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Original Research Article

Seismic Behaviour of the Low-Rise RC Buildings in Nonlinear Static and Dynamic Analysis

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Abstract

This paper presents the behavior of three different types of irregular low-rise buildings, subjected to earthquake load. The study is performed by numerically modelling the buildings for the linear static analysis. Structural parameters displacements, drift, and storey shear are checked for various time periods of the building. The same models are also analyzed using nonlinear pushover analysis. The model is made nonlinear by introducing the hinges in the beam and column. The execution of nonlinear analysis is done by applying push in X and push in Y directions in controlled displacement mode. After the execution of nonlinear pushover analysis, different colours of hinges were formed, which were used as a basis for the study. The parameters like maximum displacement, max storey drift, and storey shear were computed in both in X and Y directions. Peak ground acceleration of Gorkha earthquake, EI Centro earthquake, and Kobe earthquakes was used for time history analysis. The results for max displacement, base shear, and max storey drift are presented and the comparison is made for the different building models. The study showed that a building behaves well in seismic loading even though they have an irregular plan with a larger structure size, compared to a building that has a regular plan and a smaller structural member size.

Keywords: RC Building, Ground Motion, Earthquake, Peak Ground Accelerogram.

I. INTRODUCTION

Depending upon the nature of the variable considered, the analysis approach can be classified as: deterministic and probabilistic. In deterministic analysis, specific values of the demand and properties of the system are used, while probabilistic methods acknowledge the uncertain nature of those parameters. In the latter, probability distribution of the variables is used, thus permitting to quantify of the reliability of the result. Probabilistic methods are common in the earthquake analysis and design of the structure, as well as in those areas that provide/require an extra dimension for the interpretation and solution of the problem. By their use, computed element forces, stresses, and deformation of any other response quantities not only assume a value but also a probability associated with it. This becomes essential in the seismic analysis of buildings where important uncertainty exists in the input ground motion and properties of the soil, material, and structure. In the performance-based seismic design of structures, the use of nonlinear response history analysis has gained the most importance. It has become a prerequisite factor for controlling the level of structural and non-structural damage during an earthquake. This method of analysis requires different parameters as input along with the recorded ground motions. In the case of three-dimensional non-linear response history analysis, the pairs of records were used. Each of the models was checked for collapse capacity by evaluating it with nonlinear time history analysis under a set of prescribed ground motions. The ground motions are scaled to reflect the specified earthquake ground shaking intensities. The earthquake-induced collapse behaviour is accounted by the nonlinear assessment for tall the likely modes of stiffness and strength reduction.

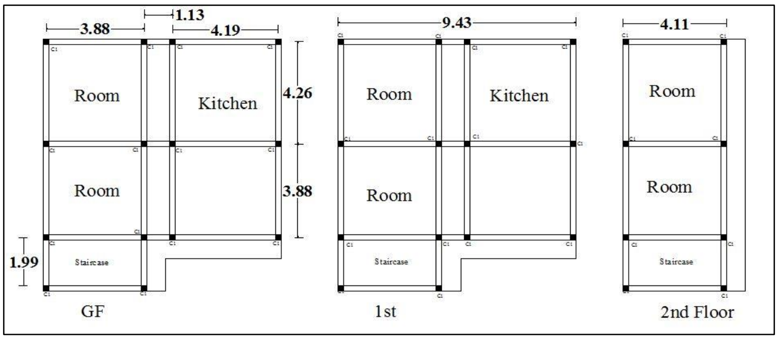
II. LITERATURE REVIEW

Nonlinear analysis methods are suitable in structural modelling and analysis when there is the presence of either material or geometric nonlinearity. Broadly there are two types of nonlinear analysis i.e. nonlinear dynamic analysis also known as time history analysis and nonlinear static analysis also known as pushover analysis. Time history analysis is conducted, in the performance-based design approach, whenever the ground motions are scaled to a certain level of shaking intensity. However, when the elastic material behavior is only considered then the linear analysis method is sufficient for the analysis, although P-Delta effect consideration may still be made. Nonlinear time history analysis can be used for more precise calculation and its accuracy depends upon the degree of accuracy of the strength and stiffness calculation which are the prime cause of the structural collapse.

