Performance Evaluation of RC Special Moment Frames Using Nonlinear Time History Analysis

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Abstract. Bangladesh, located in a seismically active region, faces significant earthquake risks, particularly in high seismic zones like Sylhet. The Bangladesh National Building Code (BNBC) provides design guidelines, including the equivalent static load method and dynamic analysis method, to ensure structural safety. While the static method is widely used for its simplicity, it often fails to capture the true structural response during seismic events. This study evaluates the seismic performance of reinforced concrete (RC) buildings in Bangladesh using nonlinear time history analysis. By incorporating nonlinear auto hinges in beams and columns, the damage levels of structures were assessed. In the absence of local earthquake data, representative time histories were generated by matching the SC soil response spectrum with global earthquakes such as El Centro, Northridge, and Kobe, scaled to simulate varying seismic intensities. Building frames designed per BNBC were assessed against performance levels for serviceability, design, and maximum earthquakes. Results show that regular frames in Sylhet meet both local and global performance criteria. However, buildings designed for gravity loads or with soft-story configurations performed poorly under maximum earthquakes, highlighting the limitations of the equivalent static method. Additionally, the study examined the impact of column tie spacing on ductility. Reduced tie spacing enhanced ductility, improving seismic performance. The findings underscore the need for advanced design methods, such as the response spectrum or time history approach, and strategies to improve ductility for irregular or soft-story buildings. These measures can enhance Bangladesh's resilience to seismic hazards. Enhanced ductility measures to ensure the safety of irregular buildings.

Keywords: Non linearity, Time history analysis, SDOF, Drift ratio, Soft story

1 Introduction

1.1 Background

Bangladesh, near the Indian-Eurasian Plate boundary, frequently faces seismic activity, especially in Sylhet, a high-risk zone. Rapid urbanization has led to buildings often lacking earthquake-resistant standards, increasing the risk of severe damage. The 2020

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revision of the Bangladesh National Building Code (BNBC) provides seismic design guidelines, and nonlinear time history analysis was conducted on RC moment frame buildings in Sylhet using scaled earthquake data to identify weaknesses. The study also validated linear analysis with ETABS and evaluated performance levels against Federal Emergency Management Agency of USA (FEMA) and Applied Technology Council (ATC) standards, contributing to safer building designs [1-3]. Studies on seismic performance through nonlinear time history analysis reveal significant findings. It has been observed that despite the claims of codes such as International Building Code (IBC) 2009 and American Concrete Institute (ACI 318- 2014) [1, 4], many buildings have shown unacceptable performance and even collapse during near-source earthquakes, highlighting a need for improved code provisions [5]. Multi-story buildings not properly designed to resist seismic forces have the potential for total collapse, underscoring the importance of rigorous seismic analysis to ensure structural safety [6]. Adding more ground-motion parameters in seismic fragility analysis has been found to significantly reduce scatter and better account for uncertainties [7-8]. Full 3D nonlinear time history analysis of dynamic soil-structure interaction has enhanced the understanding of complex seismic behavior [9]. Evaluation of a collapsed RC building during the Van earthquake revealed that the structure exceeded collapse prevention levels, emphasizing the need for robust seismic design [10]. Additionally, studies on reinforced concrete frame buildings in China further reinforce the necessity for improved seismic performance to prevent structural failure during earthquakes [2].

The objective of including earthquake resistant design provisions in this Code is to offer guidelines for designing and constructing new buildings that are exposed to seismic ground movements. The Bangladesh National Building Code (BNBC) contains distinct guidelines for calculating earthquake loads and analyzing structures that are exposed to seismic activity. For assessing the severity of earthquake BNBC divides our country in four zones. The most severe earthquake prone zone, Zone 4 is in the north-east which includes Sylhet and has a maximum PGA value of 0.36g [11].

1.2 Earthquake Hazard Levels and Analysis Methods

Earthquake hazards are classified as Maximum Credible Earthquake (MCE), with a 2% chance of occurring in 50 years causing severe but non-collapsing damage; Design Based Earthquake (DBE), with a 10% chance in 50 years causing minor, repairable structural damage; and Serviceability Limit Earthquake (SLE), with a 50% chance in 50 years primarily affecting non-structural elements. Performance-Based Earthquake Engineering (PBEE) designs structures based on expected performance, using methods like equivalent static analysis, pushover analysis, and time history analysis to minimize damage and loss of life [11].

