Dynamic Behavior of Code-Designed Existing Buildings with Non-Ductile Detailing in Pakistan

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THESIS ACCEPTANCE CERTIFICATE

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Copyright Statement

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"In the name of Allah, the most beneficent the most merciful"

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Abstract

Pakistan is a developing country with a extensive range of design and construction practices, which needs to evolve its own strategies for seismic hazard evaluation. The last two decade have pointed our shortcoming in design procedures of structures, specifically in resisting lateral forces. The October 2005 earthquake was one of the most catastrophic earthquakes in the history of the country causing immense loss of life and property. After this loss attention is now being given to the evaluation of the adequacy of strength in structures to resist strong ground motions. After 2005 earthquake Pakistan Building Code 1986 was revised and published in the year 2007. The main reason for the loss of life and property was inadequacy of knowledge of behavior of structures during ground motions. The vulnerability of the structures against seismic activity must be essentially studied.

Usual design Practice in Pakistan is to design buildings according to UBC-97/ BCP-07, which is based on Equivalent lateral force procedure and Response Spectrum Analysis. Linear Dynamic Analysis of buildings is very necessary for the true dynamic behavior of building as it utilizes actual ground motions record and consider the effects of higher modes. Finite element model of Six existing buildings was generated in ETABS and analyzed by Equivalent Lateral Force procedure, Response Spectrum Analysis and Linear Time History Analysis for same seismic parameters. Seismic responses i.e. Story Displacements, inter story Drifts Ratios, story shares and story moments from ELF and RSA were then compared with benchmark LTHA.

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List of Abbreviations

ASCE	American Society of Civil Engineers
BCP	Building Code of Pakistan
C.E.	Common Era or Christian Era
DBE	Design Bases Earthquake (10*% PE in 50 years)*
FMD	Frequency Magnitude Distribution
GSHAP	Global Seismic Hazard Assessment Program
IBC	International Building Code
ISC	International Seismological Center
MCE	Maximum Credible Earthquake (2*% PE in 50 years)*
NCB	North Collision Boundary
NESPAK	National Engineering Services, Pakistan
NGA	Next Generation Attenuation
NORSAR	Norwegian Seismic Array
PE	Probability of Exceedance
PGA	Peak Ground Acceleration
PMD	Pakistan Meteorological Department
PSHA	Probabilistic Seismic Hazard Analysis
QTZ	Quetta Transverse Zone
RP	Return Period
SA or Sa	Spectral acceleration
SLE	Service Level Earthquake *(69% PE in 50 years)*
UHS	Uniform Hazard Spectra
USGS	United States Geological Survey
WCB	West Collision Boundary

CHAPTER 1

INTRODUCTION

1.1 General

Natural disasters such as earthquakes have been a cause of losses of human life and economy since past. Pakistan is located in a region which is highly seismically active and has experienced many disastrous periods during historical times. In last 100 years it has experienced many disastrous earthquakes like Mach earthquake, M 7.3 (1931), Quetta earthquake, M 7.4 (1935), Makran cost earthquake of magnitude above 8 (1945), Pattan earthquake, M 6 (1974) and a recent disastrous event of Muzaffarabad earthquake, M 7.6 (October, 2005) which has shaken the entire nation in many ways and has enhanced the consciousness about the increasing vulnerability that a growing population is confronted with (Seismic Hazard Analysis and Zonation for Pakistan, Azad Jammu and Kashmir by Pakistan Metrological Department, July 2007). Seismic hazards also have a major impact on the earthquake resistant design of structures by justified estimate of hazard parameters like peak ground acceleration (PGA) or response spectrum amplitude at different natural periods.

