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Seismic Vulnerability Assessment of School Building

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Abstract
Seismically undermined infrastructural facilities and poor construction, specifically of school buildings, make the lives of youth severely vulnerable during earthquake. Vulnerability assessment of school building that are constructed before and after 2015 Gorkha earthquake need to be done within time.This paper has evaluated the seismic performance of three different typology building(RC frame) commonly found in Kathmandu.Both Non Linear Static and Dynamic analysis was carried out to evaluate the performance of the buildings in three different earthquake having different peak ground acceleration value.The weak links in the building from which the potential failure may be initiated is studied using pushover analysis. Probability of occurence of different damage state when subjected to different earthquake was analysed using fragility curve.These analysis has been performed using finite element analysis software SAP 2000.After analysis it has been concluded that the performance of the building constructed after 2015 Gorkha earthquake following the IS 1893:2016 code was within the targeted performance level.

Keywords
Vulnerability Assessment, Pushover Analysis, Time History Analysis, Fragility Curve

1. Introduction

Collapse and severe damage of many buildings in past earthquakes have shown the vulnerability of many old-type buildings in seismic regions around the world. Nepal is one of the most seismically vulnerable countries in the world as it is located in the high seismic region having the geographical position along the tectonic boundary of the Indian and Eurasian plates. The subduction of the Indian plate beneath the Eurasian plate has caused the accumulation of the strain energy, which results in the disastrous earthquakes of the greater magnitudes. The recent Gorkha earthquake of magnitude 7.8 in 25th April 2015 is one of those several great earthquakes which Nepal have faced causing extensive damage with loss of life and property. Most of the damages have been due to the poorly designed structure.The soil types of Kathmandu mainly consist of clay and sand deposits which makes it more susceptible to compaction during ground vibrations, resulting in the formation of excess hydrostatic pore water pressures to cause liquefaction of the soil.

Vulnerability assessment (VA) is a testing processes for determining the vulnerability of an asset or assets at risk of being lost, taken, damaged, or destroyed. Threats come in a wide variety such as flooding,earthquakes, fire, hurricanes, breakdown of equipment, collapse of a structure etc. In country like Nepal the occurrence of earthquake possesses the major risk and if we look up at the history nine major earthquake has occurred during the last 700 years. The seismic records of this region extends back to 1255AD, and suggests that Great Bihar-Nepal earthquake in 1934 occurs in every 75 years, indicating that a devastating earthquake is inevitable in a long run and likely in a near future [1].

1.1 Need and importance of the research

Seismically weak school buildings, make the lives of youth severely vulnerable. For the long term economic prosperity and social growth of the nation, youth safety and their continuation of education is of utmost importance. During its life span structure is subjected to diffrent range of seismic loading, so an adequate engineering proficiency to assess diminishing structural capacity is essential to avoid complete structural collapse or to decide any specific intervention. The 2005 Kashmir earthquake caused approximately 87000 casualties, out of which almost

19,000 were school going children, and cumulatively, it affected about 3.5 million people[2]. In Kathmandu district there are 289 public school out of which 87 public schools are in Kathmandu metropolitan city and nearly 33,000 students are reading in these schools. Most of the school building which are constructed prior to 2015 Gorkha earthquake are constructed without following any seismic resistant design while some building are constructed following the relevant code at the time of construction. After assessing the vulnerability of existing school building, this will help to put together the rational conclusion so that necessary disaster preparedness can be carried out by identifying any need for structural strengthening and interventions.