Non-Linear Time History Analysis. Nonlinear time history analysis (NTHA) is a detailed method for assessing structural response under seismic loads, accounting for material and geometric nonlinearity. It uses the Newmark method to integrate equations of motion, updating displacements, velocities, and accelerations while incorporating damping effects. Nonlinear hinge properties, defined by FEMA 356 describes moment-rotation relationships at structural joints, with performance levels indicating damage severity.

Structural Performance Level and Range. Building performance during seismic events is evaluated by FEMA 356's four structural performance levels: Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP), with two intermediate levels. IO ensures safety with minimal damage, LS involves significant damage with some collapse margin, and CP indicates severe damage with high collapse risk [12]. These levels guide risk assessment and mitigation during earthquakes. These structural performance levels are also defined by ATC 40. ATC 40 describes the performance levels based on Maximum total drift and Maximum inelastic drift. The limit of maximum total drift for IO, LS and CP is 0.01, 0.02 and 0.04 respectively whereas the limit of maximum inelastic drift for IO is 0.005, LS and CP have no limit [13]. On the other hand, FEMA 365 defines these three levels by primary damage, secondary damage and drift [12].

2 Methodology

2.1 Comparison between Linear and Nonlinear Time History Analysis

The Kobe earthquake was used to simulate seismic conditions for Sylhet to assess the maximum seismic displacement demand of six-story reinforced concrete frames [6]. Both linear and nonlinear time history analyses were performed using ETABS 2017, and the results were compared. The result of the analysis is showed in Fig. 1. It tells that target displacements estimated by approximation techniques differed from actual results, with nonlinear time history analysis yielding higher maximum displacements (15 inches) compared to linear time history analysis (10 inches). So nonlinear analysis is necessary to compare the real nonlinear behaviour of structures.

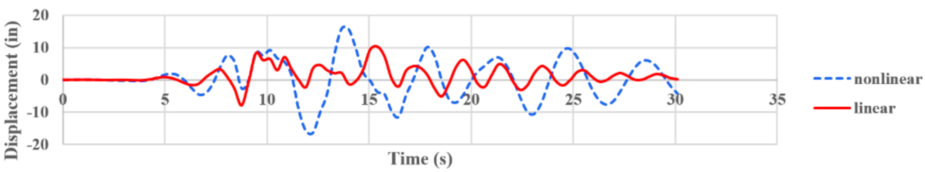

Fig. 1. Linear And Nonlinear Displacement of Generated Earthquake.

2.2 Validation of Nonlinear Time History Analysis for SDOF System

Nonlinear time history analysis (NTHA) is a detailed method for modelling structural responses to seismic events, offering more accuracy than pushover analysis despite its greater time requirement. The nonlinear response of structures is highly sensitive to both structural modelling and the characteristics of ground motion. To account for uncertainties and variations in severity, frequency, and duration, three distinct ground motion records were used to predict possible deformation modes in seismic performance evaluations. Using ETABS 2017, NTHA was conducted on scaled ground motion data to determine key response parameters, such as story displacements, inter-story drift ratios, and base shears. The analysis also identified plastic hinge locations.

The performance of structures designed per BNBC 2020 was evaluated against ATC 40 and FEMA 356 criteria to ensure compliance with safety standards. Additionally, the response of linear Single Degree of Freedom (SDOF) systems to earthquake motion was studied and validated with established results from the research of Chopra et. al. [14]. SDOF models were constructed with appropriate stiffness and mass to simulate desired time periods. The research involved analyzing a 12 ft vertical cantilever concrete column, with a cross-section of 20″ × 20″ and a 38,636 lb/ft-s² load at the apex. The column's response was examined using El Centro ground motion data, demonstrating the approach's application in seismic analysis. For a given ground motion (El Centro ground motion), the deformation response of an SDOF system depends only on the natural vibration period of the system and its damping ratio. The deformation response of five different systems due to El Centro ground acceleration was used for validation.

Table 1. The result of Time History Analysis of the column used by Chopra et. al. on ETABS.

T_n (s)	Damping ratio	Displacement (in)
0.50	0.02	2.69
1.00	0.02	5.88
2.00	0.02	7.53
2.00	0.00	9.95
2.00	0.02	7.53
2.00	0.05	5.39

According to Chopra et al. [14], the displacement was measured at varying damping ratios. The results obtained from ETABS closely matched those from the physical test, indicating that the findings are validated and that the NHTA simulation in ETABS accurately represents real-life structural behavior.

