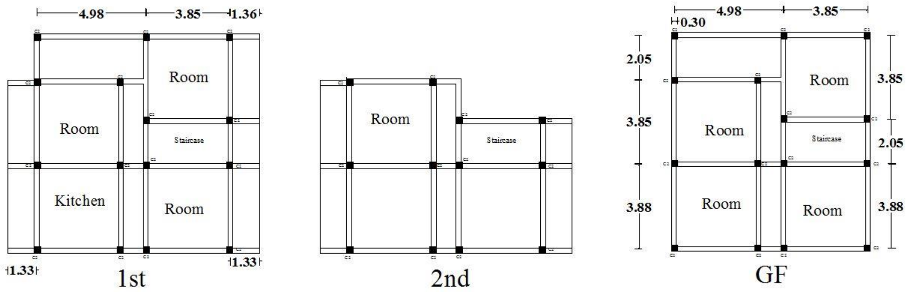

Fig. 5: Plan of the model 6 building

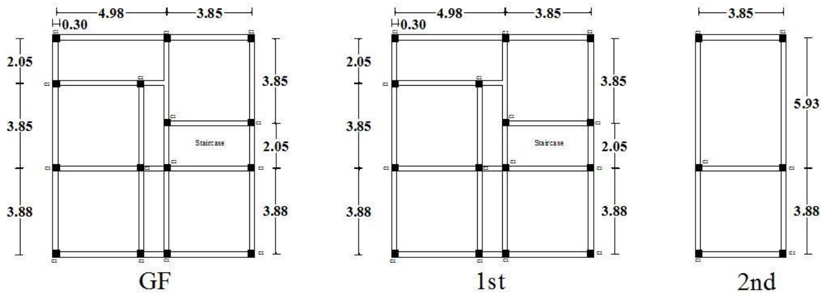

Fig. 6: Plan of the model 7 building

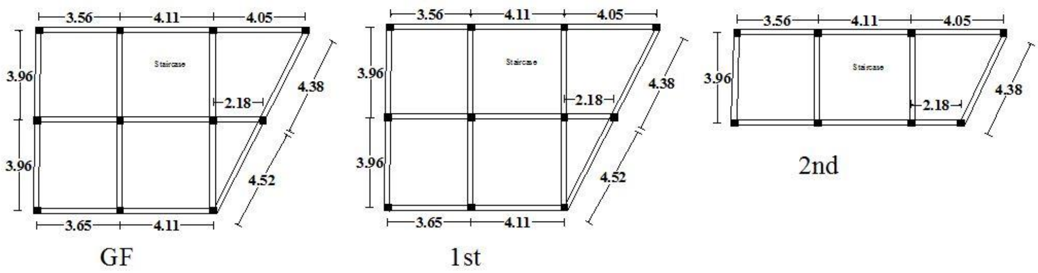

Fig. 7: Plan of the model 8 building

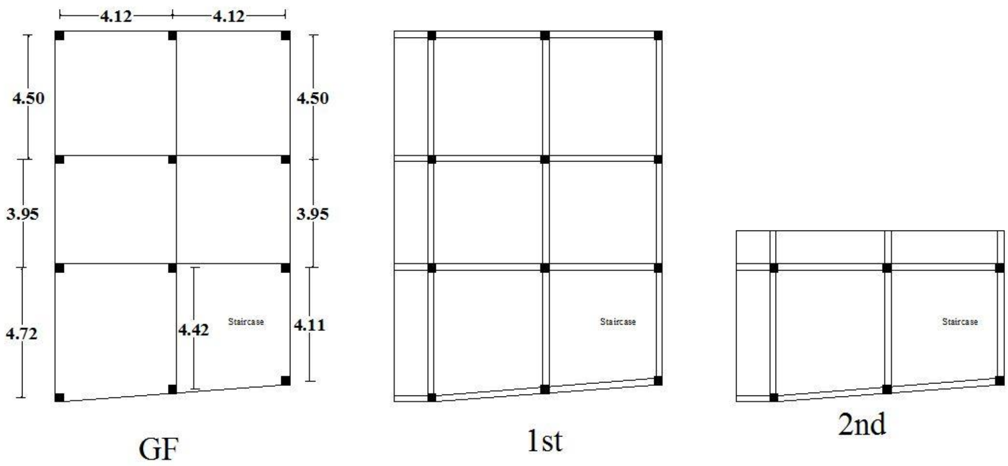

Fig. 8: Plan of the model 9 building

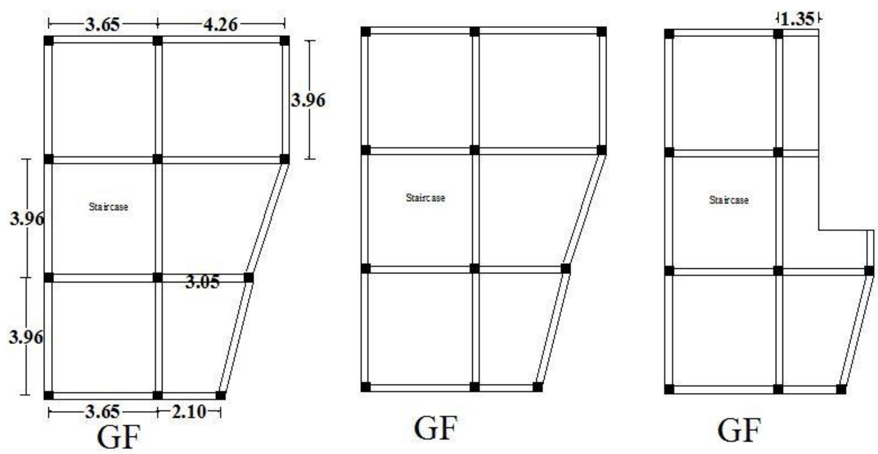

Fig. 9: Plan of the model 10 building

The numerical modelling is completed using ETABS, ten different configurations of low rise buildings each having 3 storey are modelled. Table 1 shows the details of the structural parameters considered in the numerical modelling. Beams and columns are modelled as the frame element while slab is modelled as a membrane element. During the modelling on linear and nonlinear static analysis response spectrum is used. For execution of nonlinear pushover analysis model is made nonlinear by varying nonlinear parameter and defining nonlinear hinges to models. For nonlinear time history analysis different 13 scaled ground motion were used.

