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FRAGILITY EVALUATION OF RC BUILDINGS DESIGNED BY NEPAL BUILDING CODE CONSIDERING DEFORMATION CAPACITY

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ABSTRACT

Fragility evaluation assessed for RC moment-resisting frame structures designed by the Nepal Building Code considering deformation capacity. The site soil type, the number of stories, and seismic protection intensity were supposed to be design variables. Also, the limit state (global seismic damage index) and maximum inter-story drift ratio had employed as the damage identifiers. The non-linear static push-over analysis determined the limit state of the structure. The probability of exceeding each damage level in each scaled earthquake intensities of applied 40 ground motions is determining by incremental dynamic analysis (IDA) with the aid of dynamic response time history analysis. The fragility curves of the target building and revised model building derived by the statistics analysis are considering lognormal accumulative distribution function. The fragility evaluation of the target structure and revised model against earthquakes motion were examined using the developed fragility curves. The results indicate that target RC frame structures designed by the Nepal Building Code (NBC 105:1994 and NBC 110:1994) can be achieved satisfactorily with some modification in the limit state of shear design.

Keywords: Limit state, maximum inter-story drift ratio, incremental dynamic analysis, fragility curve.

1. INTRODUCTION

Nepal lies on the Main Himalayan Thrust (MHT) fault line that divides the Indian and Eurasian tectonic plates. The movement of the Indian Plate towards north collided with the Eurasian Plate. Due to inter-plate crustal shortening activity, the probability of the occurrence of an earthquake is very high in the Himalaya Range. Therefore, Nepal is one of the world's most earthquake-prone countries. The recent big earthquake occurred in Nepal was the Gorkha Earthquake in 2015 (Mw 7.8) resulted in huge losses of lives and physical infrastructures. To overcome these types of earthquake fatalities, we need earthquake-resistant resilient structures based on a seismic performance design. The purpose of this study is to evaluate the seismic performance of a target RC structure designed by Nepal Building Code (NBC105:1994, Seismic Design of Buildings in Nepal and NBC 110:1994, Plain and Reinforced Concrete) and to know the size of an earthquake the buildings can sustain.

2. DATA

In this study, a set of 40 ground motions data are applying to the analytical model for the dynamic response time history analysis which recorded in various earthquakes in California. These data are

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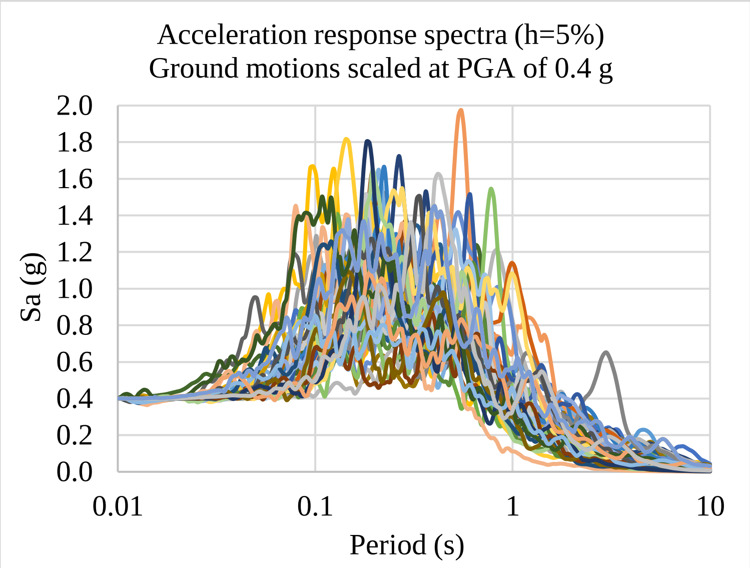

Figure 1. Acceleration response spectra of ground motions used for dynamic response analyses.

attained from the web-based PEER ground motion database and the Center for Engineering Strong Motion Data (CESMD). The ground motions data consists of Type D site of NEHRP (National Earthquake Hazards Reduction Program) having earthquake magnitude ranges from 6.5 to 6.9 M_w, and source-to-site distance ranges from 13 to 40 km. Each ground motion systematically scaled to increasing earthquake intensities from 0.1 g to 1.5 g. Altogether 600 (40 x 15) numbers of the earthquake intensities are applied for the dynamic response time history analysis in the incremental dynamic analysis (IDA). The acceleration response spectra of the selected 40 ground motions, scales at PGA of 0.4 g, are shown in Figure 1.