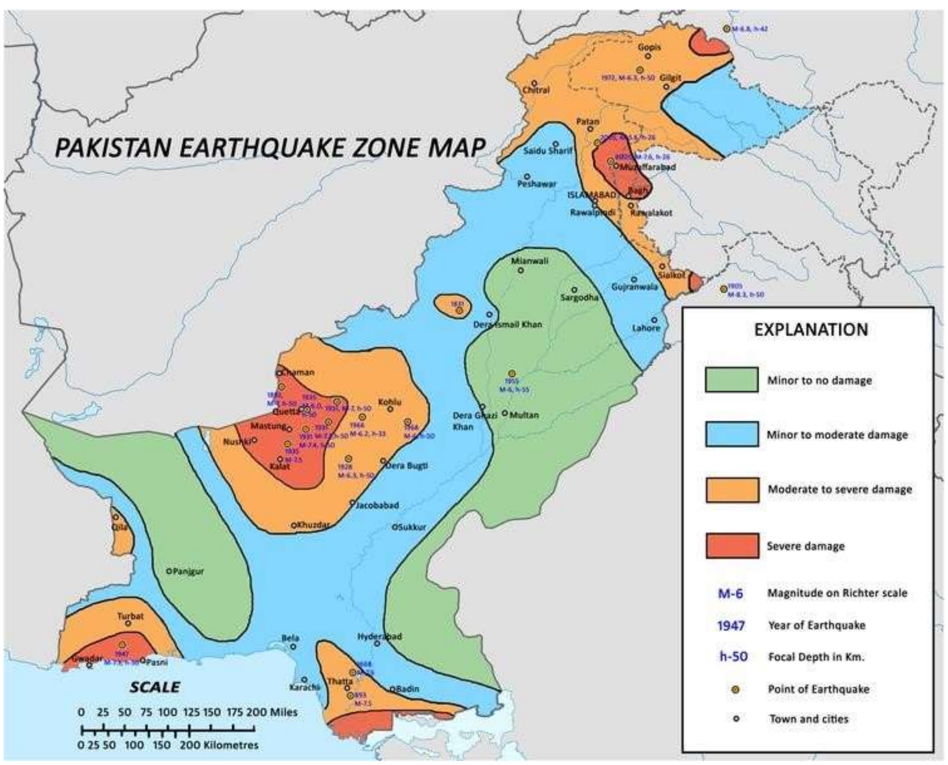


Figure 1-1: The standard seismic hazard map of Pakistan.

Pakistan Metrological Department has divided the country into 19 different zones based on the seismicity, the fault systems and the stress direction analysis. Pakistan is located on the north-west region of Indian plate which pushes into the Eurasian plate. After Muzaffarabad earthquake (October, 2005), collapse of Margalla tower showed the need of seismic design provisions for the design of earthquake resistant buildings in Pakistan to reduce the vulnerability of structures to minimize the both structural and non-structural damages. Building code of Pakistan (BCP 2007) revised after 2005 earthquake contains provisions for construction of new frame structures. For the design of buildings, BCP 2007 divides the country into 5 zones based on site specific seismic hazard levels (Peak ground acceleration PGA). Figure 1-2 describes that zoning

Table 1-1: Peak Ground Acceleration for Seismic Zones

Seismic Zone	Peak Horizontal Ground Acceleration
1	0.05 to 0.08g
2A	0.08 to 0.16g
2B	0.16 to 0.24g
3	0.24 to 0.32g
4	> 0.32g

Where "g" is the acceleration due to gravity.

