Siddiqui, M.S., & Azuhata, T. (2025), Seismic performance evaluation of a typical reinforced concrete building in Dhaka, Bangladesh, with proposed design modifications for enhanced performance, Synopsis of IISEE-GRIPS Master's Thesis, Bulletin of IISEE, 60

SEISMIC PERFORMANCE EVALUATION OF A TYPICAL REINFORCED CONCRETE BUILDING IN DHAKA, BANGLADESH, WITH PROPOSED DESIGN MODIFICATIONS FOR ENHANCED PERFORMANCE

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ABSTRACT

Low and mid-rise buildings in Dhaka are designed using the equivalent static analysis (ESA) method, which is a force-based design approach. This study aimed to evaluate the seismic performance of a typical mid-rise building for updated seismic hazard data using a displacement-based design approach. The ultimate story drift and corresponding collapse probabilities of the original model and two hypothetical models were evaluated for both current and updated seismic hazards. The collapse probability of the original model was obtained as 2.33% for current seismicity and 5.56% for updated seismicity over a 50-year return period. In contrast, the collapse probability of modified model I, with increased shear reinforcement in the columns, and modified model II, which includes RC infill walls, were reduced by 50% and 70%, respectively, for the updated seismic hazard.

Keywords: Incremental Dynamic Analysis (IDA), Ultimate Drift Limit, Collapse Probability, Capacity Spectrum Method, BNBC 2020

1. INTRODUCTION

In the ESA method, the design base shear is determined by scaling down the maximum expected earthquake force using a reduction factor, R. This factor is derived from the building's capacity to dissipate energy through inelastic deformation. While structural members are designed to withstand this seismic force, their deformation capacity is not directly assessed within this method. Therefore, employing a displacement-based design approach is essential for accurately predicting a building's seismic performance.

2. DATA

2.1 Target Building

A six-story reinforced concrete building located in the periphery of Dhaka was selected for this study. The total floor area measured 154.3 square meters, and the overall height from ground level is approximately 21 meters. Each floor consists of 14 columns, which can be classified into five categories, as shown in Figure 2. All columns' flexural and shear reinforcements meet the requirements of an intermediate moment-resisting frame system.

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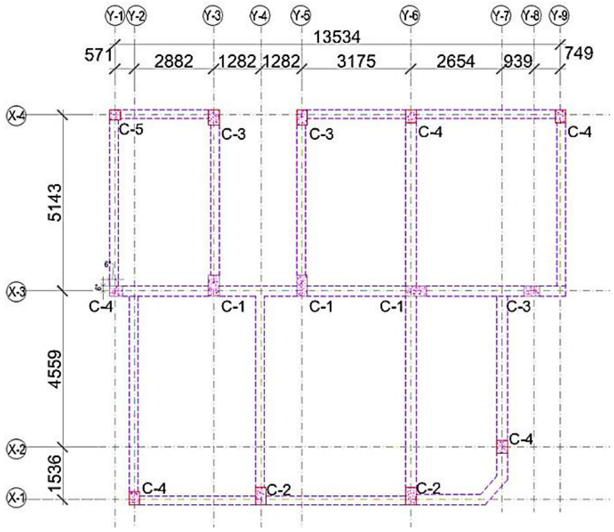


Figure 1: Column layout plan

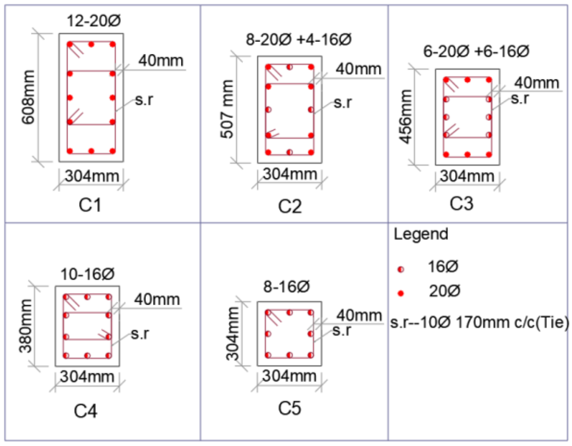


Figure 2: Column detailing
(1st floor to 6th floor)

The specified design compressive strength of the concrete is 3,000 psi (21 MPa), and the yield strength of both the longitudinal and shear reinforcing bars is 60,000 psi (413 MPa).

