PERFORMANCE BASED SEISMIC DESIGN OF REINFORCED CONCRETE BUILDING FRAMES

By

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16MEQE001P

A thesis submitted in partial fulfilment of the requirements for the degree of
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This project titled **"Performance Based Seismic Design of Reinforced Concrete Building Frames"** submitted by **Sohel Rana**, Student ID. No. **16MEQE001P**, Session: **2016-2017** has been accepted as satisfactory in partial fulfilment of the requirements for the degree of **Master of Engineering in Earthquake Engineering [M. Engg. (EQE)]** on 10th August, 2023.

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Dedication

To my parents

List of Publications

- Sohel Rana, M. Abdur Rahman Bhuiyan. (2021). "Performance-Based Seismic Design of Reinforced Concrete Building Frames", *5th International Conference on Advances in Civil Engineering (ICACE)*, Chittagong University of Engineering & Technology, Chattogram 4349, Bangladesh.

- Sohel Rana, M. Abdur Rahman Bhuiyan, Md. Rabiul Alam. (202x). "Performance-Based Seismic Design of Reinforced Concrete Building Frames Considering Effect of Soil-Structure-Interaction". Manuscript in preparation to be submitted to a journal.

Declaration by the Supervisor

This is to certify that ***Sohel Rana*** has carried out this research work under my supervision, and that he has fulfilled the relevant Academic Ordinance of the Chittagong University of Engineering and Technology, so that he is qualified to submit the following Thesis in the application for the degree of MASTER of ENGINEERING in EARTHQUAKE ENGINEERING. Furthermore, the Thesis complies with the PLAGIARISM and ACADEMIC INTEGRITY regulation of CUET.

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Abstract

Buildings and other structures must be designed to perform satisfactorily to withstand earthquake, provide safety to human lives, and to minimize the economic losses from the damages, if any. Current code-prescriptive force-based design intends to provide strength and ductility to structures for life safety, but actual performance is never assessed. Structures designed with a code-based approach experienced extensive damage leading to enormous economic loss and high repairing costs in the past earthquakes (e.g., 1994 Northridge and 1995 Kobe earthquake). With a view to enhancing safety and reducing damage, i.e., emphasizing the performance of structures accelerated the development of performance-based seismic design. This study aims at designing reinforced concrete building frames following performance-based earthquake engineering approach. An archetype eight storied RC building has been selected and a frame has been analyzed and designed following the seismic design approach of the BNBC 2020. Nonlinear time history analyses using suitable earthquake ground motion records have been performed to assess the performance of the code designed building frame. The selected ground motions have been matched with acceleration response spectra of required earthquake hazard levels to check the selected performance objectives. Story drift, an indicator of damage, has been selected as an engineering demand parameter to quantify performance. Then, the frame has been designed using the performance based seismic design approach meeting the selected performance objectives. Finally, the effects of base flexibility on the responses of the building in force-based and performance-based design approaches have also been assessed. The present study will help designers, owners, and stakeholders to make intelligent decisions in designing new or strengthening existing buildings to achieve the required performance of the structures.

বিমূর্ত

ভবন এবং অন্যান্য কাঠামো অবশ্যই সন্তোষজনক পারফর্ম করার জন্য ডিজাইন করা উচিত যাতে ভূমিকম্প সহনশীল হয়, মানুষের জীবনের নিরাপত্তা থাকে এবং আর্থিক ক্ষতি কমিয়ে আনা যায়। ফোর্স-বেইজড ডিজাইন যদিও জীবনের সুরক্ষা (লাইফ সেইফটি) নিশ্চিতের লক্ষ্যে বিল্ডিং কোড প্রদত্ত নিয়মাবলী মেনে করা হয় এবং বিল্ডিং'এ প্রয়োজনীয় শক্তি (স্ট্রেংথ) এবং নমনীয়তা (ডাকটিলিটি) প্রদান করার উদ্দেশ্য থাকে, কিন্তু বাস্তবে প্রকৃত ক্ষমতা বা পারফর্মেন্স মূল্যায়ন করা হয় না। কোড-ভিত্তিক পদ্ধতির মাধ্যমে ডিজাইন করা বিল্ডিং (ও অন্যান্য স্ট্রাকচার) বিগত ভূমিকম্পে (যেমন, ১৯৯৪ ইং নর্থরীজ ও ১৯৯৫ ইং কোবে, জাপান) ব্যাপকভাবে ক্ষতিগ্রস্থ হয় যার ফলে বড়মানের অর্থনৈতিক ক্ষতি হয়, অনেক স্ট্রাকচারের মেরামত খরচ অনেক বেশি হয় এবং ক্ষেত্রবিশেষে সম্ভবপরও হয়ে উঠে না। বিল্ডিং এর নিরাপত্তা বাড়ানো এবং ভূমিকম্পের কারণে ক্ষয়-ক্ষতি হ্রাস করা, অর্থাৎ ভবনের পারফর্মেন্সের উপর জোর দেওয়া-ই পারফর্মেন্স-ভিত্তিক সাইজমিক ডিজাইনকে ত্বরান্বিত করে। এ গবেষণা কর্মটিতে রেইন্সফোর্সড কংক্রিট বিল্ডিং ফ্রেম এর পারফর্মেন্স-বেইজড সাইজমিক ডিজাইন করা হয়েছে। একটি আট তলা বিশিষ্ট রেইন্সফোর্সসহ কংক্রিট বিল্ডিং নির্বাচন করা হয়েছে এবং নতুন বাংলাদেশ বিল্ডিং কোড (বিএনবিসি ২০২০) এর সাইজমিক ডিজাইন পদ্ধতি অনুসরণ করে এ ফ্রেম এনালাইসিস ও ডিজাইন করা হয়েছে। কোড ডিজাইন করা বিল্ডিং ফ্রেমের কার্যকারিতা, ক্ষমতা বা পারফর্মেন্স মূল্যায়নের জন্য ১১টি কোড স্পেকট্রাম এর সাথে ম্যাচ করা আর্থকোয়াক টাইম হিস্ট্রি রেকর্ড ব্যবহার করে ফ্রেমের নন-লিনিয়ার টাইম হিস্ট্রি এনালাইসিস করা হয়েছে। স্টোরি ড্রিফট, যেটি বিল্ডিংয়ের ড্যামেজ বা ক্ষতি নির্দেশক, বিল্ডিংয়ের পারফর্মেন্স বা ক্ষমতা নিরূপণে এটি নির্বাচন করা হয়েছে। অতঃপর বিল্ডিং ফ্রেমটি পারফরম্যান্স-বেইজড সাইজমিক ডিজাইন পদ্ধতি ব্যবহার করে ডিজাইন করা হয়েছে যা বিল্ডিং ফ্রেমটির "পারফর্মেন্স অবজেকটিভস" পূরণ করে। পরিশেষে, সয়েল-স্ট্রাকচার ইন্টেরেকশান এর প্রভাব ফোর্স-বেইজড এবং বর্তমান পারফর্মেন্স-বেইজড ডিজাইন পদ্ধতি অনুসরণ করে বিল্ডিং ফ্রেম এনাইলিস করে নিরুপণ করা হয়েছে। এ গবেষণা কর্মটি স্ট্রাকচারাল ডিজাইনার, বিল্ডিং মালিক এবং স্টেকহোল্ডারদের বিল্ডিং ও অন্যান্য নতুন কাঠামোর প্রয়োজনীয় শক্তি ও পারফর্মেন্স নিশ্চিতকরণ এবং পুরাতন ভবনের শক্তি নিরূপণ এর ক্ষেত্রে এবং প্রয়োজনে বিদ্যমান ভবন পুণরায় শক্তিশালী ও ভূমিকম্প সহনশীল করার ক্ষেত্রে সিদ্ধান্ত নিতে সাহায্য করবে।

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Chapter 1: INTRODUCTION

This chapter outlines the background and context of the performance-based seismic design and its necessity. The significance, scope and objective of this research are discussed. Finally, organization of this Thesis and an outline of the chapters is also provided.

1.1 BACKGROUND

Designing Structures to withstand various loads, imposed to structures by various natural hazards and service requirements, is one of the main aims of Structural and Earthquake Engineering. Structures should be designed for life safety and property damage prevention due to natural and man- made hazards (earthquakes, hurricanes, blast, fire etc.). Because of randomness in nature and unpredictability earthquakes, among the various natural hazards, have the potential to cause great damages to structures. At present, structures are designed following the prescriptive codes and standards. Although the current prescriptive design criteria through codes, guidelines, manuals and standards is intended to provide certain performance level, the actual performance of the structure is never assessed as part of commonly used code-based design method (ATC 2006 and ATC 2012, 2018). Moreover, although some performance level for various structures is obtained through the approach, responses of the structures are often found different regarding damages for the same hazard levels (ATC 2006). Designing civil structures and infrastructures for seismic hazard is yet improving. Current code-based seismic design methods are based on the Forced Based Design (FBD). Extensive damages to structures and infrastructure systems designed with current seismic design approach during previous seismic events validates that current FBD approach has some sort of deficiencies to provide required performance level (Priestley 2000). Thus, an approach with suitable reliability for design of buildings or structures against seismic loading has become necessary. In recent years, earthquake resistant design of structures is emphasizing on performance rather than strength (Priestley 2000). Code based design approach emphasizing on strength and ductility for designing buildings are not suitable enough for the seismic assessment of existing buildings. Thus, for the assessment of the performance of existing (building) structures, some preliminary assessment methods developed by engineers led to performance-based engineering approach for seismic design (ATC 2006 and Hamburger et al. 2004).

1.2 PROBLEMS WITH FORCE BASED SEISMIC DESIGN

Buildings owners, users and even design professional generally believe that structures designed with current approach with codified requirements are safe. Destructive damage to the buildings, bridges and other civil structures due to earthquakes such as 1989 Loma Prieta, 1994 Northridge and 1995 Kobe made owners, designers, and authority to rethink about the fate of their structures for future earthquakes. This made them worried and to consider that structures even designed with current code may experience severe damages (ATC 2006 and Whittaker et al. 2003). It is concluded from the severe damaging effects of significant earthquakes that seismic risk in increasing mainly in urban areas and damages due to building will be beyond thinking due to earthquakes in socio-economic perspective.

Current codes and standards intend that buildings should meet a specified performance level (e.g., life safety) at the specified level of ground shaking (e.g., design base earthquake). But there are no assessment guidelines by which designers may determine whether the assumed (or other) performance levels can be actually achieved or not. This means the actual performance level of a code-designed structure is never assessed by the prescriptive procedures (ATC

2006, 2018). Due to a design base earthquake, building designed with code and standard could prevent loss of life or severe injury to users providing intended safety level, but could experience extensive damage, and be out of service for long time. The damage may also be too costly to repair and time consuming that demolition may be the only option (ATC 2018). Thus, there is need for revision of the current seismic design guidelines, codes and standards to assess the probable performance of the structures during the design process.

1.3 CONCEPT OF PERFORMANCE BASED SEISMIC DESIGN

Performance Based Seismic Design (PBSD) is an approach of designing structures with a reliable method evaluating the probable performance of structures in future seismic events. The design process is focused on the assessment of the structure's performance along with the design. Another important point is that owners, users, design professionals and stakeholders can participate in the decision of selecting the performance objectives for the structures considering life safety and property protection (ATC 2006). Performance objective is related to the requirement considering damage that the structure may experience during a future earthquake event and the consequences (casualties, property damage, etc.) of that damage (ATC 2012, 2018).

PBSD explicitly evaluates the performance of structures, experiencing the potential hazards, considering uncertainties in hazard as well as in the assessment of response of the structure (ATC 1997, 2006). This approach may be used to design, with a better understanding of their performance, more loss-resistant and reliable individual facilities to satisfy various performance levels of the structures at specified earthquake hazard levels than typical structures designed with current code prescriptive design criteria. This design procedure, i.e., PBSD procedure can assess the performance of the existing structures that are designed with current prescriptive procedures to be quantified and to enable modification for better desired performance (Hamburger et al. 2004 and ATC 2006).

In current design codes and standards, strength and ductility demands are applicable for designing new buildings. Thus, it is not suitable for seismic assessment of existing building structures. This lead the development of PBSD for the seismic evaluation or safety assessment of existing building structures which can also be used for the design of new buildings. Its main aim is to minimise the damages of structures through evaluation of proper engineering demand parameters. The approach also provides meaning information to building owners, users and decision-makers which is necessary for safety related decisions.

Performance-based seismic design approach is an efficient design approach. The design criteria are mainly expressed as in performance objectives. These objectives are related to the performance of structures corresponding to specified levels of seismic hazard.

1.4 OBJECTIVES OF THE STUDY

The present study will attempt to design a RC building with current code and assess the performance of the code designed building through performance-based approach and then designing the building through PBSD approach. The study will also consider the soil-structure interaction effect in both the case. The aims and objectives of the present study are:

a) Seismic analysis and design of a reinforced concrete (RC) building frame following the BNBC 2020 guidelines.

b) Performance assessment of the code designed RC building frame for the selected performance objectives through nonlinear time history analyses.

c) Seismic design of the RC building frame using performance-based design (PBD) approach to fulfil the performance requirements.

d) Assessment of soil-structure interaction (SSI) effects for both the code based (BNBC 2020) and performance-based seismic design.

1.5 THESIS OUTLINE

Here is an outline of the chapters of this Thesis report.

Chapter 2 reviews background literature relating to development of performance-based seismic design, its necessity, and also the theory and literature related to this thesis.

Chapter 3 of this thesis report describes the procedures followed for the analysis, design, and assessment for force-based and performance-based seismic design approach, and also the method for the assessment of soil-structure-interaction.

Chapter 4 give the results and other outcome along with some discussion on the study.

Chapter 5 provides the conclusions and recommendation for future studies.

Chapter 2: LITERATURE REVIEW

This chapter provides highlight for the necessity of performance-based seismic design, literature and development of the conceptual framework of PBSD, and the theory, definitions, and discussions needed for the study. Theory and steps for force-based and performance-based approach along with the basic of nonlinear time history analysis and soil-structure interaction are discussed. Previous studies are also provided herewith.

2.1 GENERAL

After many severe earthquakes which caused serious damage to structures in medium to high seismic regions, continuous improvement of seismic design methods of were carried out. During the 1994 Northridge, 1989 Loma Prieta and 1995 Kobe earthquake, buildings designed with current codes though performed well in from the point of life safety performance, the degree of damage to structures was so severe that it cost enormous economic loss. So, owners, authorities, designers and society became so much worried thinking that what will the performance of their buildings, bridges, or other structures. They also realized that structures even built with current standard are prone to severe damages during seismic events (ATC 2006 and Whittaker et al. 2003).

2.2 FORCE BASED SEISMIC DESIGN (FBSD)

Current seismic design approach of buildings using building codes is Forced Based Design approach, not performance based. In the current approach of seismic design; "design professionals select, proportion, and detail building components to satisfy criteria as described in the code" (ATC 2012). In Forced Based Design (FBD), at first member sizes along with other structural geometry is estimated and elastic stiffness of members is evaluated. In some codes un-cracked stiffness and in others reduced stiffness is used to reflect the expected cracking for the consideration of the appropriate stiffness of members.

Based on the member stiffness, structural period (T) is calculated with the following equation,

$$T = 2\pi\sqrt{\frac{m}{K}} \tag{2.1}$$

Here, m and K are mass and stiffness of the structure, respectively.

In some codes, structural period (T) is calculated by,

$$T = C_t(H_n)^m \tag{2.2}$$

Where C_t is a constant depends on structural system (also on unit) and H_n is the height of the building and m depends on material and structural system.

The design base shear is calculated by,

$$V = S_a W \tag{2.3}$$

Here, S_a is seismic coefficient which depends on seismicity of the building location, soil conditions, building period, occupancy and ductility capacity of structural system.

The calculated base shear is then distributed to storey levels of the building to apply seismic forces. The building is then analyzed for the defined seismic forces. Then storey displacements for the seismic force are estimated to evaluate story drift as well as inter-storey drift ratio. The story drift or story drift ratio are then compared with the code limits.

The above procedure is the simplified forced based seismic design procedure (equivalent lateral static analysis). In many cases, modal analysis (dynamic) is also carried out in which the contributions are combined.

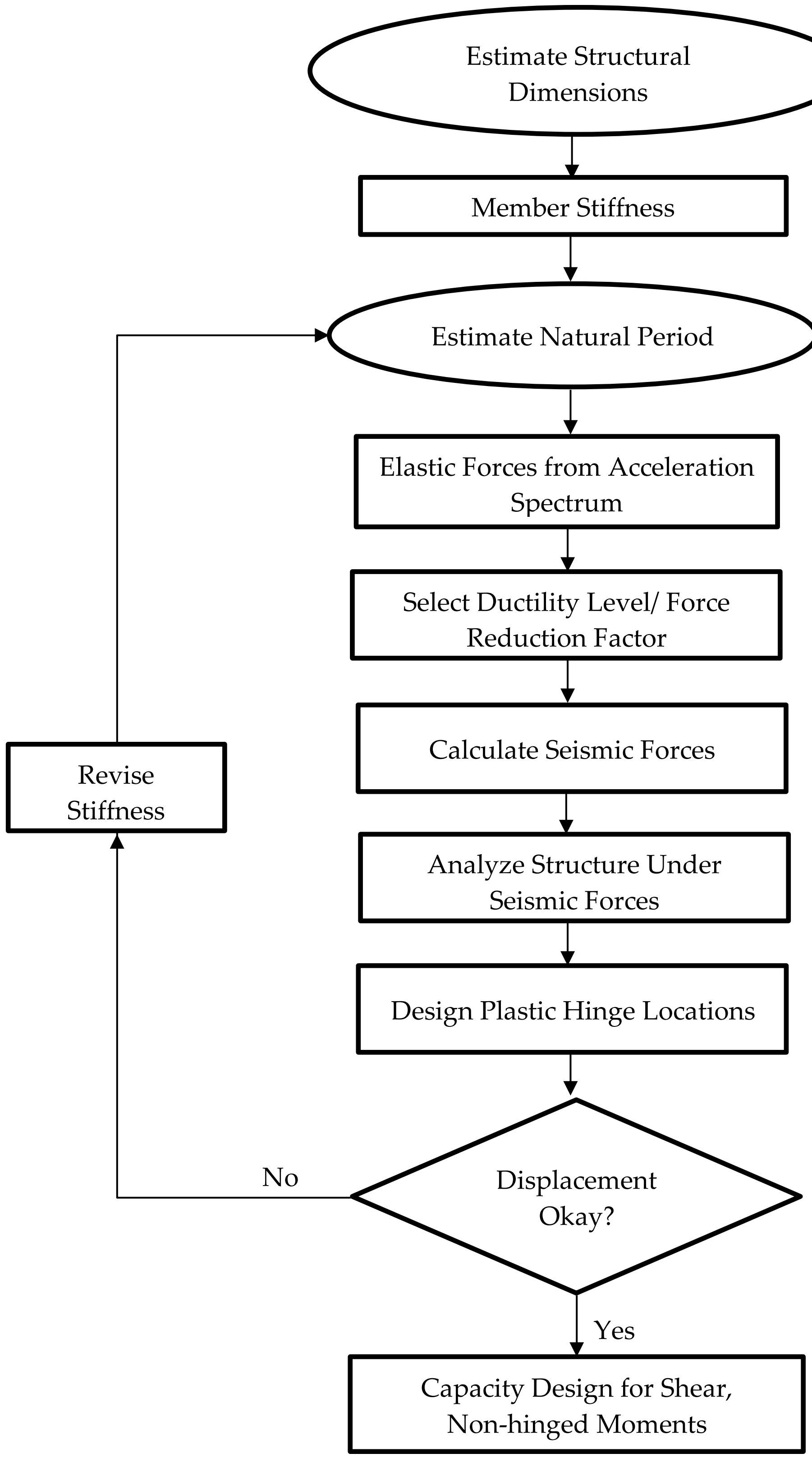


Fig. 2.1. Steps in Force Based Seismic Design (Priestley et al., 2007)

2.3 PERFORMANCE BASED SEISMIC DESIGN (PBSD)

Performance-Based Seismic Design (PBSD) is an improved procedure for reliable prediction of performance or behaviour of the structures under earthquake forces. The method is efficient for designing new building structures as well as seismic upgrade of existing buildings to achieve pre-defined performance objectives. It aims to control damages of the structures based on the defined engineering demand parameters. This approach can be used to assess or design structures to satisfy different performance levels of structures at different hazard level (e.g., DBE) with better performance than structures designed with prescriptive criteria (ATC 2006; Hamburger et al. 2004).

2.3.1 State of Development

After many powerful seismic events it was realized that increasing strength of the structure may not enhance its safety, also nor necessarily reduce the damage to the structure. This realization led to the development of performance-based engineering emphasising on performance rather than strength (Priestley 2000). The framework of performance-based engineering explicitly addresses damage limitation or functional issues, reparability and life-safety in building at corresponding levels of seismic hazards.

Documents that are primarily should be credited for the development of performance-based design are- ATC 40, "Seismic Evaluation and Retrofit of Concrete Buildings" (ATC 1996), FEMA273/274, "NEHRP Guidelines and Commentary for the Seismic Rehabilitation of Buildings" (ATC 1997) and Vision 2000, "Performance-Based Seismic Engineering of Buildings" (SEAOC 1995). SEAOC Vision 2000 developed the framework to design structures with predictable performance satisfying required seismic performance objectives. This document represents the concepts of the performance levels for structures corresponding to specified seismic hazard levels. ATC 40 emphasizes on the capacity spectrum method for concrete buildings. The acceptance criteria are expressed in performance of the structure associated with a hazard level which is referred as performance objectives. The Federal Emergency Management Agency FEMA 273 represents performance objectives using a similar framework of Vision 2000, with slightly different performance descriptions and seismic hazard levels (Porter 2003).

FEMA 273 and 274 were then converted to a single document and updated to a pre-standard level by American Society of Civil Engineers as FEMA-356, "Pre-Standard and Commentary for the Seismic Rehabilitation of Buildings" (ASCE 2000). The development of FEMA 356 includes technical updates to the requirements along with acceptance criteria. This document was again replaced and updated as ASCE 41, "Standard for Seismic Rehabilitation of Existing Buildings" (ASCE 2006). Though these documents were mainly developed for seismic assessment or evaluation and rehabilitation of existing building, the approach can also be used to design new buildings in order to achieve reliable performance objectives better than the code procedures.

For the present form of the PBEE, the contribution of the "Pacific Earthquake Engineering Research (PEER) Center" is a milestone and worth mentioning. The technical basis of the present form of PBEE is based on the framework developed by many researchers collaboratively at the Pacific Earthquake Engineering Research (PEER) Center (Porter 2003; ATC 2006; ATC 2012; Gunay and Mosalam 2013). The PEER framework applies total probability theorem for the prediction of consequences of earthquakes (Moehle 2003; Porter 2003; Moehle and Deierlein 2004; Gunay and Mosalam 2013).

FEMA 445 and FEMA P-58 are the two updated documents for the PBSD under the ATC 58 project funded by FEMA. FEMA 445, "Next- Generation Performance- Based Seismic Design Guidelines, A Program Plan for New and Existing Buildings" (ATC 2006) was performed under ATC 58 project with the modifications of FEMA 283, “Performance Based Seismic Design of Buildings-An Action Plan”, prepared by the “Earthquake Engineering Research Center” (EERC 1996) and FEMA 349, “Action Plan for Performance Based Seismic Design”. FEMA P58, “Seismic Performance Assessment of Buildings, Methodology and Implementation” (ATC 2012, 2018) is guideline for seismic performance assessment of buildings with various performance measures (repair cost, downtime, etc.) meaningful to decision making authorities. Performance objectives are related to the damage that the building may experience in an earthquake event and the consequences of that damage (ATC, 2012).

ATC-58 project developed the performance assessment methodology based on the framework provide by the PEER Center (Whittaker et al. 2003; Hamburger et al. 2004, ATC 2012). The approach is applicable to new building and also to retrofit building with performance-based procedures. It will utilize performance objectives that are both predictable for professional and meaningful to decision makers (Whittaker et al. 2003).

2.3.2 Advantages of Performance Based Seismic Design

Performance based seismic design offers efficient and effective method to design and assess buildings to avoid future earthquake losses. Further, the procedure can be applied also for performance-based design for other hazards (e.g., fire, wind, flood, and also terrorist attack) (Whittaker et al. 2003).

According to FEMA 445 (ATC 2006), performance-based seismic design can be used:

- To design buildings to achieve higher performance and lower potential losses with a better confidence satisfying the intended performance.

- To design buildings beyond code-prescribed limits considering building's configuration, materials to be used, and the structural systems meeting the intended performance.

- To assess the performance of existing structures for seismic hazard and estimate corresponding losses for the damage (if any) due to that event.

- To assess the performance of new buildings designed with code prescriptive approach, and serve for the improvement of seismic design criteria for safe and reliable buildings in future seismic events.

2.3.3 Previous Research

Inelastic strength and displacement spectrum can be obtained through nonlinear response history analysis (NRHA), as described by Fajfar (2000), to calculate the seismic demand of a structure. The procedure is known as the so-called N2 method. The technique integrates response spectrum analysis (RSA) of an equivalent single degree of freedom (SDOF) system with nonlinear static (pushover) analysis of a multi degree of freedom (MDOF) system. The basic seismic response values may be evaluated because the method is presented in an acceleration-displacement style. If the structure is mainly in the first mode, it is believed that the results are fairly accurate. The planer analysis of structures is the only application.