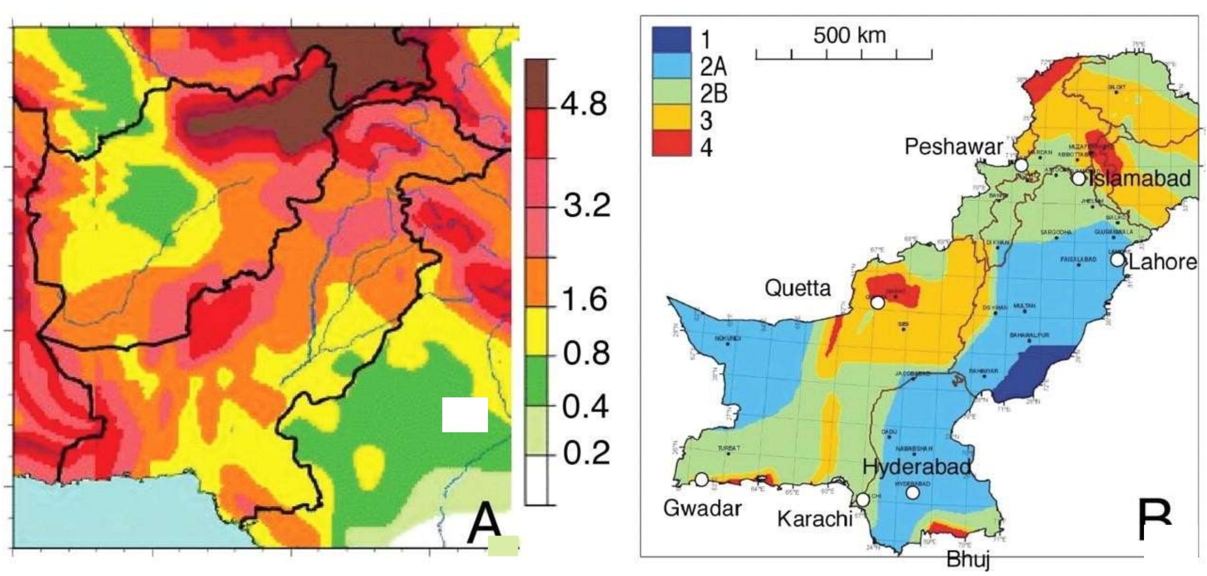


Figure 1-2: Seismic Zones of Pakistan (Bilham et al. 2008)

1.2 Research Motivation

Most of the RCC structures are constructed without major consideration towards their seismic resistance features. Moreover, recent earthquakes around the world proven the high seismic vulnerability of seismic deficient structures which were designed according to conventional code-based procedure and with nonductile detailing.

The Kashmir earthquake (also known as the South Asian earthquake or the Great Pakistan earthquake) of 2005 designates as a major earthquake (Magnitude 7.6) with its epicenter in the Pakistan-administered Kashmir at a depth of 19 Km (EERI 2006). The buildings, hospitals, schools, and rescue services were paralyzed. Approximately 138,000 peoples were injured and over 3.5 million rendered homeless (EERI 2006). According to Government figures, 19,000 children died in the earthquake, most of them in widespread collapses of school buildings (EERI 2006). The earthquake affected more than 500,000 families (EERI 2006). The motive of this research is to find most accurate seismic analysis procedure for economical earthquake resistance design.

1.3 Problem Statement

The safety of the non-engineered buildings from the fury of earthquakes is a subject of highest priority. In view of this fact, a huge stock of buildings with a non-ductile detailing are present in different moderate to severe seismic zones of Pakistan. Nowadays, the matter of concern for engineers is that many people are still living and working in such buildings, and during future earthquake many losses of lives could occur due to their collapse. Thus, there is a dire need to evaluate their seismic performance by using state-of-the-art dynamic analysis procedures.

The existing stock of designed RCC buildings in Pakistan are either based on Equivalent Lateral Force (ELF) Method or Response spectrum Analysis (RSA) method. Both said method do not consider the actual ground motion at the site. This research also aims to compare ELF and RSA method to Linear Time History Analysis and to compare how the design requirements vary with the change in analysis procedure. The linear time history analysis considers the actual ground motion of the site.

1.4 Overall and specific research objectives

The overall objective of the research program is to evaluate the seismic performance of code designed existing buildings with non-ductile detailing in Pakistan.

In this research work, the comparative evaluation of the accuracy of code procedures (equivalent lateral force method, linear response spectrum analysis) with the benchmark Linear Dynamic Analysis under the same seismic conditions is studied. Efforts were made to find most accurate procedure among ELF , RSA and LTHA for practicing structural engineer to ensure the most economical and safe design of structure.

This study compares the following structural behaviors for different analysis procedure:

- Base shear
- Story drift
- Story shear
- Story moment
- Inter story drift ratio

1.5 Research Methodology

Research methodology consists of following steps.

a. Six existing buildings were selected for this case study.
b. Finite Element model of selected buildings were generated in ETABS.
c. Each of these building was designed according to ELF and RSA.
d. The same buildings were then designed according to LDA
e. The seismic behavior of all these buildings were compared for different analysis procedures and the graphs were plotted for different structural behaviors.
f. The trends of then compared to one another and recommendations were mae for the most efficient design procedure.