2.3 Design of the Building

A six storied reinforced concrete building is designed in ETABS according to BNBC 2020 [11]. Building dimension is 60ft × 60 ft. Each story is 10 ft high. The building has three bays in each direction. The length of each bay is 20ft. It is fixed at its support. For earthquake analysis, earthquake load is calculated using static analysis procedure. From BNBC Table 6.2.20 the value of Ct = 0.0466 for concrete moment resisting frame is considered. Building height exponent K is calculated using building time period. We found it to be 1.0687. 5% damping is considered for earthquake analysis. Also, for analysis, we considered the direction and eccentricity to be in all six categories available in ETABS. The design parameters and loads considered according to BNBC for ETABS analysis is given in table 2.

Table 2. Loads and Parameters Considered for Analysis

Loads and Design Parameters	
Live load (Floor)	40 psf
Partition wall load	60 psf
Floor finish	30 psf
Additional dead load	Self-weight, Building facilities
Earthquake Design	Static Analysis Procedure
Occupancy Category	II
Importance Factor	1
Site Class	SC
Seismic Zone	Zone-IV, Sylhet city
Seismic Zone Coefficient	Z=0.36
Structural System	SMRF
Wind Load Design	Analytical Design Procedure
Exposure Category	Exposure A
Topographic Factor	Plain Terrain Area, Topographic Factor=1
Basic Wind Speed	136.67 mph
Concrete strength, fc’	4 ksi
Yield strength of steel	60 ksi

Total 25 load combinations were considered according to BNBC 2020 for the analysis [11]. After analyzing, we found the column dimension to be 18″ × 20″ and beam dimension to be 12″ × 18″. Total 8 rebars of 1.125″ diameter (#9) were provided for the column design. Tie spacing was designed according to the requirement of Special Moment Resisting Frame (SMRF) detailing.

The structural design uses concrete with a compressive strength of 4000 psi and an isotropic material with a unit weight of 150 pcf. The concrete's hysteretic behavior has an energy degradation factor of 0.5. Strain limits for performance levels are set as follows: 0.0008 for immediate occupancy, 0.0023 for life safety, and 0.006 for collapse prevention. Tension acceptance criteria are ignored in the analysis. Reinforcement bars with a yield strength of 60,000 psi are used, with kinematic hysteresis. A uniaxial material model, indicating directionally varying properties, is applied. Strain limits for the reinforcement are 0.01 for immediate occupancy, 0.05 for life safety, and 0.107 for collapse prevention. This nonlinearity is briefly illustrated in Fig. 2.

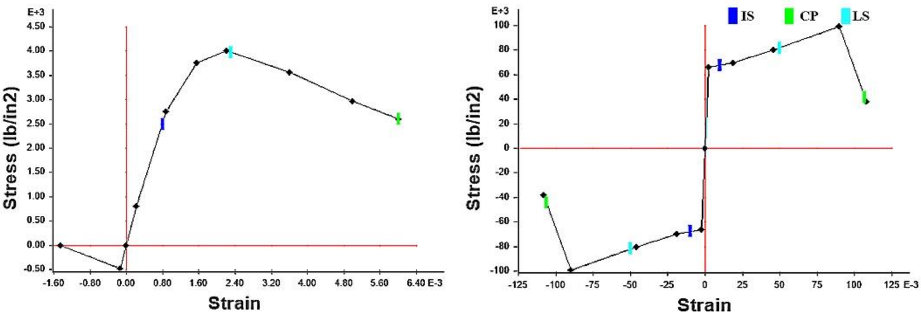

Fig. 2. Non-linearity in Stress-Strain curve for Concrete (Left) and Steel (Right)

Here, IO: Immediate Occupancy, LS: Life Safety, CP: Collapse Prevention

The automatic assignment of hinges was carried out in accordance with ASCE/SEI Table 41-17 [1]. For the analysis of the beams, M3 hinges were utilized. Two hinges were placed on each beam: one at a distance of 0.02L from one end and the other at a distance of 0.98L from the same end (Fig. 3). For the Immediate Occupancy (IO) limit state, a plastic rotation to strength factor (SF) ratio of 0.0085 was applied. For the Life Safety (LS) limit state, a value of 0.023 was used. For the Collapse Prevention (CP) limit state, a value of 0.047 was adopted.