3. METHODOLOGY

3.1. Target building structure

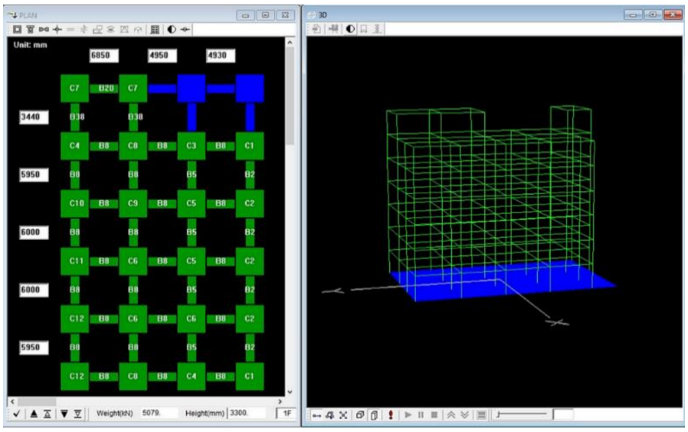
Figure 2. An analytical model of the target building by the STERA 3D computer program.

In this study, a 7-story special moment-resisting framed RC structure selected as a target structure. It consists of size 27.94 m x 17.33 m having standing height 23.1 m in medium soil site condition. The size of the column is 600 mm x 600 mm. The analytical model constructed with the aid of the STERA 3D (Version 10.1, Saito T., 2019) computer program, which shown in Figure 2. The only bare frame structure is carried out for the model analysis. The compressive strength of concrete of 20 N/mm^2 and tensile yield strength of steel rebar of 500 N/mm^2 is used in the target building and also in the analytical model.

3.2. Research Procedure

The research procedure for this study follows mainly in three steps. They are:
Step 1: Evaluate the limit state of the building by non-linear static pushover analysis,
Step 2: Execute incremental dynamic analysis proposed by Vamvatsikos and Cornell (2002), and
Step 3: Statistics analysis for the fragility evaluation by the lognormal cumulative distribution function.

3.2.1. Estimation of limit state

The evaluation of the limit state was carried out by non-linear static pushover analysis. The static push-over analysis is a technique in which the model of the structure subjected to predetermined lateral forces.

The lateral forces represent the relative inertia forces, which is equivalent to the seismic forces induced by earthquake ground motion. In the static pushover analysis, applied forces gradually increased, and the sequences of cracks, yielding plastic hinge formations, and the load at which the failure of the various structural components occur recorded. This incremental process continued until the structural model reaches a collapse or target global structural drift limit achieved.

3.2.2. Incremental Dynamic Analysis

The Incremental Dynamic Analysis (IDA) proposed by Vamvatsikos D. and Cornell C.A. (2002) and used to the collapse evaluation. To develop the analytical fragility curve, it is required to carry out the incremental dynamic analysis. IDA is executed to estimate the limit-state capacity and the seismic demand for a specified structure by performing a series of non-linear dynamic response time history analysis under a suite of multiple scaled ground motions records. For evaluating the seismic capacity, the selected ground motion intensity increased incrementally until the intended structural capacity reaches the global collapse achieved.

3.2.3. Statistics analysis for fragility evaluation

The conditional probability that the maximum inter-story drift ratio (IDR_{max}) exceeds IDR at given PGA is calculated (for example, Nagae T. et al., 2006, Jiang H. et al., 2012 etc.) by the Eq. (1) moreover, consequently it applied in the fragility evaluation.

$$P[IDRmax > IDR] = 1- P[IDR_{max} \leq IDR] = 1- \phi(\frac{\ln(x)-\ln(\mu)}{\delta}) \quad (1)$$

where ϕ is the function of the normal cumulative distribution function,
x is the limit state of the target structure, i.e. IDR,
μ is the mean of the statistical data, computed by 50th percentile data, and
δ is the standard deviation of the statistical data, which is the equivalent dispersion of the 16th percentile and 84th percentile data of maximum IDR in the normal distribution, computed by Eq. (2).

$$\delta = \frac{ln(IDRmax)84\% - ln(IDRmax)16\%}{2} \quad (2)$$

where $(IDR_{max})84\%$ is the 84th percentile and $(IDR_{max})16\%$ is the 16th percentile statistics data of the maximum inert-story drift ratio for specified scaled ground intensities.

A fragility curve is developed from the behavior model of a structure, capacity and a suite of ground motions. Fragility curves is a cumulative distribution function that describes the probability of collapse or failure. The maximum inter-story drift ratio (IDR_{max}) of the structure is adopted as an engineering demand parameter (EDP) to represent the degree of seismic response in the incremental dynamic analysis and applied for the fragility evaluation.