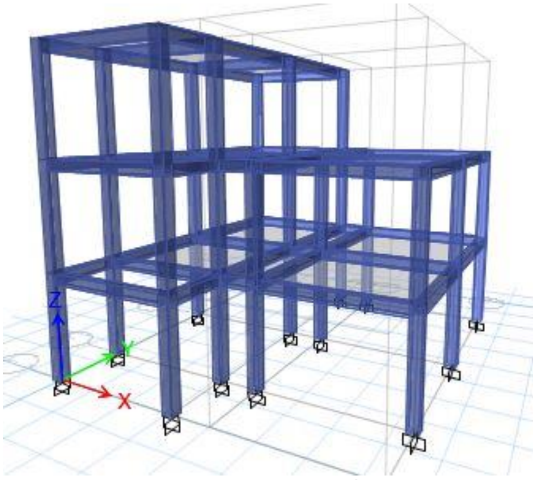

Fig. 10: Typical 3D model of the building

Models have square column of size 230mm × 230 mm except model 7 where column size is 300mm × 300mm. Similarly all models have rectangular beam size of 230mm × 300mm and slab of thickness of 125 mm. The building is considered to be in seismic zone V with importance factor 1. Concrete grade of M15 and reinforcement of Fe500 is assigned to all the frame members. Medium type of soil is considered in the numerical modelling. External load, internal load and parapet wall load of 8.5 kN/m, 5.5 kN/m and 3.9 kN/m respectively is applied to the building. Floor height of 3 m and live load of 2 kN/m2 is assigned on building. For nonlinear time history analysis mass and stiffness parameter for different models is calculated and assigned on model. For introduction of ground motion on models, ground motion is scaled as per response spectrum of modes as per Indian standard code.

