

PERFORMANCE-BASED SEISMIC DESIGN OF REINFORCED CONCRETE BUILDINGS CONSIDERING EFFECT OF SOIL-STRUCTURE-INTERACTION

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ABSTRACT

To withstand catastrophic events like earthquakes and to reduce the physical damages and socioeconomic consequences, civil structures must be designed to function satisfactorily. Although the code-prescriptive, force-based approach aims to provide structures with the necessary strength and ductility for a life safety level, actual performance is never evaluated. In previous earthquakes (such as the Northridge earthquake in 1994 and the Kobe earthquake in 1995), structures built using a code-based method suffered significant physical damage and enormous socioeconomic losses. The performance-based seismic design method, which is still being developed with an emphasis on resilience enhancement, was accelerated in order to improve safety and minimize damage to structures. This study aims to design reinforced concrete buildings following a performance-based earthquake engineering approach, considering soil-structure interaction. An archetype eight-story RC building was selected, and a frame was analyzed and designed following the seismic design approach of the Bangladesh National Building Code 2020. To evaluate the performance of the code-designed building frame, nonlinear time history analyses were conducted utilizing eleven (11) earthquake ground motion records. To verify the chosen performance objectives, the selected ground motion records were matched with acceleration response spectra of various earthquake hazard levels. Story drift, an indicator of damage, was selected as an engineering demand parameter to quantify performance. Then, the frame was designed using the performance-based seismic design approach, meeting the selected performance objectives. Finally, it was evaluated how base flexibility affected the building's responses in force-based and performance-based design approaches. The current study will assist engineers, building owners, and other stakeholders in making well-informed decisions when designing new buildings or enhancing existing ones in order to meet the necessary structural performance.

Keywords: *Performance-based engineering, RC buildings, nonlinear time history analysis, design and serviceability earthquake, SSI*

1. INTRODUCTION

Designing structures to safely withstand various types of loads with required serviceability is one of the main aims of structural engineering. Civil structures are typically designed following the prescriptive codes and standards. Although the current force-based seismic design (FBSD) criteria are intended to provide a certain performance level, actual performance is never assessed as part of the code-based approach (ATC, 2006 & 2018). Building owners, users, and even design professionals generally believe that structures designed with a modern approach with codified requirements are safe. But, destructive damage to structures due to earthquakes, such as the 1994 Northridge and 1995 Kobe earthquakes, made designers rethink that structures even designed with the latest code may experience severe damage (ATC, 2006; Whittaker et al., 2003). Thus, in recent years, earthquake-resistant design is emphasizing on performance rather than strength (Priestley, 2000). A code-based approach emphasizing strength and ductility is not suitable for the performance assessment. There are no code-provided guidelines to determine whether the assumed or better performance levels can actually be achieved.

Performance-Based Seismic Design (PBSD) is an improved procedure for the reliable prediction of the performance or behaviour of structures under earthquakes. This approach can be used to assess or design structures to satisfy different performance levels of structures at different hazard levels (e.g., DBE) with better performance than structures designed with prescriptive criteria (ATC, 2006; Hamburger et al., 2004). PBSD can be utilized in building design and evaluation to meet specific seismic performance goals, minimize possible losses, surpass regulatory restrictions, evaluate existing structures, and enhance seismic design methods in the future (ATC, 2006). Further, the procedure can be applied for performance-based design for other hazards (e.g., fire, wind, flood, and also terrorist attack) (Whittaker et al., 2003).

This study performs seismic analysis, design, and performance assessment of an eight-storied residential building located in Chattogram, Bangladesh. At first, an elastic analysis was performed following the Bangladesh National Building Code (BNBC, 2020) for DBE to satisfy the life safety performance level. Story drift ratio (SDR) was selected as the engineering demand parameter (EDP) for the global performance assessment. Design of columns and beams was done following the code. Then, the building was assessed using the PBSD method and the capacity of elements. In this regard, nonlinear time history analyses (NTHA) were performed using eleven (11) records, which were matched with various earthquake levels. BNBC recommended load combinations were used to assess the capacity of beams and columns. Then, the building was also analyzed for the serviceability level earthquake (SLE). Finally, the building was analyzed considering the soil-structure-interaction effect both with FBBD and the PBSD approach.

2. CASE STUDY BUILDING

The case study building considered in this study is situated on a location with soil type SC (BNBC, 2020) with an average shear wave velocity (V_{s30}) of 300 m/s. The framing system was considered as special reinforced concrete moment frames to fulfil the requirements of BNBC. Figure 1 represents the plan and elevation of the selected building. Bay width and story height are 4.572 m (15 ft) and 3.05 m (10 ft), respectively. Concrete compressive strength and yield strength of reinforcing bar were considered as 28 MPa and 414 MPa, respectively. Various loads were considered as per BNBC 2020. The soil properties and parameters for the soil type are shown in Table 1. The unit weight of soil was considered following ATC 40 (ATC, 1996), the Poisson ratio was considered following ASCE 41 (ASCE, 2017), and the shear modulus was calculated from unit weight and V_{s30} .

Table 1: Soil Properties and Related Parameters

V_{s30} (m/s)	Site Coefficient, S	Poisson Ratio	Unit Weight (KN/m ³)	Shear Modulus (KN/m ²)
300	1.15	0.25	18	165137.6