A method for PBSD using a displacement-based approach coupled with pre-quantified performance requirements was presented by Xue and Chen (2003). To meet the necessary performance requirements, they suggested a lower bound of yielding displacement. The study showed adaptability in considering the duration of strong motion, the near-fault effect, and the ease of numerous performance objectives. Through nonlinear time history analysis, they suggested that the technique can regulate target displacement to achieve the performance limit.

Kappos and Panagopoulos (2004) proposed a procedure to design 3D reinforced concrete (RC) buildings using PBSD procedure. The proposed method used advanced analytical tools applied to a regular three-dimensional multi-storey RC frame building. The method was found to assure better seismic performance compared with the code (Euro code 8, CEN 2004), and also a more economic design of transverse reinforcement.

Zou and Chan (2005) demonstrated a computer-based technique incorporating nonlinear static pushover analysis with numerical optimization for the design of RC buildings using PBSD approach. They came to the conclusion that reinforcing can be applied to provide the necessary ductility as well as control drift beyond the first yielding.

Harries and McNeice (2006) demonstrated a rational performance-based design and analysis of a ten-storey proto type coupled core wall system. They suggested that strength-based design approach of coupled wall system is unrealistic and they behave differently assumed in the strength-based procedure. To check the adequacy of PBSD nonlinear static (pushover) and also nonlinear dynamic analysis were performed for the critical responses at different limit states.

Whittaker et al. (2007) summarized the tools and updated procedures for next generation performance-based engineering. They considered explicit treatment of uncertainty and randomness in performance assessment by the next-generation procedures. The performance was characterized in terms of economic loss and casualties rather than deformations and accelerations.

Zou et al. (2007) developed an optimization technique for nonlinear design of RC building with multiple objectives. The approach considers inelastic drift performance under pushover loadings using appropriate damage loss model. They used optimization algorithm on the basis of the constraint method to satisfy lateral drift constraints as well as minimizing the structural life-cycle cost with effective distribution of the reinforcements in structural members. They suggested that reinforcement has significant effect to improve drift to reduce damage i.e., loss.

Xue et al. (2008) summarized the development of a draft design code for performance-based design of buildings for earthquake hazard for Taiwan. In their study, seismic design objectives were selected based on different use groups with the qualitative interpretation of performance criteria including drift limits.

Lagaros et al. (2009) proposed a PBSD methodology for RC buildings considering the influence of infill walls. The approach, based on both non-linear static and also dynamic analyses, was compared in the context of optimization using evolutionary algorithm. Life-cycle cost analysis was also considered for the reliable performance assessment. The suggested that infill has a significant impact on performance of the structures.

Cardone et al. (2010) proposed a direct displacement-based design (DDBD) approach for buildings equipped with different seismic isolation systems. Performance levels were expressed in terms of inter-storey drift and displacement of the isolation system. They provided evidence supporting the approach using Nonlinear Time History Analyses (NTHA) carried out on various base isolated building configurations, and they suggested that the DDBD method still needs some improvement.

Hajirasouliha et al. (2012) presented a practical method performance-based design of RC structures for seismic loading based on uniform distribution for deformation or damage. The method suggests redistribution of material from strong parts of structure to weak parts until uniform deformation state is achieved. The approach incorporates different parameters such as inter-storey drift and was evaluated for various target performance under seismic loading.

Welch et al. (2014) presented a simplified methodology for probabilistic loss assessment using displacement-based framework. The approach was verified using two RC frame buildings which validates similar results of the PEER comprehensive probabilistic methodology. However, they suggested for more rigorous research to select appropriate limit state displacements, cost functions also should consider residual displacement and different structural typologies.

Fatahi et al. (2014) investigated effects of pile foundations on seismic performance of the structures. From the shaking table test they suggested that response of the structures can be significantly impacted due to SSI. The rocking component can alter the dynamic properties and thus can increase the lateral deflection consequently inter-storey drifts i.e., the performance of the structure.

Özuygur (2015) demonstrated PBSD of an extremely irregular 50 storey RC residential building. The approach uses the draft Seismic Design Code of Istanbul which adopts Tall Buildings Initiative (TBI) Guidelines of PEER Center. A response spectrum analysis was used to design the structure for DBE in a seismically active area. Following that, nonlinear time history analysis was used to evaluate its seismic performance for MCE.

Vamvatsikos et al. (2015) introduced an approach for seismic performance using yield frequency spectra (YFS). This is a unique view for performance of a surrogate single-degree-of-freedom (SDOF) system oscillator. YFS can be computed for any system which can approximated with an equivalent nonlinear SDOF oscillator. This can provide accurate analytical solution with the consideration of site-specific and also structure-specific characteristics as well as uncertainties with a desired confidence.

Vamvatsikos and Aschheim (2016) implemented the Yield Frequency Spectra (YSF) as an approach for the preliminary PBSD with performance objectives. YFS has been expressed as mean annual frequency (MAF) of exceeding ductility thresholds. They suggested that using YFS can reduce the preliminary design steps as the achieved preliminary designs are very close to meet performance targets.

Arroyo et al. (2017a, 2017b) suggested PBEE framework as a novel approach in seismic design. They proposed a seismic design method based on Eigen frequency optimization and evaluated performance by PBEE framework. The buildings designed with this approach has reduced drifts at lower stories. The approach suggests different distributions of stiffness over the building height for uniform distribution of damage.

Huang et al. (2017) developed a simplified method with linear analysis for PBSD of low to medium rise buildings. They suggested the method when the structural system and seismic hazard are not defined quite well for nonlinear analysis. The method used accurately predicts the distributions of peak structural response values, such as floor acceleration and story-drift. Results from nonlinear time-history analysis of three-dimensional archetype buildings with moment frames, braced frames, and shear walls were used to calibrate the analytic approach.

Franchin et al. (2018) presented a probabilistic, risk-targeted procedure to perform seismic design of RC buildings. As performance goals, mean annual frequency (MAF) of exceedance was chosen. The method uses a gradient-based optimization strategy to control the seismic risk with a number of limit states that satisfy the demands of both structural design variables and capacity design. The procedure was validated with the application to a 15-storey plane building frame. They performed multiple-stripe analysis (MSA) with time history records matched with conditional spectra. They suggested that the method is applicable for irregular structures also.

Sattar et al. (2020) provided some insights to implement performance based seismic design (PBSD) using the procedures as provided in ASCE 41

(ASCE 2017), PEER Center TBI (PEER 2017), and the other existing guidelines/standards.

Badal and Sinha (2022) proposed a framework for performance based seismic design of reinforced concrete (RC) buildings with special moment frames. Their approach allows prescriptive code-based design standards with multi-objective risk objectives adopting probabilistic approach considering uncertainty in seismic demand and structural capacity.

PBSD is an excellent seismic design procedure for the design or assessment of buildings satisfying the performance requirements of buildings based on the associated hazard level of the building's location and the required or expected seismic performance. Still the procedure is being tried to be implemented in professional design. And thus, it is needed to compare the design procedure and also design outcomes to provide some insightful information to the engineers, owners, and design professional.

2.3.4 Procedure of Performance Based Seismic Design

Performance Based Seismic Design (PBSD) is an iterative procedure. The first step in PBSD is the selection of performance objectives. Performance objectives are associated with the risk resulting from performance levels of structures i.e., specific levels of damage of structures corresponding to specified seismic hazard level (ATC 2012; ATC 2006; Moehle and Deierlein 2004). Performance objectives provides meaningful information to the owners, users and decision officials to make safety related decisions (Whittaker et al. 2003; Hamburger et al. 2004). After the selection of performance objectives, a series of structural analyses with respect to various level of earthquake load are performed and the probable performance are assessed (ATC 2006; Hamburger et al. 2004). Performance of a building structure are assessed to meet the required objectives. Redesign is needed if the objectives are not fulfilled. Fig. 2.2 shows the major steps in the performance-based design process.

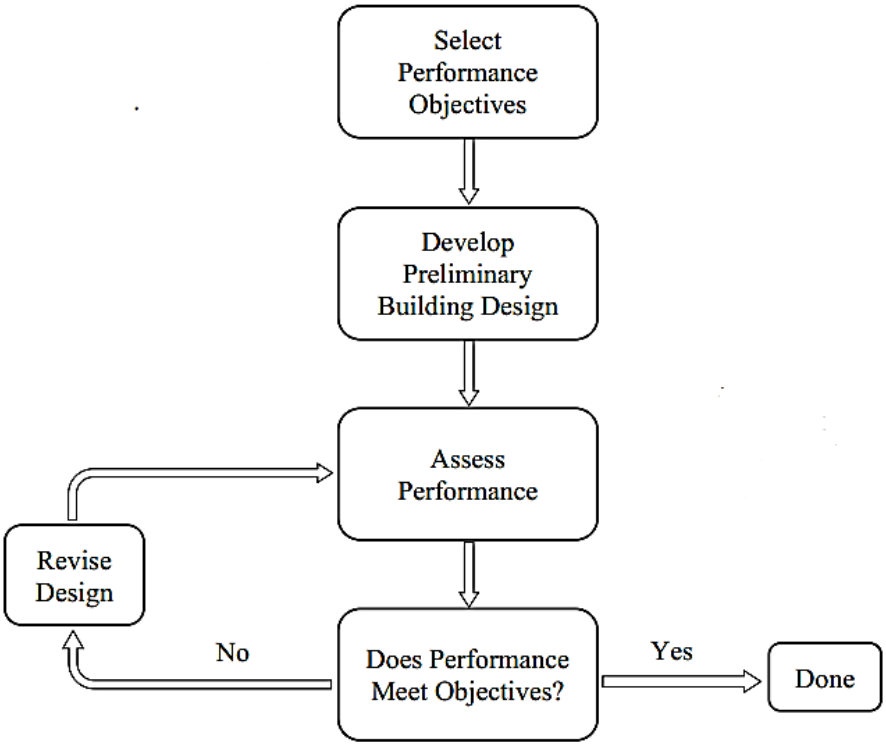


Fig. 2.2 Performance-Based Seismic Design Flowchart (ATC, 2006)

PBSD includes analyses, design and assessments of structures to fulfil the objectives or requirements of owners, users, designers, and decision-makers (ATC 2006).

2.3.4.1 Performance Objectives

Performance objectives state the expected performance levels of a structure corresponding to defined earthquake hazard levels. A group including building owners, users, stakeholders, and designers are involved in the selection of performance objectives for the structure. Stakeholders can play vital roles in assessing the hazard and also in obtaining the common agreement among all the groups participating in the decision making for acceptable performance of the structures.

Performance objective is related to the seismic performance level corresponding to specified earthquake hazard level. Fig. 2.3 represents the concepts of performance objects adapted from the SEAOC vision 2000 (SEAOC, 1995). It is also represented that performance objectives are function of earthquake hazard levels and performance level of the building.

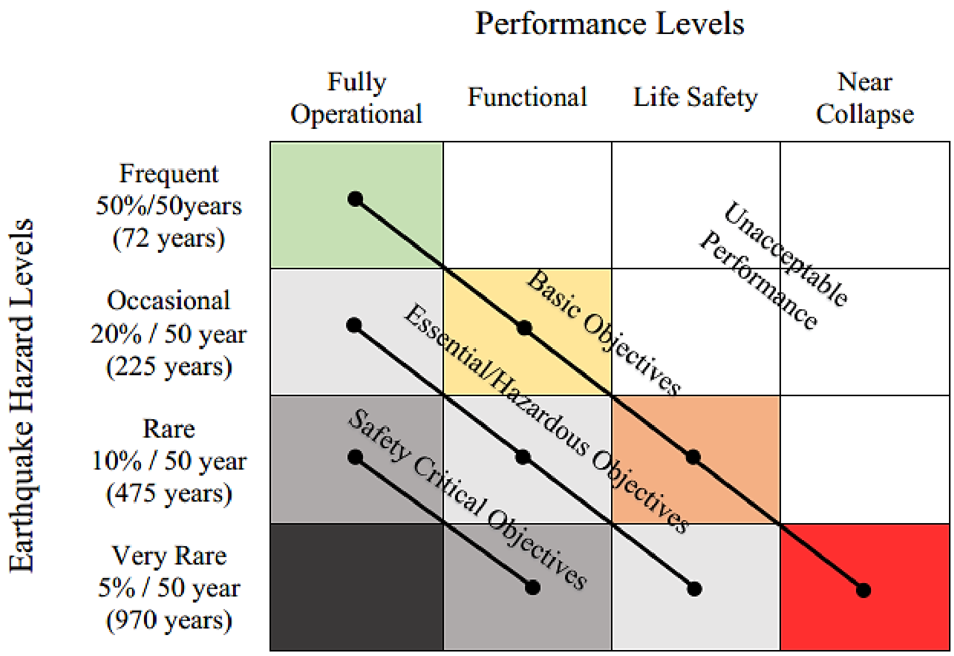


Fig. 2.3 Performance objectives as per SEAOC Vision 2000

Fig. 2.4 represents the performance objectives recommended in some other guidelines, e.g., FEMA 356 (ASCE 2000), FEMA-389 (FEMA 2004). These performance objectives originally represented for existing buildings for seismic evaluation for rehabilitation purpose can also be used for designing new buildings (FEMA 2004). The performance levels are same but there is one difference in earthquake hazard level IV, in SEAOC vision 2000 return period of the very rare earthquake is 970 years whereas in FEMA 356 or FEMA 389 the return period is 2475 years. The consideration of increased return period or the earthquake with decreased probability of occurrences is due to the present consideration of earthquake hazard assessment. In most of building codes or standards, it is found that seismic hazard assessment is performed with the consideration of Maximum Considered/ Credible Earthquake (MCE) whose probability of occurrence in 50 years is 2% (i.e., the mean return period is 2475 years).

		Target Building Performance Levels			
		Operational	Immediate Occupancy	Life Safety	Collapse Prevention
Earthquake Hazard Levels	50% / 50 years	a	b	c	d
	20% / 50 years	e	f	g	h
	BSE-1 10% / 50 years	I	j	k	L
	BSE-2 2% / 50 years	m	n	o	p

* Basic Safety Objective: k + p

Fig. 2.4 Performance objectives as per FEMA 356

2.3.4.1.1 Performance Levels

The level of damages to structures designates the behaviour or performance of the structures in earthquake hazards. This behaviour or damage condition refers to the performance level of the structure. Performance levels are the combination of structural performance and non-structural members' performance. Four common performance levels are generally found in several guidelines for evaluation of the existing and design of new buildings. Fig. 2.5 represents the four visual performance levels of a building.

Operational/ Fully Operational: Buildings designed for such performance level will perform such a way that the overall damage is negligible. Strength and stiffness of the building after the earthquake will be same as before the earthquake. The structural elements and also the non-structural members of the building are expected to have very negligible or no damage. Necessary facilities will be functional, and the threat to life safety is ignorable. In this level, structural member may have some minor cracks and minor cracks in the facades, partitions and ceilings. Buildings should satisfy this performance level under earthquakes with low intensity i.e., earthquakes with high frequency of occurrence. However, critical facility buildings should be as designed that it is intended to perform in this performance level under rare earthquake.

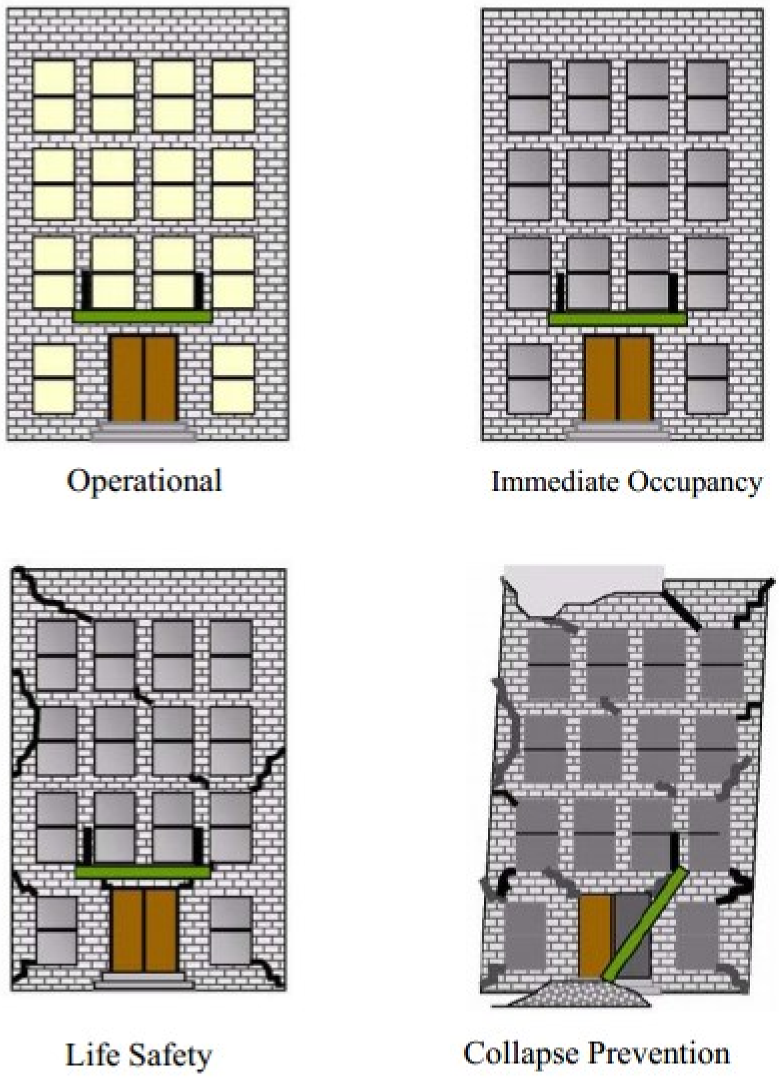


Fig. 2.5 Visual illustration of building performance level (FEMA 2004)

Immediate Occupancy: This performance level permits overall light damage to the building. The building will have almost its original stiffness after experiencing the earthquake. While non-structural components may suffer from minor damages and the risk to life safety is very low, structural elements will likely sustain little to no damage if any at all. Both structural and non-structural components will perform at the level required for immediate occupancy at this level. Utility services will be operational after the earthquake, but there may be some disruption to use other unnecessary services. Building will be safe to occupy, though some clean-up or minor repair. It is expected to obtain this performance level for moderate earthquake, but for buildings with critical facilities this performance level can also be selected under severe level earthquake.

Life Safety: The building constructed with this performance level will sustain some moderate to significant damage. Stiffness will be lost significantly, but will have some lateral strength against collapse. Gravity load carrying elements will be functional, components will experience significant damages. Life safety risk is low. After the earthquake the building can be occupied after repair though repair cost may be high. This performance level is the base in current design codes and provisions for new structures. It is expected to obtain this performance under severe level earthquake hazard.

Collapse Prevention: Building will experience severe damage at this performance level. Structural system will lose stiffness and strength significantly. The building will be near collapse state, risk to life safety will be severe though some gravity load carrying elements, and columns may be functional. Non-structural components and infill walls may collapse, and structural members sustain severe damage.

2.3.4.1.2 Earthquake Hazard Level

Earthquake hazard levels and corresponding performance levels of structures create performance objectives. Earthquake hazard levels are generally defined as acceleration response spectra developed from seismic hazard assessment. The hazard levels are categorized based on the frequency of occurrence or the mean return period of earthquakes.

Three to four earthquake hazard levels are generally provided or suggested in the guidelines. In ATC 40 (ATC 1996) three hazard levels are specified whereas FEMA 356 (ASCE 2000) provides four earthquake hazard levels. These earthquake hazard levels can be expressed by site intensity, mean return period as well as by probability of exceedance.

Serviceability Level Earthquake Hazard: In some guidelines (e.g., PEER TBI) this earthquake level is defined as 50% probability of exceedance in 30 years, i.e., the mean return period (RP) is 43 years for frequent earthquakes. However, in addition to this, this earthquake hazard level is also defined as 50% probability of exceedance in 50 years, i.e., the mean return period is 72 years for occasional earthquakes suggested as serviceability level earthquakes.

Design Base Earthquake (DBE) Hazard: The definition of the design base earthquake (DBE) hazard is an earthquake with a 10 percent probability of occurrence in the next 50 years, or an earthquake with a mean return period of 475 years. Both seismic evaluation and rehabilitation as well as new seismic building design use this level of earthquake hazard. This hazard level is also known as Basic Safety Earthquake 1 (BSE-1) for rehabilitation purposes in order to comply with basic safety objectives (BSO) (as stated in FEMA 356). Most seismic codes, including ASCE 7, BNBC 2020, refer to this threshold of earthquake as the design base earthquake (DBE) for seismic design of new buildings. The present codes recommend that buildings vulnerable to DBE hazards perform at a level consistent with life safety.

Maximum Considered/ Credible Earthquake (MCE) Hazard: This earthquake hazard level is defined as Basic Safety Earthquake 2 (BSE-2) for rehabilitation purposes in some guidelines (e.g., ATC 40, FEMA 356). In some guidelines and seismic codes, the level is defined as Maximum Considered/ Credible Earthquake (MCE) (e.g., ASCE 7, BNBC 2020). This earthquake hazard level is defined as earthquake with 2% probability of exceedance in 50 years i.e., the mean return period of the earthquake is 2475 years. This hazard level is to satisfy the Basic Safety Objectives (BSO) for rehabilitation of the buildings. However, in some standards and guidance (as in Vision 2000), earthquake with

5% probability of exceedance in 50 years (i.e., 970 years mean return period) is suggested as very rare event as maximum earthquake hazard.

2.4 PRELIMINARY DESIGN OF BUILDING

After the selection of performance objectives, the next step in PBSD is preliminary design of the building (ATC 2006). A number of attributes of the building are included in the preliminary building design step. These include the site, location and configuration of the structure i.e., number of stories, storey height, geometric irregularities, building structural system, size and location of elements, foundation type, etc. These features influence the performance capability of a building experiencing an earthquake.

As PBSD is an iterative procedure, it is required to select a suitable preliminary design to implement this approach efficiently and effectively. Improper preliminary design may lead to large iterations to have acceptable solution satisfying the selected performance objectives. Selection of improper preliminary design also may not meet enough the performance objectives (ATC 2006).

2.5 LINEAR MODAL TIME HISTORY ANALYSIS

A numerical technique called linear modal time history analysis is used in structural engineering to investigate the dynamic response of a structure to time-varying loads. Combining the ideas of modal analysis and time history analysis, the structural reaction is computed in terms of displacements, velocities, accelerations, and internal forces. A linear modal time history analysis suggests that the structural response will always remain within the linear range, which is a crucial point to remember. To determine the structure's inherent frequencies, mode shapes, and modal masses, a modal analysis must be performed first. After acquiring the mode shapes and natural frequencies, the structure must then be subjected to time history loads. The dynamic loads that the structure actually faces over a specific time period are represented by time history loads. The structure's response to the time history loads is then estimated by superimposing the responses of various modes. The modal mass, mode shape, and natural frequency are used to estimate each mode's response. The overall response of the structure is then calculated from the combined value of the modal responses.

2.6 NONLINEAR TIME HISTORY ANALYSIS

One of the most effective and trustworthy structural analysis techniques is time history analysis (THA). This analysis is used to achieve or assess performance of a building or other structure with design seismic intensity. In this dynamic analysis, the building is subjected to one or more-time earthquake ground motion time history records. Sometimes the original recorded ground motion time history is used for analysis and sometimes selected time history records are scaled according to the goals of the analysis and assessment. In addition, sometimes the time history records are matched with target response spectrum for a more reliable seismic analysis to meet the objectives.

Linear and nonlinear time history analyses (THAs) are the two types of THAs that are carried out. The result of the analysis, which is carried out in steps, is the way a building or other structure responds to dynamic earthquake loadings.

2.7 ENGINEERING DEMAND PARAMETER

Storey drift ratio (SDR) is one of the meaningful engineering demand parameters which is used to evaluate performance of building structures. For example, FEMA P58 (ATC 2018), FEMA 356 (2000), ATC 40 (ATC 1996) uses the story drift ratio to assess earthquake damage and its consequences for performance assessment of buildings. For performance assessment, the peak story drift or storey drift ratio in each storey should be assessed for each ground motion to understand the structural behaviour. The limits of various performance level of buildings as provided in various guidelines/ standards are tabulated in Table 2.1.

Table 2.1. Storey Drift Ratio for Various Performance Levels of Building

<table>
<tr><th>Guideline/ Standard/ Code</th><th>Reference</th><th colspan="12">Storey Drift Ratio Limit for Various Performance Level associated with A Hazard Level</th></tr>
<tr><td>PEER TBI</td><td>PEER 2017</td><td colspan="6">0.5% for SLE
(43 years return period)</td><td colspan="6">3% for collapse prevention
(2475 years return period)</td></tr>
<tr><td>ATC 40</td><td>ATC 1996</td><td colspan="6">1% for Operational</td><td colspan="6">2% for Life Safety</td></tr>
<tr><td>Vision 2000</td><td>SEAOC 1995</td><td colspan="3">0.2%
(43 years)</td><td colspan="3">0.5%
(72 years)</td><td colspan="3">1.5%
(475 years)</td><td colspan="3">2.5%
(975 years)</td></tr>
<tr><td>FEMA 356</td><td>ASCE 2000</td><td colspan="4">1% for IO
(225 years RP)</td><td colspan="4">2% for LS
(475 years RP)</td><td colspan="4">4% for CP
(2475 years RP)</td></tr>
<tr><td>BNBC</td><td>BNBC 2020</td><td colspan="12">2% for life safety (for present case study building)</td></tr>
<tr><td>Turkish Code</td><td>TEC 2007</td><td colspan="6">1%
(72 years RP earthquake)</td><td colspan="6">2%
(475 years RP earthquake)</td></tr>
</table>

Based on the relevant storey drift limit for various performance/ hazard level provided by guidelines/ standards, the storey drift limit for the present study for various performance level associated with a hazard level are as follows.