1.6 Thesis Outline

Chapter 1 contains introduction to the thesis. A brief introduction to Equivalent Lateral Force method, Response spectrum Analysis and Linear time History Analysis is incorporated in Chapter 2. The research methodology which includes the design parameters and modelling procedures for the selected RCC buildings are explained in Chapter 3. The results of the research are explained and discussed in Chapter 4. Chapter 5 Includes conclusions and recommendations of this study.

CHAPTER 2

LITERATURE REVIEW

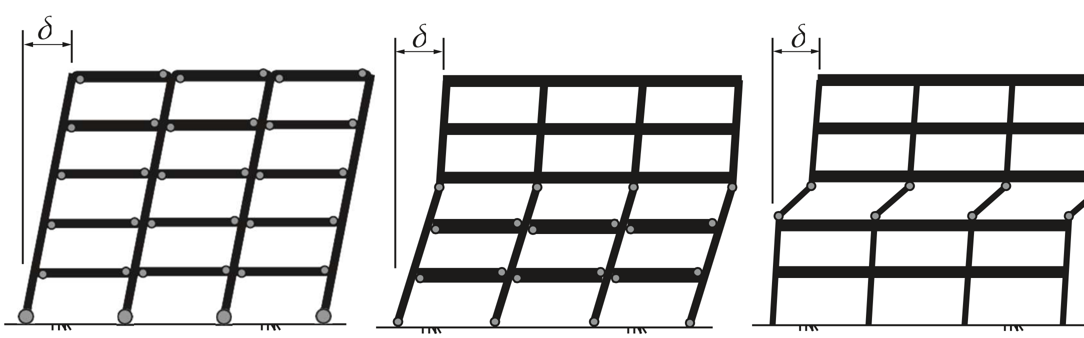
2.1 Background

Predicting the seismic damage potential of specific categories of old RC buildings, which have not been designed and constructed according to modern seismic provisions, is still a challenge for the earthquake engineering community. One of the first comprehensive attempts to quantify the expected damage potential for different intensity levels was made by Whitman et al. (1973), based on observed damage data from the 1971 San Fernando earthquake. This empirical method was used by introducing for the first time the concept of Mean Damage Ratio (MDR) as the mean ratio between repair and replacement cost which is still the most widely used economic damage indicator. Since then, various methods of vulnerability assessment have been developed (e.g. Mosalam et al. 1997, Lang 2002, Gardoni et al. 2003, Franchin et al. 2003, Crowley et al. 2004, Rossetto and Elnashai 2005, Erberik and Elnashai 2005, Erberik 2008, Celik and Ellingwood 2008) differing in level of detail and precision. In general most of these methods utilize static or dynamic analysis for the determination of the structural response and are referred to as analytical vulnerability assessment methods.

There is a fast insurgence of low to high-rise buildings around the globe. In the past few decades, there was a rapid growing ratio due to over growing population. Current structures are mostly designed on existing seismic codes at their construction time. These designs may not fulfill the different ongoing strict seismic requirements around the globe. At present, based upon current seismic regulations, different advanced techniques have been formulated to design and construct high-rise buildings. Design of these buildings on the concept *of RC core wall have been widely spread due to its extensive benefits. These structural systems get preference over the other existing sideways force resistive systems e.g., dual structural systems. The Uniform Building Code (UBC) classifies these structural systems as a building frame system. In high-rise structures, controlling structural deformation on account of the lateral load has been very challenging for designers. Different researchers have proved the effectiveness of RC core wall system to efficiently resist these lateral loads of extreme earthquakes and strong winds. The high-rise structures above thirty-five to forty stories generally depend exclusively on the core-wall structures. RC core-wall has proven to be a good structural system to design high rise buildings.