2. Methodology

2.1 Selection of Building Typology

Sample school building that represent the overall building typology that are located in Kathmandu district is selected. They are represented as MB_1, MB_2 and MB_3 . This building model represents most of the buildings located in Kathmandu district with varying year of construction. Most of the construction prior to 2015 Gorkha earthquake has similar structural members with similar concrete and reinforcement detailing. Initially they are designed for two to three storey and with the increase in student numbers further more storeys are added which make the building vulnerable resulting in both physical and economical losses which can never be compensated also the performance of every structure that are constructed with or without following the revised building code need to be assessed. Prior to such unimaginable losses the weak links in the building need to be determined from which the potential failure may be initiated and the overall performance of the structure during different level of earthquake also need to be analyzed to reduce the potential losses in future.MB_1 which is constructed in 1993 A.D. represents all those type of building constructed in Kathmandu without following any seismic design criteria. Building constructed prior to 2015 Gorkha earthquake following the relevant code of that time is best represented by MB_2 (construction year 2014 A.D.) whereas construction of school building after 2015 Gorkha earthquake using IS 1893:2016 is best represented by MB_3 . The performance of these building also need to be assessed using revised Nepal building code NBC 105:2020. MB1 is four storey while MB2 and MB3 are three storey.

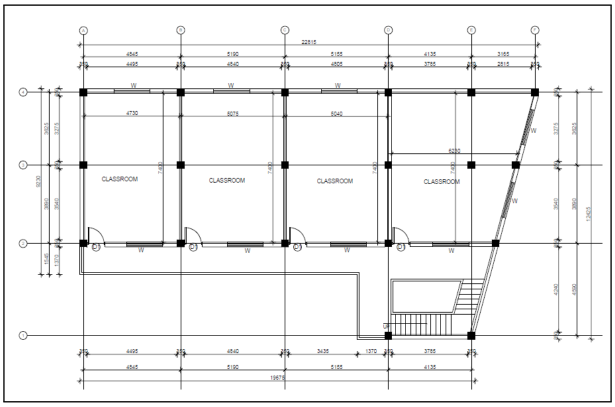

Figure 1: Ground Floor Plan of MB_1

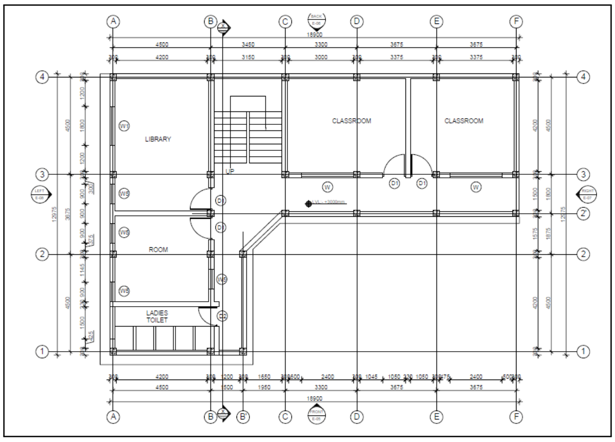

Figure 2: Ground Floor Plan of MB_2

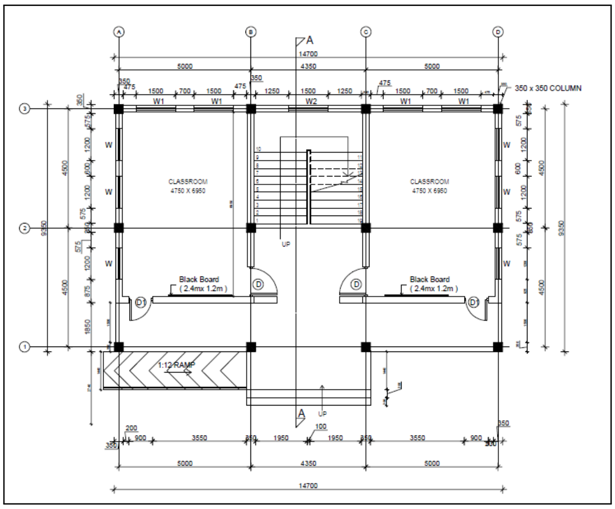

Figure 3: Ground Floor Plan of MB_3

2.2 Material and Sectional Property

Table 1: Material Properties

Description of Material	MB_1	MB_2	MB_3
Unit Weight of Brick	$20KN/m^3$	$20KN/m^3$	$20KN/m^3$
Unit Weight of RCC	$25KN/m^3$	$25KN/m^3$	$25KN/m^3$
Concrete	M15	M15	M20
Reinforcement Bar(IS 883:1994)	HYSD Fe 415	HYSD Fe 415	HYSD Fe 500