3. RESULTS AND DISCUSSIONS

The ten different low rise RC building models are analyzed and the data are extracted in the form of drift, displacement, base shear, capacity curve. Different building models exhibits different types of behavior due to the irregularities. The results of the each parameters are discussed below.

3.1 Comparison of models with respect to pushover analysis.

After push over analysis, table 1 shows displacement, drift and base shear of the model in push in X and push in Y direction. Model 5 have maximum displacement in X direction since Model 5 is L shape model, having L center of gravity and center of mass is not in same place so there is certain change in position which makes model irregular and unsafe. Model 1 and Model 2 have maximum displacement on Y direction which have irregular plan, column were not in same axis which cause eccentricity. As elastic drift ratio limit is 0.4% but inelastic drift limit is 0.2%, Model 1 to Model 10 were on inelastic limit of drift. Model 2 have maximum base shear in X direction which show it is unsafe model than other models. Model 7 have second maximum as Model 2 and Model 7 have same plan but different column size. Base shear help to estimate the maximum expected lateral force which will occur due to ground motion at the base of the structure. The presence of max base shear indicates the presence of maximum lateral forces which makes the building unsafe.

Table 1: Result after pushover analysis

Models /Direction	Dislacement (mm)		Drift		Storey Shear at Base	
	PaX	PaY	PaX	PaY	PaX	PaY
1	19.517	22.791	0.0087	0.001029	206.67	247.237
2	31.689	32.303	0.00165	0.00156	372.74	362.74
3	29.173	27.772	0.001168	0.001102	384.38	327.458
4	25.958	28.158	0.00109	0.001159	350.537	367.17
5	32.616	31.916	0.00156	0.001319	290.18	330.4221
6	27.718	27.274	0.001497	0.001174	305.807	288.1178
7	17.171	14.212	0.001041	0.00817	371.1598	304.8665
8	29.73	33.07	0.001241	0.001512	258.1301	263.441
9	28.645	28.394	0.001359	0.001302	246.99	254.2265
10	26.497	27.436	0.001191	0.001219	246.894	249.845

After nonlinear push over analysis different hinges were formed on models 1 to 10. Table 2 shows hinges details on model during pushover analysis in X and Y direction. Model 6 have formation of red color of hinges on column which shows model have weak column than beam. There is formation of hinges on column 1st on model 10. Design concept of building include stronger column weak beam but model 6 and model 10 have weak column which means model 6 and 10 where seismically venerable building. Model having green color of hinges indicate models were on collapse prevention limit. Model 7 have the maximum deviation in the Base Shear and Displacement compared to the other building models as shown in Table 3 and Table 4.

Table 2: Hinges Details

Models	Hinges Details	
	PaX	PaY
1	Formation of green color hinges on column near the staircase on 1st step.	Formation of green color hinges on column and beam.
2	Formation of green color of hinges on 1st step and in second step formation on columns and beam of ground floor and 1st floor.	Formation of pink color of hinge on ground floor beam and on last step formation of blue color of hinges on beam and then in column.
3	Formation of green color of hinges on beam on 1st step and then in column.	Formation of green color hinges on column and beam.
4	Formation of green color hinges on beam on 1st step of 1st floor and then in column.	Formation of green color of hinges in column on 1st step and then in beam.
5	Formation of green color hinges on beam on 1st step of 1st floor and then in column.	Formation of green color hinges on beam of 1st floor and then in columns on 2nd step.
6	Formation of green color hinges on column on 1st and 2nd step and then in beams.	Formation of green color hinges on column on 1st step and formation of pink color of hinges on beams and then in red color.
7	Formation of green color hinges on column on 1st step and then in beam. On last step there is formation of blue color of hinges on column.	Formation of green hinges on staircase beam in 1st step and then in column.
8	Formation of green hinges on beam 1st and then in column.	Formation of green color hinges on beam near the staircase and then in other column.
9	Formation of pink color hinges on column and green color of hinges on staircase beam.	Formation of green color of hinges on column in 1st step and then in beams.
10	Formation of green color hinges on Colum in 1st step and then in column.	Formation of blue color of hinges on column and green color of hinges on beams.