2.2 Ground motion and site hazard

A set of 20 ground motions was selected for this study, with shear wave velocities ranging from 180 m/sec to 360 m/sec, magnitudes between 6 and 8, and source-to-site distances of 10 to 250 km. The acceleration response spectrum of the selected ground motions, scaled to 0.5g, is shown in Figure 3. It indicates that the peak response acceleration occurs between 0.5 and 1 seconds for most cases.

The mean annual frequency of exceedance of ground motions is represented by seismic hazard curves. This study analyzed two curves: one based on the Bangladesh National Building Code (BNBC) zone coefficient, indicating a Peak Ground Acceleration (PGA) of 0.13 g for a 475-year return period and 0.2 g for a 2,475-year return period, reflecting moderate seismicity. The second curve, derived from Haque et al. (2019), shows a PGA of 0.26 g for a 475-year return period and 0.58 g for a 2,475-year return period, representing severe seismicity. We plotted these curves as per Equation (1), where A_r is the arbitrary PGA value, and k is the slope of the line. Both curves were plotted in log-log scale and compared with Japan's approximate hazard curve, as shown in Figure 4.

$$\lambda(IM) = \left(\frac{A_r}{A_0 T_0^{-\frac{1}{k}}}\right)^{-k} = \left(\frac{A_r}{\alpha}\right)^{-k} \tag{1}$$

3. METHODOLOGY

The study was divided into three phases. In phase 1, we evaluated the ultimate drift limit of the 1st floor of the target structure and compared it with the demand story drift, determined using the ESA, capacity spectrum (CSM), and non-linear time history (THA) methods. In the second phase, we carried out a probabilistic seismic evaluation of the target building for moderate and severe seismicity based on the results of the incremental dynamic analysis (IDA). In the final phase, we examined two revised hypothetical models and compared the collapse probability of the original model with modified models for both seismic hazard curves.

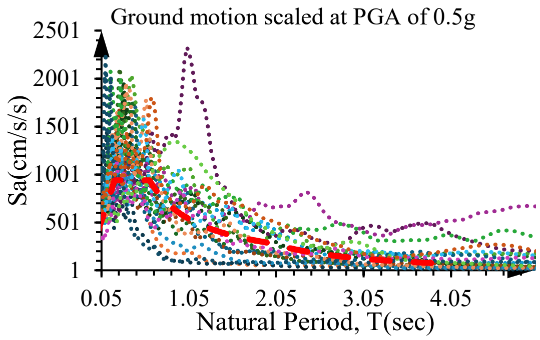


Figure 3: Acceleration Response Spectrum of Selected Earthquake (h = 5%)

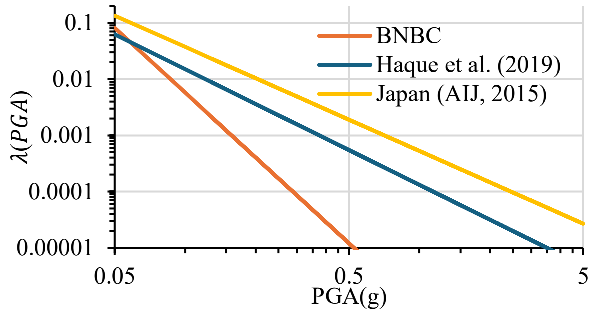


Figure 4: Approximate Seismic Hazard curve (log-log scale)

4. RESULT AND DISCUSSION

4.1 Capacity Spectrum Method

The force-deformation relationship of the bare frame model of the target building was identified using a nonlinear static push-over analysis. The structure was modeled using STERA 3D software developed by Saito, T. (2025).