Basic Safety Objective (BSO):

- 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%.

2.8 SOIL-STRUCTURE-INTERACTION (SSI)

Soil-structure-interaction (SSI) effect were ignored in designing structures for earthquake hazard since it was believed to have favourable effects. SSI plays significant role on response of structures and also affects the dynamic properties (Fatahi et al. 2014). Foundations with flexibility can affect the responses of structures under seismic loads and advantages of SSI are addressed in the current codes for seismic design e.g., ASCE 41 (ASCE 2016). Effect of various types of pile foundations on the seismic performance was investigated with a shaking table test by Fatahi et al. (2014). The dynamic properties of the system can be altered mainly due to the rocking component, and thus can increase the lateral deflection consequently inter-story drifts.

Soil-structure interaction (SSI) between the interrelated systems—the building, its foundation, the soil beneath the foundation, and the environment—affects the seismic response (FEMA 2009; NIST 2012). The collective response to free-field ground motion, or ground motions unaffected by structural vibrations or waves beneath the structure, is assessed using soil-structure interaction analysis.

Buildings are typically assumed to be fixed at their bases when evaluating their seismic response. Only when the structure is on solid rock is this assumption reasonable. Finding the difference between the structure's response and the theoretical stiff base structure's response is taken into account by SSI. There are two methods used to assess the impacts of soil-structure interaction (SSI): direct approach and substructure approach. In a direct method, the soil and the structure are modeled together and then examined as a whole. The SSI problem is divided into several components in the substructure approach, which are then integrated to generate the full solution. The direct solution for the SSI problem is difficult when the system is complex and also includes nonlinearity from a computational standpoint, so it is rarely practiced (NIST 2012). However, the direct method approach can be used only in large and critical projects like nuclear power plants, major bridges, tunnels, etc. (FEMA 2020). Fig. 2.6 and Fig. 2.7 illustrates these two the approaches for SSI (adapted from NIST 2012).

Direct Approach: In the direct approach, the foundation and structural elements are represented alongside the soil as a continuum (e.g., finite elements) (FEMA 2020, NIST 2012). The soil modeling encompasses the building and its foundation in sufficient detail to take site characteristics into account. At the soil's edge, seismic waves are transmitted. As a result, the soil's constituent elements get excited, that in turn excites the structure.

Substructure Approach: In the substructure technique, springs are used to represent the soil. In order to capture foundation rotations, which are frequently the main cause of SSI effects, the springs are normally vertically orientated. The foundation is frequently fixed to prevent horizontal translation. However, dampers can be added to capture foundation dampening, and horizontal springs can be employed to capture the foundation's ability to move horizontally in relation to the free-field.

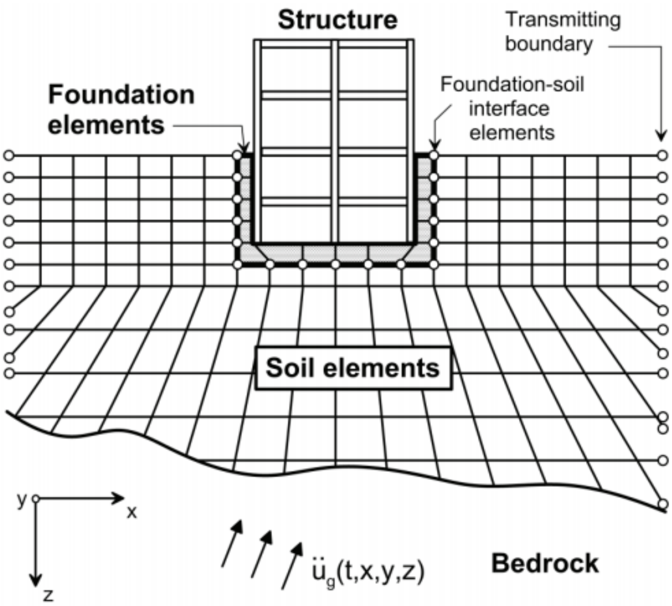


Fig. 2.6 Direct approach of soil-structure interaction model (NIST 2012)

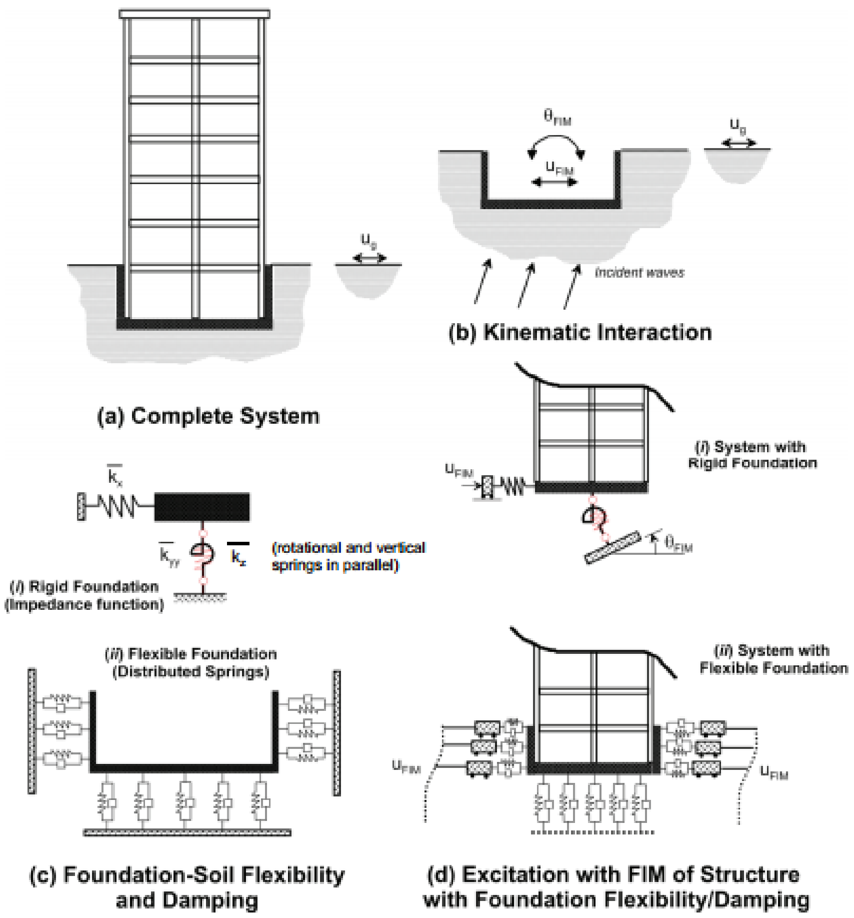


Fig. 2.7 Substructure approach of soil-structure interaction (NIST, 2012)

Chapter 3: METHODOLOGY FOR ANALYSIS, ASSESSMENT AND DESIGN

This chapter of the Thesis provides outline of the methodology used for the modelling, analyses, and design of the building frame, details regarding the nonlinear time history analyses with spectrum matched ground motion acceleration, performance assessment for the selected objectives corresponding to the each considered hazard level, modelling and assessment of effects for the consideration of SSI, etc. The outline provided in this chapter will help to understand the overall concept and methodology of the study as described in the following sections and sub-sections.

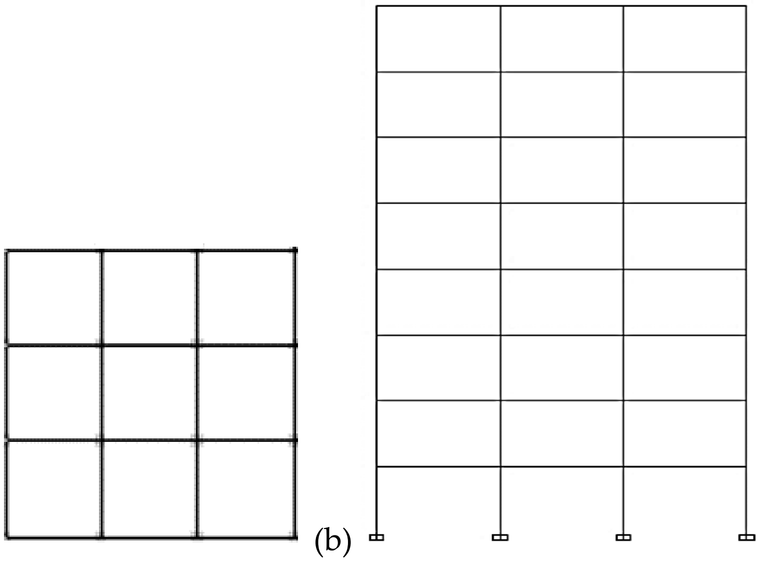
3.1 GENERAL

The present study includes seismic Analysis, design and assessment of an eight (8) storied residential building located at Chattogram. At first, analysis of the building frame has been done following the updated Bangladesh National Building Code (BNBC 2020) for design basis earthquake (DBE) to satisfy life safety performance level. Design of the selected building frame has been performed following the BNBC 2020. Then, the building frame has been assessed using the procedure of performance-based seismic method checking the performance objectives and also checking the capacity of beams and columns. Performance objectives have been selected considering performance level of structure corresponding to specified earthquake hazard level. The building has been analyzed with NTHA using eleven (11) ground motion records matched with the DBE and MCE acceleration response spectrum of BNBC. BNBC recommended load combinations have been used to assess the capacity of beams and columns. Finally, the building has been analyzed considering soil-structure-interaction effect both with FBD and PBSD approach.

Story drift ratio (SDR), an indicator of structural damage has been selected as demand parameter (EDP) for performance assessment of the building.

3.2 ANALYSIS USING BNBC 2020

The selected building is square in plan with three (3) bays in each direction. Bay width, and storey height are 4.572 m (15 ft), and 3.05 m (10 ft), respectively. Fig. 3.1 represents plan and elevation of the selected building.



(a) (b)

Fig. 3.1. Plan and Elevation of the selected building

Table 3.1 Soil Properties and Related Parameters

Soil Class	V_{S30} (m/s)	Site Coefficient, S	Poisson Ratio, υ	Unit Weight (KN/m³)	Shear Modulus, G (KN/m²)
SC	300	1.15	0.25	18	165137.6

The building is situated at Chattogram city at a location with soil condition SC having average shear wave velocity (V_{s30}) as 300 m/s. The soil properties and parameters for the soil type are shown in Table 3.1. The unit weight of soil is considered following ATC 40 and the Poisson ratio is considered following ASCE/SEI 41 (ASCE 2017), and Shear modulus is calculated from unit weight and shear wave velocity using Eq. 3.1.

$$G = \frac{\gamma V_s^2}{g} \tag{3.1}$$

An interior frame of the selected regular RC building has been analyzed using the procedure of Equivalent Static Analysis (ESA) of the BNBC 2020. Various dead and live loads for the buildings have been considered as per BNBC 2020. The framing system is considered as special reinforced concrete moment frames (SRCMF) to fulfil all the requirements of BNBC 2020. The natural period, T is determined as using Eq. (2). According to BNBC, Ct is 0.0466 (while h in m) and m is 0.9 for concrete moment- resisting frame. The seismic design base shear is evaluated using the Eq. (3). According to code, the spectral acceleration for the DBE hazard (return period of 475 years) is estimated using Eq. (3.2).

$$S_a = \frac{2}{3}\frac{ZI}{R}C_s \tag{3.2}$$

Where, S_a, Z, I, R, C_S are the design spectral acceleration, seismic zone coefficient, structure importance factor, response reduction factor, and normalized acceleration response spectrum, respectively. C_S is normalized acceleration response spectrum which depends fundamental period and soil type as defined by Eq. (3.3a-d).

$$C_s = S\left(1 + \frac{T}{T_B}(2.5\eta - 1)\right) \text{ for } 0 \le T \le T_B \tag{3.3a}$$

$$C_s = 2.5S\eta \text{ for } T_B \le T \le T_C \tag{3.3b}$$

$$C_s = 2.5S\eta\left(\frac{T_C}{T}\right) \text{for } T_C \le T \le T_D \tag{3.3c}$$

$$C_s = 2.5S\eta\left(\frac{T_C T_D}{T^2}\right) \text{for } T_D \le T \le 4\text{sec} \tag{3.3d}$$

C_S depends on S and T_B, T_C and T_D. S, T, and η are soil factor, building period, and damping factor. The damping factor, η is calculated using Eq. (3.4) taking damping ratio as 5%. Site dependent soil factor is 1.15 and T_B, T_C, and T_C for soil type S_C are 0.2 sec, 0.6 sec, and 2.0 sec, respectively.

$$\eta = \sqrt{\frac{10}{5+\xi}} \geq 0.55 \qquad (3.4)$$

BNBC 2020 load combinations have been used for analyses and design to have most severe effects. In addition to the consideration of the horizontal earthquake components, vertical earthquake loading has been also considered as per Eq. (3.5) as suggested in BNBC. This dead load component from the vertical earthquake loading should be included in the load combinations of BNBC 2020.

$$E_v = 0.5 a_h D \qquad (3.5)$$

where a_h is horizontal PGA which is equal to $^2/_3ZS$.

The deformation or deflections (δ_x) of level x shall be determined using Eq. (3.6a) and storey drift at each storey level should be computed as per Eq. (3.6b). Limit of storey drift as per code is $0.02h_{sx}$ for the considered building.

$$\delta_x = \frac{C_d \delta_{xe}}{I} \qquad (3.6a)$$

$$\Delta_x = \delta_x - \delta_{x-1} \qquad (3.6b)$$

Here, C_d, δ_{xe} and I are deflection amplification factor, deflection obtained from an elastic analysis and building importance factor, respectively.

3.3 ANALYSIS USING PBSD APPROACH

For performance assessment following the PBSD approach, at first, the performance objectives should be selected. Performance objectives are associated with performance levels of structure corresponding to defined earthquake hazard levels. For the present study, hazard levels and performance levels corresponding to Basic Safety Objective (BSO) have been selected. The performance level, hazard level, and performance objectives are represented in Table 3.2.

Table 3.2 Hazard and Performance Level and Performance Objectives

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

Basic Safety Objective

As the structure is a residential building, the basic safety objective for life safety and collapse prevention performance level has to be checked as recommended in FEMA 356. This means that the building frame should be analyzed with DBE and MCE matched ground motion records for life safety (LS) and collapse prevention (CP) performance level, respectively.

Peak ground acceleration (PGA) for DBE and MCE hazard levels have been evaluated following the updated BNBC 2020. Acceleration response spectrum for those hazard levels have been prepared. Ground motion time history records of the selected earthquake are matched with the acceleration response spectrum corresponding to the required hazard level. These matched ground motion records for the nonlinear time history analyses (NTHA), both for DBE and MCE matched records for LS and CP performance level. From the response of these analyses, performance have been performed checked for the selected performance objectives with the selected preferred criteria.

3.4 EARTHQUAKE GROUND MOTION TIME HISTORY RECORDS

Eleven acceleration ground motion time history records have been used for nonlinear time history analysis (NLTHA) of the building frame. These ground motion records were collected from COSMOS Virtual Data Center

(Strong Motion VDC (https://www.strongmotioncenter.org/vdc) and PEER NGAWEST 2 ground motion database (https://ngawest2.berkeley.edu). Magnitude, and peak ground acceleration (PGA) of the records are represented in Table 3.3. The PGA of the selected earthquake records ranges from 0.245g–1.219. Fig. 3.2 illustrates the ground motion intensity records of all the earthquakes.

Table 3.3 Selected Earthquake Ground Motion Records

Sl. No.	Earthquake Name	Magnitude	Recording Station	PGA (g)
EQ 1	Imperial Valley 1940	6.9 Mw	El Centro Array	0.348
EQ 2	Northridge 1994	6.7 Mw	Canyon Rd, Canyon	0.455
EQ 3	Kobe, Japan 1995	6.9 Mw	Nishi-Akashi	0.509
EQ 4	Uttarkashi, India 1992	7.0 Ms	Uttarkashi	0.310
EQ 5	Landers 1992	7.3 Mw	Yermo Fire station	0.245
EQ 6	San Fernando 1971	6.61 Mw	Pacoima Dam	1.219
EQ 7	Imperial Valley 1979	6.53 Mw	El Centro Array	0.466
EQ 8	Loma Prieta 1989	6.93 Mw	Gilroy Array #1	0.485
EQ 9	Kobe, Japan 1995	6.9 Mw	Takatori	0.618
EQ 10	Duzce, Turkey 1999	7.14 Mw	IRIGM 496	0.739
EQ 11	Northridge 1994	6.7 Mw	Saticoy, Northridge	0.453

3.5 SPECTRAL MATCHING OF THE TIME HISTORY RECORDS

In nonlinear dynamic analyses, the seismic input is ground acceleration time history records. The chosen time histories' response spectra should match the given target response spectrum, such as the design acceleration response spectrum for a hazard level, for a more trustworthy analysis. The main purpose of spectral matching is to adjust the hazard level with the required one, e.g. DBE or MCE hazard level. Spectral matching also has the advantage that the selection of time histories is not stringent as for the scaling approach (Al-Atik and Abrahamson 2010).

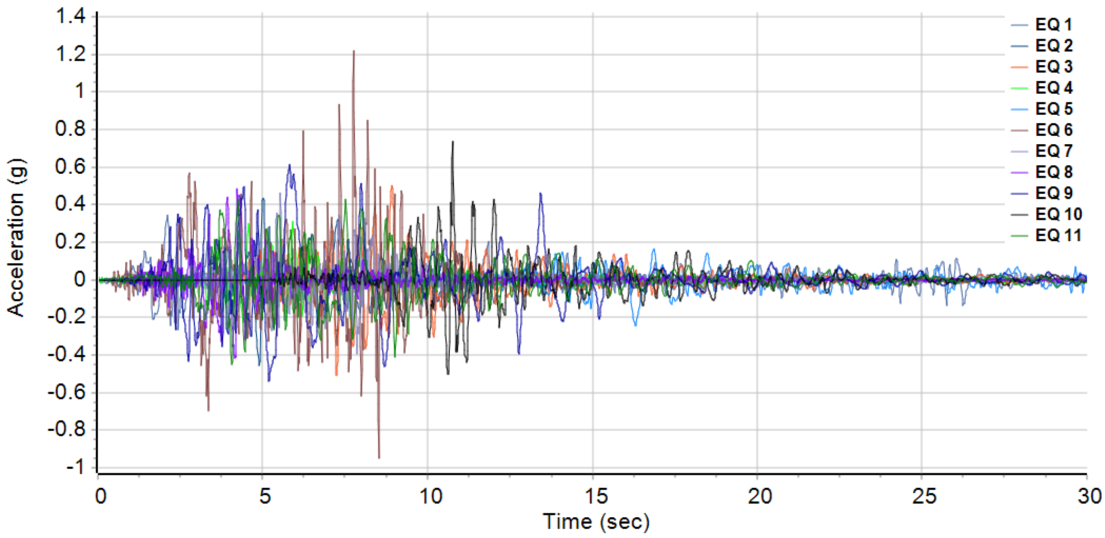
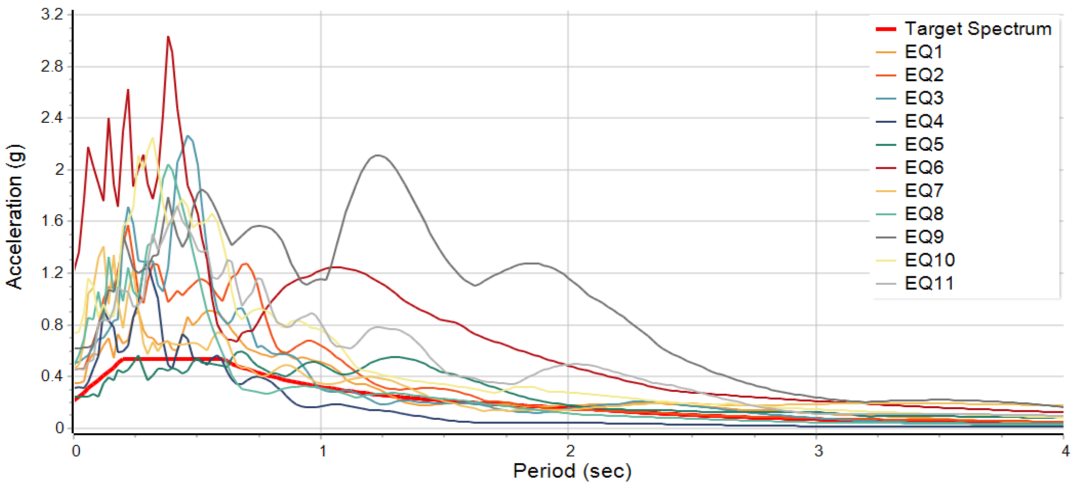


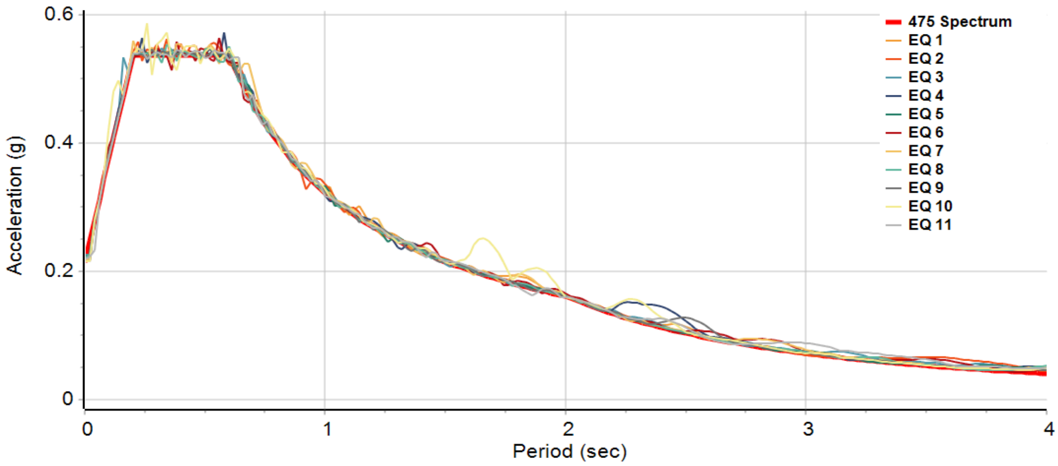
Fig. 3.2. Ground motion acceleration time history of the earthquakes

Spectral matching using frequency domain approach is the straightforward one though it frequently changes the nonstationary nature and has poor convergence properties. The time domain spectral matching approach overcomes these. In the present study spectral matching was performed in time domain approach using the approach of Al Atik and Abrahamson (2010). This was done using "SeismoMatch" (Seismosoft 2020), which is a tool developed by SeismoSoft- Earthquake Engineering Software Solutions.

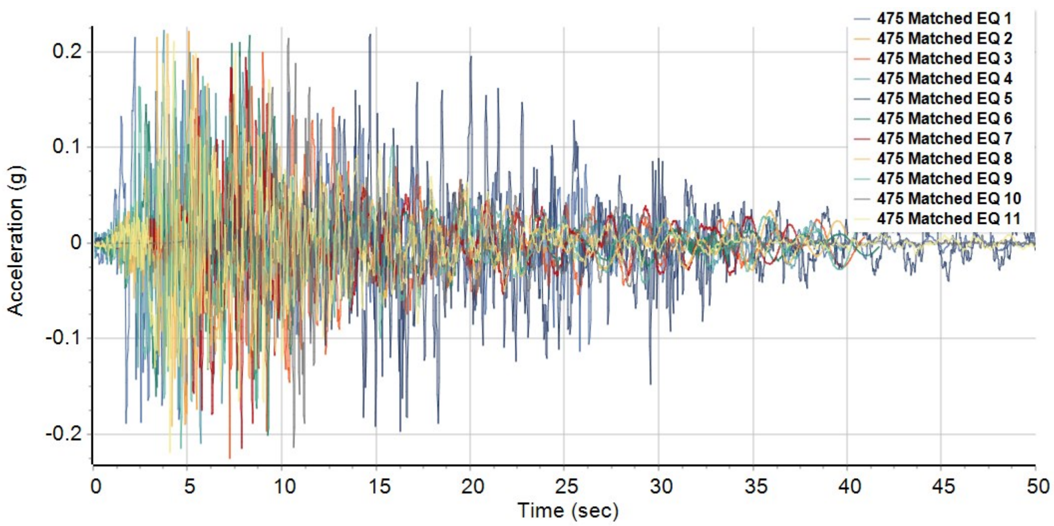
Fig. 3.3a, 3.3b, and 3.3c represent the unmatched acceleration response spectrum, matched acceleration spectrum, and matched earthquake ground motion time history records of 11 earthquakes, respectively for DBE. Fig. 3.3d, Fig. 3.3e and Fig. 3.3f represent the unmatched acceleration response spectrum, matched acceleration spectrum, and matched earthquake ground motion time history records of 11 earthquakes, respectively for MCE hazard. For the performance assessment under SLE level hazard, matching of acceleration history is also performed which is not shown here, but the matched acceleration time history records are shown in Fig. 3.3g. In this study, nonlinear direct integration approach is used for the time history functions defined with the DBE-matched and MCE-matched accelerograms using the "Newmark Beta" method. SLE-matched earthquake records are used for linear modal time history analyses.