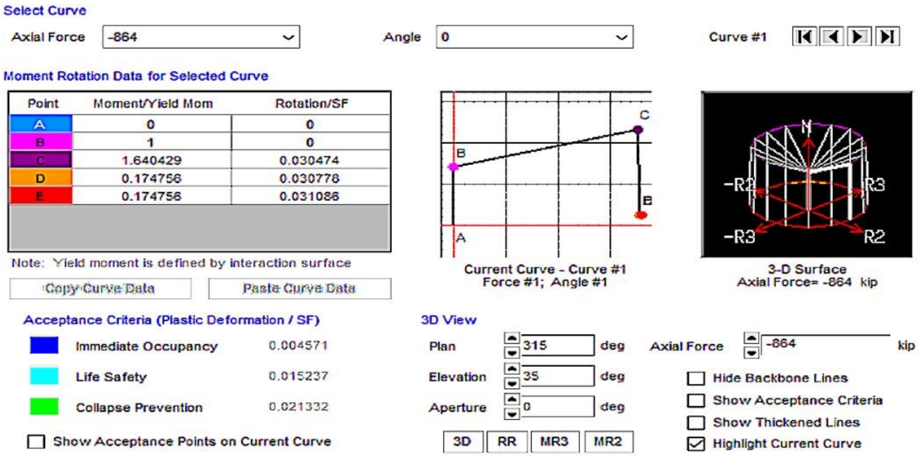

Fig. 3. Hinge Property considered for of Beams.

For the analysis of the columns, P-M2-M3 hinges are applied, with two hinges assigned to each column. One hinge is placed at a distance of 0.02h from the base, and the other at 0.98h from the base (Fig. 4). The plastic rotation/section factor (SF) limits for different performance levels are set as follows: 0.004571 for Immediate Occupancy (IO), 0.015 for Life Safety (LS), and 0.021 for Collapse Prevention (CP).

The analysis of the designed model was conducted using ETABS 2017. Mode shape 1, with a time period of 0.857 seconds, and mode shape 2, with a time period of 0.272 seconds, were utilized for the time history analysis. Lateral sway corresponding to these mode shapes was considered, and the time period associated with each mode shape was used in the analysis.

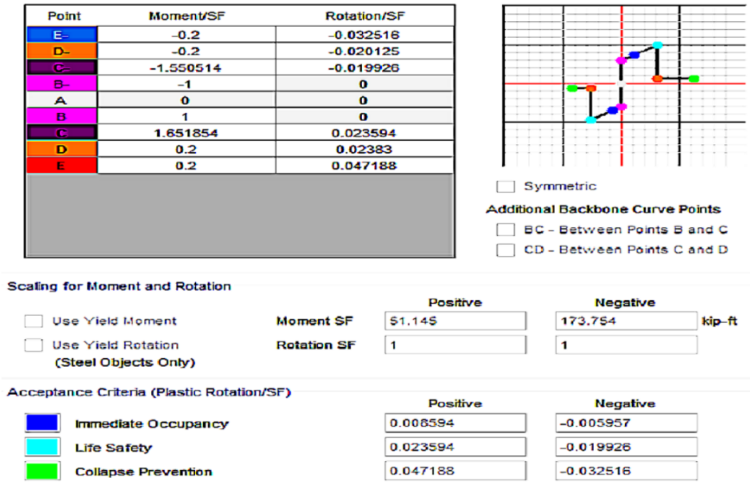

Fig. 4. Hinge Property considered for of Columns.

2.4 Time History Analysis Using Selected Earthquake

Time history analysis involves applying ground motion acceleration to a structure's base to model its dynamic response over time. This analysis can be either elastic or inelastic, with nonlinear time history analysis (NTHA) providing more accurate results for reinforced concrete (RC) frames under dynamic loading. Unlike linear analysis, which assumes constant material and geometric properties, NTHA accounts for changes in stiffness, mass, and damping due to significant deformations during seismic events. For this study, three well-recorded earthquakes were selected to analyse the ETABS model:

- El Centro (Imperial Valley, May 18, 1940): North-South component with a peak ground acceleration (PGA) of 0.31882g, a magnitude of 7.1, and a duration of 31.18 seconds.
- Kobe (Japan, January 17, 1995): PGA of 0.8g and a duration of 20.95 seconds.
- Northridge (Los Angeles, January 17, 1994): PGA of 0.4g, magnitude of 6.7, and a duration of approximately 20 seconds [6].

These ground motions were scaled appropriately for analysis, with specific "zone" factors applied: Z = 0.31 for El Centro, Z = 0.8 for Kobe, and Z = 0.4 for Northridge. The data was sourced from the PEER Ground Motion Database. The study focused on the response of the RC frames, including story displacements, inter-story drift ratios, base shears, and plastic hinge locations.