Table 2: Sectional Properties(mm)

Sectional Size	MB_1	MB_2	MB_3
Column	350X350	300X300	350X350
Beam	230X350	230X350	Main: 230X350 Secondary: 230X300
Slab	125	125	125
Floor Height	3000	3000	3000
Total Height	12000	9000	9000

2.3 Reinforcement Detailing

MB_1 and MB_2

Column

4 -12mm diameter rebar (minimum reinforcement bar assumed)

Stirrups: 8 mm diameter rebar at 200 mm c/c

Beam

2 -12mm diameter rebar (top) + 2 -12mm diameter rebar (bottom)

Stirrups: 8 mm diameter rebar at 200 mm c/c

MB_3

Column

Ground floor

Grid B-2, C-2, A-1, A-3, D-1, D -3: 4-16mm+4-20mm diameter rebar

Other Grid: 8-16mm diameter rebar

First /Second Floor

All Column: 8-16mm diameter rebar

Stirrups: 8 mm diameter rebar at 100 mm c/c

Beam

First Floor

Main Beam

5 -16mm diameter rebar (top) + 5 -16mm diameter rebar (bottom)

Stirrups: 8 mm diameter rebar at 100 mm c/c

Secondary Beam

5 -16mm diameter rebar (top) + 5 -16mm diameter rebar (bottom)

Stirrups: 8 mm diameter rebar at 100 mm c/c

Second /Third Floor

3 -16mm diameter rebar (top) + 3 -16mm diameter rebar (bottom)

Stirrups: 8 mm diameter rebar at 100 mm c/c

2.4 Material Hysterisis Modelling

Performance evaluation of RC structures under repeated cyclic loading, prediction of the structural hysteresis behavior is of utmost importance [3]. Under repeated cyclic loading,load deformation behavior/characteristics of a structural members can be produced using hysteresis model. It can be broadly classified into smooth and polygonal hysteresis models. The stiffness change in polygonal hysteresis models, are considered at cracking, yielding, strength, and stiffness degradation stages. The trilinear primary curve of the Takeda model represents the un-cracked, cracked, and post–yielding stages as shown in Figure 1.

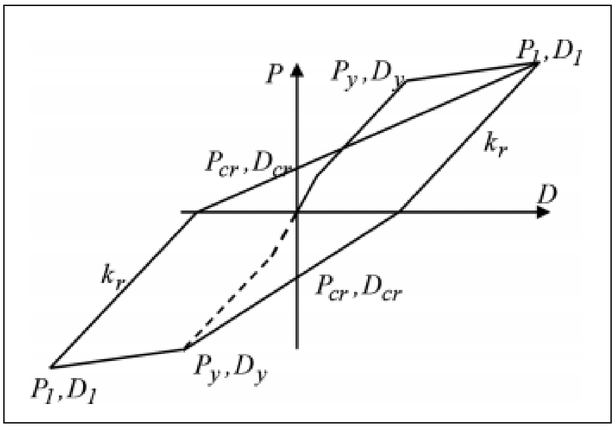

Figure 4: Load Deflection diagram of trilinear stiffness degrading model (Source:Sengupta and Li, 2017)

The unloading stiffness k_r is calculated in terms of yielding stiffness k_y, yield deflection D_y, and maximum deflection D_{max} as follows .In this model, the reloading branch projects toward the previous unloading point of the loading history

$$Kr = K_y \left(\frac{D_y}{D_{max}} \right)^{0.64}$$

2.5 Modal Analysis

The time periods and modal mass participation factors obtained using eigen vector are discussed and presented below. For MB_1, the first mode of vibration shows a time period of 1.538 seconds with major mass participation (56.89%) towards the longer direction of structure. The second mode shows the time period of 1.4995 second with modal mass participation of 62.14% also toward shorter direction. Similar condition arise in next two model where the time period of first and second mode are 1.024 seconds and 0.983 seconds for MB_2 and 0.587 seconds and 0.584 seconds for MB_3 More than 98% of the mass has participated within first 12 modes for all model type building.