2.2 Seismic Performance of Reinforced Concrete Frame Structures

Modern RC structures are built with ductility in their main elements. Therefore, such RC structures are able to move back and forth during a seismic event, and to survive with acceptable damage, but without total collapse. Moment resisting RC frames are used as seismic force resisting system for design of earthquake-resistant structure. These members can be classified based on materials and geometry. Columns, beams and their joints are detailed with such amount of reinforcements that resist shear, flexural and axial actions. As a reaction the building sway back and forth multiple cycles in an event of earthquake ground shaking. During an earthquake as the building moves backward and forward, the damage is distributed over the height. If the structure holds weak columns, drift gravitate to focus on particular stories (Figure 2.1 (a)). Resultantly the drift may surpass the columns drift capacity. On the contrary, if columns provide firm support throughout the height of the building, drift will be distributed uniformly (Figure 2.1 (b)) thus minimize the chances of localized damage. Further, it is necessary to understand that the columns in a particular storey carries the entire building weight above those columns. On the other hand beams supports only gravity of that particular storey, therefore column failure is of more danger than beam failure. Due to this phenomena, building codes states that columns must be built stronger in frame as compare to beams. This principle is known as strong column/weak beam which is essential to accomplish safe behavior during seismic hazards. Studies have shown that the full structural mechanism of Figure 2.1 (c) can be only achieved if column to beam strength ratio is relatively large.



(a) Story Mechanism (b) Intermediate Mechanism (c) Beam Mechanism

Figure 2-1: The displacement mechanisms experienced by frame buildings under lateral shaking.

Inducing ductility requires that members yield in flexure and avoid shear failure. Shear failure in column is avoided because it leads to brittle failure and can result into loss of axial load carrying capacity. Using capacity design method it is decided which object within a structural system will be designed as ductile component and permitted to yield and which object as brittle component and remain elastic. Ductile components are designed with sufficient deformation capacity while brittle components are designed to achieve sufficient strength levels.

The following is the ACI definition for a beam-column connection in a monolithic reinforced concrete structure: “A beam-column joint is defined as that portion of the column within the depth of the deepest beam that frames into the column…. A connection is the joint plus the columns, beams, and slab adjacent to the joint. A transverse beam is one that frames into the joint in a direction perpendicular to that for which the joint shear is being considered (ACI 352).”

As previously mentioned current design code enforces all plastic hinges to form in the beam in order for the structure to absorb most of the seismic energy through inelastic deformation. Current design suggests that column hinges should be avoided because they result in a high ductility demand and can cause collapse of buildings. Many investigators have studied the effects of varying relative beam to column flexural strength ratios, Mr. For example, Ehsani and Wight 1985a, 1985b, Durrani et al. 1987, French and Moehle 1991, and Di Franco et al. 1995. After evaluating the results from these experiments, ACI 352 R-02 (2002) announced that the ratio of the sum of the flexural strengths of the column sections connecting to the joint divided by the sum of the flexural strengths of the beam sections connecting to the joint should not be less than 1.2. This prevention assures that plastic hinges occurs in the beam creating a “strong column weak beam” structural system.

2.3 Non-Ductile Detailing of RC Frame Structures

The importance of understanding the behavior of non-ductile RC structures beam-column connections has been especially crucial with the damage caused by recent earthquakes. When these types of buildings are subjected to seismic action it is observed that the most critical element of the structure is the beam-column joint. In recent decades a number of experimental and analytical studies have been done to better understand the behavior of beam–column joints. Most of the studies investigate the shear behavior of the joints, but additional data is necessary to accurately assess the behavior of lightly transverse reinforced joints subject to early column failure.

The relative beam to column flexural strength ratios for buildings built before the enforcement of current deign codes are not greater than 1.2 this creates a “weak column strong beam” structural subassemblies. After testing two beam-column joints of low column axial load, in 1974 Megget concluded that the reinforcing in the transverse beams adds little confinement to the connection region. Though, the reinforcement didn’t add much confinement the actual transverse beam had a great contribution to the joint confinement. This helped to strengthen the joint moving the plastic hinge away from the joint into the beam. The ACI 352-02(2002) suggests that when evaluating the beam’s flexural strength, the slab should also be considered. When a building is subjected to earthquake motion a portion of the slab flexural reinforcement interact with the beam’s reinforcement to take the load. Therefore, to acquire more realistic results from the research the slab should be included in the specimen.