Using the Response spectrum of our location and response spectrum of the mentioned earthquakes, a matched response spectrum was generated for Sylhet region [6]. From Fig. 5, the blue coloured trend represents the response spectrum of the original earthquake while the red coloured trend represents the matched response spectrum for Sylhet region.

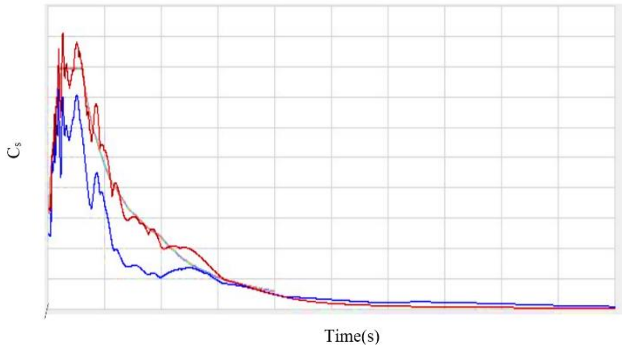

Fig. 5. Matched Response Spectrum for El Centro.

Using the matched response spectrum, we created representative earthquakes shown in Fig. 6 for El Centro, Kobe and Northridge, with the blue graph showing the original earthquakes and the red graph representing these earthquakes for the Sylhet region.

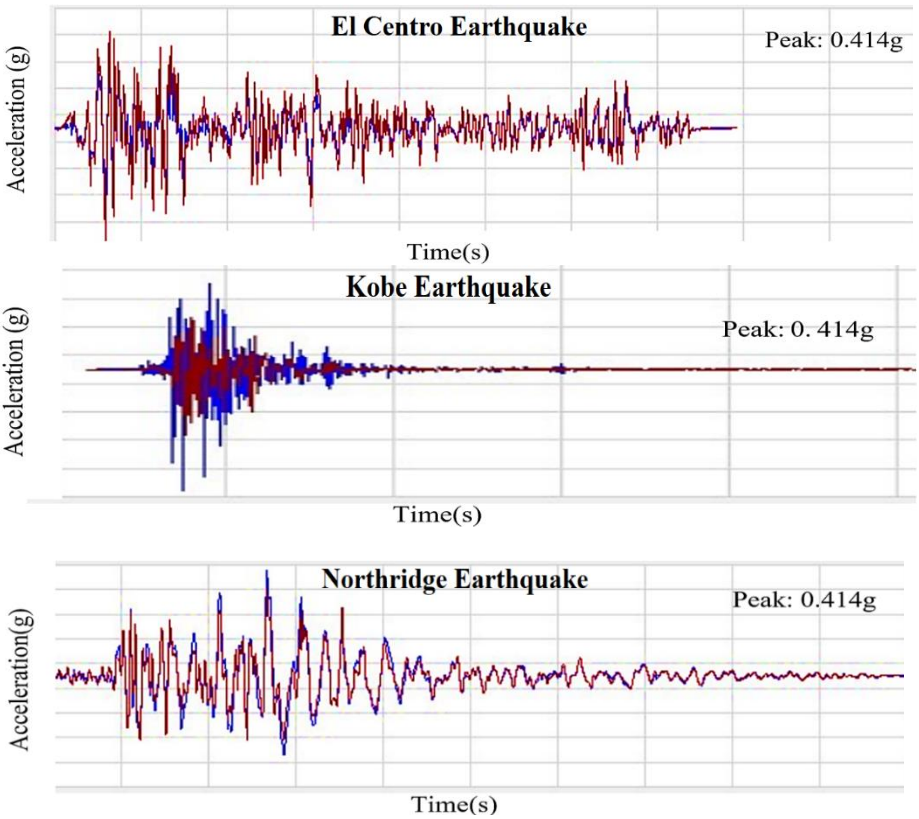

Fig. 6. Generated Earthquakes using reference Earthquakes [6].

3 Result and Discussion

3.1 Local Level Performance

Considering the model under different seismic events, hinge formation was analysed for Maximum Credible Earthquake (MCE), Design Basis Earthquake (DBE), and Serviceability Earthquake (SLE) levels using the El Centro, Northridge, and Kobe ground motions (Fig. 7). No color in the beam column joint means no hinge is formed, the green color means the performance is below LS level and the cyan color means below CP level. The red color means the corresponding joint is failed.