Table 3: Displacement of models after push over analysis

Models /Direction	Displacement (mm)	Maximum Displacement (mm)	Increase Percent in Displacement
1	19.517	57	65.00
2	31.689	54	41.29
3	29.173	67	56.41
4	25.958	52	50.00
5	32.616	59	44.74
6	27.718	88	69.09
7	17.171	100	82.80
8	29.73	66	56.00
9	28.645	95	69.80
10	26.497	91	70.81

Table 4: Base shear of models after push over analysis

Models /Direction	Base Shear of Models (kN)		Max Base Shear (kN)		Increase in Percent	
	X	Y	X	Y	X	Y
1	206.67	247.237	330	375	37.3	34.08
2	372.74	362.74	620	650	39.8	44.2
3	384.38	327.458	760	570	49.42	42.54
4	350.537	367.17	510	520	31.27	29.38
5	290.18	330.4221	515	470	43.65	29.7
6	305.807	288.1178	465	400	34.23	27.9
7	371.1598	304.8665	720	570	48.44	46.5
8	258.1301	263.441	475	440	45.66	40.13
9	246.99	254.2265	392	383	36.98	33.62
10	246.894	249.845	373	282	33.8	11.41

3.2 Comparison of models with respect to nonlinear time history analysis.

After conducting nonlinear time history analysis, figure 11 shows the displacement on models on El Centro, Gorkha and Kobe earthquake of peak ground acceleration of 0.2188g, 0.3447g and 0.177g respectively. Model 6 shows maximum displacement and model 8 shows minimum displacement on X direction during introduction of El Centro ground motion but model 2 have maximum displacement on X direction during introduction of Kobe ground motion. Since El Centro have more peak ground acceleration than Kobe ground motion but time period of excitation of Kobe is fast than electro so there is max displacement on Kobe earthquake, whereas model 4 have minimum displacement Gorkha ground motion.

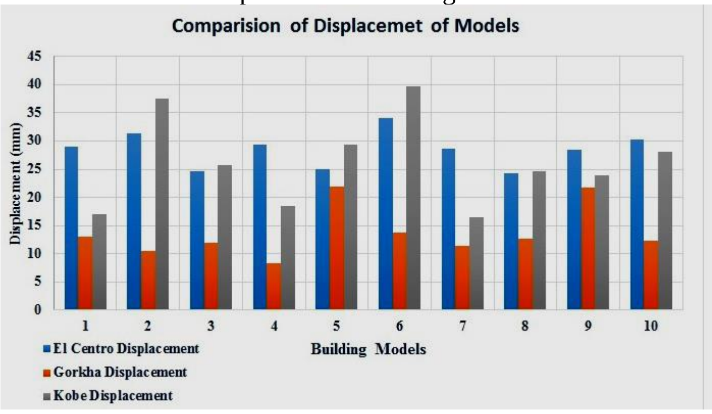

Fig. 11: Displacement of models

After conducting nonlinear time history analysis, figure 12 shows the drift on models on El Centro, Gorkha and Kobe earthquake of peak ground acceleration of 0.2188g, 0.3447g and 0.177g respectively. Elastic drift of model is 0.4% (0.004) and here design drift inelastic drift is 0.2% (0.002) .Model 7 exceed drift limit on El Centro. Gorkha ground motion drift is within inelastic limit but in Kobe model 6 maximum drift ratio. As per (Elnashai. et.al. 2015) there is no damage up drift limit 0.2 to 0.5%.