Reinforce concrete structures designed before enforcement of current design codes are primarily intended to support gravity loads and they lack capacity for lateral loads. Due to a deficiency in the reinforcing detailing in the joint region and other members, these structures are commonly characterized as non-ductile.

The figure below shows typical non- ductile detailing of structural elements.

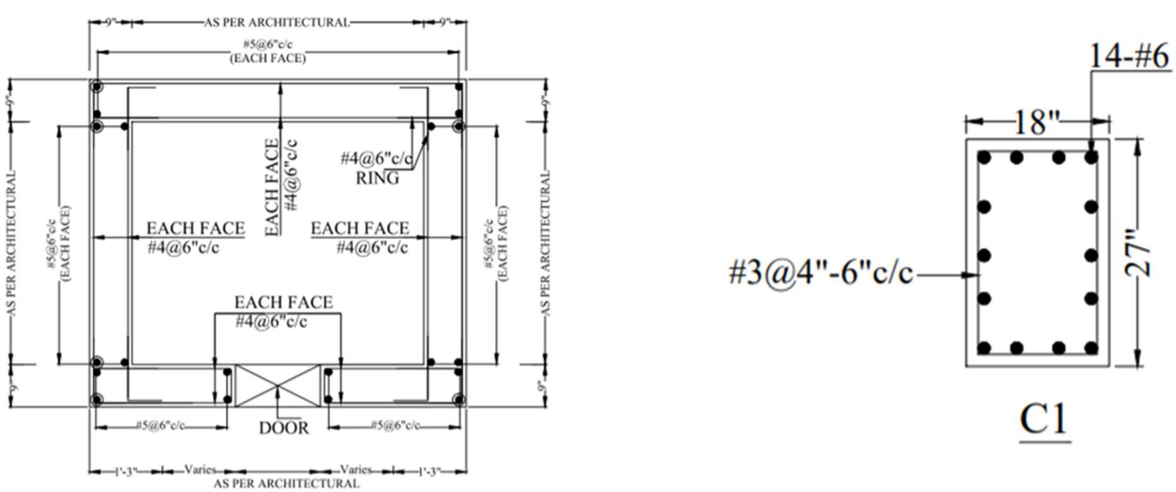


Figure 2-2: An example of non-ductile detailing in RC structures.

2.4 Equivalent Lateral Force Procedure (as Prescribed in BCP-07)

Equivalent Lateral Force (ELF) is the most frequently used method for analysis of structures. Buildings are modeled as a Single Degree of Freedom (SDF) system in linear static methods along with linear elastic stiffness and equivalent viscous damping and the input of seismic excitation is modeled by an equivalent lateral force.

For Zone 1 to Zone 3 base shear can be calculated using.

$$V = \frac{C_v\, I}{RT} W$$

The base shear shall not exceed.

$$V = \frac{2.5 C_a\, I}{R}\, W$$

The base shear shall not b less than.

$$V = 0.11\ C_a\ I\ W$$

In addition, for Seismic Zone 4, the total base shear shall also not be less than.

$$V = \frac{0.8\ Z\ N_v\ I}{R}\ W$$

Where:

C_a = seismic coefficient (as set forth in Table 5.16., BCP-07)

C_t = numerical coefficient (given in Section 5.30.2.2., BCP-07)

C_v = seismic coefficient (as set forth in Table 5.17, BCP-07)

I = importance factor (given in Table 5.10., BCP-07)

R = numerical coefficient representative of the inherent over strength and global ductility capacity of lateral force-resisting systems (as set forth in Table 5.13 or 5.15., BCP-07)

N_v = near-source factor used in the determination of C_v in Seismic Zone 4 related to both the proximity of the building or structure to known faults with magnitudes and slip rates (as set forth in Tables 5.19 and 5.20, BCP-07)

W= Total Weight of the Structure

$$T = C_t h_n^{\frac{3}{4}}$$

h_n= Height of Structure.