The performance point in the x-direction, representing the demand deformation and corresponding seismic force, was determined using the capacity spectrum method, as illustrated in Figure 5.

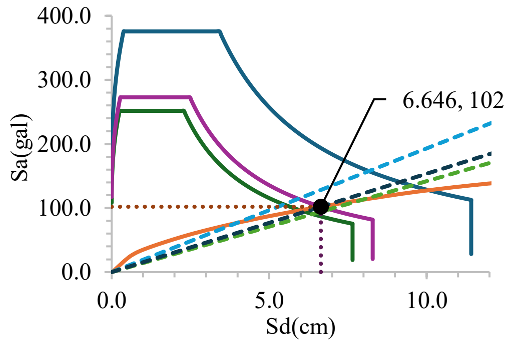


Figure 5: Performance point (x-direction)

4.2 Ultimate Drift Limit

For this study, the deformation capacity of the least ductile column was assumed as the story drift limit, which was determined using the arch and truss method outlined in the AIJ Structural Design Guidelines for RC Buildings, 1994.

Among 13 columns, X3-Y6 and X3-Y8 had the lowest ductility factors. The difference between the shear and flexural capacities of both columns is minimal, and the shear capacity in the plastic hinge zone decreases as plastic rotation increases. This interaction, illustrated in Figures 6 and 7, depicts that the ultimate drift limit of columns X3-Y6 is 0.015 rad, and X4-Y8 is 0.0185 rad.

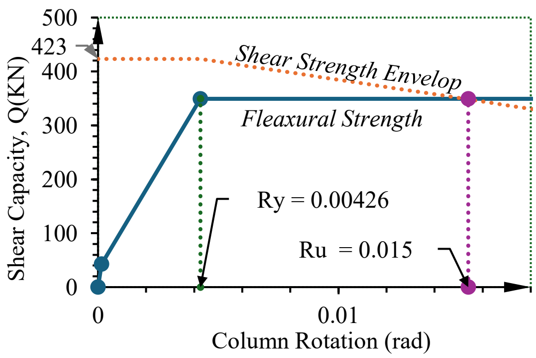


Figure 6: Shear force-rotation relationship of column X3-Y6 (for X-Direction loading)

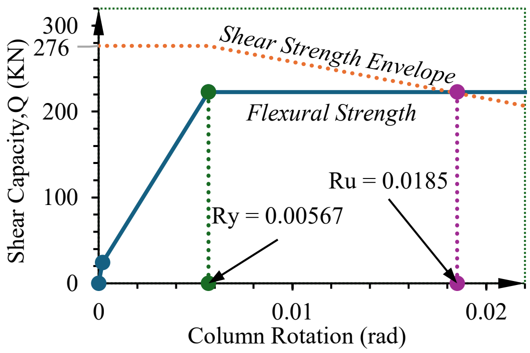


Figure 7: Shear force-rotation relationship of column X3-Y8 (for X-Direction loading)

4.3 Comparison of Demand Drift

The demand deformation associated with the ESA method is calculated by amplifying the elastic deformation based on the design seismic force, using a deflection amplification factor, Cd. For the target structure, the Cd value is 4.5 (BNBC 2020). The demand story drift ratios, related to this value and the performance point in the CSM, were determined using the output of STERA 3D analysis. The median of the time history responses of 20 earthquake motions, scaled to 0.15g, was compared with the response of the ESA and CSM, as illustrated in Figure 8.