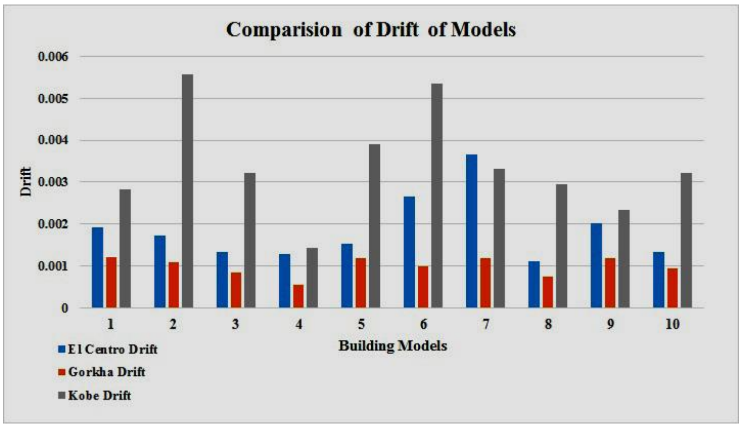

Fig. 12: Drift of models

After conducting nonlinear time history analysis, figure 13 shows the base shear on models on El Centro, Gorkha and Kobe earthquake of peak ground acceleration of 0.2188g, 0.3447g and 0.177g respectively. Model 7 have maximum base shear on electro and model 10 have minimum base shear value, model having lower base shear have low resistance to lateral force during nonlinear time history analysis. Model 7 show maximum value of base shear on Gorkha and Kobe ground motion. Model 8 show minimum base shear on Gorkha ground motion and model 10 shows minimum base shear on Kobe ground motion.

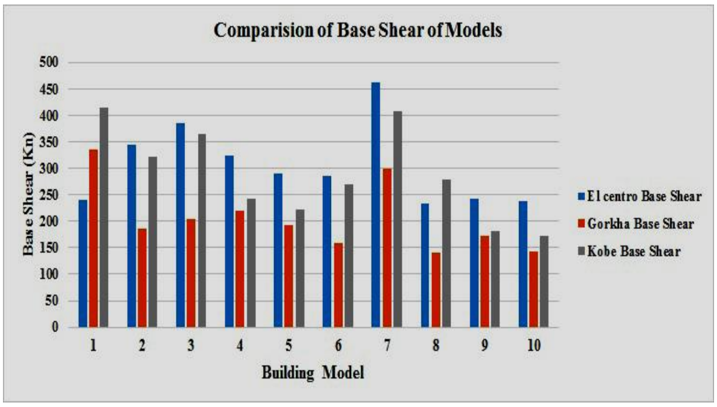

Fig. 13: Base Shear of models

3.3 Comparisons of models with respect to Demand, Design and Capacity.

After model execution, different parameter value is obtained, figure below shows the demand, design and Capacity of the models on different ground motion. Capacity of models is more than design and demand of the models but in some model demand is more than design. In model 1 demand is more than design. Model 1 need to be strengthen. Displacement demand is more than displacement design which need to change design of model 6 by strengthening model. Figure 14, 15, 16 shows demand, design and capacity of models with respect to displacement of models on El Centro, Gorkha and Kobe ground motion. Model 3, 5 and 8 have less demand then design displacement which indicates model 3, 5 and 8 were safer than other models.