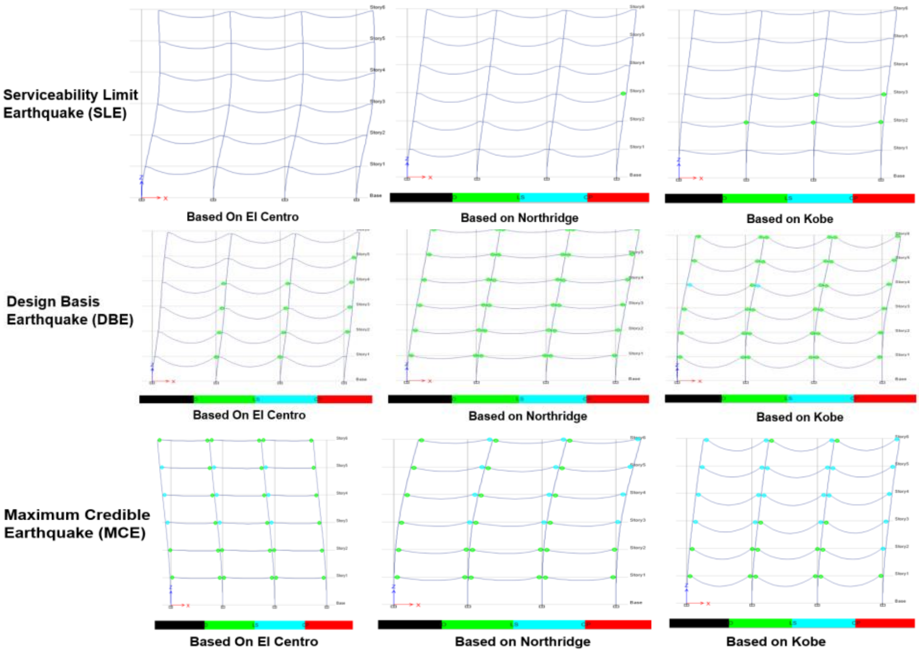

Fig. 7. Local Level Performance for three reference earthquakes.

- *MCE Level:* No hinges formed in the columns, but numerous hinges appeared inn the beams. Most beam hinges ranged from IO to LS, with some between LS and CP, but none exceeded CP. This indicates that while the structure will experience damage, it will not collapse during an MCE. Damage was most significant in the Kobe earthquake due to its higher PGA.
- *DBE Level:* Similar to MCE, no column hinges were observed, and beam hinges predominantly fell within the IO to LS range. Only two hinges reached the LS to CP range, particularly in the Kobe event. The structure will endure minor beam damage but remains safe, fulfilling local performance criteria per ATC-40.
- *SLE Level:* No column hinges were detected, and all beam hinges stayed within the IO to LS range. While minor damage was noted, especially in the Kobe scenario, the

structure will not collapse, although it doesn't entirely meet local criteria due to beam damage. However, the overall safety of the building is maintained.

3.2 Global Level Performance

From Fig. 8, it is clearly visible that the performance point of the designed model has maximum story drift less than allowable IO level 0.01 for El Centro earthquake. But for Northridge and Kobe earthquake drift ratio exceed IO level but less than LS level 0.02. So, for Northridge and Kobe earthquake the structure will face minor damage at MCE and DBE level earthquake but will be okay at SLE earthquake. So, this structure satisfies the requirement of ATC 40. Therefore, it has been said that 6-story frame structure fulfils the performance objective at global level in SLE, DBE and MCE. After the analysis, it was found that the frames designed by BNBC for Sylhet city can easily satisfy the local and global performance requirements. So, designing a building using BNBC 2020 code is safe and convenient.