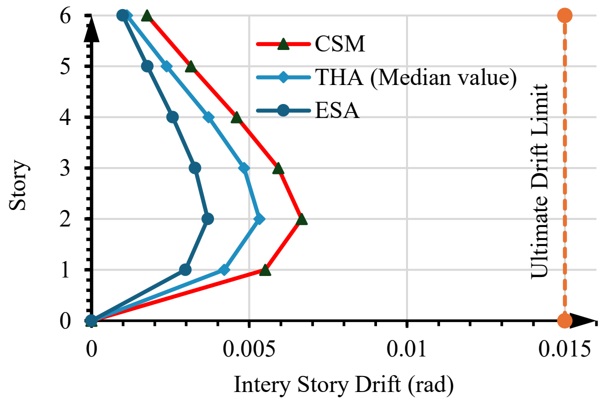


Figure 8: Comparison of Demand Story Drift Ratio

4.4 Result of Incremental Dynamic Analysis

The nonlinear static analysis methods demonstrate a complete range of structural behavior from elasticity to yielding and finally collapse. However, this does not reflect variability in structural response. Therefore, to predict seismic response, the nonlinear dynamic analysis is instrumental (Vamvatsikos, 2002). For the incremental dynamic analysis (IDA), we utilized twenty ground motion data sets, obtained from the PEER ground motion database. The PGA of the ground motions was scaled from 0.05 g to 0.8 g, with constant intervals of 0.05 g. The results of the IDA are shown in Figure 9

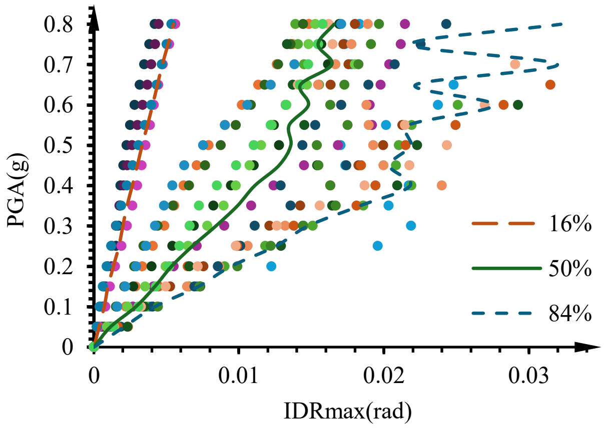


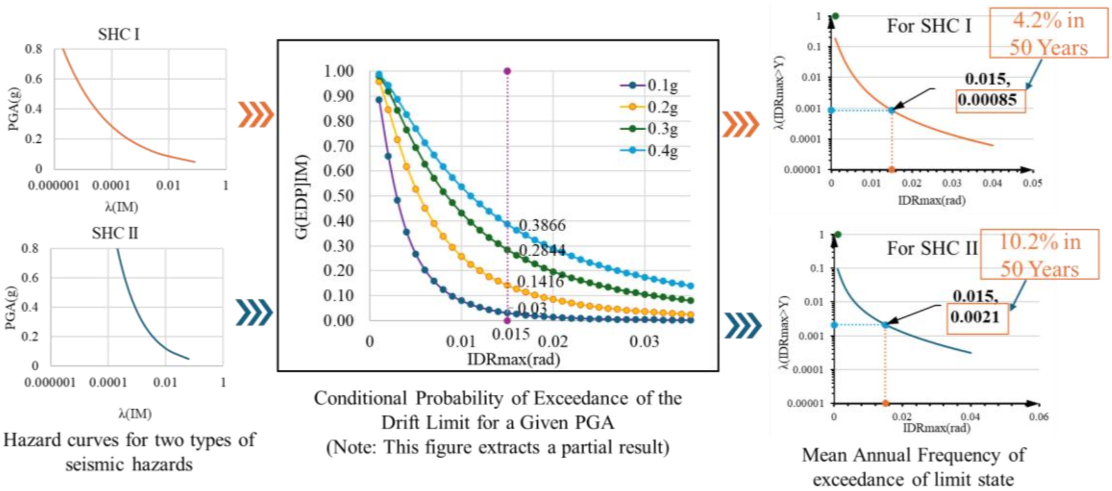
Figure 9: Result of IDA of the target building