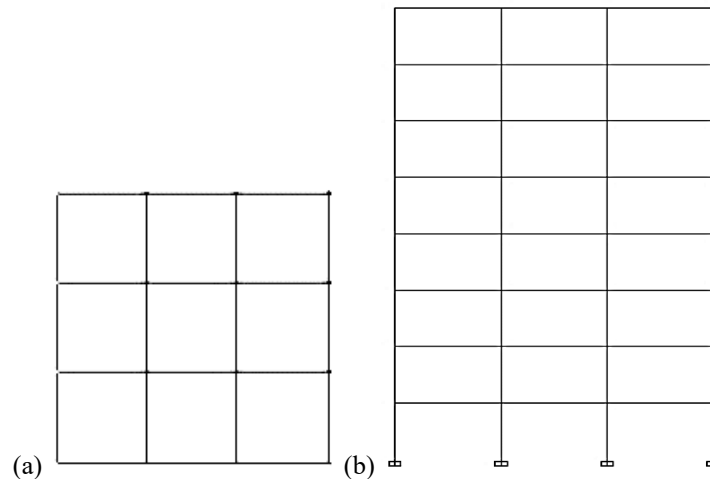


Figure 1: Plan and Elevation of the selected building

3. FORCE-BASED SEISMIC DESIGN

An interior frame of the selected regular RC building was analyzed using the equivalent static analysis (ESA) of the BNBC. The spectral acceleration for the DBE hazard, to estimate design base shear, was evaluated using BNBC. The considered column size was 508mm x 508mm (20"x20") and beam size was 305mm x 508mm (12"x20"). Several other size combinations were also considered to satisfy the SDR and other design requirements. The seismic weight, time period, spectral acceleration, and base shear for ESA are represented in Table 2. This base shear was dispersed over the building's story level. The story displacements were estimated to evaluate story drift as well as SDR. Structural members were designed to resist these forces, with expected inelastic behaviour addressed indirectly through code-specified detailing provisions.

Table 2: Parameters for Equivalent Static Analysis

Seismic Weight, W (kN)	Time Period, T (sec)	Spectral Acceleration, Sa	Base Shear, V (kN)
5139.8	0.826	0.0487	250.4

4. PERFORMANCE-BASED SEISMIC DESIGN

The frame designed with the BNBC approach was then analyzed to assess for life safety performance under DBE matched records and collapse prevention performance for MCE matched records. Material and geometric nonlinearity were considered in the analyses. Nonlinear material modelling was developed using concentrated M3 moment hinges for beams, using ASCE 41, and fiber-based P-M-M interaction hinges were used for columns as per NIST guideline (NIST, 2017). Damping was determined as recommended in various guidelines, e.g., NIST (2017), PEER TBI (PEER, 2017), ATC 72-1 (ATC, 2010). After the analysis, the performance level and strength requirements were checked. Finally, the frame was designed with modified sections based on the results of NLTHA, satisfying all the strength and performance requirements.

4.1 Performance objective

Performance objectives (POs) are related to the performance level of structures corresponding to the earthquake hazard level. Engineers define performance levels for specified seismic intensity levels, e.g., serviceability for SLE, life safety for DBE, and collapse prevention for MCE, which is generally discussed with building owners and users. Figure 2 illustrates POs and mentioned basic safety objective, which generally can be considered for a typical residential building.

Performance Levels Hazard Levels	Operational	Immediate Occupancy	Life Safety	Collapse Prevention
50% in 30 Years	⊙			
20% in 50 Years	Δ	○		
10% in 50 Years (DBE)	⊗	Δ	○	
2% in 50 Years (MCE)		⊗	Δ	○

Figure 2: Hazard level, performance level and performance objectives

Based on the relevant SDR limit for various performance levels provided by standards, SDR limit for various performance level associated with a hazard level were considered as follows:

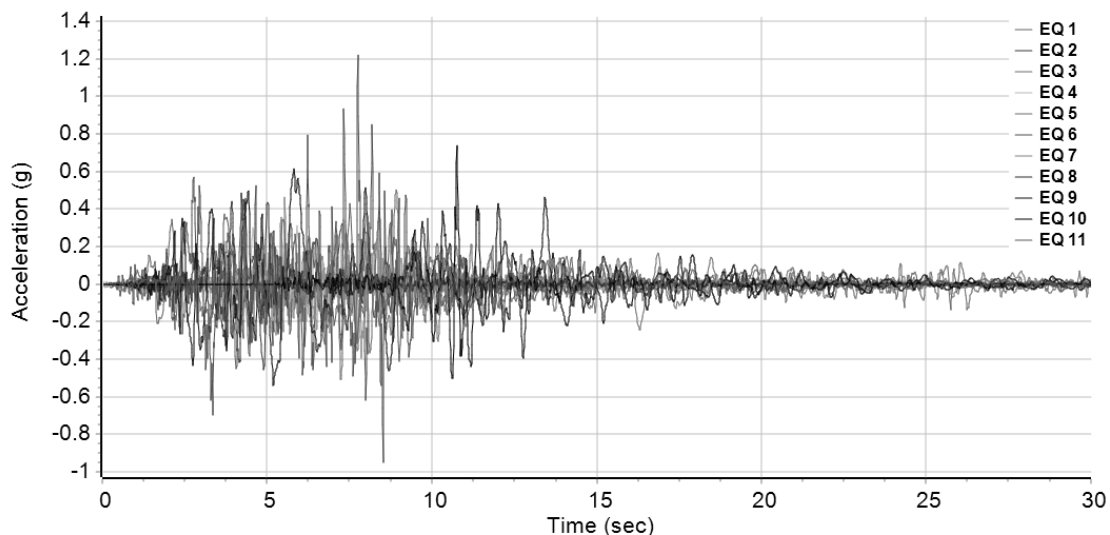
- Serviceability Level (for 43 years RP): SDR limit is 0.5%
- Life Safety (for 475 years RP): SDR limit is 2%.
- Collapse Prevention (for 2475 years RP): SDR limit is 3%.