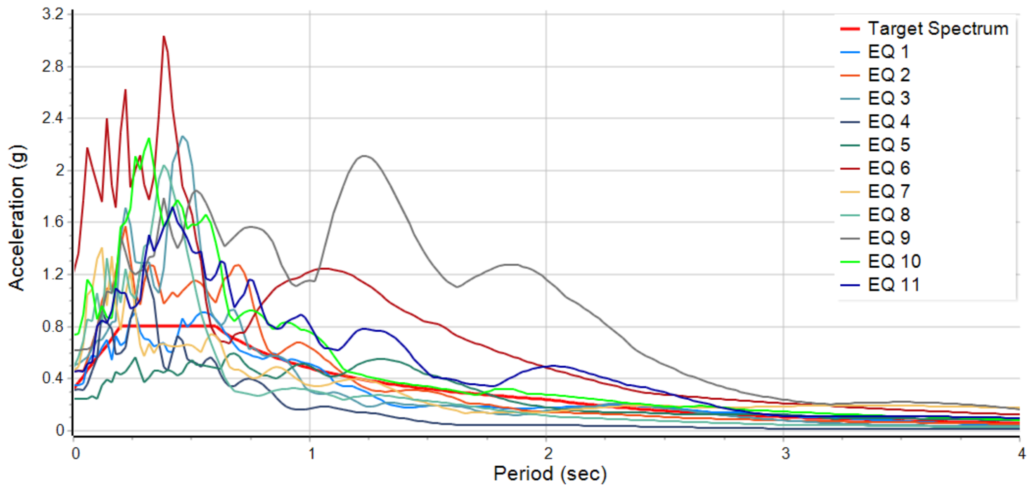
a) acceleration response spectrum of the earthquakes with DBE target spectrum



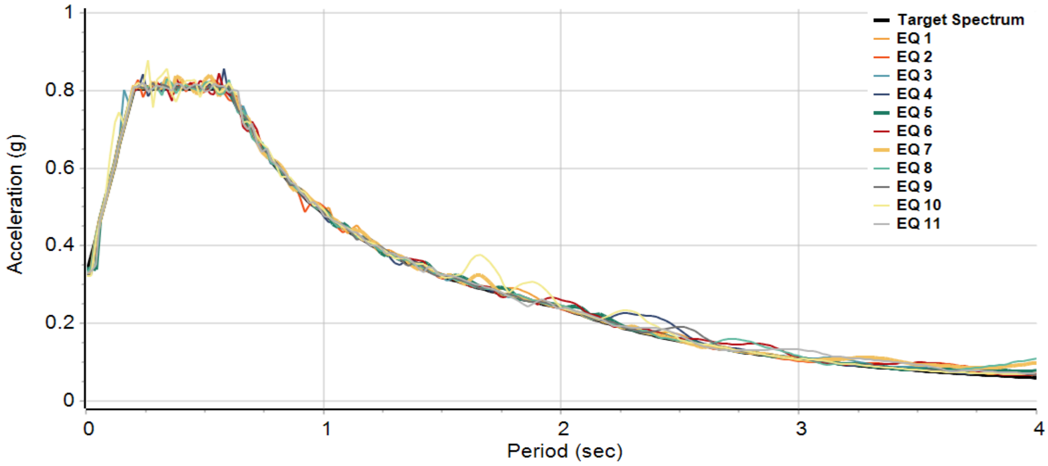
b) DBE matched acceleration response spectrum



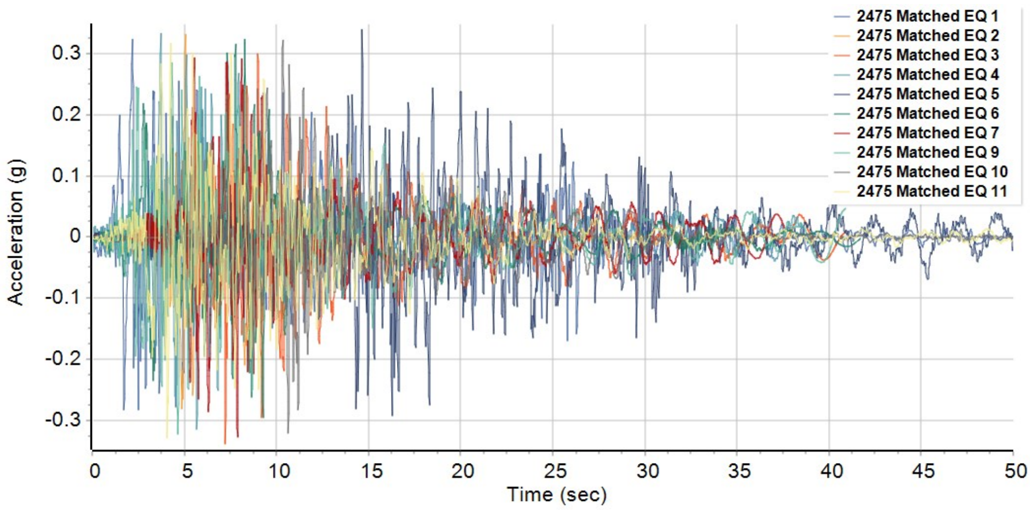
c) DBE matched acceleration time history records



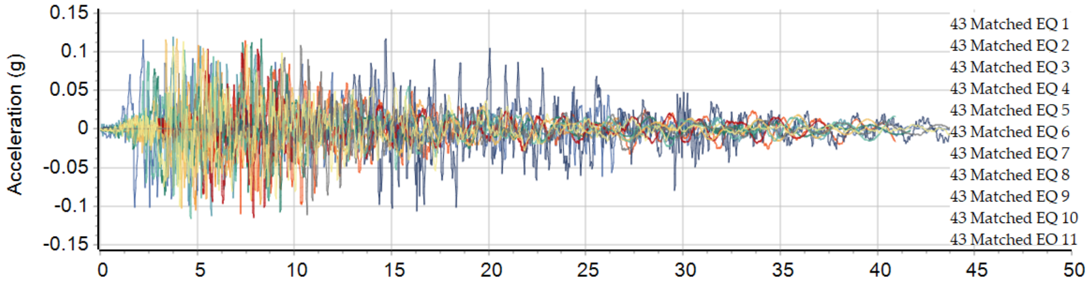
d) acceleration response spectrum of the earthquakes with MCE target spectrum



e) MCE matched acceleration response spectrum



f) MCE matched acceleration time history records



g) SLE (43 years RP) matched acceleration time history records

Fig. 3.3. Spectral matching and matched ground motion records

It is clear from the figures that the PGA of DBE and MCE acceleration matched earthquake records are now about 0.22g and 0.32g which are also found in BNBC 2020. For the PGA estimation of 43 years return period earthquake and also for the development of acceleration spectrum corresponding to this earthquake hazard (SLE), the seismic coefficient is evaluated as 0.10g from the MCE hazard seismic coefficient (0.28) for Chattogram using the approach as shown in Eq. 3.7 from Euro Code 8 (CEN 2004).

$$\gamma_I \sim \left(\frac{T_{LR}}{T_L}\right)^{-1/k} \tag{3.7}$$

where T_{LR} is reference return period of peak seismic action, T_L is return period of a seismic action, k is a factor depending on seismicity. Here, k was found to be 4.07 using seismicity corresponding to return periods of 2475 years and 475 years.

3.6 MODELING OF NONLINEARITY

Nonlinear model of a structure is suitable to clearly identify the structural damage and performance. The linear elastic analysis can be used to estimate design demands with the requisite level of precision in the majority of real-world situations. For a variety of reasons, the discipline of structural engineering frequently considers the nonlinear modeling and analysis of structures as difficult. It first requires a detailed understanding of the myriad complicated interactions and phenomena related to individual inelastic components. Second, nonlinear analysis calls for a significant amount of computational effort as well as the use of specialist tools. However, since structural engineers always strive to arm themselves with the most recent technical breakthroughs, the necessity for nonlinear modeling and analysis is expanding quickly with the introduction of the most recent "Performance-based Design" methodology. The advent of the most recent seismic analysis solvers, software tools, and guidelines (e.g., ASCE/SEI 41) significantly aid in comprehending and putting into practice the nonlinear modeling of structural components.

Nonlinearity's effects of structures can be introduced at the material level, cross-section level, or member level. To directly account for the effect of inelastic materials, the material's stress-strain curve can be defined. Or, inelastic component (or element) can be added for the inelastic effects at the cross-section and member levels. Nonlinear material modeling can be divided into three categories: continuum finite element models, distributed fiber models, and concentrated hinge type models (ATC 2010, NIST 2017a, b). The most popular models for simulating the material nonlinearity of reinforced concrete moment frames are concentrated hinge models (NIST 2017b. Fig. 3.4 represents an idealized frame model with concentrated hinges inserted at expected locations to yield during analysis. In the present study, nonlinear material modelling is performed using concentrated hinges in beam using ASCE 41 guideline using M3 moment hinges. For columns, nonlinear material modelling is performed using distributed fiber-based P-M-M interaction hinges as per NIST guideline (NIST 2017b). Material stress-strain diagram for concrete and steel are defined for material non-linearity at material level for fiber-based nonlinear modeling.

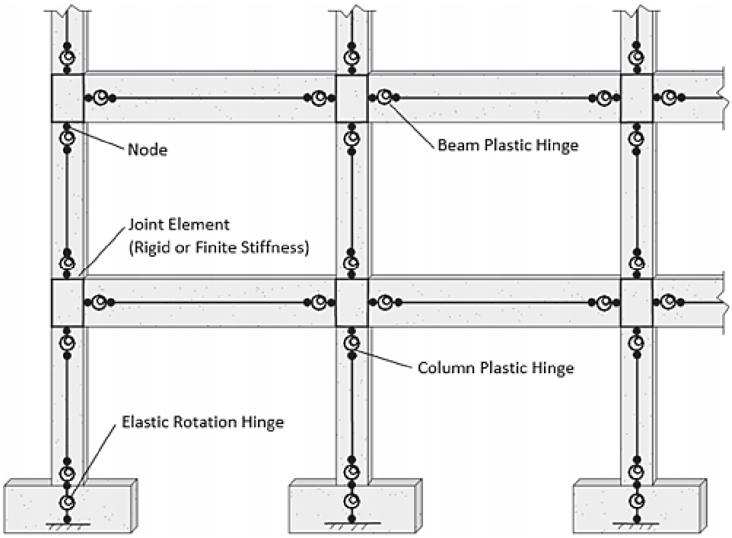


Fig. 3.4. An idealized RC frame with concentrated hinges (NIST 2017b)

Fig. 3.5 illustrates the modelling of frame element which needs defining three components, a) inserting the concentrated hinges at or near the frame ends, b) defining force-displacement relationship (backbone curve) and c) the cyclic response modelling.

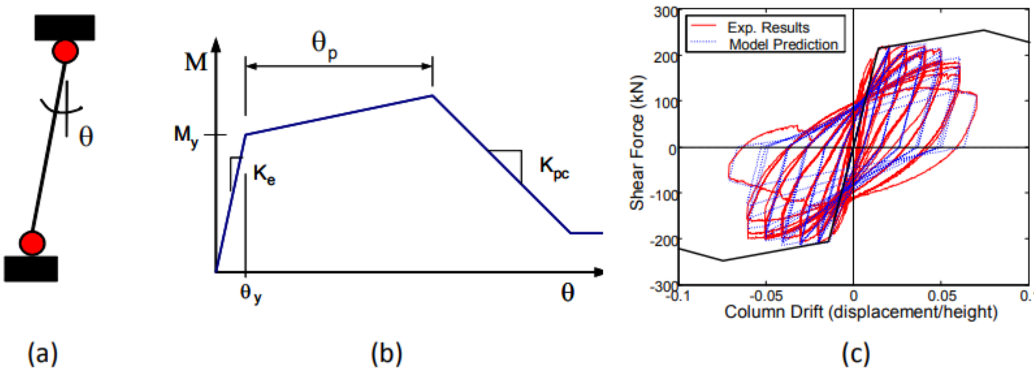


Fig. 3.5. Modeling of a reinforced concrete beam-column (ATC 72-1, ATC 2010)

Material stress-strain diagram and performance points for immediate occupancy, life safety, and collapse prevention level are shown in Fig. 3.6 a and b. To define the parameters for nonlinear modelling of RC flexural members (beams), i.e., for the back-bone curve, hysteresis, etc. strategies, and properties as suggested in chapter 10 of ASCE 41-17 (ASCE 2017) have been followed. Fig. 3.7 provides screen of defined beam's M3 hinges.

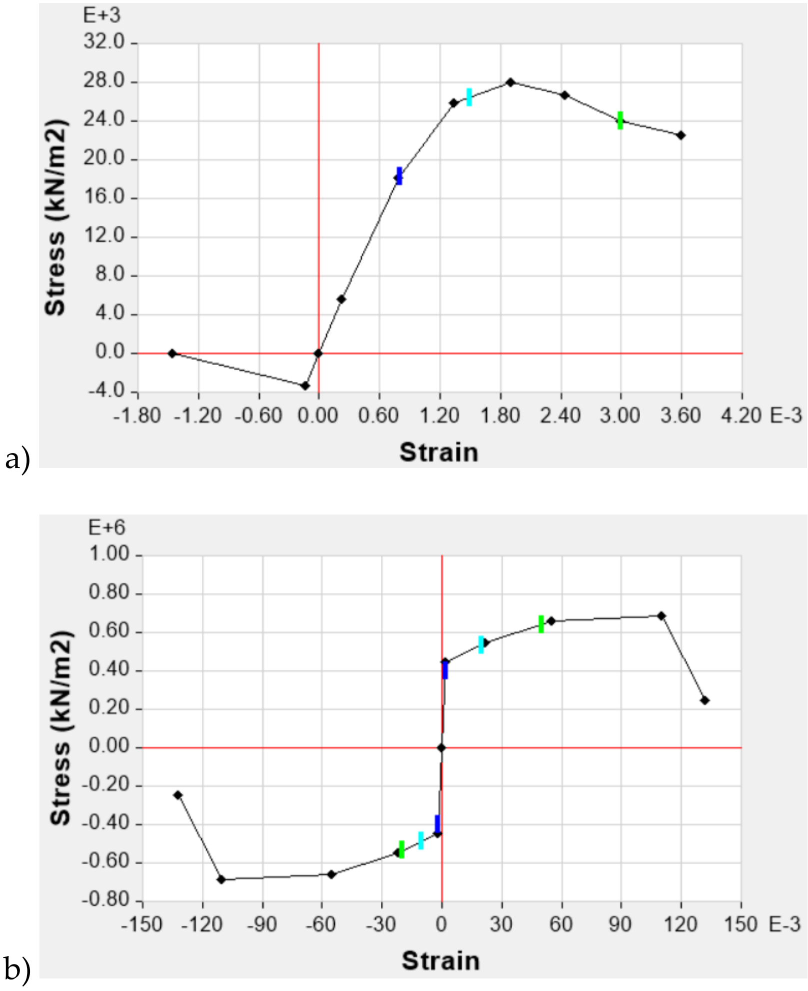


Fig. 3.6. Stress-strain curves and performance point – a) concrete, b) steel

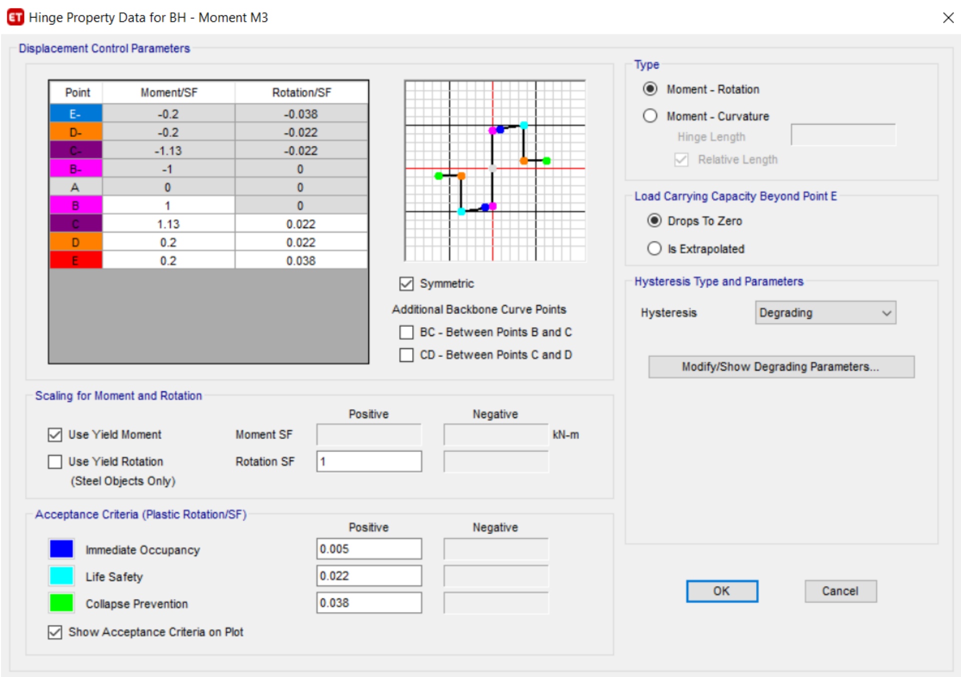


Fig. 3.7. Modeling of concentrated nonlinear M3 hinges

3.7 DEFINING MATERIAL STRAIN FOR PERFORMANCE LEVELS

Material strain can be found from material testing, especially for concrete. Yield strain for 414 MPa (60000 psi) reinforcing steel will be 0.002 (i.e., f_y/E_s). For plain concrete it is widely known that the peak unconfined strain can be taken as 0.002 and the peak confined strain varies with typical values 0.004 to 0.01 (NIST 2017b). Kowalsky (2000) reported limiting buckling strain for reinforcing steel as 0.06, whereas ASCE 41-06 (ASCE 2007) limits the usable strain of reinforcing steel to 0.05. Performance criteria for structures in terms of material strain as found in manual of SeismoStruct (SeismoSoft 2018) are- for RC concrete, spalling strain as -0.002 and core concrete strain for crushing as -0.006 and for reinforcing steel, fracture strain as +0.06. Table 3.4 shows material strain for concrete and reinforcing steel which can be considered for various performance levels.

Table 3.4 Material strain for various performance levels

Performance Level	Concrete Strain		Reinforcement Strain
	Unconfined	Confined	
Life Safety	-0.0015	-0.0025	+0.02
Collapse Prevention	-0.003	-0.005	+0.05

3.8 MODELING OF DAMPING

In a building system, damping can come from a variety of factors, including friction. The amplitude of the structural system's deformations determines how much each damping source contributes to the overall energy dissipation. Studies have shown that while many factors contribute to the system's overall energy dissipation at low levels of excitation, the hysteretic damping from regions of the structure that respond inelastically does so at higher levels of excitation when the structure is responding in the elastic range.

Components in the building system, including the structural frame, foundation, and non-structural components that may dissipate energy. Damping can be modelled as equivalent viscous damping (EVD) (NIST 2017a).

Energy dissipation is modeled in Equivalent Viscous Damping (EVD) as an equivalent velocity-dependent force that tends to be out of phase with the structure's motion and, as a result, reduces or dampens the motions. Mathematically, Rayleigh (proportional to mass and/or stiffness) damping, modal damping, or other discrete damping terms are typically used to express EVD in the [C] matrix. The two damping constants are often chosen to represent the critical damping percentage at two vibration frequencies, T_1 (first-mode) and $0.1T_1$ (third or higher mode) (NIST 2017a). Equation 3.8 has been used to estimate the fraction of critical damping as recommended in several standards and guidelines (e.g., NIST 2017, PEER 2017).

$$\xi_{critical} = \frac{0.36}{\sqrt{H}} \leq 0.05 \tag{3.8}$$

where, H is the height of the structure in feet.

Fig. 3.8 represnts the equivalent viscous damping of building structure as per height of the structure for earthquake. It is recommended in various guidelines, e.g., NIST 2017, PEER 2017, PEER/ATC 72-1 (ATC 2010).

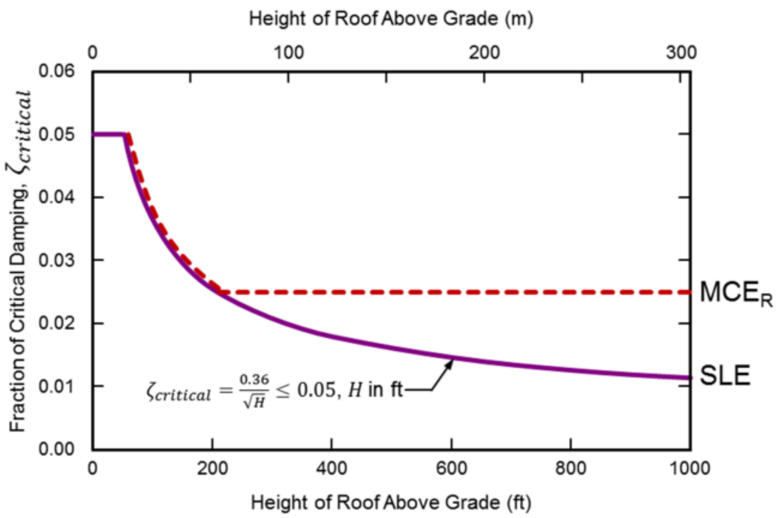


Fig. 3.8. Equivalent viscous damping versus building height

3.9 MODELING SOIL-STRUCTURE INTERACTION

The flexibility of the foundation system, which includes the structural components of the foundation as well as the supporting soil, can significantly affect the building's dynamic characteristics and overall reaction. The analytical model of the structure should take into account the foundation's and the soil's vertical, horizontal, and rotational flexibility when accounting for soil-structure interaction (SSI) (ASCE 7- ASCE 2017). In this regard, soil properties, e.g., unit weight, Poisson ratio, shear wave velocity, shear modulus are needed. These properties are already defined in earlier sections. However, while considering SSI during nonlinear earthquake analysis, guidelines recommend use of reduced shear modulus, G using a reduction factor (NIST 2012, FEMA 2020). The reduction factor based on earthquake hazard level is tabulated in Table 3.5. BNBC soil class S_C is equivalent to NEHRP soil type D and for this soil type and site seismicity, S_{DS} = 0.215, the reduction factor is found to be 0.75.

Table 3.5 Reduction Factor of Shear Modulus in Earthquake

Site Class	Shear Modulus Reduction Factor		
	$S_{DS}/2.5 \leq 0.1$	$S_{DS}/2.5 = 0.4$	$0.8 \leq S_{DS}/2.5$
BNBC S_C	0.90	0.50	0.10

For the evaluation of foundation stiffness based on the foundation geometry and the soil stiffness for three translational and three rotational degrees of freedom, Equations proposed by Pais and Kausel (1988) for rectangular footing at surface are used. These Equations, as tabulated in Table 3.6 are recommended in famous guidelines such as NIST 2012, FEMA 2020. This spring constants need to be modified using the embedment correction factor while the foundation in embedded in soil for a specific depth (Pais and Kausel 1988, NIST 2012, FEMA 2020). For the present study, a mat/raft foundation is considered.

Table 3.6 Static Stiffness of Rigid Footings at Surface (From NIST 2012)

Degree of Freedom	Pais and Kausel (1988)
Translation along z-axis	$K_{z,\,sur} = \frac{GB}{1-\nu}\left[3.1\left(\frac{L}{B}\right)^{0.75} + 1.6\right]$
Translation along y-axis	$K_{y,\,sur} = \frac{GB}{2-\nu}\left[6.8\left(\frac{L}{B}\right)^{0.65} + 0.8\left(\frac{L}{B}\right) + 1.6\right]$
Translation along x-axis	$K_{x,\,sur} = \frac{GB}{2-\nu}\left[6.8\left(\frac{L}{B}\right)^{0.65} + 2.4\right]$
Torsion about z-axis	$K_{zz,\,sur} = GB^{3}\left[4.25\left(\frac{L}{B}\right)^{2.45} + 4.06\right]$
Rocking about y-axis	$K_{yy,\,sur} = \frac{GB^{3}}{1-\nu}\left[3.73\left(\frac{L}{B}\right)^{2.4} + 0.27\right]$
Rocking about x-axis	$K_{xx,\,sur} = \frac{GB^{3}}{1-\nu}\left[3.2\left(\frac{L}{B}\right) + 0.8\right]$

3.10 DESIGN OF BEAM

From the analysis results of elastic analysis (life safety level) and time history analysis with DBE matched acceleration time history records, response parameters of beams are checked for the BNBC 2020 load combinations. For beam moments and shear at both ends of beams and moment at mid of beam are checked. Then, the beams are designed for the governing combination for moment and shear. Design check of beams are performed using developed design checking tool in excel prepared by Fernandez (2011) using 318-08 which is followed by BNBC 2020.

3.11 DESIGN OF COLUMN

Design of columns requires load-moment interaction diagram and consideration of biaxial bending. A typical load-moment interaction diagram is represented in Fig. 3.9. The analysis and design of columns under biaxial bending is not simple. When a normal force is applied on a reinforced concrete (RC) column, axial compression, pure tension, pure flexure, balance failure, etc. can occur (Fig. 3.9).