ELF procedures find their extensive use in most building codes for the seismic analysis and design. They are applicable only to regular buildings having predominant first mode of vibration i.e. responds in its fundamental lateral mode. Contrary to the linear static procedures, in linear dynamic procedures the buildings are modeled as a multi degree of freedom (MDF) system with linear elastic stiffness and equivalent viscous damping like static procedures. Seismic inputs are modeled as time–history analysis or modal spectral analysis.

UBC-97 permits design of a structure using equivalent static force procedure or a dynamic analysis for not more than 240 feet tall in case of regular structures and 65 feet tall in case of irregular structures. When the structure height exceeds the limit of 240 feet in case of regular structures, 65 feet in case of irregular structures and in case of buildings which are located on soil type-SF and having time period, T more than 0.7 seconds, dynamic response spectrum analysis is required. The equivalent static force procedure is most commonly used for the case of regular structures. For irregular structures dynamic analysis must be adopted.

2.5 Response Spectrum Analysis (RSA) for RC Building (BCP-07)

By progressing speedy refinements in seismic design regulations, numerous guiding principles and evaluation procedures to design RC buildings have been reproduced during the past decades. These guidelines not only provide the procedures for conventional code-based design but also gives the guidelines for the performance based seismic assessment of high-rise structures. Prominent reports for performance-based assessment but not limited to the mentioned studies have been published. These reports allow structures to be designed beyond elastic limit for economical design using either the DBE or MCE level. The flexural and plastic hinges were usually permitted to generate at the bottom of core wall for these strong earthquake levels. As per code provisions, remaining wall portions over the hinge region were predicted to behave elastic. The plastic rotation for these plastic hinges must be complying with the code requirements, as the development of plastic hinge necessarily be preferred to locate near the base area of the core wall.

The RSA process is considered an effective approach in the past decades to design taller RC core walls. To perform this process, the elastic behaviors of different vibrational modes is decreased by a response modification coefficient R to estimate the anticipated design level response for each mode. Usually the design demands are decreased by a same R coefficient for each mode. Numerous investigators have illustrated that the development of plastic hinge at cantilever wall bottom essentially decreases seismic response of the first mode, whereas greater vibration modes were not linked to decrease the identical amount as in the first mode. Hence, the RSA process has not been believed to be an effective method to design cantilever RC walls having plastic hinge at the wall base. New researches on the sixty-storied and the forty-storied RC core wall structures in highly active seismic regions also investigated that the RSA gives significant under estimation of seismic response across the full elevation of core wall for both the DBE and MCE levels

As per BCP-07 following guidelines have been established:

- Response spectrum analysis: An elastic dynamic analysis of a structure utilizing the peak dynamic response of all modes having a significant contribution to total structural response. Peak modal responses are calculated using the ordinates of the appropriate response spectrum curve which correspond to the modal periods. Maximum modal contributions are combined in a statistical manner to obtain an approximate total structural response.
- Response spectrum representation and interpretation of results are as follows:

  The ground motion representation shall as a minimum be one having a 10-percent probability of being exceeded in 50 years shall not be reduced by the quantity R and may be one of the following:

  1. An elastic design response spectrum constructed in accordance with Figure 5.1, using the values of C_a and C_v consistent with the specific site. The design acceleration ordinates shall be multiplied by the acceleration of gravity, 9.815 m/sec^2 (386.4 in/sec^2).

  2. A site-specific elastic design response spectrum based on the geologic, tectonic, seismologic and soil characteristics associated with the specific site. The spectrum shall be developed for a damping ratio of 0.05, unless a different value is shown to be consistent with the anticipated structural behavior at the intensity of shaking established for the site.

  3. Ground motion time histories developed for the specific site shall be representative of actual earthquake motions.

  4. The vertical component of ground motion may be defined by scaling corresponding horizontal accelerations by a factor of two-thirds. Alternative factors may be used when substantiated by site specific data. Where the Near Source Factor, Na, is greater than 1.0, site-specific vertical response spectra shall be used in lieu of the factor of two-thirds.