4.2 Earthquake ground motion records

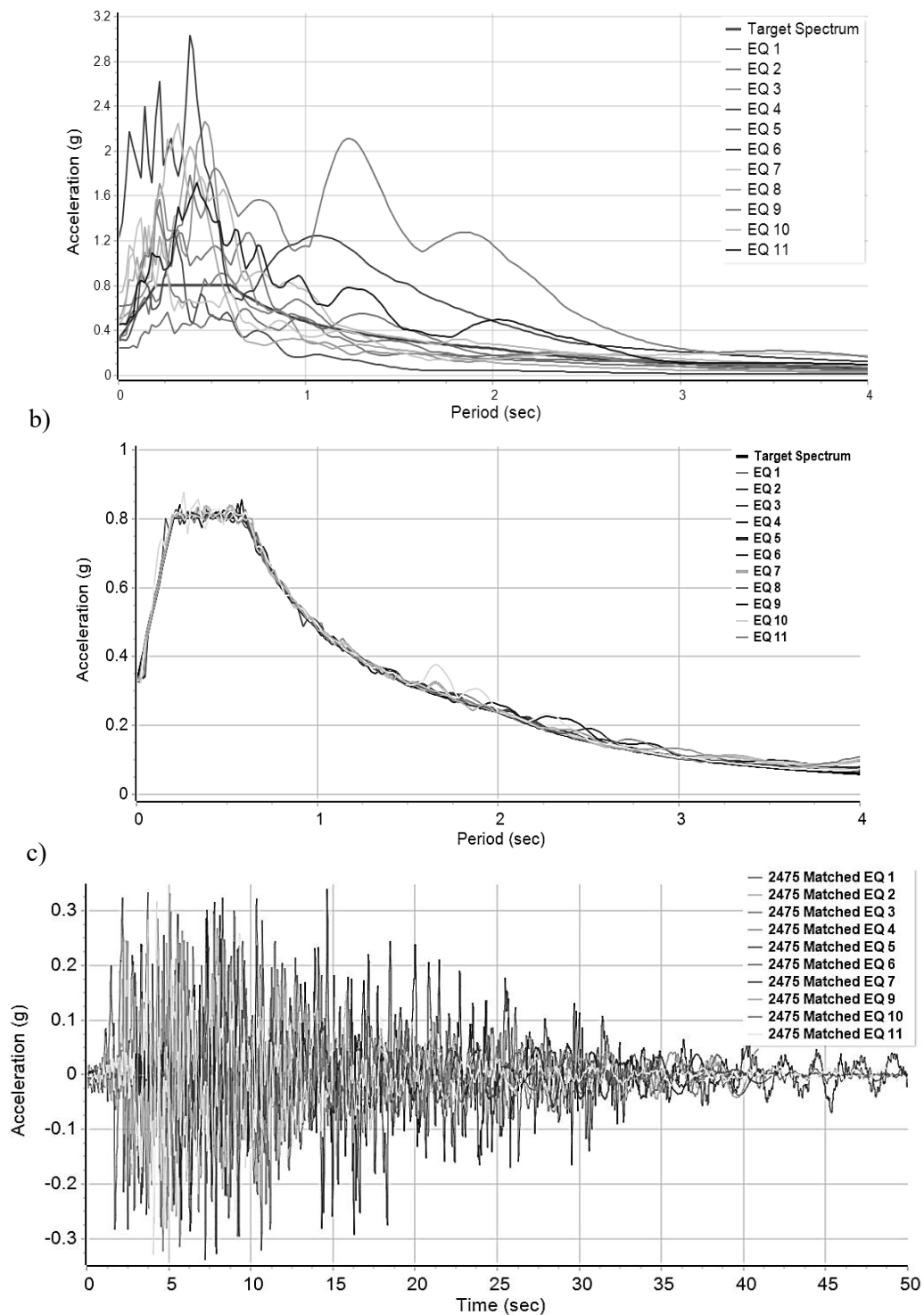
Eleven acceleration ground motion records were used the NLTHA of the building. These records were collected from COSMOS Virtual Data Center (<https://www.strongmotioncenter.org/vdc>) and PEER NGAWEST2 database (<https://ngawest2.berkeley.edu>). Magnitude, and peak ground acceleration are represented in Table 3. The acceleration response spectra for the earthquake records were matched with the target acceleration response spectrum to adjust with the hazard level, e.g., DBE, MCE or SLE. In this study, spectral matching was performed using “SeismoMatch” (Seismosoft, 2020) as per the approach of Al Atik and Abrahamson (2010). Figure 3a illustrates the ground motion intensity records and Figure 3b-d represents unmatched and matched MCE acceleration response spectrum and matched earthquake records, respectively.

Table 3: Selected Earthquake Ground Motion Records

Sl.	Earthquake	Mw	PGA (g)	Sl.	Earthquake	Mw	PGA (g)
EQ 1	Imperial Valley 1940	6.9	0.348	EQ 7	Imperial Valley 1979	6.53	0.466
EQ 2	Northridge 1994	6.7	0.455	EQ 8	Loma Prieta 1989	6.93	0.485
EQ 3	Kobe, Japan 1995	6.9	0.509	EQ 9	Kobe, Japan 1995	6.9	0.618
EQ 4	Uttarkashi, India 1992	7 (Ms)	0.310	EQ 10	Duzce, Turkey 1999	7.14	0.739
EQ 5	Landers 1992	7.3	0.245	EQ 11	Northridge 1994	6.7	0.453
EQ 6	San Fernando 1971	6.61	1.219				



a)



d) Figure 3: Earthquake record– a) original, b) MCE spectra, c) matched spectra, c) matched record

5. SOIL-STRUCTURE-INTERACTION (SSI)

Soil-structure interaction (SSI) between the interrelated systems affects the seismic response and dynamic properties of structures (FEMA, 2009; NIST, 2012; Fatahi et al., 2014). The analytical model of the structure should model the foundation's and the soil's flexibility (ASCE 7, 2017). While considering SSI during nonlinear analysis, guidelines recommend the use of reduced shear modulus,

G, using a reduction factor (NIST, 2012; FEMA, 2020). In this study, SSI for the mat/raft foundation was modelled in a simplified way with three translational and three rotational degrees of freedom, proposed by Pais and Kausel (1988). The soil spring constants are tabulated in Table 4. The spring constants were modified using the embedment correction factor while the foundation was embedded in soil for a specific depth (Pais and Kausel, 1988; NIST, 2012; FEMA, 2020), in this case, which was considered as 1 for surface mat/raft foundation.