2.6 Hinge Modelling

The generalized load-deformation relation for beam and column element is represented by linear response from A (unloaded stage) to an effective yield B, then a linear response at reduced stiffness from point B to C, then sudden reduction in seismic force resistance to point D, then response at reduced resistance to E, and final loss of resistance thereafter as shown in Figure. Parameters that are needed to define the load-deformation curve are obtained from ASCE 41-13. Performance criteria are set based on the material properties, section properties, transverse reinforcement detailing and shear force.

For Column P-M2-M3 auto hinges using ASCE 41 -13 table 10-8 are defined from sap 2000 at relative location of 10% and 90% assuming the concrete failure condition as flexure and shear. For beam M3 auto hinges using ASCE 41 -13 table 10-7 are defined at relative location of 10% and 90%. As the spacing of the stirrups is greater than 115 mm i.e. d/3 in the flexure plastic hinge region for Model 1 and 2,the transverse reinforce is assumed as non conforming while for Model 3 the spacing of trasverse reinforcement is is within the limit specified in ASCE 41-13 and hence assumed as conforming.

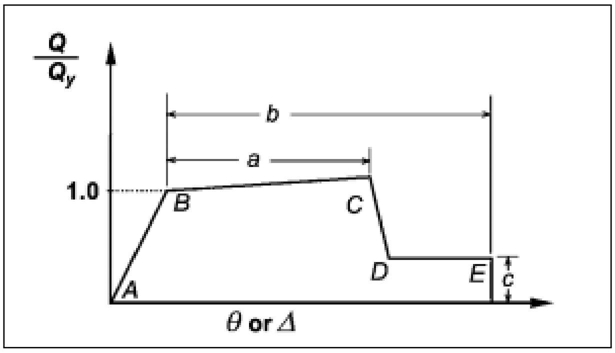

Figure 5: Typical Force Deformation Curve

2.7 Non-Linear Time History Analysis

Nonlinear Direct-integration time-history method which involves the integration of structural properties and behaviors at a series of time steps which are small relative to loading duration is performed using the following accelerogram data of three different earthquake as specified in NBC 105:2020

Table 3: Earthquake Data

S.No	Description of Earthquake	PGA
1	Gorkha Earthquake	0.177g
2	El-Centro Earthquake	0.348g
3	Imperial Earthquake	0.61g