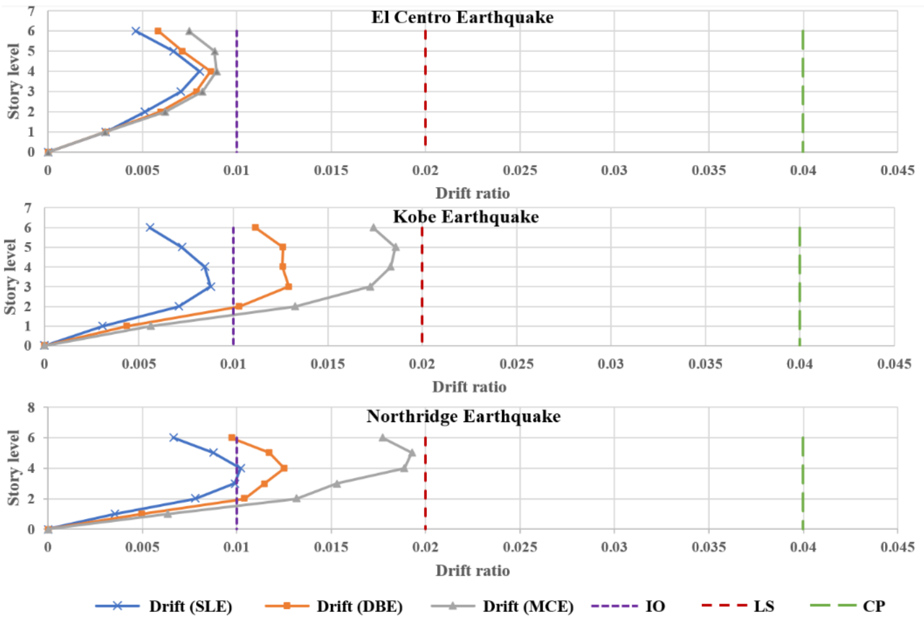

Fig. 8. Global Level Performance for different level Earthquakes.

3.3 Effect on Soft Story Building

A soft-story building is a multi-story structure with a ground floor that has significantly less structural stiffness or strength than the floors above, often due to an open or partially open design. This makes such buildings particularly vulnerable during earthquakes, as the weaker ground floor may not provide adequate lateral support. To analysis the seismic performance of a soft-story building, a five-story structure was designed following BNBC 2020 guidelines. The ground floor was intentionally made twice as tall as the other floors, creating the soft-story condition. The building was designed with a column section of 20″ × 20″ and a beam section of 12″ × 18″. Nonlinear hinges were assigned to both beams and columns to assess potential damage under various earthquake scenarios.

The building's local performance was evaluated under SLE, DBE, and MCE. At SLE, hinges formed in the beams within the IO to LS range, indicating minor damage. During DBE, hinges appeared in all beams, with some reaching the CP range, indicating significant damage but no collapse. At MCE, severe damage occurred, with hinges in the interior columns surpassing the CP limit, suggesting the building would likely collapse. Fig. 9 shows us the local level performance for soft story building.

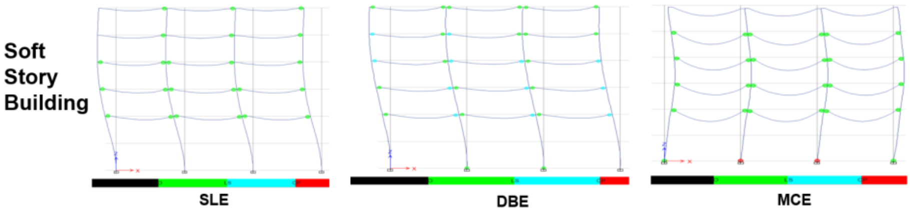

Fig. 9. Local Level Performance of soft story building.

Fig. 10 tells us that the performance point of the soft-story frame structure during serviceability and design basis earthquakes remains below 0.02, indicating it falls between the IO and LS limits. During a maximum credible earthquake, the drift ratio exceeds 0.02 but stays below 0.04, not crossing the CP limit. While the structure meets ATC 40 requirements for DBE and MCE levels, it fails to satisfy the criteria at the SLE level [13].

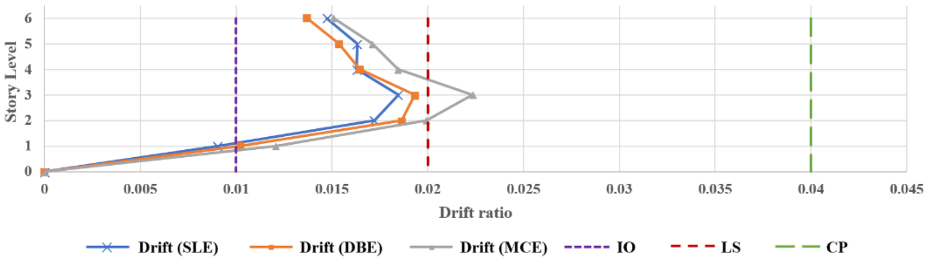

Fig. 10. Global Level Performance for Different Level of Earthquake of a soft story building.