4.5 Collapse Probability of the Target Building

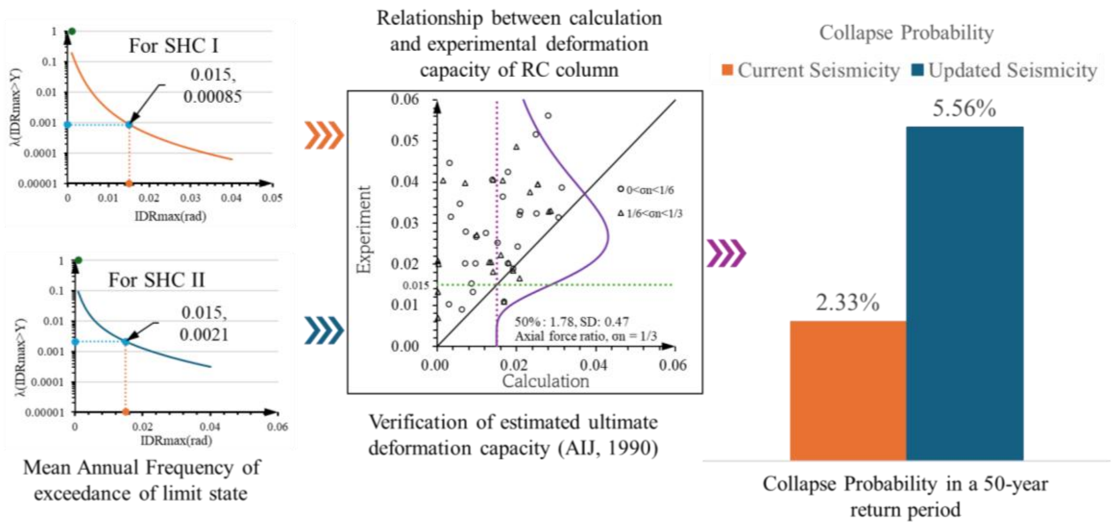
The collapse probability was determined by numerical integration using Eq. (2) and Eq. (3) for both current (SHC I) and updated (SHC II) seismicity from the response of IDA, as shown in Figure 10. The expression *G(EDP|IM)* represents the conditional probability of exceedance for a given intensity measure (IM), and $\lambda(EDP)$ denotes the mean annual frequency of exceedance of the limit state. ECP is the Engineering capacity parameter (Design limit), φ denotes the probability density function. The corresponding value of exceedance probability of ultimate limit state in a 50-year return period for moderate and severe seismicity was found to be 2.33% and 5.56% respectively.

$$\lambda(EDP) = \int_0^\infty G(EDP|IM)d\lambda(IM) \approx \sum_{i=1}^{n} k\left(\frac{IM_i}{\alpha}\right)^{-k-1} . \frac{1}{\alpha} . G(EDP|IM_i)\Delta IM \tag{2}$$

$$\lambda(DM) = \sum_{i=1}^{n} \lambda(EDP). \varphi\left(\frac{\ln(\frac{EDP}{ECP}) - \ln(\frac{EDP}{ECP50\%})}{d_{eq}}\right) \times \Delta(\frac{EDP}{ECP}) \tag{3}$$



(a) Calculation of $\lambda(EDP)$



(b) Calculation of $\lambda(DM)$

Figure 10: Procedure for collapse probability analysis with two kinds of seismic hazard

4.5 Design Modification

Two modified models were analyzed for this study: i) Design modification by decreasing tie spacing in the plastic hinge zone of column X3-Y6 and X3-Y8, and ii) Design modifications by inserting an RC infill wall from the 1st floor to the 6th floor. For modified model-I, the spacing of shear reinforcement in the column plastic hinge zone was revised from 127 mm to 100 mm and 115 mm for column X3-Y6 and X3-Y8, respectively. For the modified model-II, RC walls are infilled between X3-Y6 and X3-Y8, X3-Y3 and X4-Y3, and X3-Y5 and X4-Y5. The median response of