Table 4: Soil Spring Constants for SSI

Soil Spring Constants	Edge Column	Interior Column
Translation along x-axis, K_x (kN/m)	184173.6	263105.1
Translation along y-axis, K_y (kN/m)	184173.6	263105.1
Translation along z-axis, K_z (kN/m)	219540.2	313628.9
Rocking about x-axis, K_{xx} (kN-m/rad)	11537195.7	16481708.1
Rocking about y-axis, K_{yy} (kN-m/rad)	11537195.7	16481708.1
Rocking about z-axis, K_{zz} (kN-m/rad)	17976393.0	25680561.5

6. RESULTS AND DISCUSSION

6.1 Analysis and design using FBSD

The selected RC building frame was first analyzed and designed using the FBSD as per BNBC. The calculated base shear was distributed to the story levels as illustrated in Figure 4. The maximum displacements at every story level and the story drift ratio (SDR) are represented in Table 5, which satisfy the code limit.

From the ESA results, the governing combinations and maximum moment, axial load, shear, etc., were checked. The sections were then investigated and designed using governing responses. The sections were then investigated and designed using governing responses. The maximum shear, moment at the beam end, and mid were found to be 124 kN, 178 kN-m, and 40 kN-m, respectively. Hoops and stirrups were provided with 10 mm rebar. 1st hoop was provided at 50 mm, and the others at 100 mm spacing, and stirrups spacing was 150 mm. The governing load combination was found with column axial load 2271 kN and moment 179 kN·m. The column section was finalized as 508 mm x 508 mm. Figure 5 illustrates beam details, column details, and column capacity.

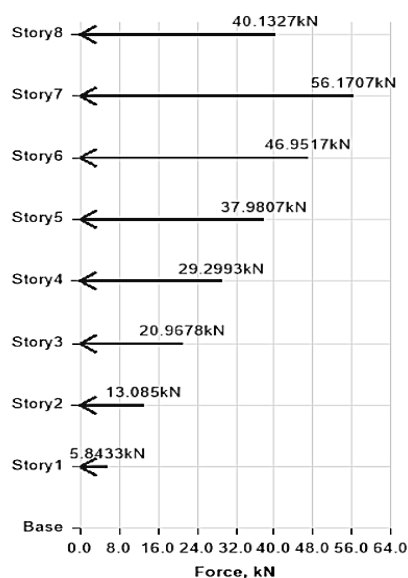


Figure 4: Distribution of Base Shear Force

Table 5: Story Displacement and SDR

Story Level	Displacement (mm)	SDR (%)
Roof	322.3	0.51
Story6	306.6	0.87
Story5	280.0	1.24
Story4	242.2	1.55
Story3	194.7	1.78
Story2	140.3	1.89
Story1	82.5	1.76
GF	28.9	0.95

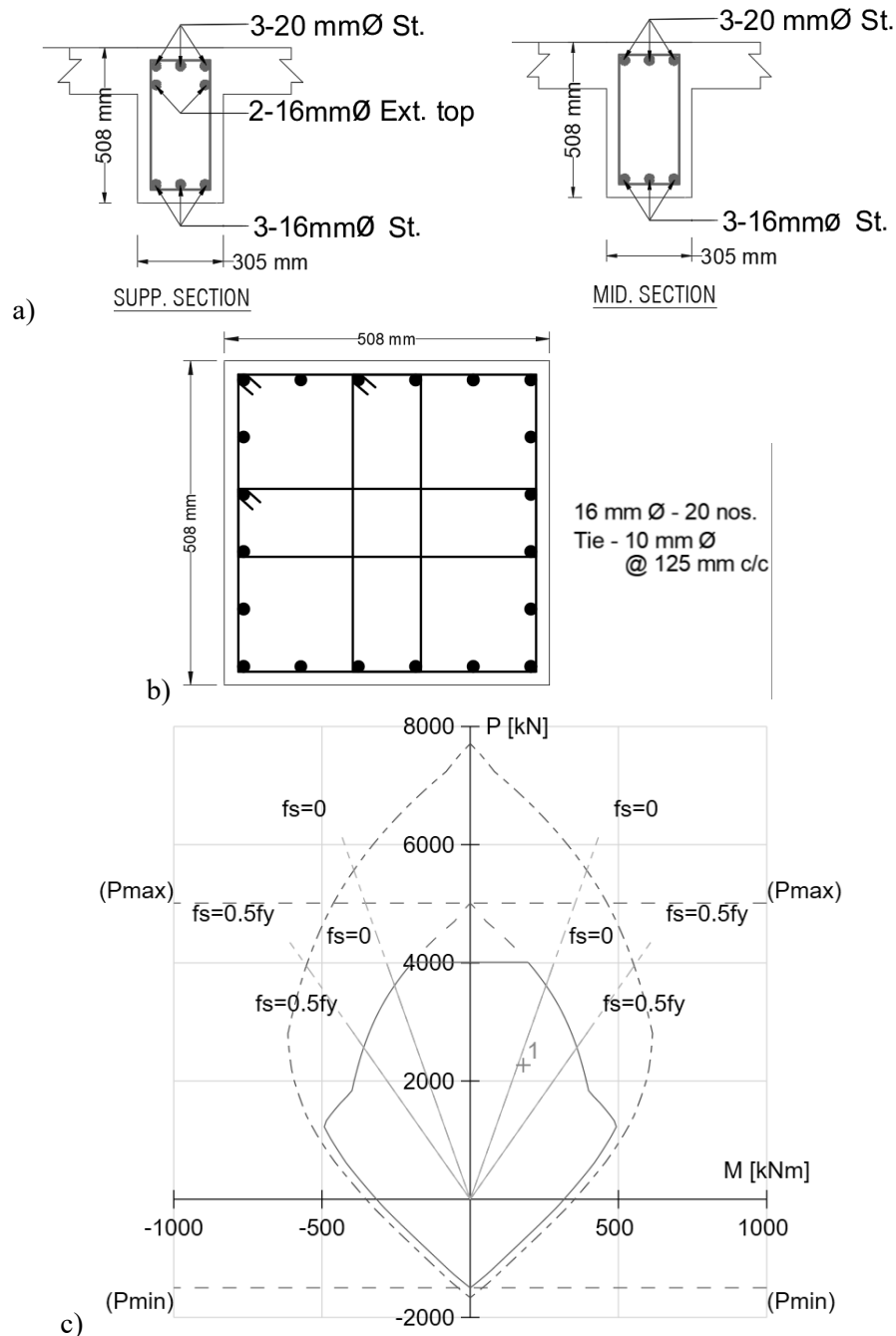


Figure 5: Beam and column details designed with FBSD– a) beam, b) column, c) column capacity