In the current study, design check of the columns is performed with the help of spColumn V7 (Structure Point), a software from "Structure Point-Concrete Software Solutions" for design and investigations of reinforced concrete column sections, using the axial forces and biaxial moments. This is a wonderful too to investigate and design the columns with the interaction diagram. spColumn V7 evaluates the section with provided reinforcement for all the control points to draw the interaction diagrams (i.e., P-M, MM). Sections can be investigated for the capacity comparing with the load and moment found from the nonlinear response history analysis with the selected 11 DBE based acceleration records. The critical or governing load and moment and/or load combinations can be used for checking the capacity based on moment capacity or critical capacity of the section.

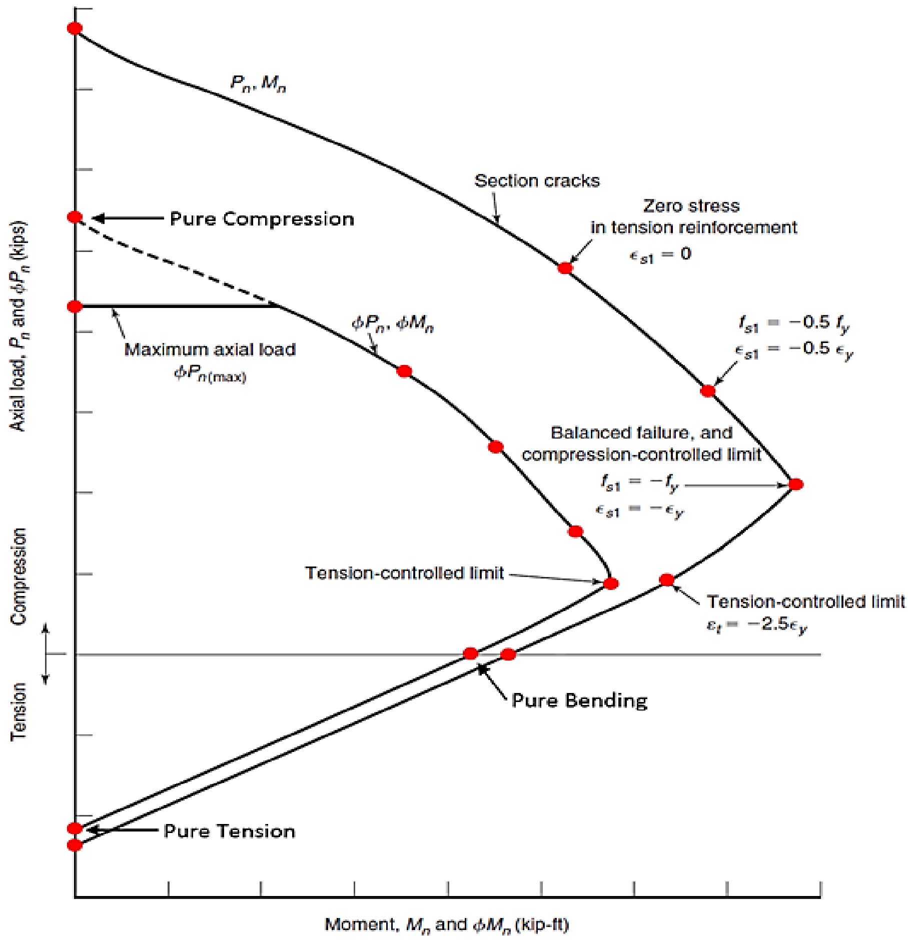


Fig. 3.9. Load–moment interaction diagram with cases (Structures Point)

3.12 BNBC LOAD COMBINATIONS

The considered load combinations from the BNBC 2020 based on the considered analysis and loading cases are as follows:

1. 1.4D

2. 1.2D+1.6L+0.5Lr

3. 1.2D+1.6Lr+1.0L

5a. 1.2D+1.0EQx+0.1073D+1.0L

5b. 1.2D-1.0EQx+0.1073D+1.0L

7a. 0.9D+1.0EQx-0.1073D

7b. 0.9D-1.0EQx-0.1073D

Chapter 4: RESULTS AND DISCUSSION

This chapter illustrates the results from the linear elastic and nonlinear analyses, response of the analyzed frame under the considered loading, performance check, and also the design of beams and columns. All the results are found from the analyses using the methodology as discussed in Chapter 3 in details. At first, the storey drift ratio from linear and nonlinear analyses are analyzed and discussed. Second, axial force, moment, shear, etc. for columns and beams are reported and discussed. Third, the performance check based on the objectives as per the storey drift ratio and the strength requirement of beams and columns are checked and discussed. Fourth, design of beams and columns (sections and reinforcement are represented. And fifth, effects of soil-structure interaction are mentioned.

4.1 GENERAL

The selected RC building frame is analyzed and designed using the force-based approach, BNBC 2020. The code-designed building is assessed using the PBSD approach and then also designed with the PBSD approach. The effect of SSI is also assessed both for force-based and performance-based approach. The results for all the cases are represented and discussed in the following sections.

4.2 EQUIVALENT STATIC ANALYSIS

Equivalent Static Analysis is performed using the methodology of force-based approach following BNBC 2020 as described in previous chapter. The considered column size is 508mm x 508mm (20″x20″) and beam size is 305mm x 508mm (12″x20″). It is to be mentioned that several other combinations are also checked during the trails for consideration which are not mentioned here. Those trails are performed to satisfy the storey drift ratio limit and other design requirements of the BNBC 2020.

The seismic weight, time period, spectral acceleration, and base shear are calculated using BNBC. These evaluated parameters needed for equivalent static analysis is represented in Table 4.1. The calculated base shear is then distributed to the storey levels for analysis with seismic lateral loads which is illustrated in Fig. 4.1. The maximum displacements at every storey level and the storey drift ratio (SDR) evaluated from analysis using ETABS Ultimate 18 (Computer and Structures, Inc.) is represented in Table 4.2. It should be noted that the displacements from the equivalent static analysis are amplified with the amplification factor (5.5) as with recommended in BNBC 2020. From the Table 4.2, it is clear that the selected sections satisfy the storey drift ratio limit. Then, the sections are then investigated and designed with the BNBC approach using the load combinations and governing response for elastic earthquake load.

Table 4.1 Parameters for Equivalent Static Analysis

Seismic Weight, W (kN)	Time Period, T (sec)	Spectral Acceleration, Sa	Base Shear, V (kN)
5139.83	0.826	0.04873	250.43

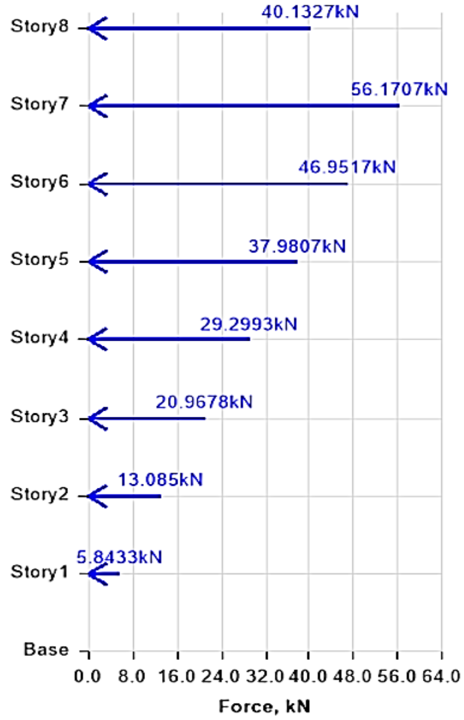


Fig. 4.1. Distribution of Base Shear Force at Storey Level

Table 4.2 Storey Displacement and Drift Ratio from Elastic Analysis

Storey Level	Displacement (mm)	Storey Drift Ratio (%)
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

The following section provides design of beam and column using BNBC force-based approach for the responses from the equivalent static analyses.

4.3 FORCE-BASED DESIGN

From the analysis of the building frame using equivalent static method, the governing load combinations and maximum moment, axial load, shear, etc. are checked. Using the load, moment, shear capacity requirement from the outcome, beams and columns are designed, i.e., section and reinforcement.

4.3.1 Design of Beams

From the analysis of the building frame using equivalent static analysis, the maximum shear, maximum moment at beam end, and maximum moment at mid of beams are found to be 124 kN, 178 kN-m, and 40 kN-m, respectively. For these response requirements, the beam section (305 mm x 508 mm) is designed using excel tool. 3 nos. of 20 mm rebar are provided at the top as longitudinal rebar with 2 nos. of 16 mm rebar as extra top. For the bottom longitudinal reinforcement 3 nos. of 16 mm rebar are provided. Hoops and stirrups are provided with 10 mm rebar. 1st hoop is provided at 50 mm and others (11 nos.) are provided at 90 mm and stirrups are provided at 150 mm. With this design, the moment capacity is found 206 kN-m at end, 99.5 kN-m at mid, and shear capacity is found as 240 kN. Fig. 4.2a, b and c represent excel beam design check for end moment capacity, mid-moment capacity and shear capacity, respectively. The hoops and stirrup and their spacing are found adequate. Fig. 4.3 illustrates details of the designed beam.

LEFT SUPPORT

β1	=	0.85		β1 = max. of: 0.85-(0.05*(f'c-28)/7) and 0.65
c	=	71.14	mm	Solve for "c" in: As*fy = 0.85*f'c*b1*c*b+A's*(c-d')/c*ec*Es
a	=	60.47	mm	a = β1*c
ρ	=	0.0099		ρ = As/(b*d)
$\rho_{(min)}$	=	0.0034		ρ(min) >= 0.25*SQRT(f'c)/fy >= 1.4/fy
ρb	=	0.0289		ρb = 0.85*β1*f'c/fy*(600/(600+fy)
ρg	=	0.0126		ρg = (As+A's)/(b*h)
$\rho_{(max)}$	=	N.A.		ρ(max) = As(max)/(b*d)
$As_{(min)}$	=	459.12	mm^2	As(min) = ρ(min)*b*d
$As_{(max)}$	=	3,110.7	mm^2	As(max) = (0.85*f'c*β1*c*b+A's*(c-d')/c*εc*Es)/fy for c = εc*d/(εc+0.005)
ε_c	=	0.003		εc = 0.003 (assumed concrete strain)
ε'_s	=	0.0010		ε's = εc*(c-d')/c < fy/Es, f's does not yield
f's	=	195.16	Mpa	f's = ε's*Es
As2	=	284.35	mm^2	As2 = A's*f's/fy for f's > 0
As1	=	1060.25	mm^2	As1 = As-As2 for f's > 0
εt	=	0.0158		εt = εc*(d-c)/c = (fy/Es+εc)/(ρ/ρb)-εc
ϕ	=	0.90		ϕ = 0.65+0.25*(εt-fy/Es)/(0.005-fy/Es) <= 0.90
ϕMn	=	205.98	kN-m	> Mu = 178 kN-m, O.K.

(a) end moment

MIDSPAN

β1	=	0.85		β1 = max. of: 0.85-(0.05*(f'c-28)/7) and 0.65
c	=	46.78	mm	Solve for "c" in: As*fy = 0.85*f'c*b1*c*b+A's*(c-d')/c*ec*Es
a	=	39.76	mm	a = β1*c
ρ	=	0.0043		ρ = As/(b*d)
$\rho_{(min)}$	=	0.0034		ρ(min) >= 0.25*SQRT(f'c)/fy >= 1.4/fy
ρb	=	0.0289		ρb = 0.85*β1*f'c/fy*(600/(600+fy)
ρg	=	0.0100		ρg = (As+A's)/(b*h)
$\rho_{(max)}$	=	N.A.		ρ(max) = As(max)/(b*d)
$As_{(min)}$	=	474.44	mm^2	As(min) = ρ(min)*b*d
$As_{(max)}$	=	3,540.9	mm^2	As(max) = (0.85*f'c*β1*c*b+A's*(c-d')/c*εc*Es)/fy for c = εc*d/(εc+0.005)
ε_c	=	0.003		εc = 0.003 (assumed concrete strain)
ε'_s	=	-0.0002		ε's = εc*(c-d')/c < fy/Es, f's does not yie
f's	=	-41.30	Mpa	f's = ε's*Es
As2	=	N.A.	mm^2	As2 = A's*f's/fy for f's > 0
As1	=	N.A.	mm^2	As1 = As-As2 for f's > 0
εt	=	0.0311		εt = εc*(d-c)/c = (fy/Es+εc)/(ρ/ρb)-εc >= 0.005, Tension-controlled
ϕ	=	0.90		ϕ = 0.65+0.25*(εt-fy/Es)/(0.005-fy/Es) <= 0.90
ϕMn	=	99.52	kN-m	> Mu = 40 kN-m, O.K.

(b) mid moment

SHEAR CAPACITY CHECKS

@ 2H for Seismic Provision for transverse Reinforcements per Sec.21.5.3.2

S(max)= smallest of ==>				S(max) = d/4
				S(max) = $8d_b$ (smallest longitudinal bar)
S(max)	=	113.15	mm.o.c.	S(max) = $24d_b$ (diameter of hoop bar)
		> 88mm, O.K.		S(max) = 300mm

For Shear Reinforcements

ϕVc	=	93.14	kN	ϕVc = 0.17*λ*SQRT(f'c)bd
ϕVs	=	147.17	kN	ϕVs = ϕ*fy*d*Av(stirrup)/s >= 0
ϕVn	=	240.31	kN	>=Vu = 124.00 kN, O.K.
$\phi Vs_{(max)}$	=	361.59	kN	>= Vu-(phi)Vc = 30.86 kN, O.K.
$Av_{(prov)}$	=	157.08	mm^2	
$Av_{(req'd)}$	=	57.79	mm^2	<= Av(prov) = 157.08 mm^2, O.K.
$Av_{(min)}$	=	38.68	mm^2	<= Av(prov) = 157.08 mm^2, O.K.
$s_{(max)}$	=	226.31	mm	>= s = 150 mm, O.K.

(c) shear

Fig. 4.2. Beam design in excel

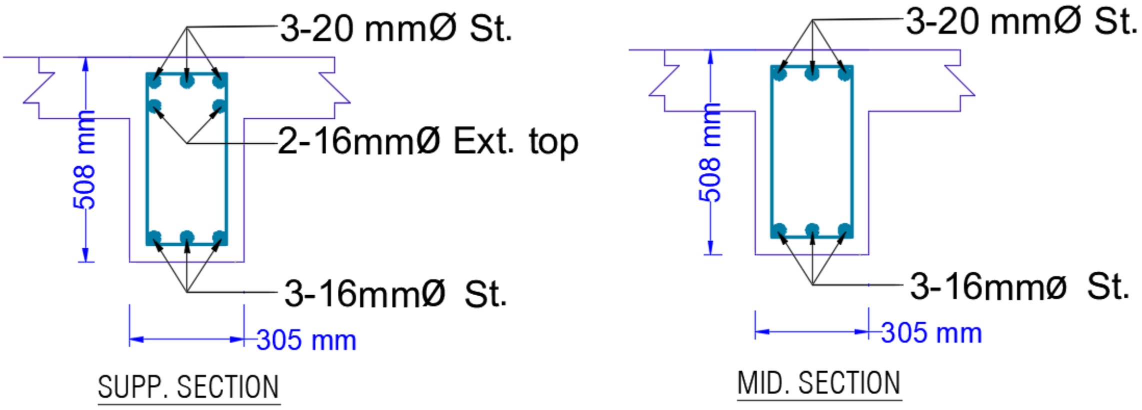
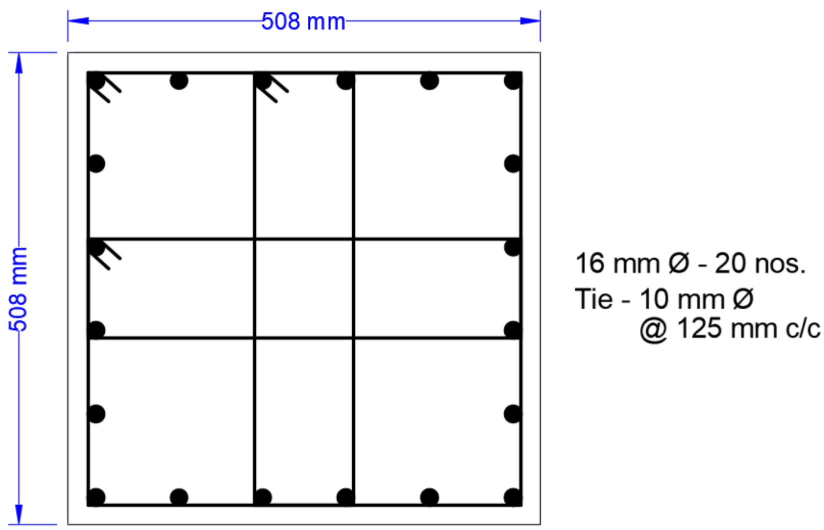


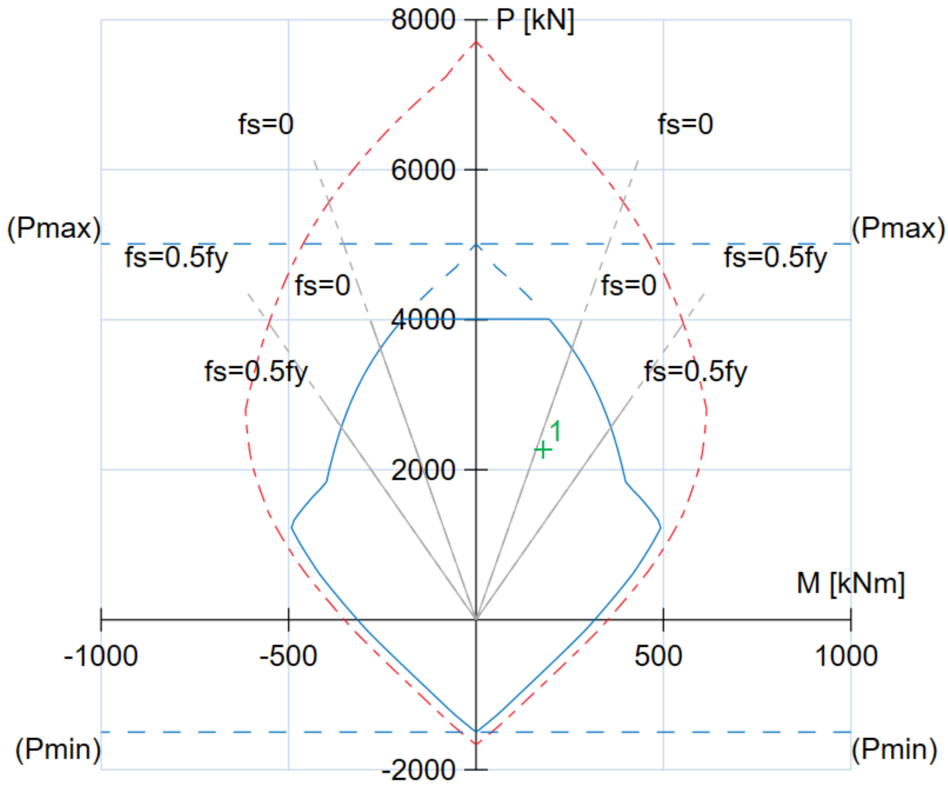
Fig. 4.3. Beam details designed with BNBC elastic analysis

4.3.2 Design of Columns

Columns are designed with spColumn V7 as discussed previously. spColumn perform to draw the PM interaction diagram through the determination of control point based on the failure cases. After that, it checks the column capacity for the defined load, moment, or load combinations, etc. The governing load combination is found with column axial load 2271 kN and moment 179 kN-m. The column section is provided as 20" sq. (508 mm sq.). Use of 20 nos. of 16 mm bar (1.56% reinforcement) as longitudinal rebar and 10 mm tie bar at 125 mm found the columns adequate for the governing load combination. Fig. 4.4a represent the column section and reinforcement details and Fig. 4.4b illustrates the check of capacity with the PM interaction diagram.



(a)



(b)

Fig. 4.4. Column design- details and interaction diagram for BNBC analysis

Now, this code-based designed building frame should be analysed through nonlinear time history analyses using code acceleration spectrum matched ground motion records.

4.3.3 Demand to Capacity Ratio (DC Ratio)

Using the specified sections and reinforcements of beams and columns for the case study building frame, demand-to-capacity (DC) ratios are checked. DC ratios can be checked through checking the column P-M-M interaction ratios in ETABS. It can also be checked in the spColumn tool as the capacity ratio. ETABS will provide the DC ratios for the load combinations which are defined for the analysis and design. The governing load combination, from the all combinations which are mentioned before and defined in ETABS, provides the maximum DC ratios for all the columns in ETABS. The DC ratios found in ETABS are represented in Fig. 4.5.

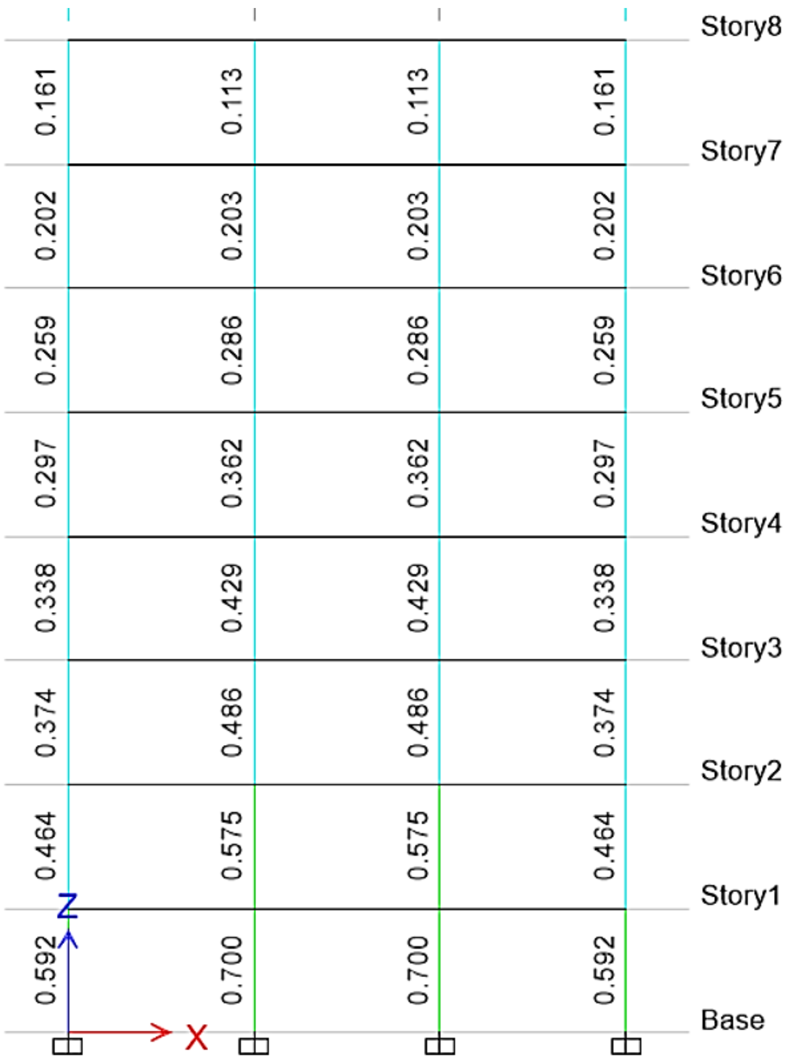


Fig. 4.5. Column DC ratios for BNBC force-based design

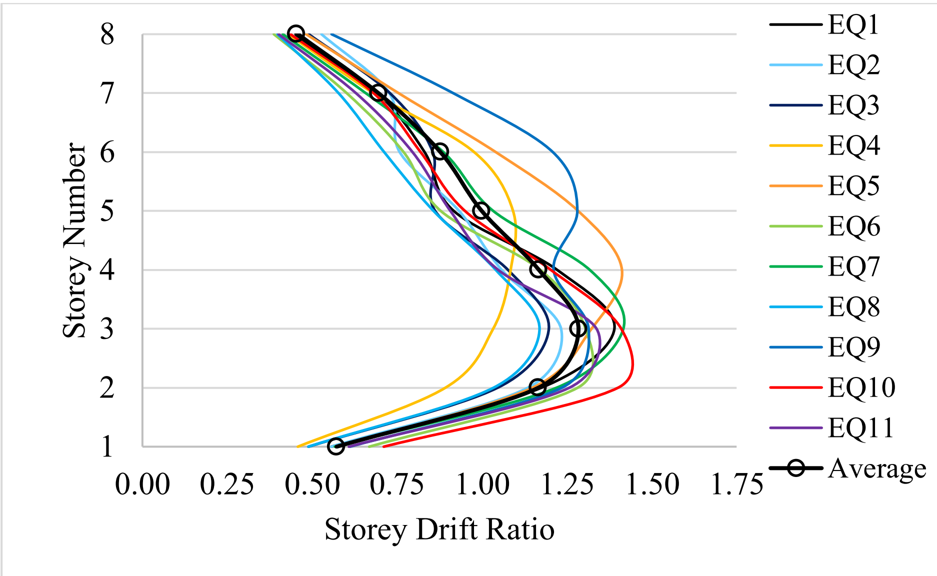
4.4 NONLINEAR TIME HISTORY ANALYSES

The building frame designed with the BNBC 2020 approach is then analyzed to assess for Life Safety performance level under earthquake ground motions matched with DBE hazard spectrum. It is also analyzed with MCE matched earthquake ground motions to check Basic Safety Objective (BSO) and also Enhanced Safety Objectives (ESO) selected as the building's performance objective. Nonlinear material modelling and geometric nonlinearity modelling are performed for the analyses. Damping ratio is found to be 4% for the building. After that, it is checked for performance level and the strength requirements.