There is always a discrepancy, either on a small or a large scale, in the recommended building design code provision and building construction practices. Chaulagain *et al.*, (2013), studied the seismic response of the RC buildings in Nepal by considering current construction practices, Nepal National Building Code, and well-designed structures. The results reported the variations in the building performance due to the various above-mentioned parameters. The seismic behaviour of the two existing buildings was studied by Mosleh *et al.*, in 2016 by performing the pushover analysis and time history analysis. It was observed that the different structural parameters were responsible for the seismic performance of the building. Chaulagain *et al.*, (2016) presented an intensive case study of existing RC buildings in Nepal, during this case study they use non-linear analyses on a bare frame with masonry infill. The five buildings with three storey each having different structural configurations and detailing were selected to evaluate the failure mechanism due to the influence of infill walls on the buildings. Seismic performance was evaluated with relation to global strength, stiffness, energy dissipation, inter-story drift, and total deflection of the structure. The results show that masonry infill increases the global strength and stiffness of the structures and decreases the inter-story drift, thus influencing the overall displacement of the structure. Varum *et al.*, (2018), have performed a seismic evaluation of the performance of the buildings in Nepal after the Gorkha earthquake and concluded that most of the damages and collapse are mostly due to the vertical irregularities in the construction, which attributed to the stiffness differences and subsequently leading to the soft story failure mechanisms. Reyes *et al.*, (2017) used nonlinear response history analysis for validating the proposed design of new or performance assessment of existing structures. The idealized models of different multistoried RC (5, 10, 15, and 20) buildings were considered and the seismic demands were determined by nonlinear response history analysis of the structure. The building was excited by several ground motion acceleration records and the test results based on 3-D computer models demonstrated that the proposed method was viable and capable of controlling discrepancies in estimates of engineering demand parameters like peak roof displacement. Tiwari and Adhikari, (2020) have done seismic analysis on mass and stiffness variation of models and found that with an increase in the column stiffness, the axial forces in column and base shear of the building increase and top story displacement are more in the building where there is more mass on the top storey resulting in the increase of lateral forces. The study on the ten different irregular buildings by Tiwari *et al.*, (2020) showed that the building with various sizes of structural members performed well during the seismic analysis. Shah *et al.*, (2021) performed the linear and nonlinear static analysis of RC buildings with and without the shear wall. It was observed that the building with a shear wall performed well when subjected to seismic loads. Chhetri and Adhikari (2021) studied the linear static method for the calculation of different parameters due to earthquake load for low-rise buildings which are generally constructed in the hilly regions of Nepal. They concluded that the shear wall shares the column loads in an effective way to reduce the seismic vulnerability associated with hillside buildings.

III. NUMERICAL MODELS OF BUILDING USING ETABS

In this study, the numerical models of three different low-rises, reinforced concrete, and moment-resisting frame building, models are considered. Each model has a different floor plan and shape. The plan of the model considered is shown in Figure 1. The size of the column is made smaller on the top floor compared to the size of columns on other floors. Figure 2 shows the typical 3D model of the building. The seismic behavior is compared between the models by assigning different loads value and load combinations in the plan of various buildings.



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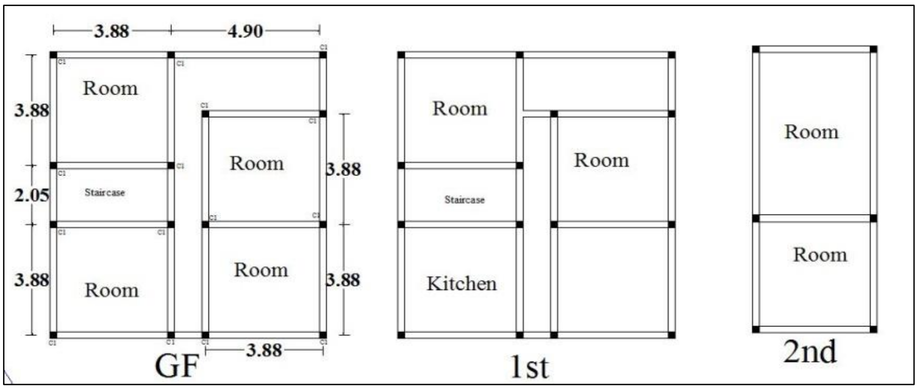