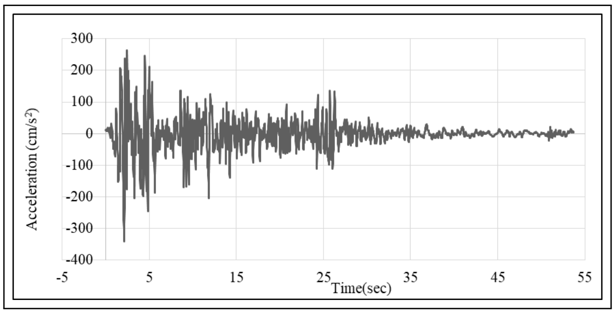

Figure 6: Accelerogram of Imperial Earthquake

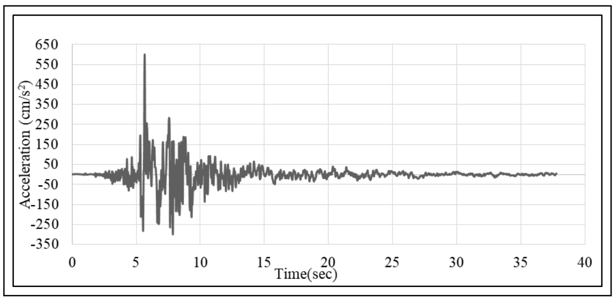

Figure 7: Accelerogram of El-Centro Earthquake

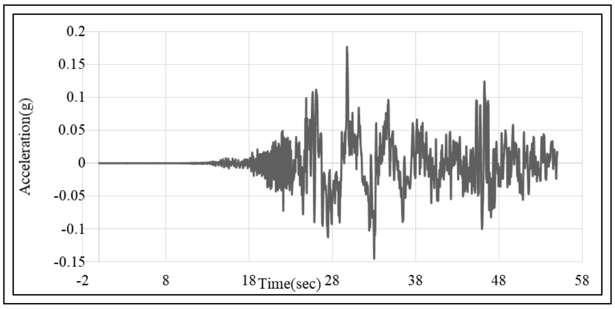

Figure 8: Accelerogram of Gorkha Earthquake

2.7.1 Development of Fragility Curve

The probability of being in or exceeding a given damage state is modeled as a cumulative lognormal distribution. For structural damage, given the spectral displacement demand, S_d, the probability of being in or exceeding a damage state ds, is modelled as:

$$P[ds \mid S_d] = \emptyset \left[\frac{1}{\beta_{ds}} ln \left(\frac{S_d}{\overline{S}_{d,ds}} \right) \right]$$

where:

$\overline{S}_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of the damage, ds

β_{ds} is the standard deviation of the natural logarithm of spectral displacement of damage state, ds, and

$\emptyset$ is the standard normal cumulative distribution function.

The median value of spectral displacement for four different damage state for MB_1 is computed by considering the Pre- Code damage function as this building is constructed without following any seismic design criteria. Similarly the displacement criteria for MB_2 is computed by considering the Moderate- Code damage function as this typology building is constructed following the codes prevalent during the time of construction. Building Model MB_3 displacement criteria is selected by considering the High –Code damage function as this building typology represent those constructed following the modern seismic code provisions.

Table 4: Median value of Spectral Displacement (mm)

S.N.	Model Type	Damage state			
		Slight	Moderate	Extensive	Complete
1	MB_1	30.48	48.77	121.9	304.80
2	MB_2	22.86	39.62	106.68	274.32
3	MB_3	22.86	45.72	137.16	365.76

3. Results and Discussion

3.1 Pushover Analysis

After assigning the hinges to the beam and column at critical location as per ASCE 41-13, the structure is pushed to 300 mm. The resulting hinge behavior at critical location along with the performance point base shear and displacement of the structure as per ATC-40 at design basis earthquake was noted. For building model MB_1, it was clear from Table 7 that at a performance point displacement of 114.38 mm which is near to step 4, 80% of the hinges are within the desired performance level of Immediate Occupancy (IO). Major portion of the remaining hinges fall within Life Safety (LS) performance level while 3% of the hinges has crossed the Collapse Prevention Level (CP). The main reason for 20% of the hinges exceeding the desired performance level is due to the insufficient member size, poor ductile detailing at column and beam element. The model building type MB_3 which is designed as per IS 1893:2016 justified this result as the size of column is same with increased reinforcement percentage although the height of building is less than building model type MB_1 . Performance evaluation of model building type MB_2 portrays that at performance point displacement of 81.69 (nearly similar to step 7 of Table 8) nearly 22.5% of the hinges has surpassed the desired performance level of immediate occupancy at rare earthquake. Building model MB_3 which was designed and constructed after the occurrence of 2015 Gorkha earthquake following the revised codes shows the better performance with all the hinges falling within the desired performance level. The response of the buildings such as structural time period,drifts, base shear, and storey forces varies significantly with the application of different codes [4]. These parameters significantly affects the demand on the structure resulting in increased size of members. Model building type MB2 and MB3 although being of same height, the reinforcement detailing and sectional sizes of member varied significantly. The main reason for this deviation is due to the provision given in seismic code are based on the observations, experiments and analytical case studies made during past earthquake in particular region. This code deemed to be just sufficient for the time being and with the occurrence of catastrophic earthquake and advances in the field of seismic designs, their limitations and inadequacy gets exposed. In order to fulfill this gap the code gets revised and structural demands varies significantly.