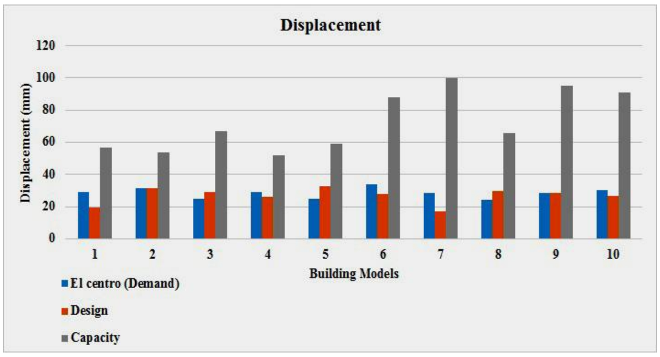

Fig. 14: Displacement of model on El Centro

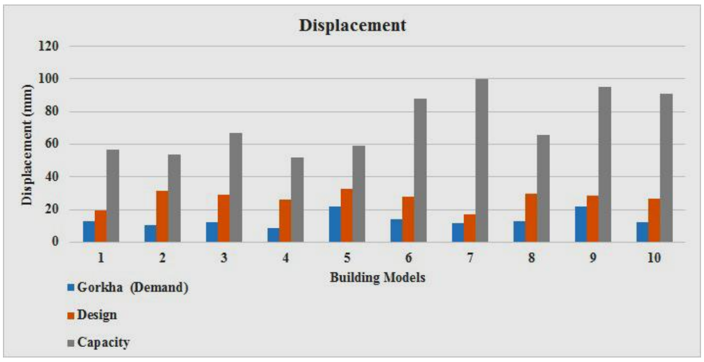

Fig. 15: Displacement of model on Gorkha

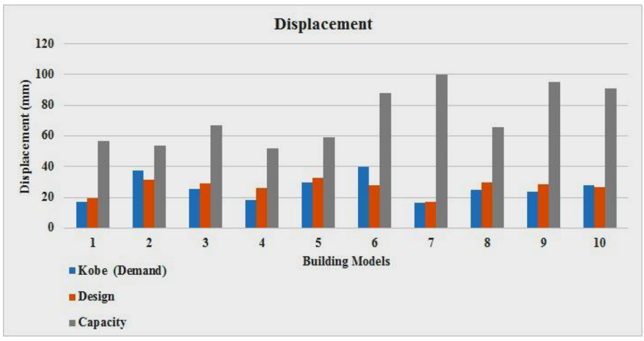

Fig. 16: Displacement of model on Kobe

3.4 Comparison of drift of models after nonlinear time history analysis.

During model execution of 12 different ground motion of different PGA, different drift ratio on different storey level is obtained. As per Amr.s (Elnashai.et.al. 2015) there is mild damage on drift (%) within 0.2 to 0.5, and there is moderate damage when drift (%) is within 0.5 to 1.5 and sever damage when drift (%) within 1.5 to 3.0. Building having more drift indicated building is in damage state. In model 1, there is maximum drift of 0.38 which is in limit of 0.2 to 0.5 so there is mild damage on model during earthquake ground motion. But average value of drift is within inelastic limit. Model 2 have average value of 0.26. Model 3 have average drift 0.29 which is more than in elastic drift limit so there is damage on model on 1st floor level. Model 4 have average value of drift of 0.2 which shows model is in inelastic limit but in some ground motion there is more value of drift value. Model 5 have average drift 0.4 which is more than in elastic drift limit so there is damage on model on 1st floor level. Model 5 have maximum drift value of 0.8% on 0.278 PGA ground motion due to long range of ground motion shaking. Model 6 have average value of drift of 0.31 which shows model is in not inelastic limit. Model 7 have average drift 0.13 which is on inelastic drift limit so there is no damage on model on 1st floor level. Model 7 have minimum drift ratio. Model 8 have maximum drift value of 0.54% on 0.278 PGA ground motion due to long range of ground motion shaking. Model 8 have average value of drift of 0.35 which shows model is in not inelastic limit. Here model 9 have average drift 0.35 which is not in inelastic drift limit so there is mild damage on model. Model 9 have maximum drift value of 0.86% on 0.278 PGA ground motion due to long range of ground motion shaking. Model 10 have average value of drift of 0.28 which shows model is in not inelastic limit.

3.5 Comparison of models using different 12 set of ground motion.

After conducting nonlinear time history analysis on 12 different ground motion having different PGA .Figure 17 shows average displacement on models. During analysis model 5 shows maximum displacement of 24.78 and model 7 have minimum displacement of 10.60mm. But model 1 have maximum base shear value of 260.83kn and model 10 have minimum base shear value of 142.31.