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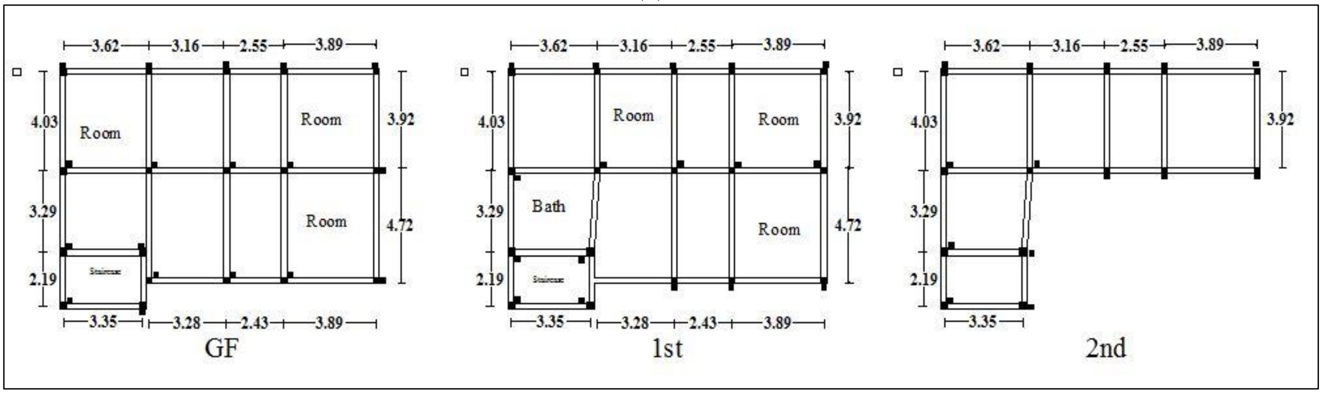

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Figure 1: Plan of the buildings for (a) Model 1, (b) Model 2, (c) Model 3

The numerical modelling of the three different configurations of low-rise buildings for three storey is completed using ETABS. The slab is modelled as a membrane element and the beams-columns are modelled as a frame element. The nonlinear parameters given in Table 1 were introduced to study the nonlinear parameters of the model. Paramters in Table 1 was calculated as per period of different models to introduce model dampness using general formula. The response spectrum method, a nonlinear static analysis method, was used to model the building. For performing the nonlinear pushover analysis, the building models are made nonlinear by varying the nonlinear parameters and defining nonlinear hinges to models as shown in Figure 3. Similarly, the three different ground motions were used to perform the nonlinear time history analysis.