Table 5: Performance Point

S.N.	Model Type	Base Shear (KN)	Displacement (mm)
1	MB_1	1381.03	114.38
2	MB_2	1062.72	81.69
3	MB_3	937.28	47.66

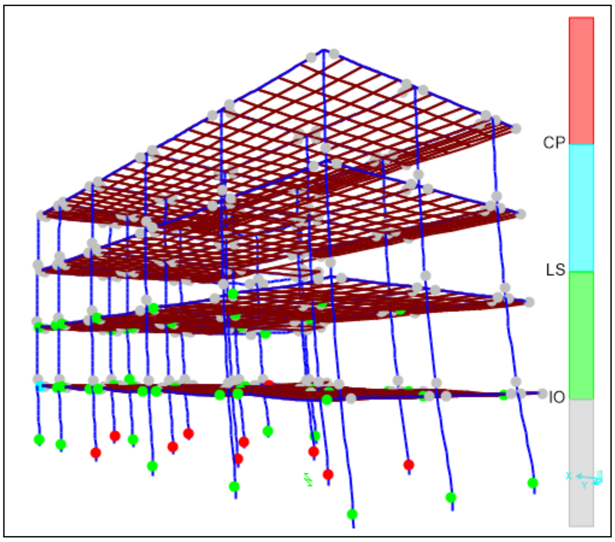

Figure 9: Hinge Behaviour of MB_1 at Performance Point

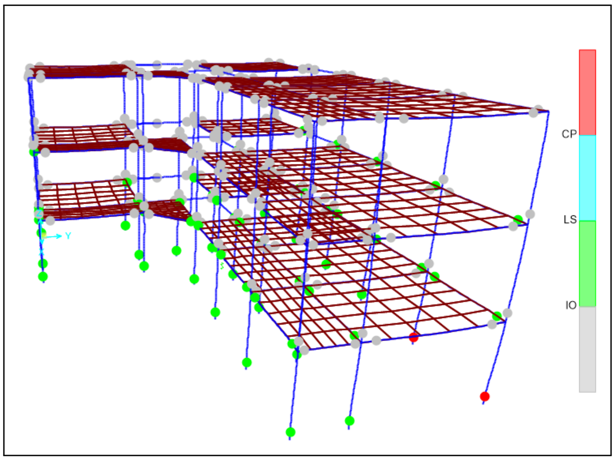

Figure 10: Hinge Behaviour of MB_2 at Performance Point

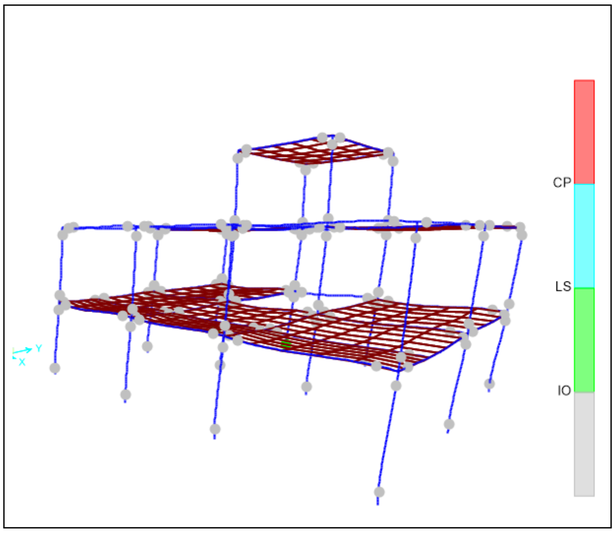

Figure 11: Hinge Behaviour of MB_3 at Performance Point

3.2 Time History Analysis

3.2.1 Result of Non Linear Time History Analysis

After performing the Nonlinear time history analysis, the roof displacement vs time period was plotted and the maximum roof displacement was noted. The maximum top displacement of MB_1 has increased significantly with increase in peak ground acceleration. Similar condition arise in building model MB_2 and MB_3 as shown in Table 6. MB_2 and MB_3 although being of same height, the maximum top displacement value varies significantly. The main reason behind this variation is due to the effect of plan irregularity and section size of the column in x and y direction.The seismic analysis of plan irregular L shaped building has has shown that it has large top storey displacement as compared to symmetrical building [5]

Table 6: Maximum Top Displacement(mm)

S.N.	Model Type	Earthquake		
		Gorkha	El-Cento	Imperial
1	MB_1	85.73	109.66	331.00
2	MB_2	56.28	84.44	149.20
3	MB_3	40.34	77.53	96.18

3.3 Fragility Curve

Fragility Curve for three different typology building at three different earthquake for four different damage state is plotted as shown below by considering the median value of spectral displacement as capacity and maximum top displacement as demand for particular earthquake. Figure 12, Figure 13 and Figure 14 indicates the fragility curves of MB_1, MB_2 and MB_3 which help engineers/decision makers to assure the seismic vulnerability condition of structure and to predict the damage of