4.4.1 Performance Assessment of the Code-Designed Building

Storey drift ratio (SDR) is a true indicator of damage to the building which can represent performance level of the building. The selected performance parameter, SDR at each storey level as found from the analyses under DBE and MCE ground motions are averaged as recommended in several codes/ standards/ guidelines (e.g., BNBC 2020, ASCE 7-16). Fig. 4.6a and Fig. 4.6b represent the storey drift ratio for DBE and MCE level earthquake response history analyses, respectively.

a)



b)

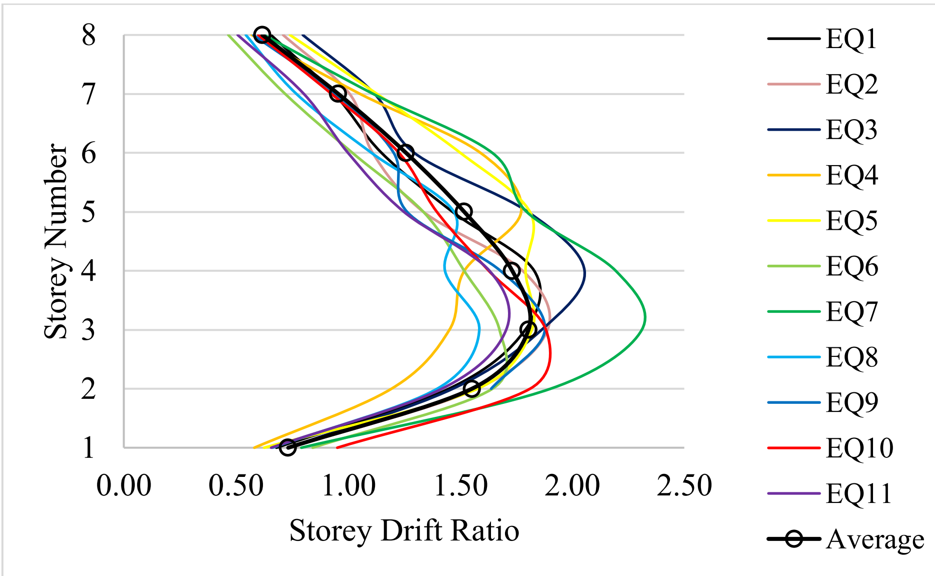


Fig. 4.6. Storey drift ratio from nonlinear earthquake analyses– a) DBE, b) MCE

From the Fig. 4.6, it is found that the building frame with specified beam and column sections and reinforcement designed through the BNBC 2020 code-based procedure is adequate satisfying the performance objective. The maximum storey drift ratio (mean of all responses for the 11 earthquake ground motion records) for DBE and MCE hazard level is 1.28 % and 1.81 %.

4.4.2 Beam and Column Responses in Nonlinear Analyses

Moment at support and mid, shear-force of beam for all the matched earthquake records are assessed and the mean responses are evaluated. Column axial load and moment are also assessed same way. These checks are performed for all load cases and the governing combinations are found and the maxim responses are checked. The maximum top and bottom moment at any storey and maximum shear are found as -307 kN-m, 220 kN-m and 157.5 kN, respectively for the beam. Table 4.3 shows the responses at each storey of beams. Column axial force and moment are maximum at the bottom storey, which are found 2279 kN and 467 kN-m, respectively. Investigating the beam using the excel tool and column with spColumn, the sections and the reinforcement are found inadequate for the responses from nonlinear analyses as illustrated in Fig. 4.7 and Fig. 4.8, respectively.

Table 4.3 Code- designed beam responses from nonlinear analysis

	Left			Right		
Storey	V (kN)	M-top (kN-m)	M-bot (kN-m)	V (kN)	M-top (kN-m)	M-bot (kN-m)
8	-82.9	-128.9	68.7	82.2	-127.4	67.4
7	-132.4	-192.4	98.0	132.3	-194.5	100.2
6	-149.3	-270.7	177.7	148.7	-261.6	168.7
5	-155.4	-297.4	205.4	155.4	-299.7	207.7
4	-156.8	-303.6	213.1	156.7	-304.9	214.6
3	-157.5	-307.1	218.7	157.4	-307.4	219.1
2	-157.1	-306.7	220.3	156.9	-306.7	220.6
1	-153.8	-296.5	213.9	153.4	-296.4	214.5

LEFT SUPPORT

β1	=	0.85		β1 = max. of: 0.85-(0.05*(f'c-28)/7) and 0.65
c	=	71.14	mm	Solve for "c" in: As*fy = 0.85*f'c*b1*c*b+A's*(c-d')/c*ec*Es
a	=	60.47	mm	a = β1*c
ρ	=	0.0099		ρ = As/(b*d)
$\rho_{(min)}$	=	0.0034		ρ(min) >= 0.25*SQRT(f'c)/fy >= 1.4/fy
ρb	=	0.0289		ρb = 0.85*β1*f'c/fy*(600/(600+fy)
ρg	=	0.0126		ρg = (As+A's)/(b*h)
$\rho_{(max)}$	=	N.A.		ρ(max) = As(max)/(b*d)
$As_{(min)}$	=	459.12	mm^2	As(min) = ρ(min)*b*d
$As_{(max)}$	=	3,110.7	mm^2	As(max) = (0.85*f'c*β1*c*b+A's*(c-d')/c*εc*Es)/fy for c = εc*d/(εc+0.005)
ε_c	=	0.003		εc = 0.003 (assumed concrete strain)
ε'_s	=	0.0010		ε's = εc*(c-d')/c < fy/Es, f's does not yield
f's	=	195.16	Mpa	f's = ε's*Es
As2	=	284.35	mm^2	As2 = A's*f's/fy for f's > 0
As1	=	1060.25	mm^2	As1 = As-As2 for f's > 0
εt	=	0.0158		εt = εc*(d-c)/c = (fy/Es+εc)/(ρ/ρb)-εc >= 0.005, Tension-controlled
ϕ	=	0.90		ϕ = 0.65+0.25*(εt-fy/Es)/(0.005-fy/Es) <= 0.90
ϕMn	=	205.98	kN-m	< Mu = 307 kN-m, Failed!

MIDSPAN

β1	=	0.85		β1 = max. of: 0.85-(0.05*(f'c-28)/7) and 0.65
c	=	46.78	mm	Solve for "c" in: As*fy = 0.85*f'c*b1*c*b+A's*(c-d')/c*ec*Es
a	=	39.76	mm	a = β1*c
ρ	=	0.0043		ρ = As/(b*d)
$\rho_{(min)}$	=	0.0034		ρ(min) >= 0.25*SQRT(f'c)/fy >= 1.4/fy
ρb	=	0.0289		ρb = 0.85*β1*f'c/fy*(600/(600+fy)
ρg	=	0.0100		ρg = (As+A's)/(b*h)
$\rho_{(max)}$	=	N.A.		ρ(max) = As(max)/(b*d)
$As_{(min)}$	=	474.44	mm^2	As(min) = ρ(min)*b*d
$As_{(max)}$	=	3,540.9	mm^2	As(max) = (0.85*f'c*β1*c*b+A's*(c-d')/c*εc*Es)/fy for c = εc*d/(εc+0.005)
ε_c	=	0.003		εc = 0.003 (assumed concrete strain)
ε'_s	=	-0.0002		ε's = εc*(c-d')/c < fy/Es, f's does not yiel
f's	=	-41.30	Mpa	f's = ε's*Es
As2	=	N.A.	mm^2	As2 = A's*f's/fy for f's > 0
As1	=	N.A.	mm^2	As1 = As-As2 for f's > 0
εt	=	0.0311		εt = εc*(d-c)/c = (fy/Es+εc)/(ρ/ρb)-εc >= 0.005, Tension-controlled
ϕ	=	0.90		ϕ = 0.65+0.25*(εt-fy/Es)/(0.005-fy/Es) <= 0.90
ϕMn	=	99.52	kN-m	< Mu = 110 kN-m, Failed!

Fig. 4.7. Inadequate beam for nonlinear earthquake responses

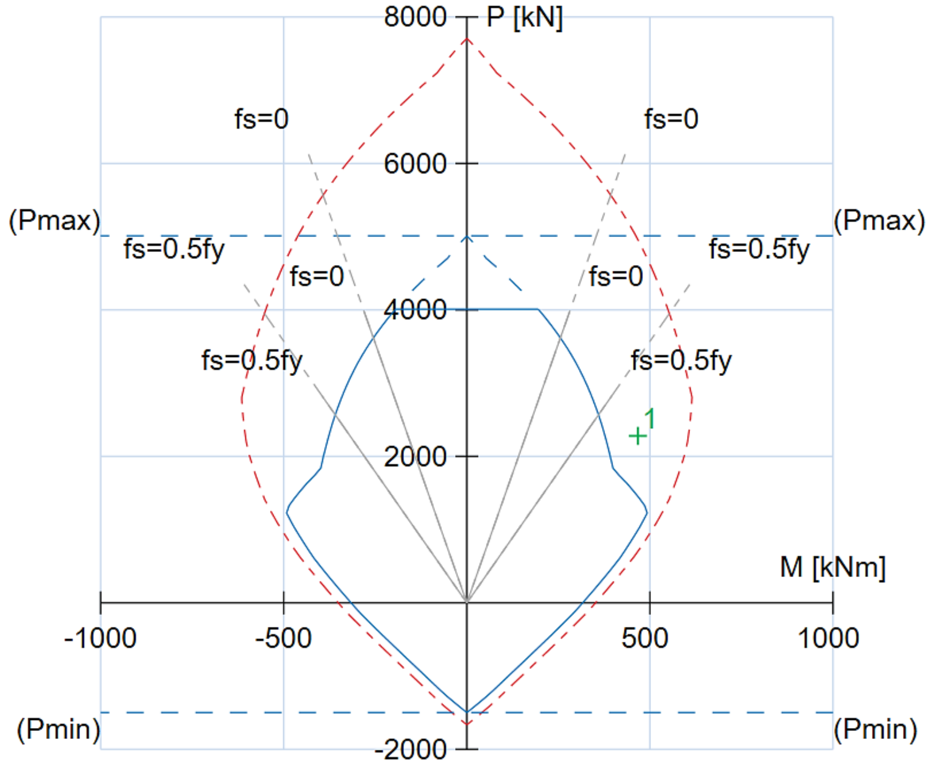


Fig. 4.8. Inadequate column for the nonlinear responses

4.5 ANALYSES FOR PBSD DESIGN APPROACH

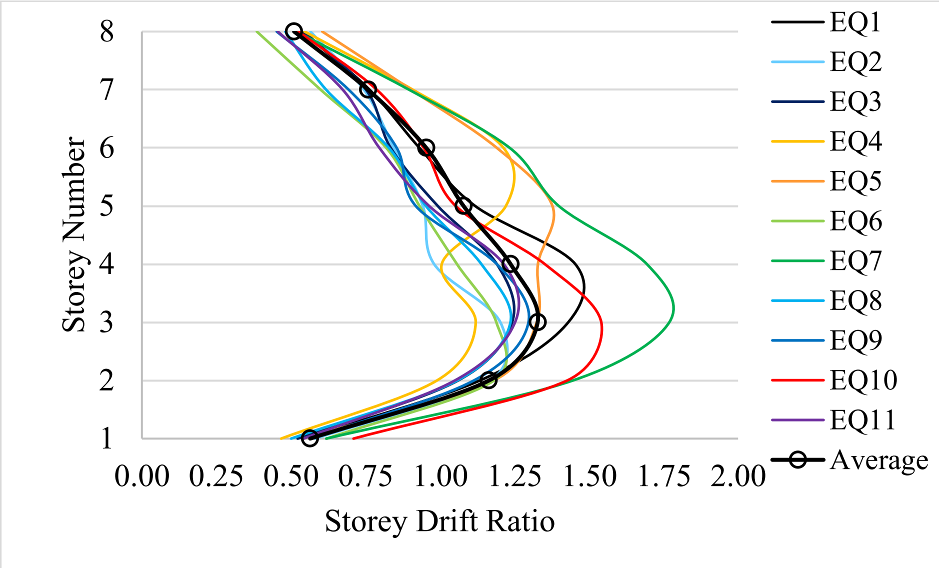
The building frame designed with BNBC 2020 procedure is found adequate for the storey drift requirement, but inadequate from the point of strength limit. The columns are found as inadequate section for the axial load and moment requirement for the governing load case. And beams are also found inadequate in moment carrying capacity. It seems that the moment is very low for the elastic analysis case and from the nonlinear analyses the averaged moment is very much greater than the capacity. Thus, the building frame is then analyzed with modified beam and column sections to satisfy the performance objective and strength requirements. Several trails have been performed with the modification of beam and columns sections and damage state performance limit and strength limit are assessed. After several trails, the sections have been finalized after meeting the strength objective mainly (and also satisfying the damage limit state objective). The frame with modified beam and column sections analyzed with DBE and MCE matched ground motions. Again, modelling is revised needed for nonlinear modelling and also time history analyses. Then, the frame is designed with the mean responses from DBE hazard-based analyses using PBSD approach. The finalized sections of beam and column are shown in Table 4.4.

Table 4.4 Final Column and Beam Section

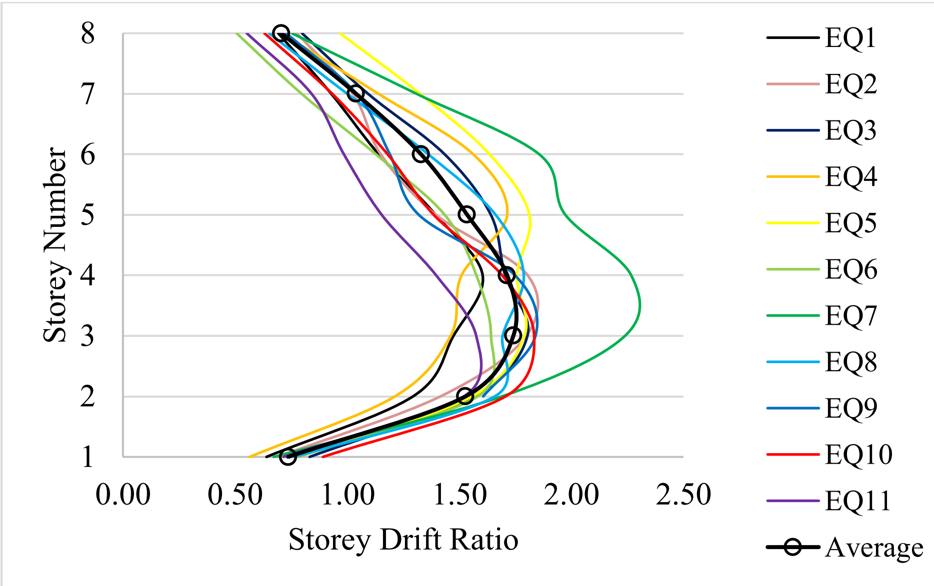
Beams		Columns	
Initial Section	Final Section	Initial Section	Final Section
305mm X 508mm (12″ X 20″)	381mm X 457mm (15″ x 18″)	508mm X 508mm (20″ x 20″)	508mm X 508mm (20″ x 20″)

4.5.1 Performance Check Using Storey Drift Ratio

The storey drift ratio (SDR) of the building with modified final sections under DBE and MCE hazard are represented in Figure 4.9 a and 4.9 b, respectively. The storey drift ratio is within the limit both MCE and DBE level earthquake ground motion intensity. For DBE and MCE based analyses, the SDR is 1.33 % and 1.74 %, respectively.



a) DBE



b) MCE

Fig. 4.9. Storey drift ratio from nonlinear analyses (final)

4.5.2 Final Frame Responses in Nonlinear Analyses

Moment at support and mid of beam, beam shear force, column axial force and moment for all the matched earthquake records are assessed and the mean responses are evaluated. From the governing load combination (combination 5 is found governing), response moment and shear for beams (bay 1) and moment and axial load for columns (storey 1, 2, and 3 of left column) are tabulated in Table 4.5 and 4.6, respectively.

Table 4.5 Beam forces from nonlinear analyses (final section)

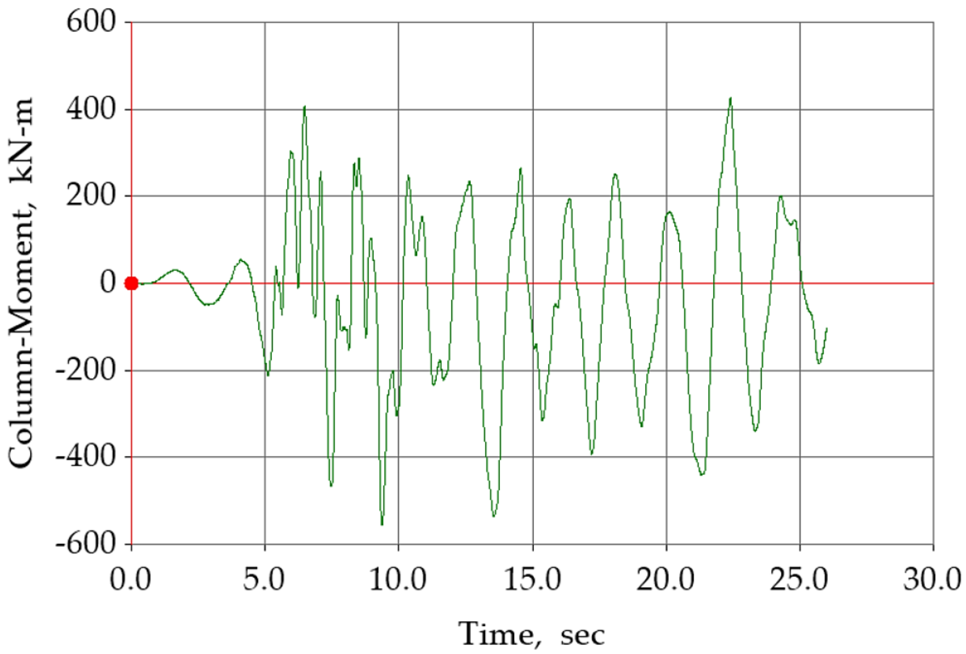
	Shear (kN)	Moment at Left Support (kN-m)		Moment at Mid (kN-m)	Moment at Right Support (kN-m)	
Story8	-85.4	-131.9	70.9	32.8	-113.4	70.4
Story7	-131.6	-201.2	106.1	65.1	-171.4	98.6
Story6	-146.3	-265.6	171.8	97.9	-232.5	158.7
Story5	-149.3	-278.0	185.1	108.5	-263.3	188.5
Story4	-150.0	-280.6	189.1	114.2	-269.1	192.9
Story3	-150.4	-283.2	193.7	115.8	-273.3	195.2
Story2	-149.8	-281.7	151.0	111.9	-275.1	195.0
Story1	-146.7	-274.7	146.3	108.8	-269.0	185.9

Table 4.6 Column forces from nonlinear analyses (final section)

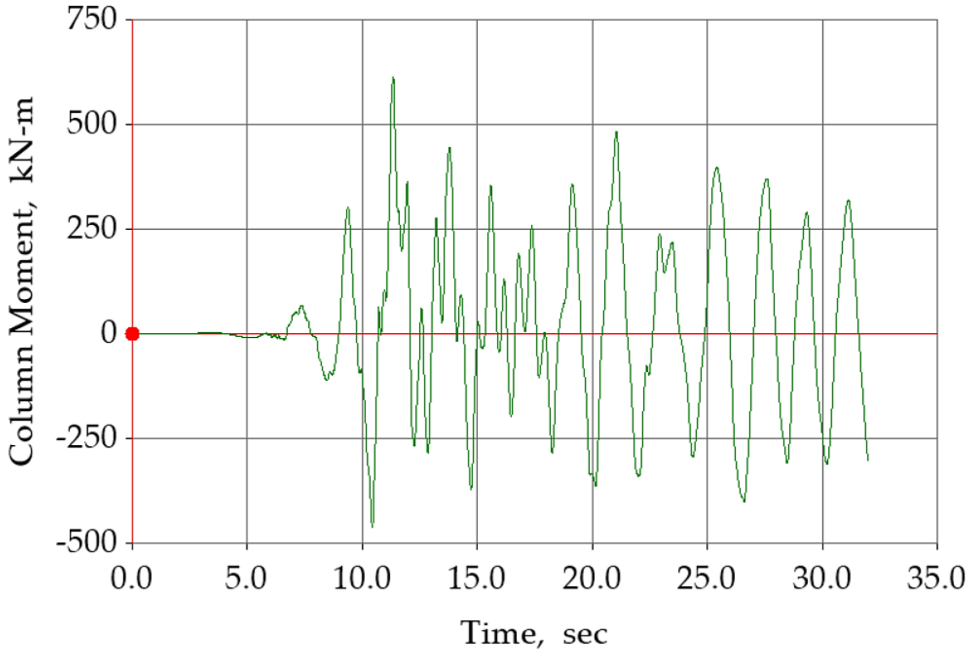
Load Combination	Storey 1		Storey 2		Storey 3	
	Axial Load (kN)	Moment (kN-m)	Axial Load (kN)	Moment (kN-m)	Axial Load (kN)	Moment (kN-m)
1.	-2123.1	0.7	-1844.3	-1.4	-1566.3	-1.4
2.	-2293.8	0.7	-1988.1	-1.6	-1683.1	-1.6
3.	-2142.7	0.7	-1862.0	-1.5	-1582.0	-1.5
5a.	-2297.3	-445.1	-1988.8	-331.9	-1681.7	-233.0
5b.	-2318.6	-443.5	-2008.1	-325.3	-1696.3	-227.6
7a.	-1227.0	-445.4	-1062.9	-331.1	-899.6	-232.2
7b.	-1248.4	-443.8	-1082.1	-324.5	-914.2	-226.8

4.6 NONLINEAR RESPONSES

In the current study nonlinear dynamic analyses are performed with eleven (11) code specified acceleration response spectrum matched earthquake ground motion records. For the nonlinear analyses, nonlinear concentrated M3 hinges and Fiber P-M2-M3 hinges are defined for beams and columns, respectively, as discussed earlier. Moment for an exterior C1 column due to DBE-matched EQ 7 and EQ 10 is presented in the Fig. 4.9.



a)



b)

Fig. 4.10. Column moment at storey 1 for DBE-matched- a) EQ7, b) EQ 10

In nonlinear analyses frame elements behave based on the defined nonlinearity and maximum members must go nonlinear state based on the expectation. Hinge response for a beam for DBE-matched and MCE-based EQ7 and EQ 10 are illustrated in Fig. 4.10 and Fig. 4.11, respectively. Hinge response for a column for DBE-matched and MCE-based EQ 7 and EQ 10 are illustrated in Fig. 4.12 and Fig. 4.13, respectively. These figures illustrate that the members are in the nonlinear states as expected for DBE-based and MCE-based earthquakes.