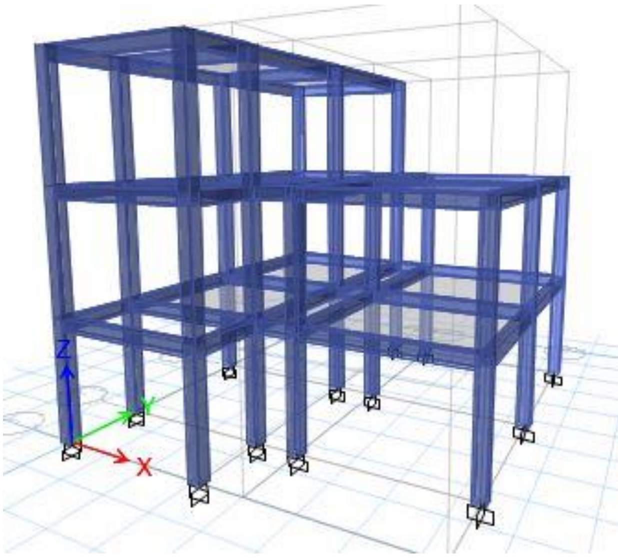


Figure 2: Typical 3D model of the building

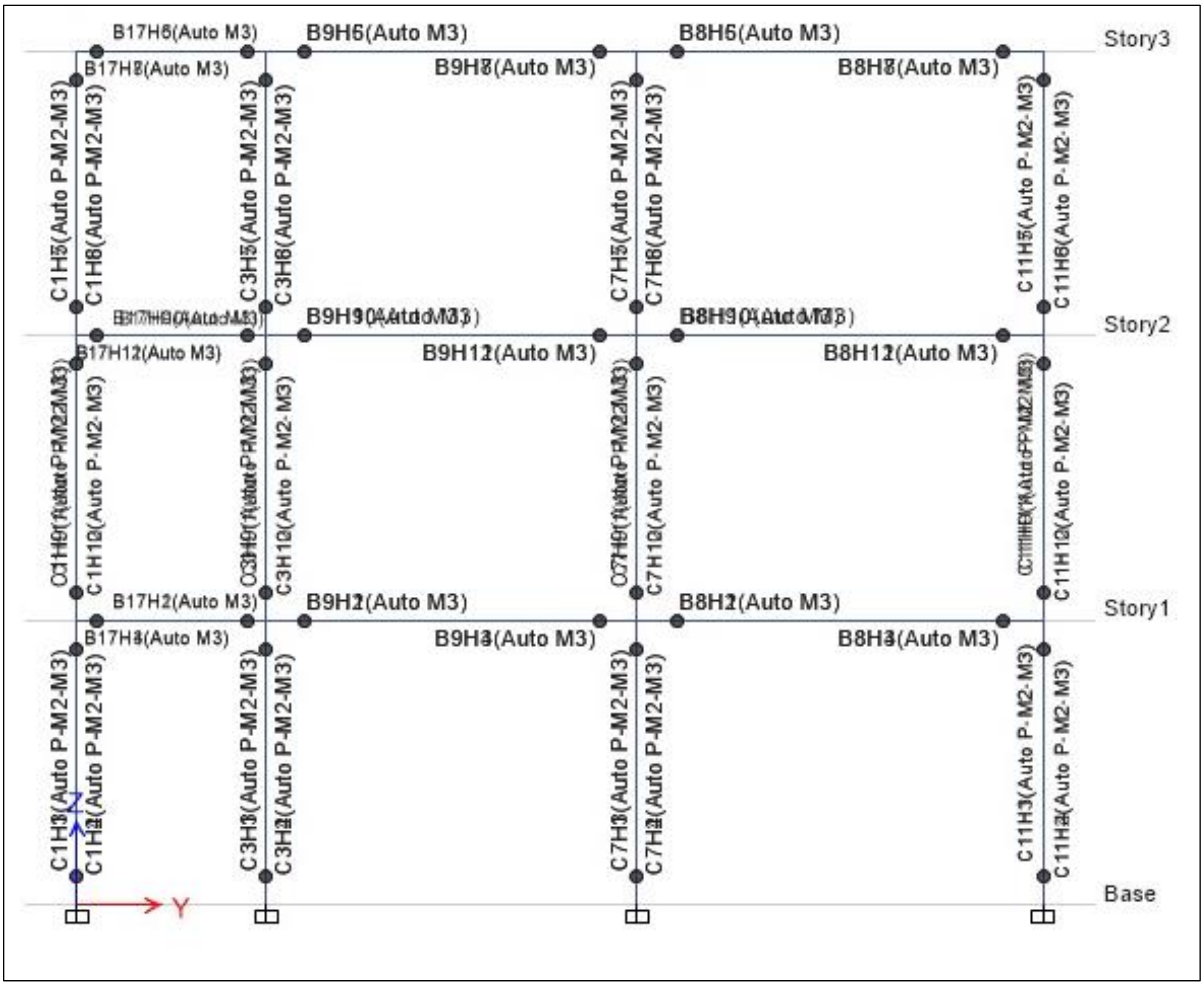
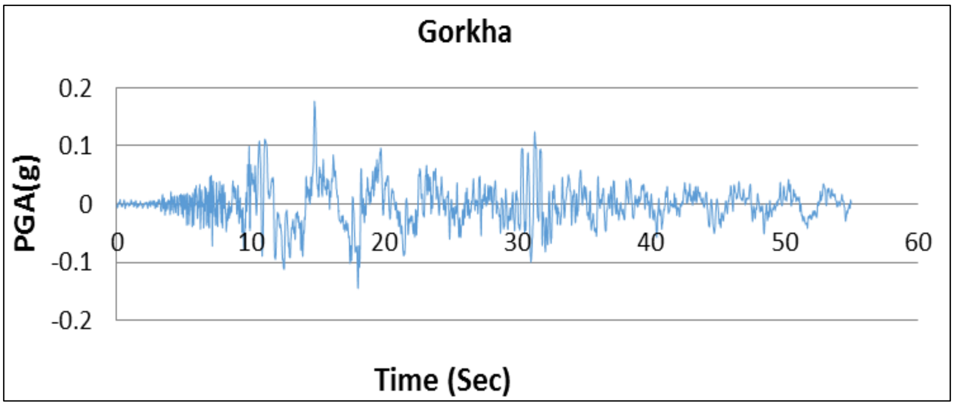