building structure from possible future earthquake. The seismic zoning factor (Z) which represents the peak ground acceleration for 475 year return period of Kathmandu is taken as 0.35g. The probability of occurrence of damage state corresponding to 0.35g is studied from fragility curve of corresponding building and is summarized in Table 7, Table 8 and Table 9. From this table it is apparent that for a particular building, the response of two earthquake are near about and the average of these values are taken into consideration for describing the occurrence of damage state. For Building Model MB_1 the probability of occurrence of slight, moderate, extensive and completed damage are nearly 99.7%, 97.8%, 72.5% and 20.4% while for building model MB_2 these value changes to 99.9%, 88.4%, 36.3% and 3.4%. Building Model type MB_3 which is designed as per IS 1893:2016 code the probability of occurrence of slight, moderate, extensive and complete damage state are 97.4%, 80.4%, 36.6% and 0.8%. These result makes clear that section size of beam and column along with percentage of reinforcement bar and proper ductile detailing plays a vital role in reducing the probable damage during particular earthquake.

Table 7: Probability of Occurrence of Particular Damage State of BM_1 at 0.35g

S.N.	EQ	Slight	Moderate	Extensive	Complete
1	Imperial	99.79%	98.32%	75.57%	22.99%
2	El-Centro	97.78%	89.89%	43.78%	5.61%
3	Gorkha	99.63%	97.42%	69.67%	17.97%

Table 8: Probability of Occurrence of Particular Damage State of MB_2 at 0.35g

S.N.	EQ	Slight	Moderate	Extensive	Complete
1	Imperial	99.99%	88.56%	36.55%	3.44%
2	El-Centro	99.99%	88.32%	36.08%	3.35%
3	Gorkha	99.99%	94.67%	52.63%	7.93%

Table 9: Probability of Occurrence of Particular Damage State of MB_3 at 0.35g

S.No	EQ	Slight	Moderate	Extensive	Complete
1	Imperial	91.58%	61.56%	7.74%	0.16%
2	El-Centro	97.27%	79.91%	19.00%	0.80%
3	Gorkha	97.46%	80.79%	19.86%	0.87%