4. RESULTS AND DISCUSSION

4.1. Results of the target building structure

The limit state of the structure evaluated by the non-linear static pushover analysis is depicted in Figure 3. From the push-over analysis, the first shear failure occurred at first floor in x-direction resulted in inter-story drift ratio of 0.011 (≈ 1/90). So, the limit state of the target structure assumed 1/90. The first RC member undergoes in shear failure occurred at column no. 17 of the first floor in step no. 495 of STERA 3D which can be seen clearly in Figure 4.

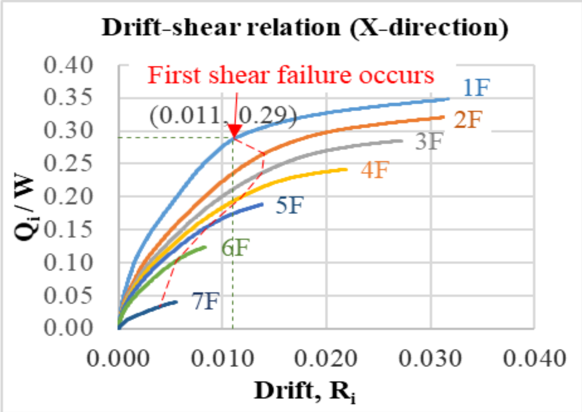

Figure 3. Drift-shear relation in x-direction. Each colored line indicates the drift-shear relation of the floors. The dashed red line indicates the drift-shear relation in each floor at the first shear failure.

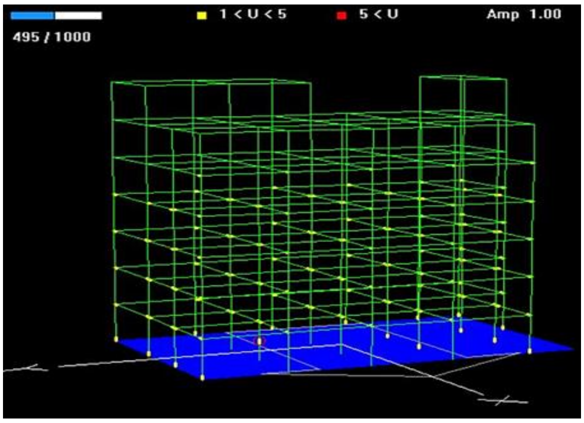

Figure 4. Failure mode in the analytical model of the target building. The figure shows the first member undergoes in shear failure is Column no. 17 at the first floor in step number 495 out of 1000.

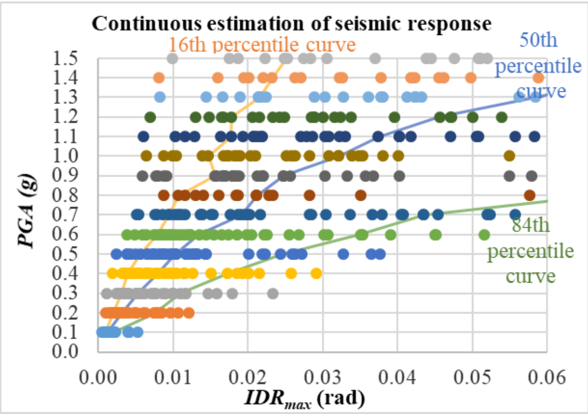

Figure 5. Statistics of seismic responses of IDA. The orange, blue and green lines indicate the 16th, 50th and 84th percentile curves of each level of PGA respectively. The different colored circles indicate the maximum IDR responses of each scaled intensity of the ground motions.

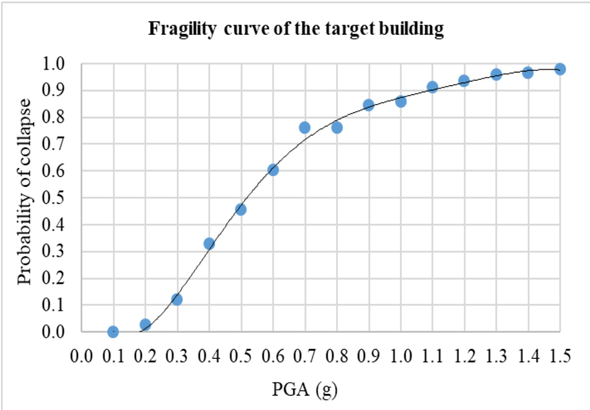

Figure 6. Fragility curve of the target building. Each solid circle point in the curve represents the probability of collapse (exceedance of the damage parameter, which is related with maximum inter-story drift ratio over the limit state value at a given ground motion intensity parameter.