6.2 Performance of the frame designed with the FBSD approach

Performance of the frame designed with the FBSD approach was assessed based on the results of the NLTHA. SDR, being a true indicator of damage, which can represent the performance of the building, was used for global performance. The capacity of frames was assessed using various load combinations as considered.

SDR at each story level, as found from the analyses under DBE and MCE ground motions were averaged as recommended in several codes, e.g., BNBC (BNBC, 2020), ASCE 7 (ASCE, 2016). Figure 6 represents the story drift ratio from analyses for DBE and MCE. Thus, the frame was found to be adequate, satisfying the performance objective. The maximum SDR for DBE and MCE hazard levels is 1.28 % and 1.81 %.

For all the DBE matched records, beam and column capacity were assessed. Investigating the beam using the excel and the column with spColumn, the sections and reinforcement were found inadequate for beams and columns, based on the responses from nonlinear analyses.

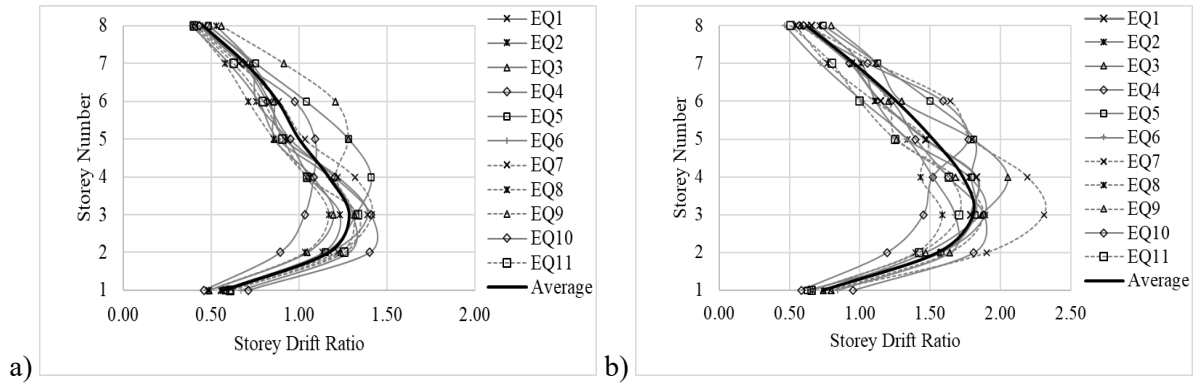


Figure 6: Story drift ratio from nonlinear earthquake analyses– a) DBE, b) MCE

6.3 Analysis and design using the PBSD approach

The frame designed with FBSD was found to be inadequate for the strength limit. So, it was modified, and several trials were performed, and the sections were then finalized. The modified frame was then analyzed with DBE and MCE records. The frame was designed for the mean DBE responses. The sections are shown in Table 6.

Table 6: Final Column and Beam Section

Beams		Columns	
FBSD Section	PBSD Section	FBSD Section	PBSD Section
305 mm × 508 mm	381 mm × 457 mm	508 mm × 508 mm	508 mm × 508 mm

6.3.1 Performance check using the story drift ratio

SDR of the building with modified final sections under DBE and MCE hazard is represented in Figure 7. The SDR is within the limit for both MCE and DBE level earthquake ground motion intensity. For DBE and MCE-based analyses, the SDR was found as 1.33 % and 1.74 %, respectively.

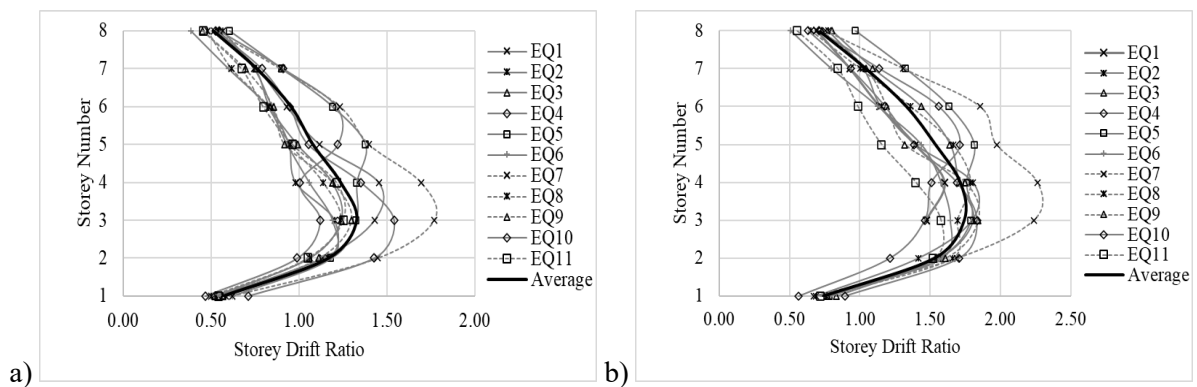


Figure 7: Story drift ratio of a frame designed with PBSD– a) DBE, b) MCE

6.3.2 Nonlinear responses

From the NLTHA, the moment for a column due to DBE-matched EQ7, hinge response for a beam and column for MCE-matched EQ7 are illustrated in Figure 8a-c, respectively. These figures represent nonlinear responses, nonlinear behavior of beams and columns as depicted from the hinge responses for all the earthquakes.