3.4 Analysis of a Building Designed for Gravity Load

A building was designed only for gravity load using the geometric and material property used for designing in the methodology. After analysis, cross section of the column found 15″ × 15″ and cross section of the beam found 12″ × 15″. The local level performance of the structure was assessed using an earthquake generated by matching the response spectrum of the Kobe earthquake. From Fig. 11, we can observe that at the SLE level, hinges formed in the beams but remained within the IO to LS range, indicating minor damage. During the DBE, hinges formed primarily in the beams, with some exceeding the LS limit, suggesting major damage but no collapse. However, at the MCE level, hinges in the columns exceeded the CP limit, indicating that the structure would collapse.

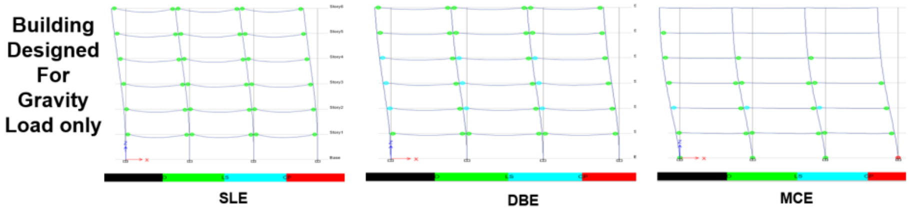

Fig. 11. Local Level Performance of a building designed for only Gravity Load

The soft-story frame structure meets the ATC 40 requirements for SLE, DBE, and MCE levels, with performance points below 0.01 for SLE, between IO and LS for DBE, and a drift ratio below 0.04 for MCE, not exceeding the collapse prevention limit as shown in Fig. 12 [2].

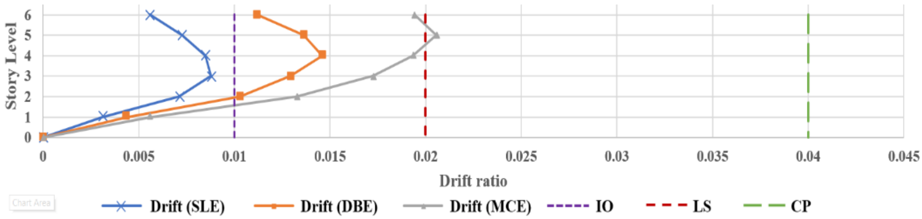

Fig. 12. Global Level Performance of a Building Designed for Gravity Load (ATC 40)

3.5 Effect of Tie Spacing on Ductility

Ductility is crucial for buildings in seismic regions, allowing them to deform and absorb energy without collapse. The study analyzed the effect of tie spacing on ductility, using tie spacings of 6, 8, and 12 inches while keeping column reinforcement constant (Fig. 13). It was found that increasing tie spacing reduces the area of the moment rotation curve, indicating decreased ductility and a higher risk of sudden collapse during a major earthquake.

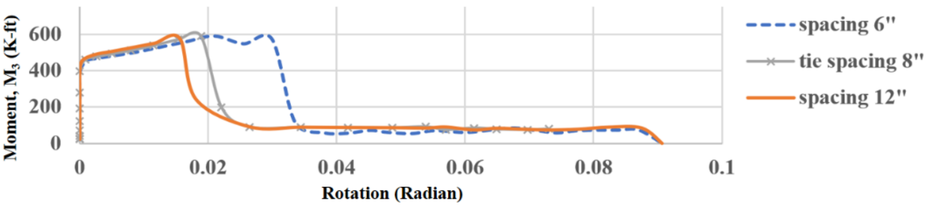

Fig. 13. Moment Rotation Curve for Column at different Tie Spacing

4 Conclusions

In conclusion, the study conducted a comprehensive seismic analysis of regular and irregular RC buildings in Sylhet, Bangladesh, using nonlinear time history analysis. The following key points summarize the findings:

1. Regular buildings designed per BNBC demonstrated strong seismic performance, meeting both local and global performance criteria during small to maximum credible earthquakes.
2. Soft-story buildings, especially with double-height ground floors, exhibited significant vulnerability, with severe damage occurring under maximum earthquakes. This highlights the inadequacy of the equivalent static method for such structures. Buildings designed solely for gravity loads performed poorly, with columns collapsing under maximum seismic conditions, emphasizing the need for earthquake-resistant design in all building types.
3. Reducing column tie spacing improved structural ductility, which enhanced the building's capacity to withstand seismic loads without sudden failure.
4. Nonlinear time history analysis revealed larger displacements and hinge formations compared to linear analysis, suggesting that nonlinear methods provide more accurate assessments for seismic performance, especially for irregular structures.

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