The maximum IDR (inter-story drift ratio) for scaled input of 40 ground motions applied in the IDA (incremental dynamic analysis) was sorted in ascending order. Then, 6th, 20th and 34th lowest data were picked for the 16th, 50th and 84th percentile data to a continuous estimation of seismic response. The result of the incremental dynamic analysis (IDA) depicted in Figure 5. From the result of IDA, finally, we derived the probability of collapse of the structure by using Eq. (1), and consequently plotted the fragility curve, as shown in Figure 6. The conditional probabilities become high as PGA becomes large.

The design aims to achieve acceptable probabilities that the structure will not become unfit for the use, which is ensured by bending or flexural failure mode. However, shear failure appears in the model of the target structure. To overcome the shear failure, the target structure is re-modelling by adopting the following techniques, and all the research procedure apply in the revised model.

• Reducing the yield strength of reinforcement from 500 to 415 N/mm^2.
• Increasing the size of shear reinforcement (12 mm and 10 mm for column and beam respectively).
• Providing additional shear reinforcement in the column in terms of legs, wherever is necessary.

4.2. Results of revised target building model structure

From the non-linear static pushover analysis, no shear failure was observed in the revised model. Second floor and x-direction were found the weakest floor, and weaker direction and the failure mode was flexural type. The result of the push-over analysis is depicted in Figure 7. For flexural failure, the limit state is assumed to be the ultimate limit, that is 0.03 though the target drift was 1/50. The flexural failure behavior in the analytical model can be seen clearly in Figure 8.

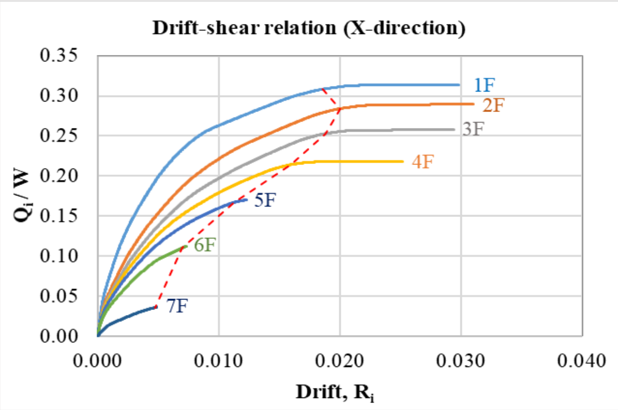

Figure 7. Drift-shear relation in the x-direction of the revised model of the target building. Each colored curvature indicates the drift-shear relation of the floors. The dashed red line indicates the drift-shear relation in each floor at 1/50 target drift.

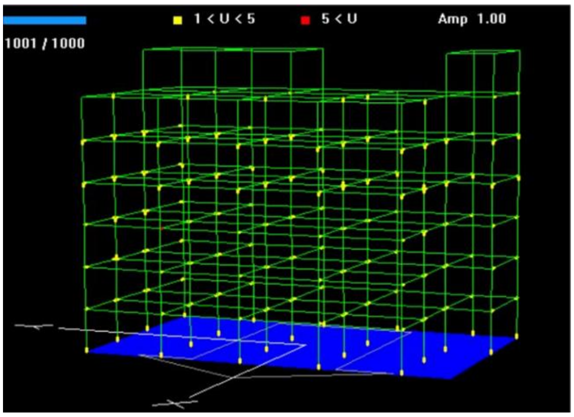

Figure 8. Failure mode in the revised analytical model of the target building. Yellow mark at the joints indicates the flexural failure after yielding. 1<U<5 and 5<U indicate the ductility index for moderate damage and severe damage.

The maximum IDR (inter-story drift ratio) for scaled input of 40 ground motions applied in the IDA was sorted in ascending order. With the help of 6th, 20th and 34th lowest data, 16th, 50th and 84th percentile data were estimated continuously. The result of the incremental dynamic analysis (IDA) for the revised model depicted in Figure 9. The probability of a collapse was computed by using Eq. (1) moreover, consequently plotted the fragility curve for the revised model, as shown in Figure 10.

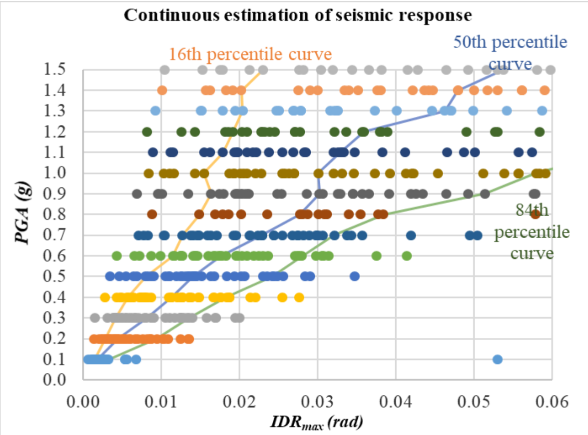

Figure 9. Statistics of seismic responses of IDA of the revised model of the target structure.