Figure 3: Hinges defined on the building model

The building model considered in the numerical analysis consists of a rectangular beam size of 230mm × 300mm and a slab of thickness 125 mm. All the columns are considered square with a size of 230 mm. An importance factor of 1 is assigned to all the models considered and the buildings are assumed to lie in seismic zone V. M15 grade of concrete and Fe500 grade of rebar are considered on all the frame members. The medium type of soil is considered for the analysis. The equivalent dead load of the parapet wall, external wall, and internal wall of 3.9 kN/m, 8.5 kN/m, and 5.5 kN/m respectively are applied to the building models. The center-to-center floor height is considered as 3 m and a live load of 2 kN/m^2 is assigned to the building models. For nonlinear time history analysis mass and stiffness parameter for different models is calculated and assigned to the model as shown in Table 1. The ground motion is scaled according to the response spectrum of modes as per the Indian standard code to introduce a ground motion in the building models. Ground motion Gorkha, Kobe, and El Centro Ground motion considered in the building models is shown in Figure 4.

Table 1: Calculation of mass and stiffness for modal damping

Model	Time Period T1	Time Period T2	Damping Constant ξ	Frequency ω1 (2π/T1)	Frequency ω2 (2π/T2)	Stiffness Coefficient δ = 2ξ/(ω1+ω2)	Mass Coefficient η =ω1*ω2* δ
1	0.818	0.0818	0.05	7.677	76.773	0.00118	0.698
2	0.739	0.0739	0.05	8.498	84.980	0.00107	0.773
3	0.802	0.0802	0.05	7.830	78.304	0.00116	0.712



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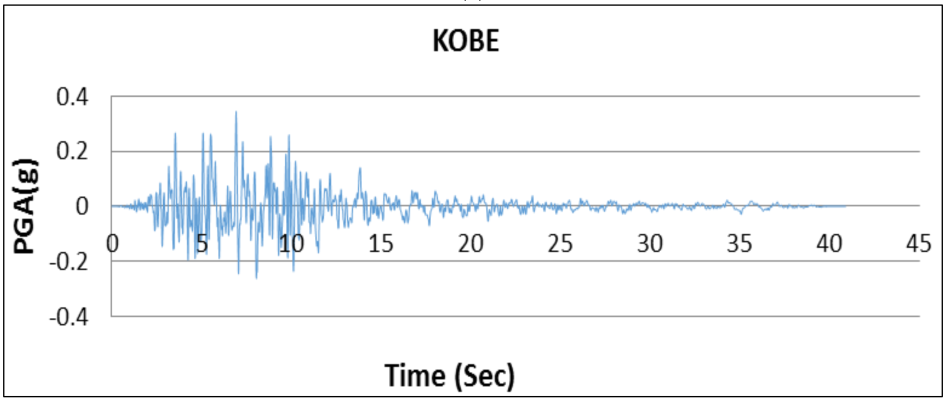

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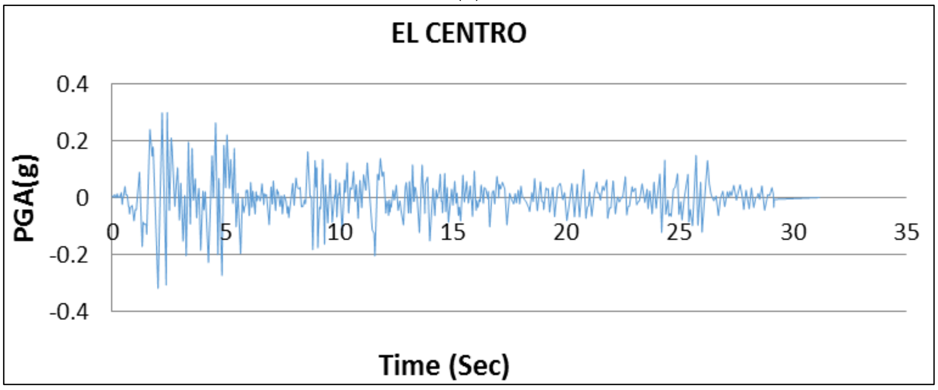

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Figure 4: Time history graph for different earthquakes (a) Gorkha (b) Kobe (c) El Centro (Source: Peer Ground Motion)

IV. RESULTS AND DISCUSSIONS

The three different low-rise RC building models with plan irregularities are numerically analyzed. Results are presented in the form of base shear, displacement, and capacity curves. Various building models exhibit different behavior due to the irregularities inherited in them. The results of each parameter are discussed in the subheading below.