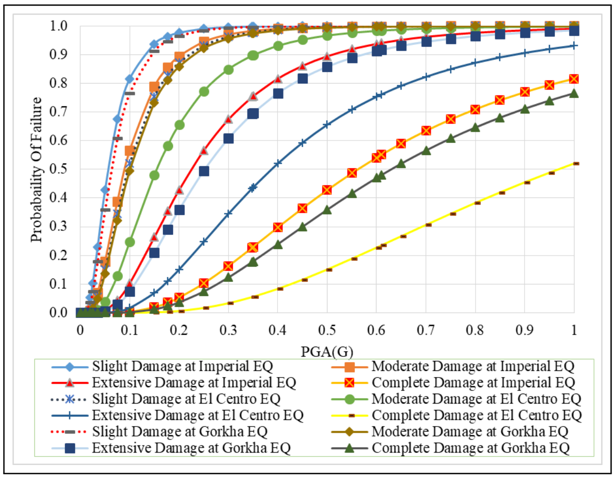

Figure 12: Overall Fragility Curve of MB_1

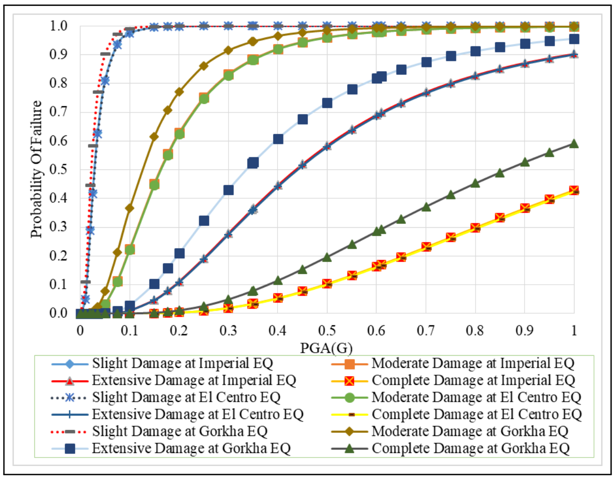

Figure 13: Overall Fragility Curve of MB_2

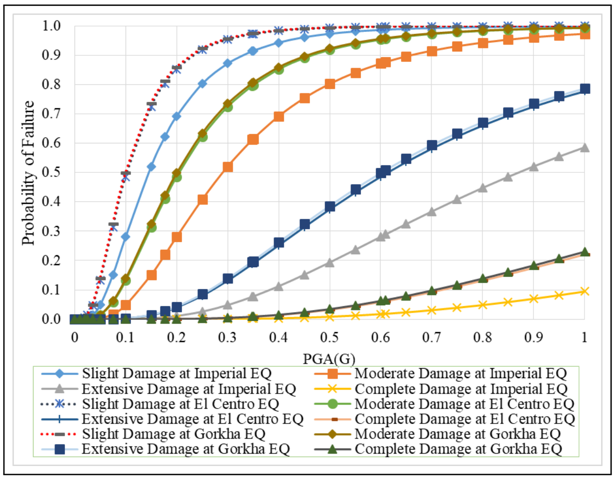

Figure 14: Overall Fragility Curve of MB_3

4. Conclusion

This study explores the seismic vulnerability of existing RC frame school of Kathmandu district as per revised NBC 105:2020.In order to achieve the result, case study on three different typology building was done. Both Non Linear static and dynamic analysis was performed using finite element software Sap2000.The main conclusion of the result are summarized below.

From pushover analysis it was clear that performance of MB_3 was found within the desired performance level of immediate occupancy at design basis earthquake while that of MB_1 and MB_2 typology building need immediate intervention to prevent possible disaster in future.

Building model type MB_3, constructed after 2015 Gorkha earthquake following IS 1893:2016 code shows the better performance at design basis earthquake as the probability of extensive and complete damage decreased significantly as compared to MB_2 and MB_1 .

Fragility curve of different building mostly depends upon the sectional size, percentage of reinforcement and ductile detailing while the earthquake time history has minimal effect on its performance.

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