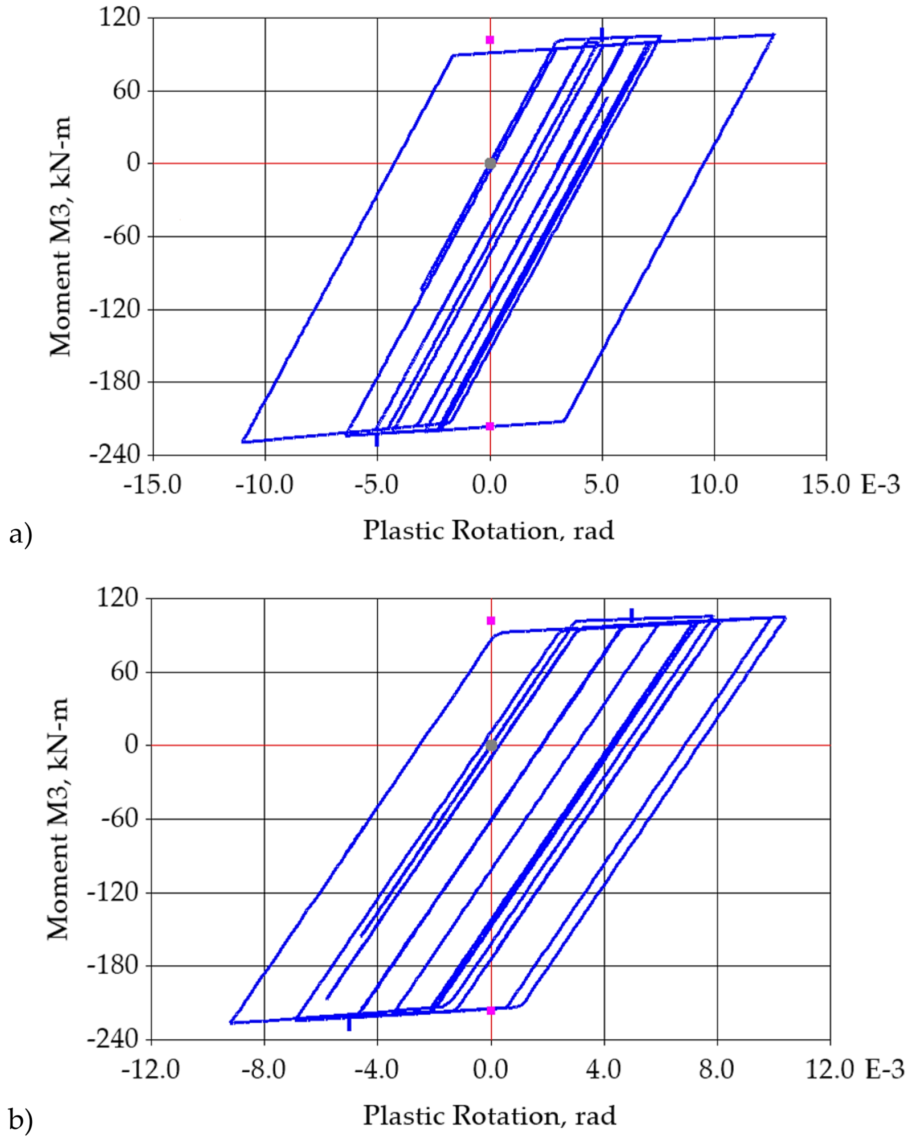
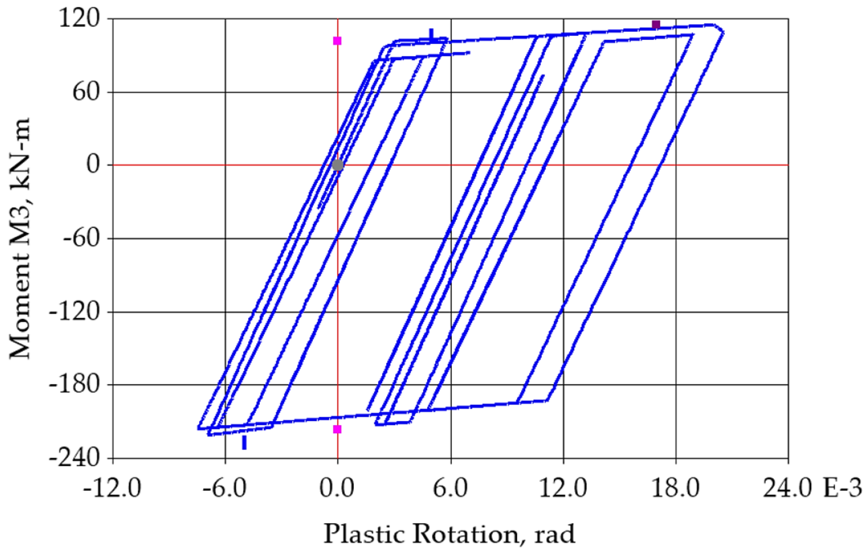
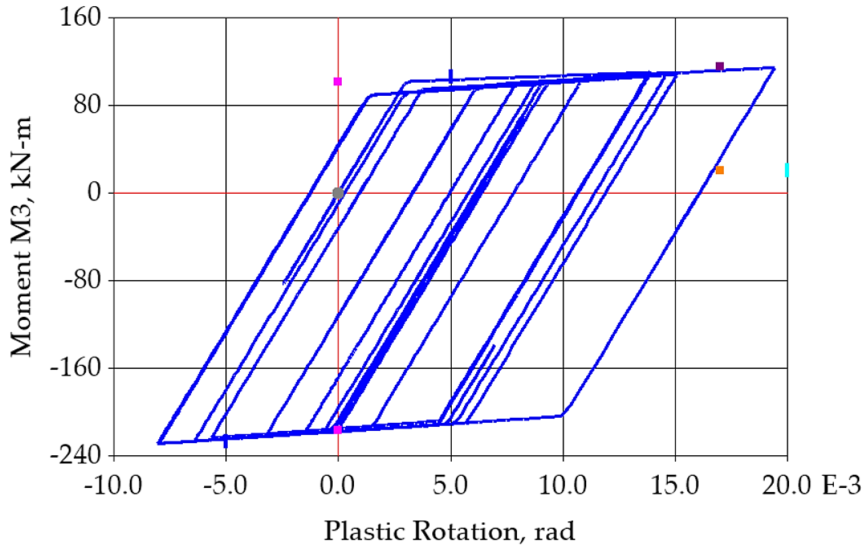


Fig. 4.11. Hinge response of a beam for DBE-matched- a) EQ7, b) EQ 10

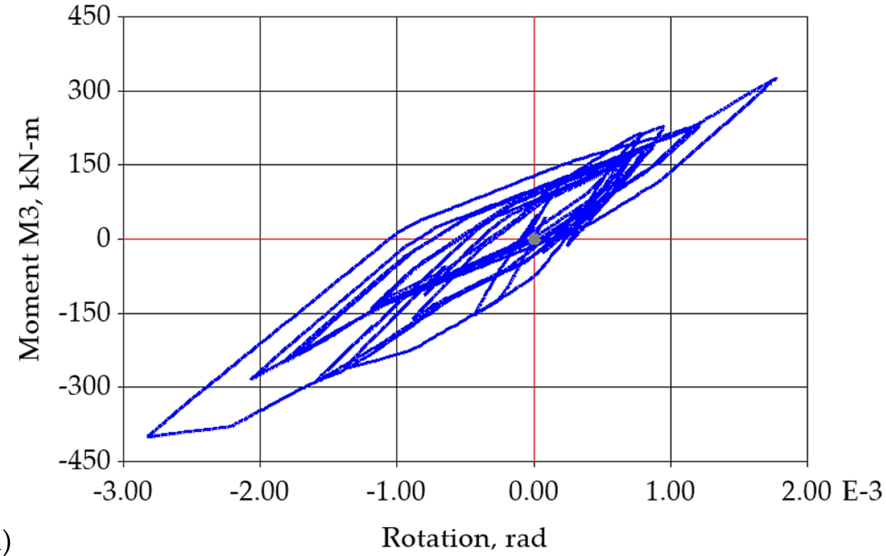


a)

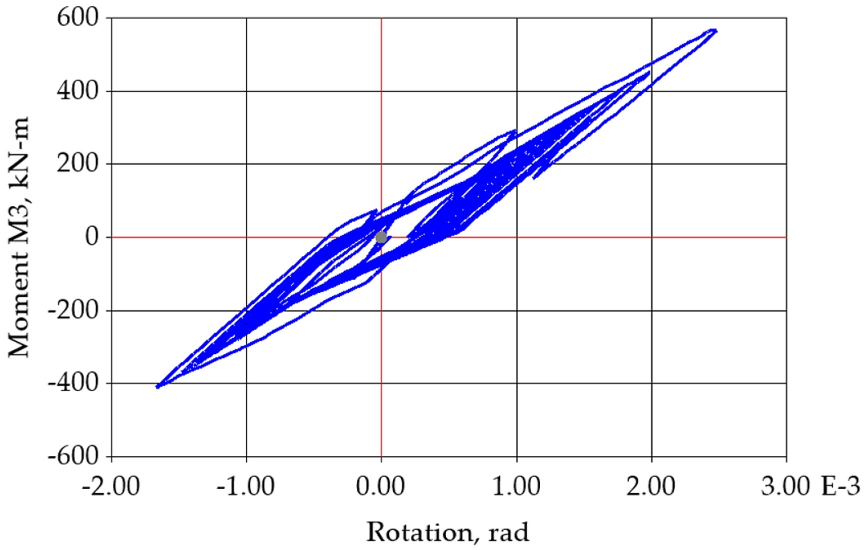


b)

Fig. 4.12. Hinge response of a beam for MCE- matched- a) EQ7, b) EQ 10

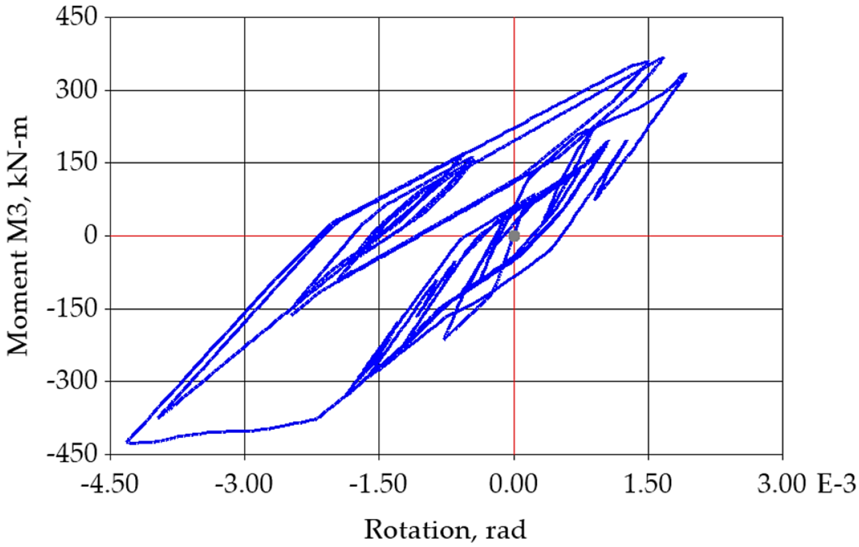


a)

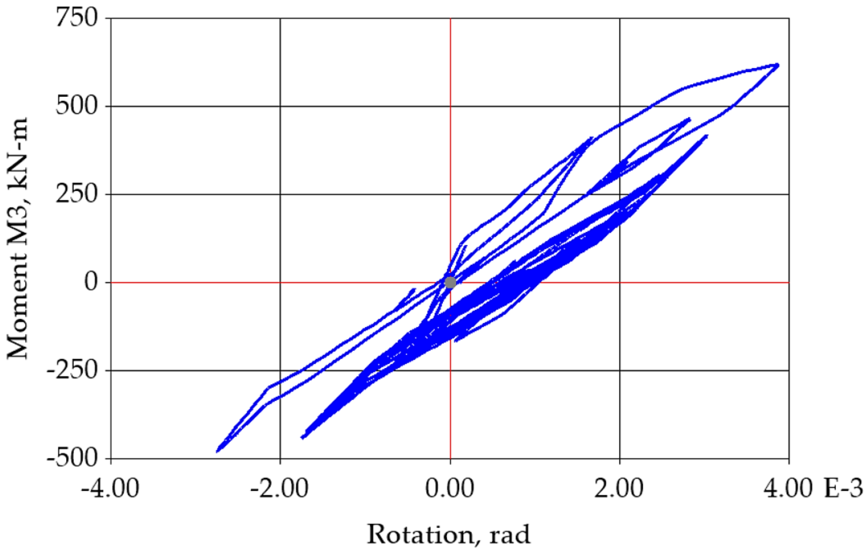


b)

Fig. 4.13. Hinge response of a column for DBE- matched- a) EQ7, b) EQ 10



a)

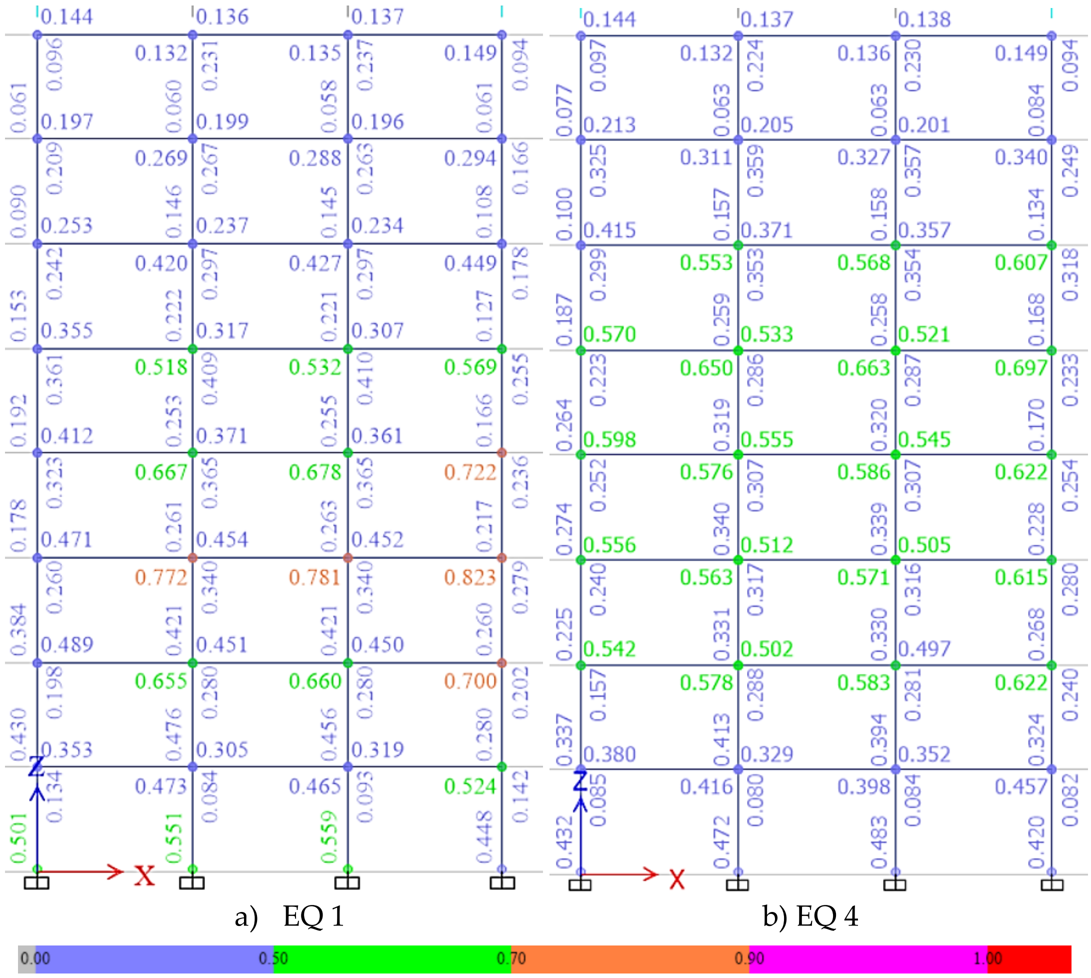


b)

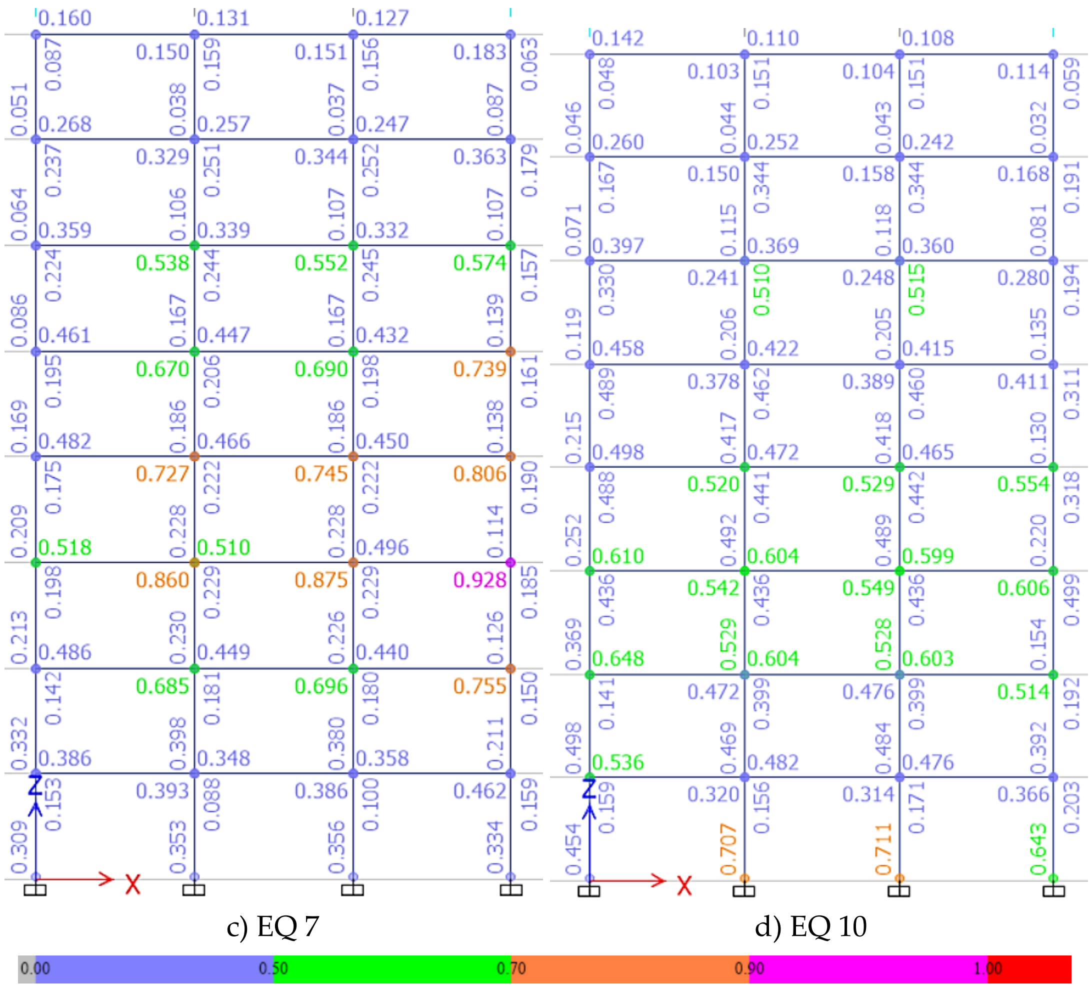
Fig. 4.14. Hinge response of a column for MCE-matched- a) EQ7, b) EQ 10

4.7 ELEMENT-BASED PERFORMANCE CHEK

From the results of nonlinear time history analyses with DBE-matched and MCE matched earthquake records, performance of the building can be checked with performance of frame elements at each storey for various performance levels. Fig. 4.14 shows life safety performance checks for the frame elements for DBE-matched EQ 1, EQ 4, EQ 7 and EQ-10. The figure shows that the building frame's performance is as expected for this level of earthquake. For MCE-matched earthquakes, performance of the frame should be like that it should cross the life safety performance level and should have good performance for collapse prevention level. From Fig. 4.15, it is clear that the frame has good performance for the collapse prevention performance level for MCE-matched EQ 1, EQ 4, EQ 7 and EQ-10.



a) EQ 1 b) EQ 4



c) EQ 7 d) EQ 10

Fig. 4.15. Life safety performance for DBE-matched earthquakes

The figures as represented in Fi. 4.14 shows that some elements are close or very close for life safety level of performance for DBE-based spectrum-matched earthquakes. Our objectives are also to have this level of performance of buildings for this level of earthquake hazard. So, the frame designed with performance-based approach to fulfill the specified performance objectives is quite satisfactory.

4.8 PERFORMANCE-BASED FINAL DESIGN

From the analysis of the building frame with final section of beam and column, the governing responses of beam and column are checked and their design are finalized. Here again, excel tool is used for beam design and spColumn is used for column design.

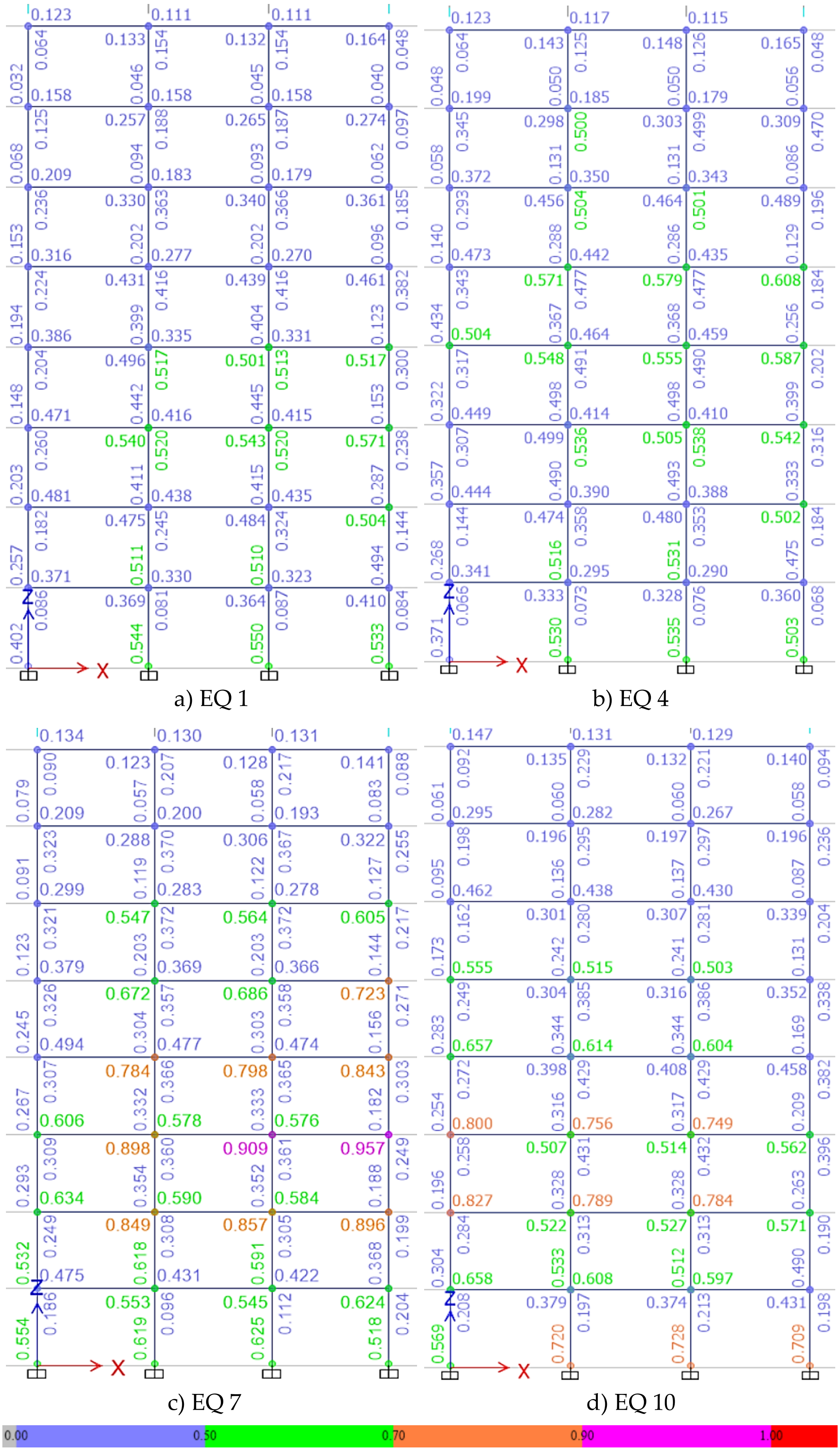


Fig. 4.16. Collapse prevention performance for MCE-matched earthquakes

4.8.1 Design of Beams

From the analysis of the building frame using nonlinear time history analysis, the maximum shear requirement, maximum moment at beam end, and maximum moment at mid of beams are found as illustrated in previous section. For the selected 381mm x 457mm (15"x18") beam section and the responses from the analyses, beam section is designed using excel. The required reinforcement and details are shown in Fig. 4.11. Hoops and stirrups remain same. The moment capacity is found 317 kN-m at end, 177 kN-m at mid, and shear capacity is found as 223.86 kN, which are found satisfactory.

LEFT SUPPORT

$\beta 1$	=	0.85		β1 = max. of: 0.85-(0.05*(f'c-28)/7) and 0.65
c	=	91.31	mm	Solve for "c" in: As*fy = 0.85*f'c*b1*c*b+A's*(c-d')/c*ec*Es
a	=	77.62	mm	a =β1*c
ρ	=	0.0172		ρ = As/(b*d)
$\rho_{(min)}$	=	0.0034		ρ(min) >= 0.25*SQRT(f'c)/fy >= 1.4/fy
ρb	=	0.0289		ρb = 0.85*β1*f'c/fy*(600/(600+fy)
ρg	=	0.0240		ρg = (As+A's)/(b*h)
$\rho_{(max)}$	=	N.A.		ρ(max) = As(max)/(b*d)
$As_{(min)}$	=	495.39	mm^2	As(min) =ρ(min)*b*d
$As_{(max)}$	=	4,081.0	mm^2	As(max) = (0.85*f'c*β1*c*b+A's*(c-d')/c*εc*Es)/fy for c = εc*d/(εc+0.005)
ε_c	=	0.003		εc = 0.003 (assumed concrete strain)
ε'_s	=	0.0010		ε's = εc*(c-d')/c < fy/Es, f's does not yield
f's	=	202.97	Mpa	f's = ε's*Es
As2	=	813.23	mm^2	As2 = A's*f's/fy for f's > 0
As1	=	1700.04	mm^2	As1 = As-As2 for f's > 0
εt	=	0.0096		εt = εc*(d-c)/c = (fy/Es+εc)/(ρ/ρb)-εc >= 0.005, Tension-controlled
ϕ	=	0.90		ϕ = 0.65+0.25*(εt-fy/Es)/(0.005-fy/Es) <= 0.90
ϕMn	=	317.17	kN-m	> Mu = 283 kN-m, O.K.