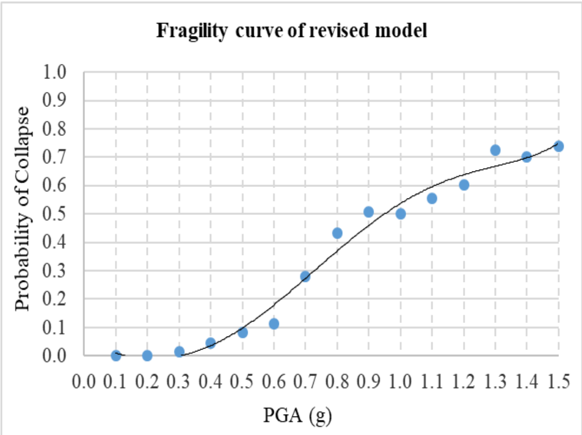

Figure 10. Fragility curve of the revised model of the target structure.

4.3. Comparison of fragility evaluation between target and revised target model structure

The comparison of the fragility evaluation between target structure and revised target model structure are depicted through conditional probabilities of collapse in Table 1 and fragility curves in Figure 11.

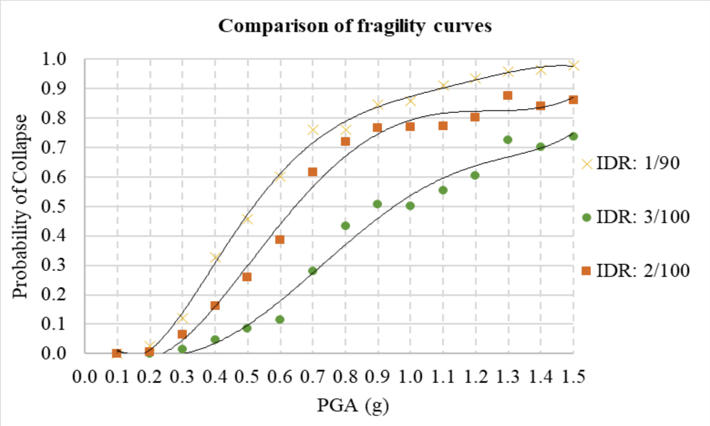

Figure 11. Comparison of the fragility curves. Orange, red and green cross and solid circles in the figure show the probability of collapse of the fragility curves for IDR 1/90, 2/100, and 3/100 respectively.

Table 1. Comparison of the probability of collapses.

PGA (g)	Prob. of Collapse: P[IDR_{max} > IDR]		
	IDR: 1/90	IDR: 2/100	IDR: 3/100
0.1	0.0000000	0.0000000	0.0000000
0.2	0.0261124	0.0061600	0.0007214
0.3	0.1211039	0.0632539	0.0134521
0.4	0.3259242	0.1624157	0.0459187
0.5	0.4564643	0.2604920	0.0841483
0.6	0.6020452	0.3845832	0.1141742
0.7	0.7592376	0.6156317	0.2802328
0.8	0.7603281	0.7182669	0.4343393
0.9	0.8447182	0.7654655	0.5058956
1.0	0.8588498	0.7680893	0.5016481
1.1	0.9098734	0.7722407	0.5551495
1.2	0.9335545	0.8007193	0.6029611
1.3	0.9592782	0.8756382	0.7245253
1.4	0.9638909	0.8385219	0.7020487
1.5	0.9789438	0.8598877	0.7381041

From Figure 11, the 33% of probability of collapse is found in the target structure (shear failure type) while it is 5% probability of collapse in the revised model (flexural failure type) for 400 gals. The significant difference of probability of collapse can see from the comparison of fragility curves accounting shear and flexural failure.

5. CONCLUSIONS

The Fragility curves of the target building and revised model building were derived with the aid of IDA and statistics analysis. The probabilities of maximum inter-story drift ratio of RC frames of both models against earthquakes were examined using the developed fragility curves. From the fragility evaluation, it is concluded that the seismic performance of the target structure can be increased by ensuring flexural type failure with respect to shear failure. The results indicate that RC frame structures designed by the NBC can be achieved satisfactorily with some modification in limit state of shear design.

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