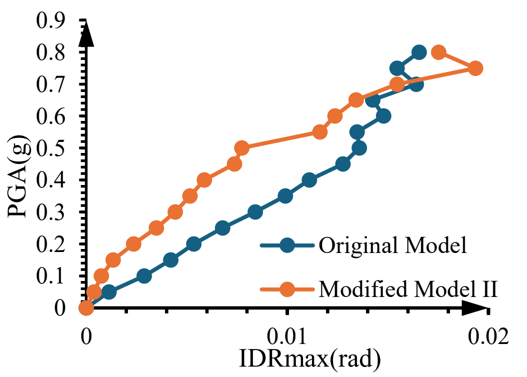


Figure 11: Comparison of the 50th percentile curve of the original and modified model-II

the IDA of the modified model-II has been compared with the response of the original model, as shown in Figure 11. The ultimate drift limit for the modified model I was found to be 0.02 rad, and for the modified model II, 0.004 rad., using the AIJ guideline's arch and truss method.

4.6 Comparison of the collapse probability

To assess collapse probability, we analyzed the ratio of experimental to calculated deformation capacity. The median ratio for columns is 1.78 (dispersion: 0.479), while for RC infill walls, the values are 3.1 (dispersion: 0.46). The final collapse probabilities are summarized in Figure 12.

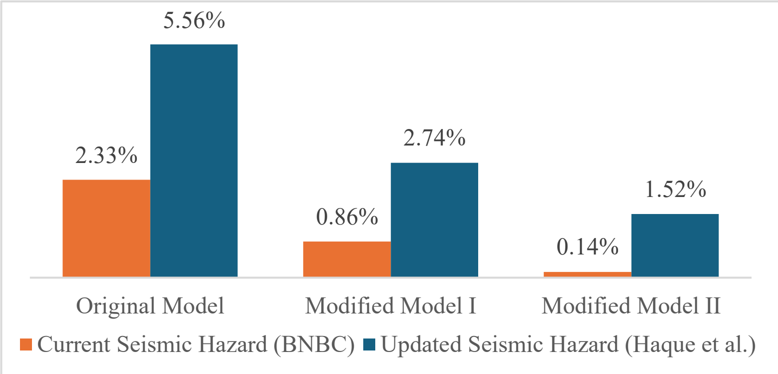


Figure 12: Collapse Probability of Original and Modified Models

5. CONCLUSION

The comparison of the deformation demand between the ESA, CSM, and THA indicates that the linear static analysis has limitations in evaluating the nonlinear deformation of low-strength structures. In addition, the probabilistic seismic evaluation depicts that the collapse probability of the target building for updated seismicity is 5.56% which is unacceptable. However, it can be significantly reduced only by increasing the shear reinforcement in the plastic hinge zone without changing the column size, as well as by inserting RC infill walls. RC infill walls are also very effective in enhancing structural safety by minimizing damage to non-structural elements.

ACKNOWLEDGEMENTS

I want to extend my deep appreciation to Dr. Tatsuya Azuhata for his exceptional supervision and unwavering support throughout my research journey. I also express gratitude to the Japan International Cooperation Agency (JICA) for giving me the opportunity to study Earthquake Engineering at the National Graduate Institute for Policy Studies (GRIPS) and the International Institute of Seismology and Earthquake Engineering – Building Research Institute (IISEE-BRI).

REFERENCES

Haque, D. M. E., Khan, N. W., Selim, M., Kamal, A. M., & Chowdhury, S. H. (2020). Towards improved probabilistic seismic hazard assessment for Bangladesh. *Pure and Applied Geophysics*, *177*(7), 3089-3118.

Pacific Earthquake Engineering Research Center (2005). PEER Strong Motion Database. Berkeley. [online] https://ngawest2.berkeley.edu/

Saito, T. (2025) 'Structural earthquake response analysis', *STERA_3D Version 11.5.*[online] https://rc.ace.tut.ac.jp/saito/software

Vamvatsikos, D., & Cornell, C. A. (2002). Incremental dynamic analysis. *Earthquake engineering & structural dynamics*, *31*(3), 491-514.

Bangladesh code: Bangladesh National Building Code, 2020

Japan code: AIJ Structural Design Guidelines for Reinforced Concrete Building, 1994