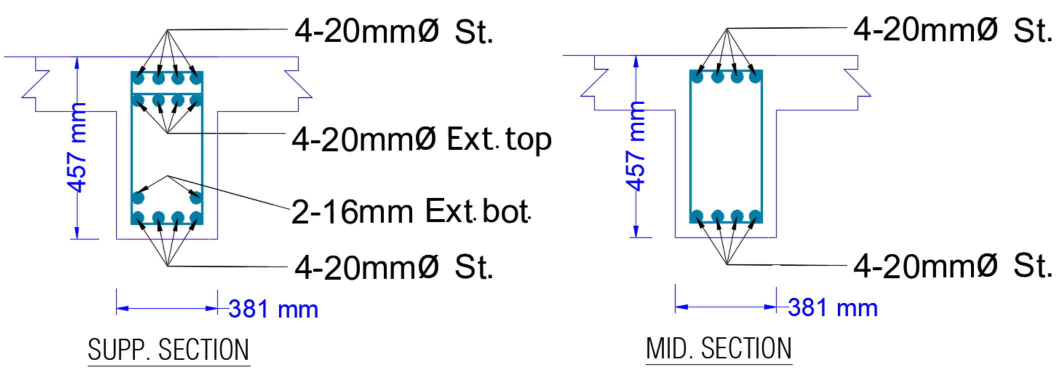
MIDSPAN

$\beta 1$	=	0.85		β1 = max. of: 0.85-(0.05*(f'c-28)/7) and 0.65
c	=	56.40	mm	Solve for "c" in: As*fy = 0.85*f'c*b1*c*b+A's*(c-d')/c*ec*Es
a	=	47.94	mm	a =β1*c
ρ	=	0.0081		ρ = As/(b*d)
$\rho_{(min)}$	=	0.0034		ρ(min) >= 0.25*SQRT(f'c)/fy >= 1.4/fy
ρb	=	0.0289		ρb = 0.85*β1*f'c/fy*(600/(600+fy)
ρg	=	0.0144		ρg = (As+A's)/(b*h)
$\rho_{(max)}$	=	N.A.		ρ(max) = As(max)/(b*d)
$As_{(min)}$	=	524.38	mm^2	As(min) =ρ(min)*b*d
$As_{(max)}$	=	4,066.1	mm^2	As(max) = (0.85*f'c*β1*c*b+A's*(c-d')/c*εc*Es)/fy for c = εc*d/(εc+0.005)
ε_c	=	0.003		εc = 0.003 (assumed concrete strain)
ε'_s	=	0.0003		ε's = εc*(c-d')/c < fy/Es, f's does not yielc
f's	=	68.08	Mpa	f's = ε's*Es
As2	=	206.63	mm^2	As2 = A's*f's/fy for f's > 0
As1	=	1050.01	mm^2	As1 = As-As2 for f's > 0
εt	=	0.0186		εt = εc*(d-c)/c = (fy/Es+εc)/(ρ/ρb)-εc >= 0.005, Tension-controlled
ϕ	=	0.90		ϕ = 0.65+0.25*(εt-fy/Es)/(0.005-fy/Es) <= 0.90
ϕMn	=	177.34	kN-m	> Mu = 116 kN-m, O.K.

```
SHEAR CAPACITY CHECKS
    @ 2H for Seismic Provision for transverse Reinforcements per Sec.21.5.3.2
    S(max)= smallest of ==>        |S(max) = d/4
                                   |S(max) = 8d_b (smallest longitudinal bar)
    S(max)      =  96.13    mm.o.c. |S(max) = 24d_b (diameter of hoop bar)
                   > 88mm, O.K.    |S(max) = 300mm

    For Shear Reinforcements
    φVc         =  98.83    kN      φVc = 0.17*λ*SQRT(f'c)bd
    φVs         =  125.02   kN      φVs = φ*fy*d*Av(stirrup)/s >= 0
    φVn         =  223.86   kN      >=Vu = 150.00 kN, O.K.
    φVs(max)    =  383.71   kN      >= Vu-(phi)Vc = 51.17 kN, O.K.
    Av(prov)    =  157.08   mm²
    Av(req'd)   =  95.33    mm²     <= Av(prov) = 157.08 mm^2, O.K.
    Av(min)     =  48.32    mm²     <= Av(prov) = 157.08 mm^2, O.K.
    S(max)      =  192.25   mm      >= s = 150 mm, O.K.
```

a) design check

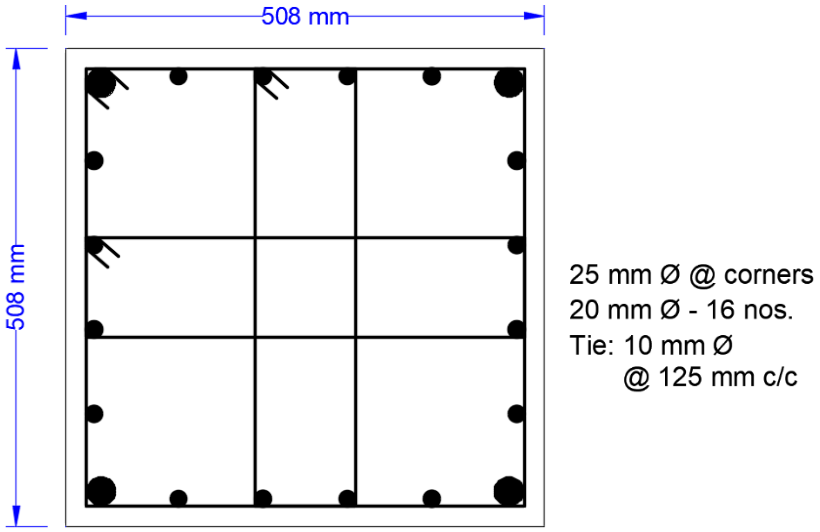


b) detailing of beam

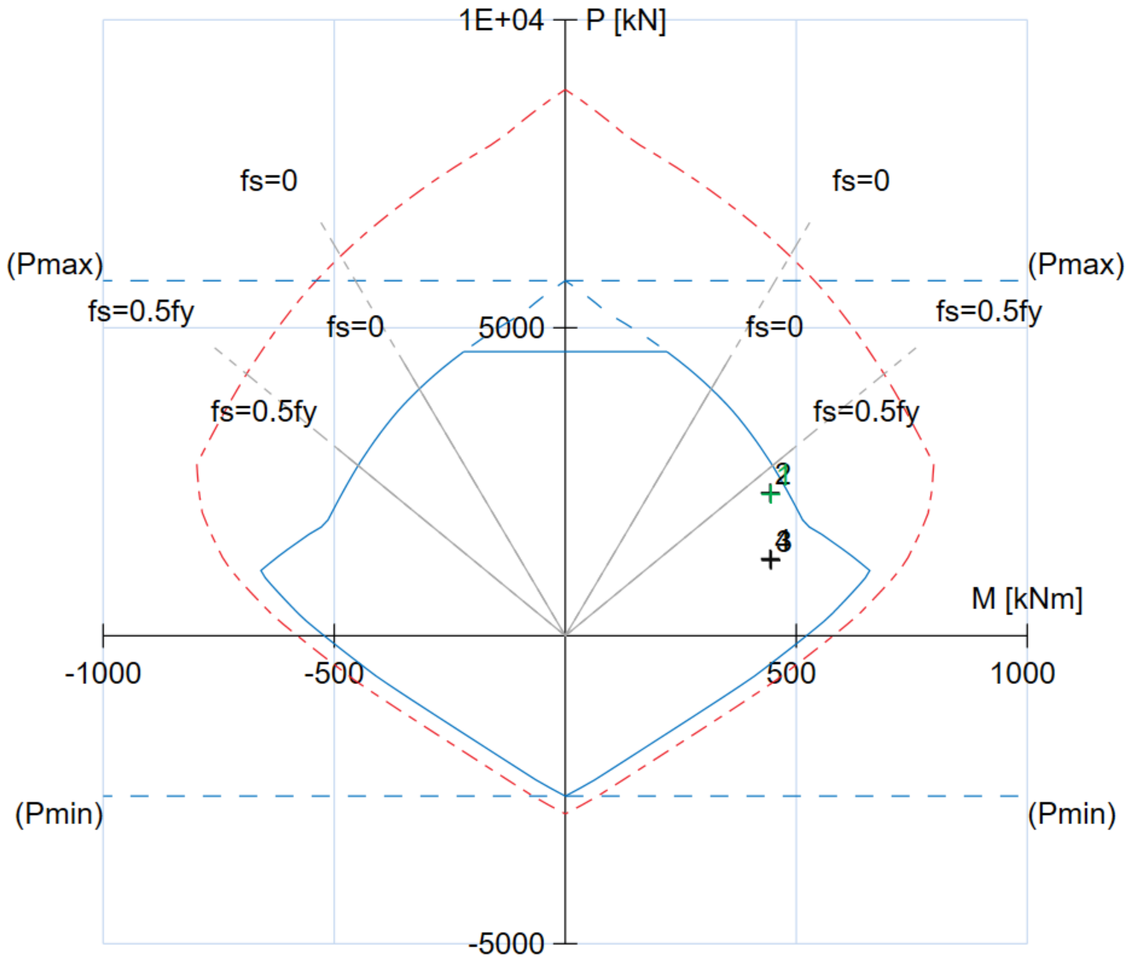
Fig. 4.17. Final design of beam using PBSD approach

4.8.2 Design of Columns

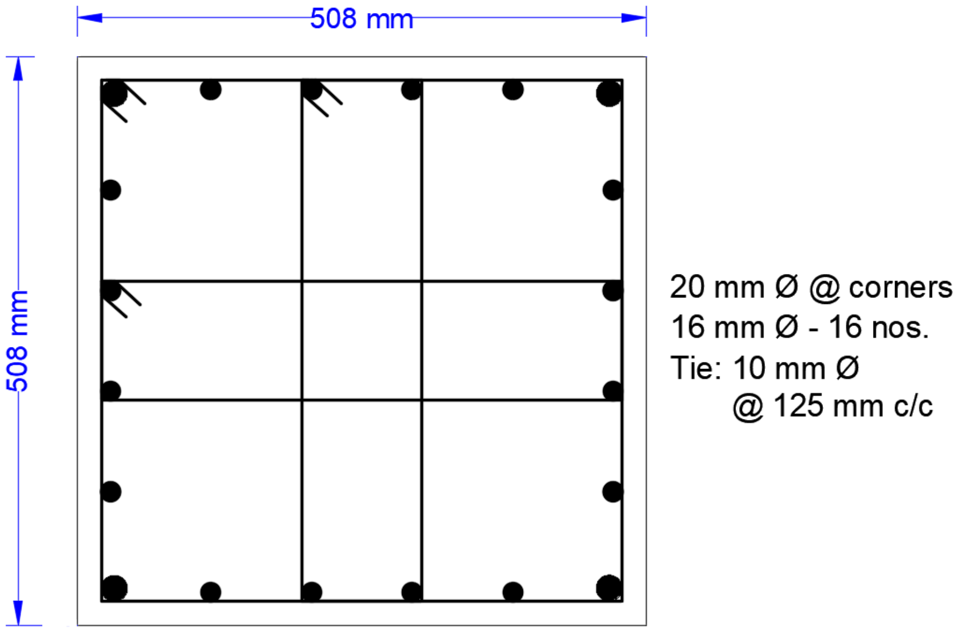
Columns are again designed with spColumn with the PM interaction diagram. The governing load combination is found with column axial load and moment are defined in spColumn. The column section is provided as 508 mm sq. section. Here, the bottom storey requires more reinforcement as the moment response is very high there. Column axial force and moment from the governing load cases for storey 1, 2 and 3 are 2262 kN, 450 kN-m; 1955 kN, 310.4 kN-m; 1651 kN, 232 kN-m; respectively. Fig. 4.12 a, b shows the column details and design check with the PM interaction diagram for the storey 1 and 2 and Fig. 4.12 c, d represent the same for the columns of upper storeys.



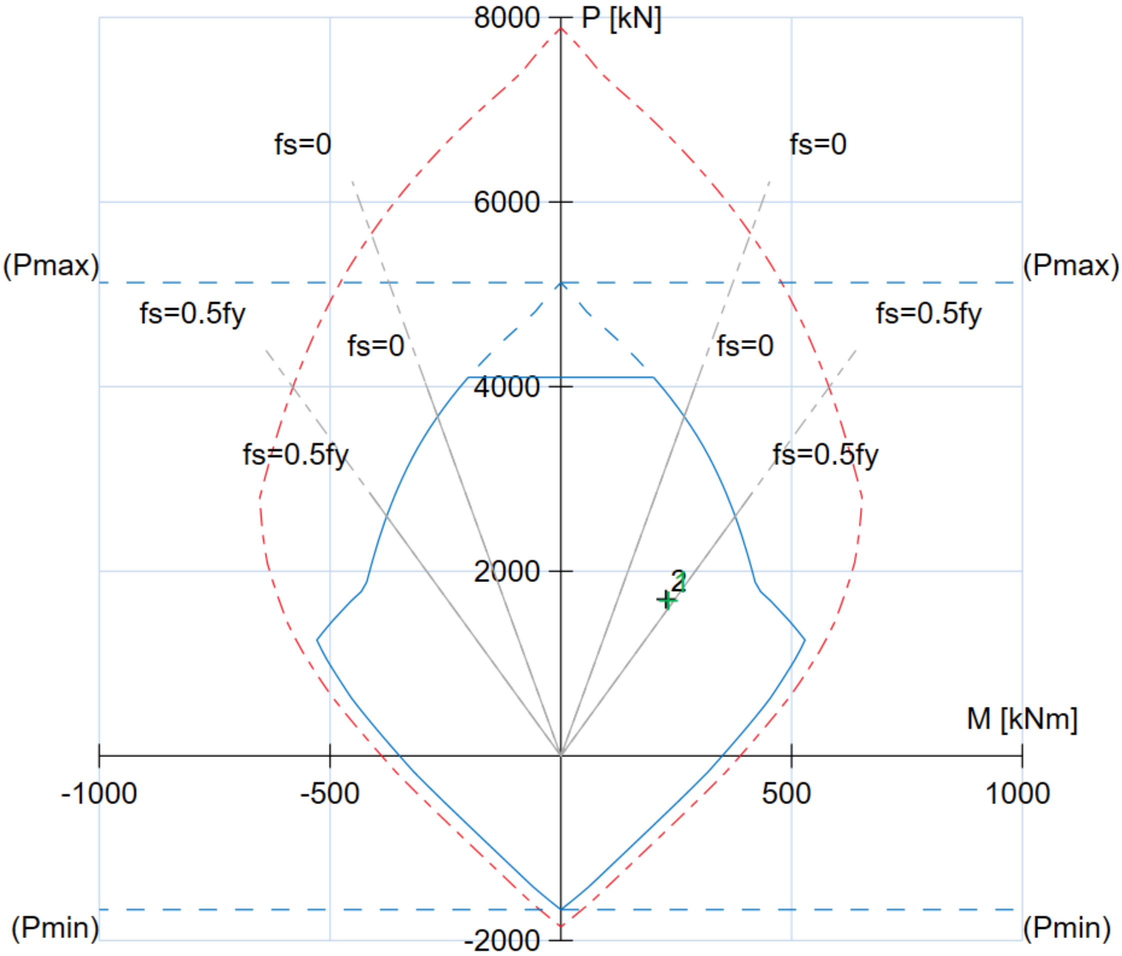
a) Column details for storey 1 and 2



b) Column interaction diagram check for storey 1 and 2



c) Column details for upper stories



d) column interaction diagram check for upper stories

Fig. 4.18. Column details designed with PBSD approach

4.9 SDR FROM LINEAR MODAL TIME HISTORY ANALYSES

After the final frame design using performance-based approach, the building frame is also analyzed under serviceability level earthquake (SLE) hazard. For this, linear modal time history analyses with the matched earthquake time history with 43-year return period spectra are used. 5% modal damping is considered here. From the analyses results, storey drift ratio is assessed which is shown in Fig. 4.13. The SDR for the average of the responses due to the earthquake loading matched with SLE hazard spectra are found within the SLE drift ratio limit (0.5%).

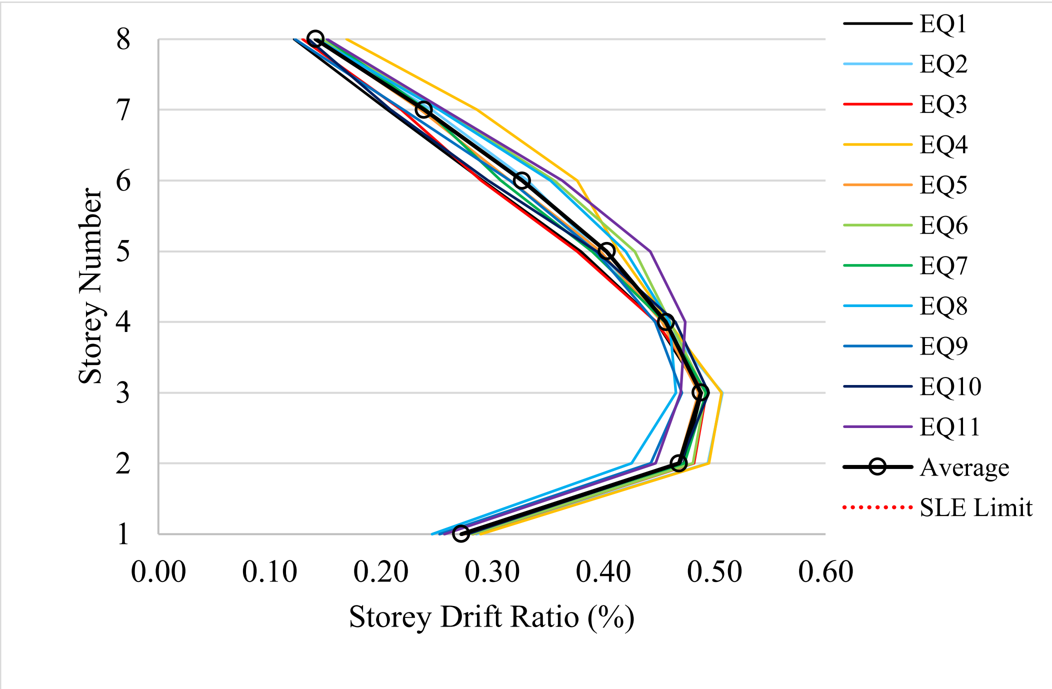


Fig. 4.19. Storey drift ratio from linear modal time history for SLE

4.10 PERFORMANCE ASSESSMENT SUMMARY

After the final design of the frame (beams and columns) and results from all the analyses using acceleration time history for SLE, DBE, and MCE hazard level, SDR is assessed and represented in previous section. The storey drift is averaged for the responses of storey drift ratio from the time history analyses using eleven (11) matched earthquake time history acceleration records for each hazard level (i.e., SLE, DBE, MCE). Here, in Table 4.7, SDR for selected performance objectives associated with various earthquake hazard levels is summarized.

Table 4.7 Summary of Performance Assessment from Final Analyses

Performance Objective	SLE		DBE		MCE	
	SDR	SDR Limit	SDR	SDR Limit	SDR	SDR Limit
BSO	0.49 %	0.5 %	1.33 %	2.0 %	1.74 %	3.0 %

4.11 EFFECTS OF SOIL STRUCTURE INTERACTION

Soil-structure interaction effects are now checked following approach discussed in previous chapter. The soil spring constants for the column are tabulated in Table 4.8. Effects on storey drift ratio (to check damage state performance level) and response modification i.e., beam and column moment, shear, axial load, etc. (for strength performance requirements) are checked. SSI effects on storey drift ratio is illustrated in Fig. 4.14. It is found from the analyses that the storey drift ratio is increased for maximum acceleration records. However, the SDR is yet within the limit of performance objectives. Fig. 4.15 represents the mean storey drift ratio with and without SSI for life safety performance level (associated with DBE hazard level).

Table 4.8 Soil Spring Constants for SSI

Soil Spring Constants	Edge Column	Interior Column
Translation along x-axis, Kx (kN/m)	184173.6	263105.1
Translation along y-axis, Ky (kN/m)	184173.6	263105.1
Translation along z-axis, Kz (kN/m)	219540.2	313628.9
Rocking about x-axis, Kxx (kN-m/rad)	11537195.7	16481708.1
Rocking about y-axis, Kyy (kN-m/rad)	11537195.7	16481708.1
Rocking about z-axis, Kzz (kN-m/rad)	17976393.0	25680561.5

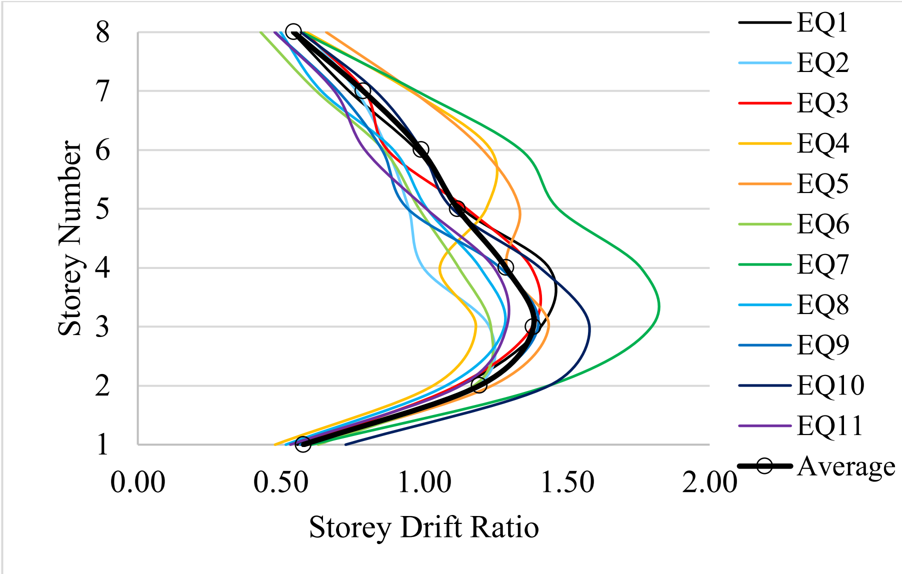


Fig. 4.20. SSI effect on SDR found by nonlinear time history analysis

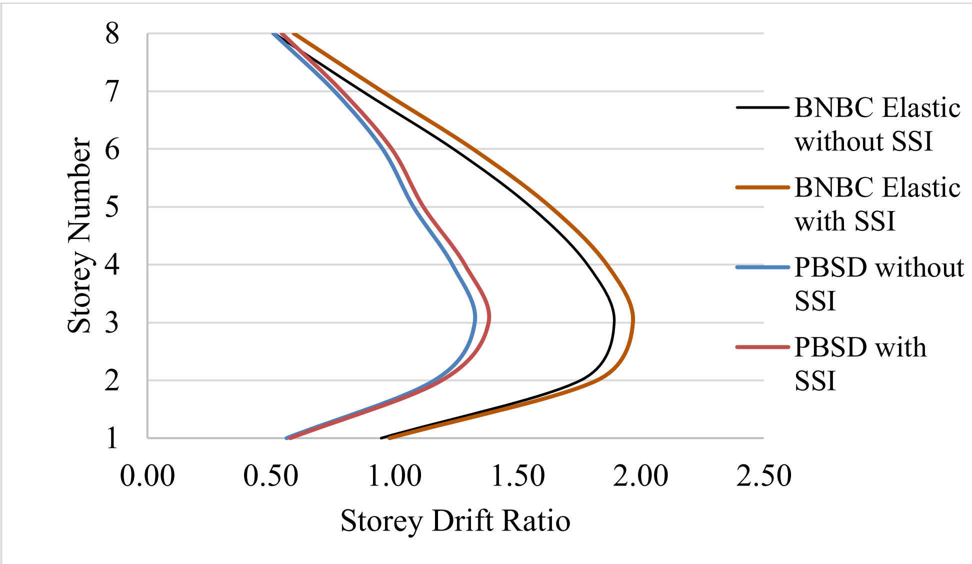


Fig. 4.21. Mean Storey Drift Ratio with and without SSI

Chapter 5: CONCLUSIONS

This chapter provides summary of the findings based on the outcome of Chapter 4. Moreover, the limitation and future research direction are also mentioned. Novelty and scope of the present research studies are also indicated.

5.1 GENERAL

In the present study an archetype 8-storey RC building frame is analyzed and designed using equivalent static procedure following the BNBC 2020. The building is then assessed for life safety and collapse prevention performance level through nonlinear time history analyses with eleven (11) ground motion time history records. The used time history records are matched with design acceleration response spectra both for DBE (475 years return period) and MCE (2475 years return period) hazard corresponding to life safety and collapse prevention performance level, respectively. After that, the building is again analyzed and designed using the procedure of performance-based seismic design approach. Element-by-element-based performance of the building frame are also checked both for life safety and collapse prevention performance level corresponding to DBE and MCE level hazards, respectively. Finally, the effect of soil-structure interaction on the performance level is assessed both for elastic analysis as per BNBC 2020 and the performance-based approach.

5.2 KEY FINDINGS

From the outcome of the present study the following key findings are illustrated:

- The code-designed building frame is found satisfactory from the storey drift performance point of view. But the columns and beams are found insufficient in terms of strength requirements. Specifically, larger beam width is required with increased reinforcement in beams and columns, while beam depth can be reduced. Thus, BNBC elastic linear static analysis may not be sufficient enough for damage state performance and/ or strength requirement fulfillment.

- Performance-based seismic design based on nonlinear time history analyses with sufficient no. of earthquake ground motion records can be suitable to fulfil the damage state performance and strength requirements of buildings. Both global level drift performance as well as element-based performance can be checked with the aid of performance-based approach. 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. Storey drift ration can be more for some earthquake records or less for some. However, the average effect on storey drift ratio is found negligible.

The present study will be helpful in new building design with satisfactory performance against damage and also strength. This will be also important in seismic restrengthening of old existing buildings with strength improvement and performance level enhancement.

5.3 RECOMMENDATION FOR FURTHER STUDY

Further study can be performed develop fragility curves for probabilistic damage state assessment. And also, some performance metric can be checked, e.g., downtime, casualties, and economic loss which will be more meaningful to decision-makers, owner, and other authorities. The present study is with frame from a regular structure. Future study can include 3D irregular frame building.

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