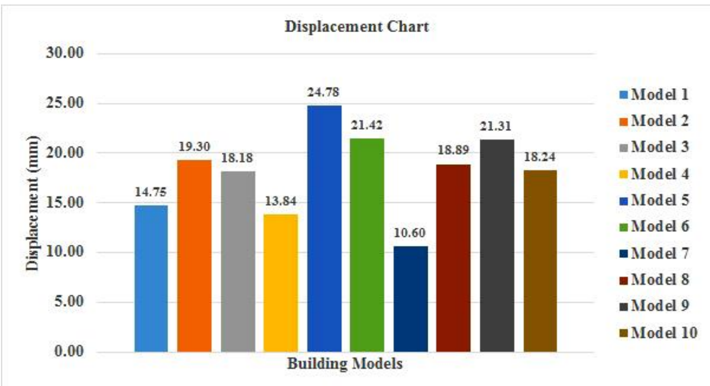

Fig. 17: Average displacement of models

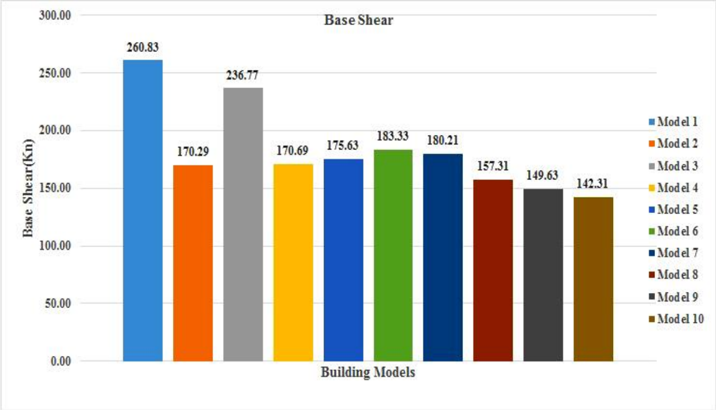

Fig. 18: Average base shear of models

4. CONCLUSIONS

In the present study, after linear and nonlinear modeling of 10 different model of low rise building of Pokhara valley and different parameter of study was done. Max storey displacement, storey drift and storey shear value on both direction X and Y is obtained from linear and nonlinear static and dynamic analysis on models.

(a) Model 5 Shows maximum time period during linear static analysis, as model 5 is in L shape irregular building.
(b) Model 6 have cantilever projection, due to this irregular behaviour it has max top displacement than other irregular models.
(c) The formation of hinges depends on demand to capacity ratio (DCR). The members in which the seismic demand exceeds its capacity, hinge formation takes place. Most of the model have green color hinges which indicated collapse preventions of models. Duration exaction model 7&10 have blue color hinges on columns which indicates strength hardening point. Duration exaction model 9 have pink color hinges on columns which indicates Collapse line. Duration exaction model 6 have red color hinges on columns which collapse, after which the column will collapse. In model where there is hinge formation first in column then in beams its means model have weak column.
(d) From nonlinear static analysis, model 7 have max increase in displacement of 82.8% but have 48.44% increase in base shear value as it have higher section of column then other model and model 3 have max increase in base shear value of 49.42% and have 56.41% increase in displacement, which shows model 7 is more seismically strong than other models. And model 2 have less increase % in displacement as it is seismically weak model.
(e) From nonlinear dynamic analysis, model 5 have maximum displacement and model 7 have minimum displacement and Drift ratio limit of model 7 is within inelastic limit then other models, which show model 7 is seismically strong than other models.
(f) Model 1 have more demand than design of model, so model 1 is week with respect to other models. Model having more demand than design value, model is to be strengthen.

Conflict of interest

There is no conflict to disclose.

5. ACKNOWLEDGEMENT

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