Comparision of Models with Respect to Pushover Analysis

After performing the pushover analysis on the building, the results are obtained for the displacement, drift, and base shear for all the models. Table 2 shows the results of the model in push in X and push in Y directions. The elastic drift ratio limit for the building is 0.4% whereas the inelastic drift limit is 0.2%. All the building models were within the inelastic drift limit. Base shear is the maximum expected lateral force that will occur due to the ground motion at the base of the structure. The large value of base shear indicates the presence of maximum lateral forces in the building making the building unsafe when subjected to seismic load. Thus, the higher the base shears of the building, the higher the seismic vulnerability of the building will be. Model 3 has maximum base shear in the X direction which indicates that it is an unsafe building model when compared with the other two models.

Table 2: Displacement drift and Base Shear

Models /Direction	Displacement (mm)		Drift		Base Shear	
	PaX	PaY	PaX	PaY	PaX	PaY
1	19.5	22.8	0.0087	0.00103	206.7	247.2
2	31.7	32.3	0.00165	0.00156	372.7	362.7
3	29.2	27.8	0.00117	0.0011	384.4	327.5

The different hinges (indicated by various colour in Figure 5) were formed in the building models after performing the pushover analysis. Table 3 shows hinges details on the model during pushover analysis in X and Y directions. Model 1 has a formation of green color of hinges on a column which shows the model has a weak column than the beam. The first hinge is formed on the column in Model 2 which indicates column was weak than beam. Model 1 has weaker column than beam. Models having the green color of hinges indicate models were on collapse prevention limit.

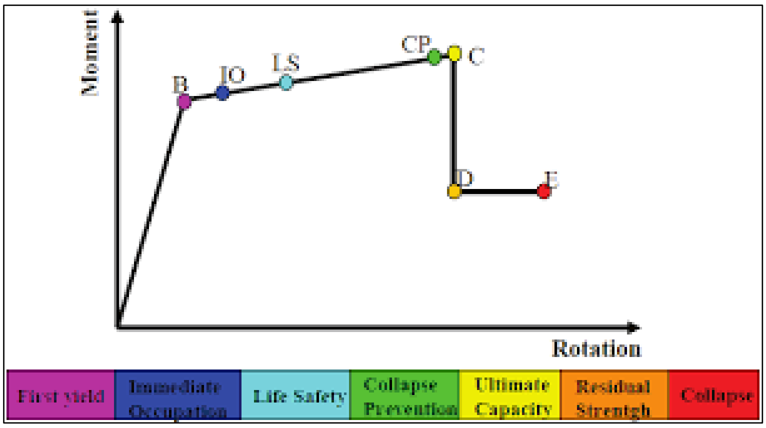

Figure 5: Different Properties of hinges (Reference: IS1893:2016 Code)

Table 3: Hinges Details

Model/Directions	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.

From pushover analysis, it was observed that, Model l has a maximum displacement of 57mm which is 65% more than the normal displacement of the building, and a base shear value of 330kN which is 37.3% more than the design base shear of the building in X directions. Similarly, in the Y direction, the base shear is 375kN which is 34.08% more than the design base shear of the building.

Model 2 has a maximum displacement of 54mm which is 41.29% more than the normal displacement of the building and a base shear value of 620kN which is 39.8% more than the design base shear of the building in X directions. Similarly, in the Y direction, the base shear is 650kN which is 44.2% more than the design base shear of the building.

Model 3 has a maximum displacement of 67mm which is 56.41% more than the normal displacement of the building and a base shear value of 760kN which is 49.42% more than the design base shear of the building in X directions. Similarly, in the Y direction, the base shear is 570kN which is 42.54% more than the design base shear of the building.

Comparison of Models with Respect to Nonlinear Time History Analysis

The nonlinear time history analysis was conducted for all the building models, on El Centro, Gorkha, and Kobe earthquakes of peak ground acceleration of 0.2188g, 0.3447g, and 0.177g respectively.