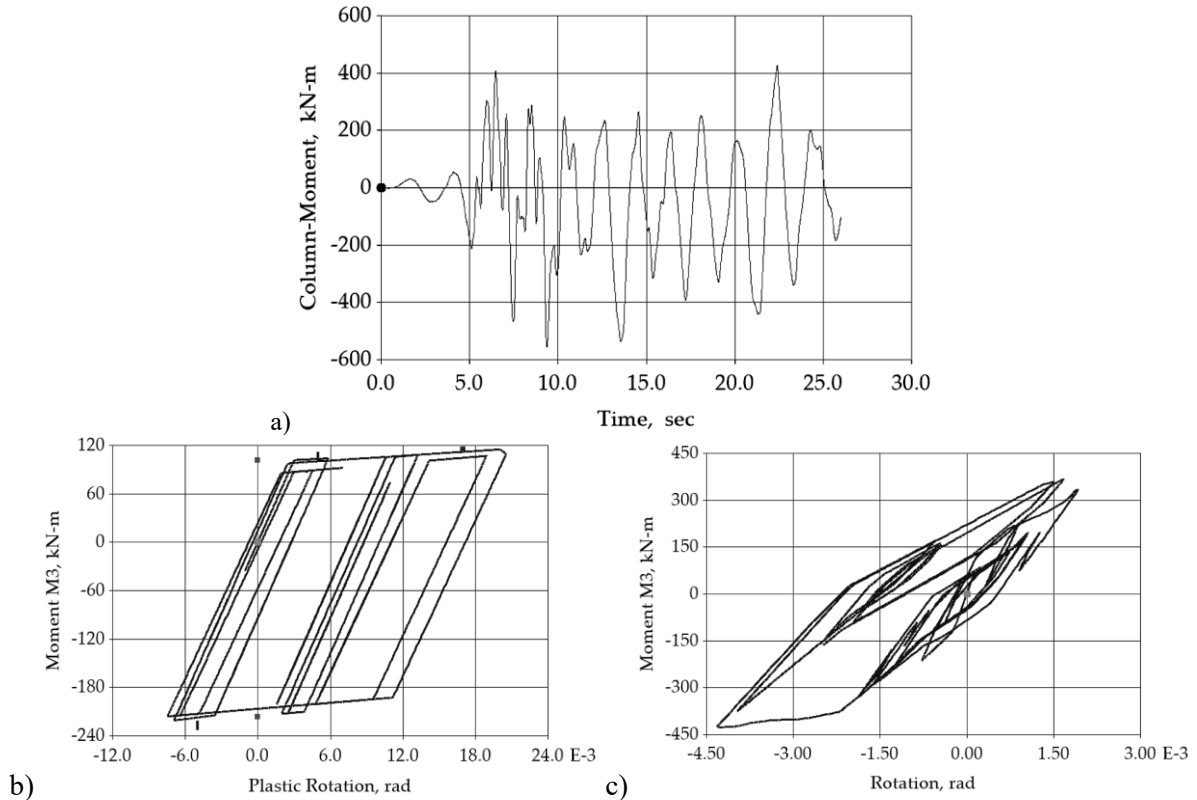


Figure 8: Nonlinear responses– a) column moment at ground floor for DBE-matched EQ7, b) beam hinge response for MCE- matched EQ7, c) column hinge response for MCE- matched EQ7

6.3.3 Details of section design with PBSD

For the responses from the analyses with matched DBE, the beam was designed using Excel again. The required reinforcement and details are shown in Figure 9. Hoops and stirrups remain the same. The moment capacity was found to be 317 kN-m at the end, 177 kN-m at the mid, and the shear capacity was found as 223.86 kN, which was satisfactory.

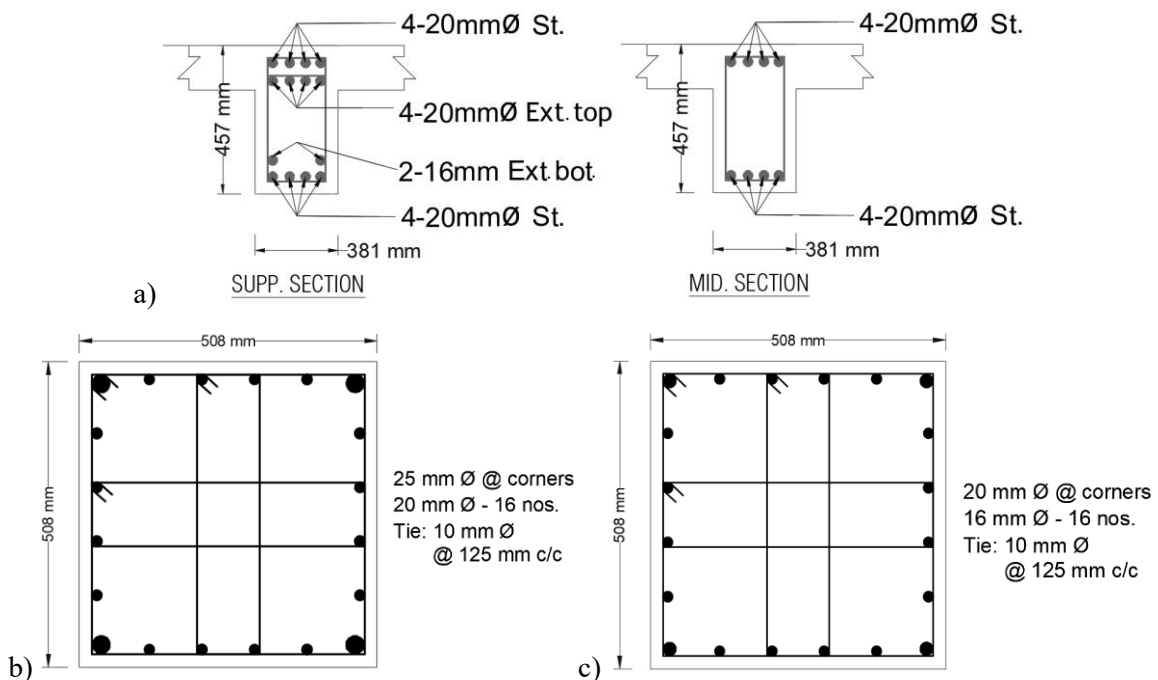


Figure 9: PBSD design details– a) beam, b) column up to 2nd story, c) upper story column