The displacement values for the three building models are obtained and are shown in Figure 6. The displacement of building models was obtained for the El Centro, Gorkha, and Kobe earthquakes. It was observed that Model 2 has maximum displacement and Model 3 has minimum displacement in the X direction when of El Centro ground motion was considered.

Similarly, Model 2 has maximum displacement in Kobe and El Centro but less in the Gorkha earthquake because PGA of Kobe and El Centro is higher than Gorkha earthquake. The displacement results show that Model 2 is more vulnerable than the other two models.

Table 5: Comparison of displacement of models (displacements in mm)

Ground Motion:	**El Centro**		**Gorkha**		**Kobe**	
Model	**X**	**Y**	**X**	**Y**	**X**	**Y**
1	28.94	27.77	12.98	12.07	17.00	18.51
2	31.38	28.98	10.42	9.38	37.44	27.24
3	24.65	23.85	11.97	12.42	25.78	25.51

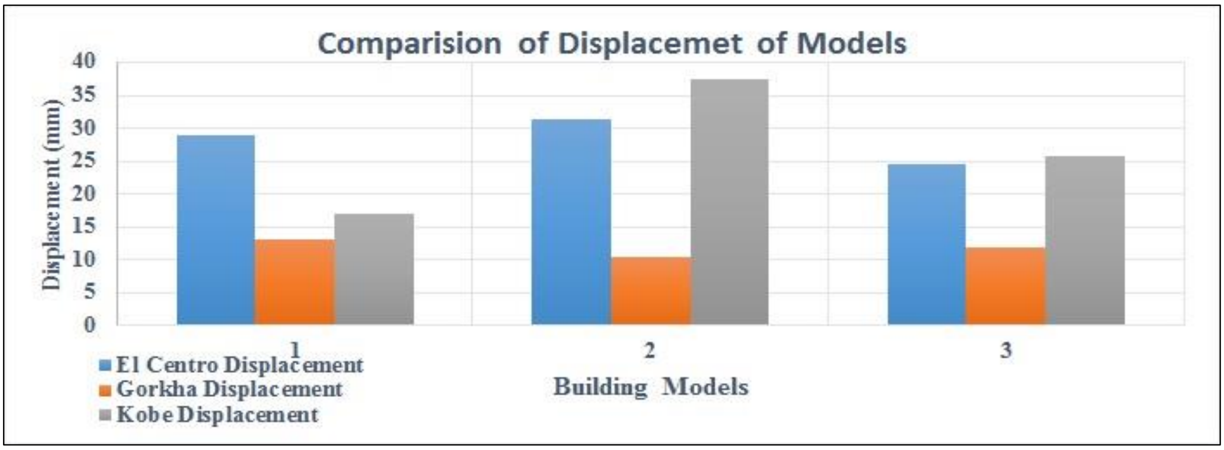

Figure 6: Displacement of models

Figure 7 shows the drift on models of El Centro, Gorkha, and Kobe earthquakes respectively. The elastic drift limit of the model is 0.4% (0.004) and here design drift inelastic drift is 0.2% (0.002).As per IS 1893:2016 , IS 456 maxmimun elastic and inelastic drift wa 0.4% and 0.2% respectively. Model 1 exceeds the inelastic drift limit on El Centro ground displacement. Similarly, for the Gorkha earthquake ground motion, drift is within the inelastic limit whereas, in Kobe earthquake Model 2 suffers the maximum drift ratio. As having max PGA and weak column models siffer max displacement, kobe earthquake have more PGA than gorkha earthquake.