6.3.4 Element-level performance

From the NLTHA, the performance of elements can be checked for various performance levels. The performance limit of beam hinges was defined as per ASCE 41. For column hinges, performance limits were defined at the material level following NIST (2017), Kowalsky (2000), ASCE 41 (ASCE, 2007), and SeismoSoft (2018). For life safety, unconfined concrete strain, confined concrete strain, and rebar strain were defined as -0.0015, -0.0025, and +0.02, respectively. For collapse prevention, unconfined concrete strain, confined concrete strain, and rebar strain were defined as -0.003, -0.005, and +0.05, respectively. Figure 10 shows performance checks for the elements.

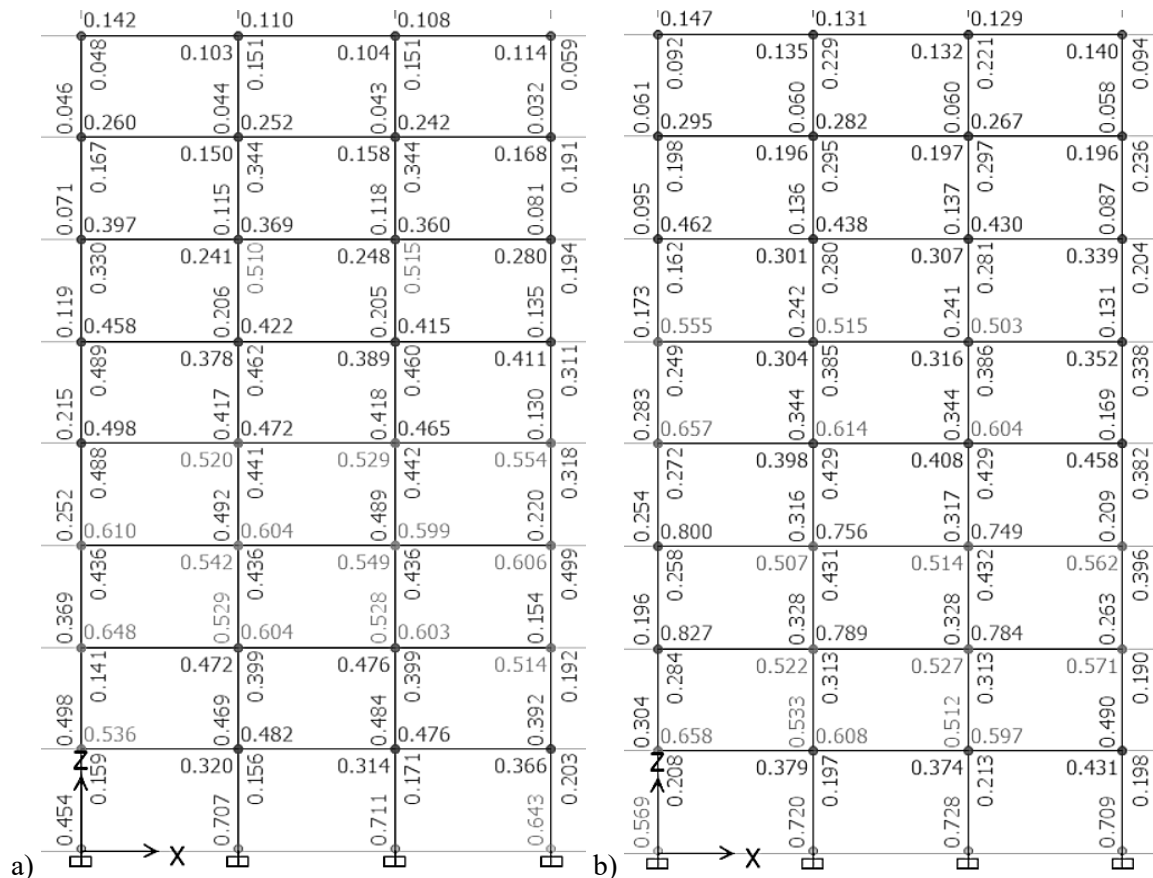


Figure 10: Element performance for EQ10– a) life safety for DBE, b) collapse prevention for MCE

6.3.5 Check for serviceability level earthquake (SLE)

After the final design with the PBSA approach, the frame was analyzed under SLE hazard. SLE intensity was defined for a 43-year return period earthquake using an important factor as per Euro Code 8 (CEN, 2004) for the regional seismicity. For SLE level assessment, linear modal time history analyses with the matched earthquake time history were used. 5% modal damping was considered here. Figure 11 illustrates SDR found from the analyses with SLE records. The SDR for the average of the responses due to the matched SLE was found 0.49% within the SLE drift ratio limit (0.5%).

6.4 SSI effect

Soil-structure interaction effects are now checked following the approach discussed in the previous chapter. Effects on storey drift ratio to check damage state and response modification, i.e., beam and column moment, shear, axial load, etc. (for strength performance requirements) are checked. It is found from the analyses that the SDR is increased for maximum acceleration records. However, the SDR is yet within the limit of performance objectives. Figure 12 represents the mean SDR with and without SSI for the life safety performance level (associated with DBE hazard level).