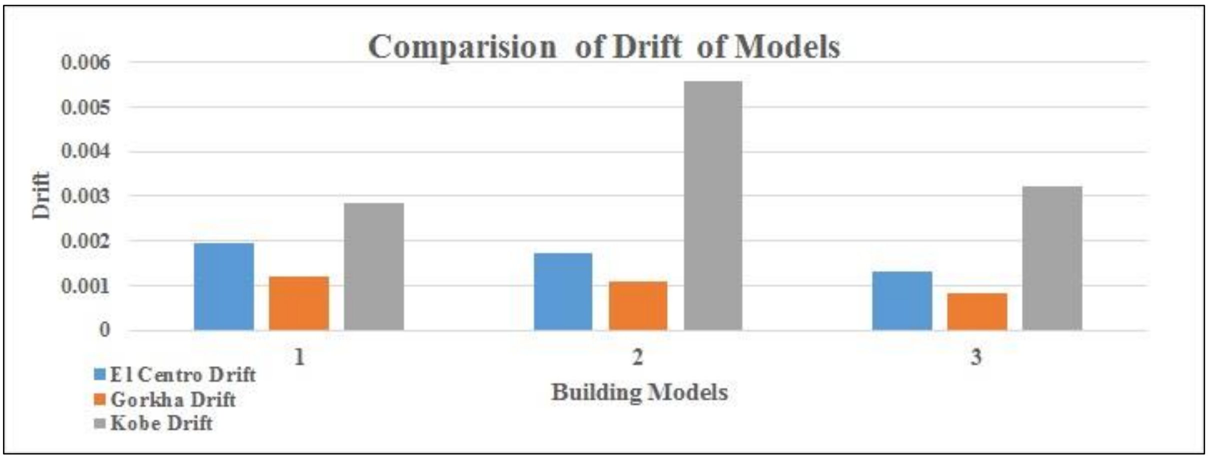

Figure 7: Drift of models

Figure 8 shows the base shear on models on El Centro, Gorkha, and Kobe earthquakes of peak ground acceleration. Model 3 has maximum base shear on El centro earthquake, and Model 1 has a minimum base shear value. Models having lower base shear has low resistance to lateral force during nonlinear time history analysis. Model 1 shows the maximum value of base shear on Gorkha. Model 2 shows the minimum base shear on Gorkha ground motion and Model 3 shows the minimum base shear on Kobe ground motion.

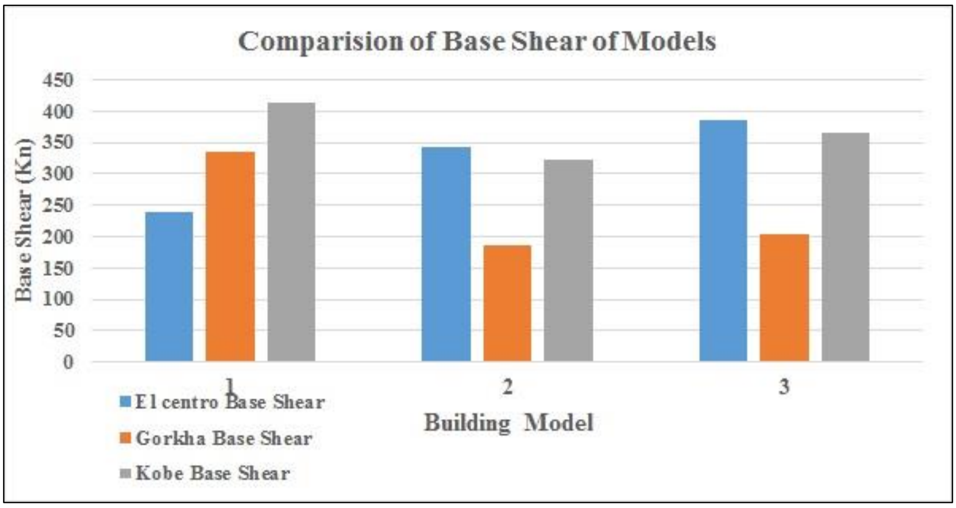

Figure 8: Base Shear of models

V. CONCLUSIONS

This paper presented the study on the various parameter obtained after the linear and nonlinear modelling on the low-rise building models. Maximum storey displacement, storey shear, and storey drift values in both directions X and Y are obtained from linear and nonlinear static and dynamic analysis on models. Based on the results obtained for the low-rise RC buildings the conclusions are made:

1. The column axis is not straight in Model 2, thus it has a maximum displacement than the other two models. Also, the irregular behaviour caused a maximum top displacement than other irregular models.
2. The nonlinear dynamic analysis showed that the maximum displacement is present in Model 2 whereas Model 3 has the minimum displacement. Owing to the irregular plan geometry, the drift ratio limit of Model 2 exceeds the inelastic limits for building model. Thus making Model 2 seismically weaker than the other models.
3. The hinges are formed in columns than the beam Model 2. This indicates that the model contains weaker columns compared to the beam, making the model a seismically failed structure.

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