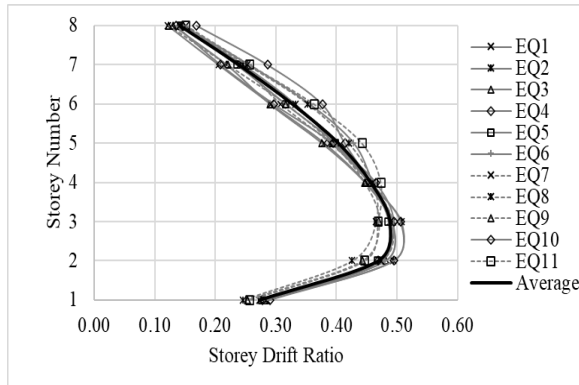


Figure 11: SDR for analyses with SLE

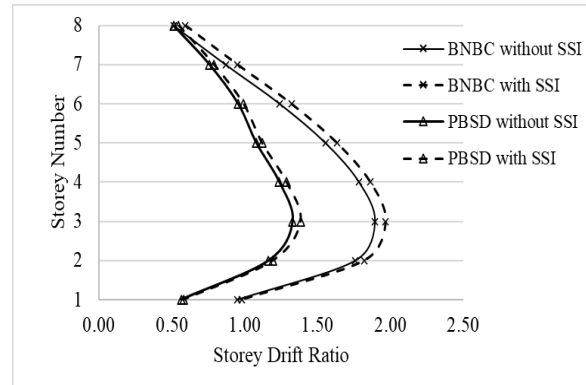


Figure 12: SSI effect on SDR

7. CONCLUSIONS

In this study, an archetype RC building frame was analyzed and designed using equivalent static procedure following the BNBC. The building was then assessed for life safety and collapse prevention performance level through nonlinear time history analyses. The used earthquake time history records were matched with acceleration response spectra, both for DBE and MCE hazards. After that, the building was again analyzed and designed using the performance-based seismic design approach. Element-by-element-based performance of the building frame was also checked both for life safety and collapse prevention performance level corresponding to DBE and MCE, respectively. Then, the building's functionality level performance was assessed from the analyses with SLE-matched records. Finally, the effect of soil-structure interaction on the performance was assessed both for forced-based and the performance-based approach.

From the outcome of the study, the following findings are illustrated:

- The code-designed building was found satisfactory for global performance, but the elements were found insufficient in terms of strength requirements. Specifically, a larger beam width was required with increased reinforcement in beams and columns, while the beam depth can be reduced.
- Performance-based seismic design based on nonlinear time history analyses with a sufficient number of earthquake ground motion records can be suitable to fulfil the damage state performance and strength requirements of buildings. Both global-level drift performance and element-based performance can be checked. However, the selected earthquake ground motion records should represent the seismic intensity of the building location.
- Soil-structure interaction consideration can affect the performance of the building frame. The story drift ratio can be more for some earthquake records or less for others. However, the average effect on the story drift ratio is found to be negligible.

The present study will help design new or strengthen old buildings with satisfactory performance against damage and strength. Further study can be performed to develop fragility curves for probabilistic damage state assessment, and also some performance metrics can be checked, e.g., casualties, economic loss, functionality recovery time, etc., which will be more meaningful to decision-makers, owners, and other authorities. A real 3D building can be considered for performance-based design and assessment.

REFERENCES

Applied Technology Council (ATC). (2006). Next-Generation Performance-Based Seismic Design Guidelines: Program Plan for New and Existing Buildings. Prepared for Federal Emergency Management Agency (FEMA 445). Washington, DC, USA.

- Applied Technology Council (ATC). (2018). Seismic Performance Assessment of Buildings. Prepared for Federal Emergency Management Agency (FEMA P58). Washington, DC, USA.
- Bangladesh National Building Code (BNBC). (2020). Housing and Building Research Institute (HBRI), Dhaka.
- CEN. (2004). Eurocode 8: Design of structures for earthquake resistance – Part I: General rules, seismic actions, and rules for buildings. European Committee for Standardization, Brussels, Belgium.
- COSMOS Virtual Data Center. Center for Engineering Strong Motion Data (CESMD), available from <https://strongmotioncenter.org/vdc/>. Last accessed on May 25, 2025.
- Federal Emergency Management Agency (FEMA). (2020). A Practical Guide to Soil-Structure Interaction. FEMA P-2091. Washington, DC, USA.
- Hamburger, R.; Rojahn, C.; Moehle, J.; Bachman, R.; Comartin, C.; Whittaker, A. (2004). The ATC-58 Project: Development of Next-Generation Performance-Based Earthquake Engineering Design Criteria for Buildings. 13th World Conference on Earthquake Engineering. Vancouver, B.C., Canada.
- National Institute of Standards and Technology (NIST). (2012). Soil-structure interaction for building structures. NIST GCR 12-917-21 Report.
- National Institute of Standards and Technology (NIST). (2017). Guidelines for Nonlinear Structural Analysis for Design of Buildings. Part IIb – Reinforced Concrete Moment Frames. NIST GCR 17-917-46v3 Report.
- Pais, A., and Kausel, E. (1988). Approximate formulas for dynamic stiffnesses of rigid foundations. *Soil Dynamics and Earthquake Engineering*, 7 (4), pp. 213-227.
- PEER NGAWEST 2. PEER Ground Motion Database. Pacific Earthquake Engineering Research Center (PEER). Web: peer.berkeley.edu/ngawest2. Last accessed on May 25, 2025.
- PEER. (2017). Guidelines for Performance Based Seismic Design of Tall Buildings. Tall Buildings Initiative. Pacific Earthquake Engineering Research Center (PEER).
- Priestley, M.J.N. (2000). Performance Based Seismic Design. 12th World Conference on Earthquake Engineering. Paper No. 2831, Auckland, N.Z.
- SeismoSoft. (2018). SeismoStruct- A computer program for static and dynamic nonlinear analysis of framed structures. <https://seismosoft.com/products/seismostruct/>
- SeismoSoft. (2020). SeismoMatch, Earthquake Software for Response Spectrum Matching. <https://seismosoft.com/products/seismomatch/>
- STRUCTUREPOINT. spColumn, Analysis and Design of Concrete Sections.
- Whittaker, A., Hamburger, R.; Comartin C., R. Bachman, R., and C. Rojahn, C. (2003). Performance based engineering of building structures. Proceedings of 73rd Shock and Vibration Symposium. San Diego, CA.