- All significant modes be included may be satisfied by demonstrating that for the modes considered, at least 90 percent of the participating mass of the structure is included in the calculation of response for each principal horizontal direction.
- The peak member forces, displacements, storey forces, storey shears and base reactions for each mode shall be combined by recognized methods. When three-dimensional models are used for analysis, modal interaction effects shall be considered when combining modal maxima.
- Elastic Response Parameters may be reduced for purposes of design in accordance with the following items, with the limitation that in no case shall the Elastic Response Parameters be reduced such that the corresponding design base shear is less than the Elastic Response Base Shear divided by the value of R.
- The analysis shall account for torsional effects, including accidental torsional effects. Where three-dimensional models are used for analysis, effects of accidental torsion shall be accounted for by appropriate adjustments in the model such as adjustment of mass locations, or by equivalent static procedures.
- Where the lateral forces are resisted by a dual system, the combined system shall be capable of resisting the base shear determined in previous steps.

2.6 Linear Time History Analysis (LTHA) of RC Buildings (BCP-07)

The LTHA has been one and only extensively recognized and precise process at the times for seismic assessment of high-rise structures. The design regulations permit LTHA procedure for design of RC core wall structure systems and also provide the modeling requirements for performance assessments of their discrete elements including walls, coupling beams, slab-column connections etc. The LTHA process requires an extensive level of practice to get the real non-linear seismic demands. The frequent LTHA investigations have been done for performance evaluation of various high-rise RC core wall structures against different seismic hazards and were not limited to these prominent investigations. The LTHA has also been proven to be utmost rigorous, time taking but the most accurate technique for seismic assessment of structures. The lengthy and time taking computation process of LTHA replicate the real performance of structures against the application of site-specific ground motions. The real recorded ground motions are collected from different earthquake databases and require prior modification to use in LTHA. The modification of site-specific time histories to match with the target spectrum calls for an evaluation of existing scaling and spectral matching practices. By the growing research on LTHA, different techniques have been established for modification of real recorded ground motion histories. A summarized overview of these developments has given in the succeeding paragraph.

As per BCP-07 following guidelines have been established:

- Time-history analysis shall be performed with pairs of appropriate horizontal ground-motion time history components that shall be selected and scaled from not less than three recorded events. Appropriate time histories shall have magnitudes, fault distances and source mechanisms that are consistent with those that control the design-basis earthquake (or maximum capable earthquake). Where three appropriate recorded ground-motion time-history pairs are not available, appropriate simulated ground-motion time-history pairs may be used to make up the total number required. For each pair of horizontal ground motion components, the square root of the sum of the squares (SRSS) of the 5 percent-damped site-specific spectrum of the scaled horizontal components shall be constructed. The motions shall be scaled such that the average value of the SRSS spectra does not fall below 1.4 times the 5 percent-damped spectrum of the design-basis earthquake for periods from 0.2T second to 1.5T seconds. Each pair of time histories shall be applied simultaneously to the model considering torsional effects. The parameter of interest shall be calculated for each time history analysis. If three time histories analyses are performed, then the maximum response of the parameter of interest shall be used for design. If seven or more time-history analyses are performed, then the average value of the response parameter of interest may be used for design.

2.7 An Overview of Existing Ground Motion Modification Methods

In order to use real records for performance evaluation of diverse structures, lots of ground motion modification methods have been investigated in the past few decades. In order to apply suitable method for the structure of interest, these methods have been further refined by different investigators. The findings of these investigations varied significantly from one investigator to another in a few cases. All of these methods are mainly divided into two principal groups. The first type is named as amplitude or magnitude scaling and the second one is termed as Spectral Matching (SM). The amplitude scaling is used as a single factor multiplier to linearly scale ground motion records with the target spectrum. The frequency content of scaled records does not change in this method. While in the SM method, the time histories were modified over the range of interest of time period to match with the target spectrum. The frequency component as well as the amplitude of the ground motion observe alteration in this approach. These components include accelerogram velocity, displacement and frequency contents. It was investigated that the standard deviation may be reduced up to a factor of 2 by using spectral matching technique instead of linear scaling. In another study it was observed that the scaled records decreased the response inconsistency by 20% to 75%. On the other hand, spectrally matched records decreased the response inconsistency by 60% to 80%, which increased the accuracy of the median response with same or reduced number of ground motions. It was observed that the scaling procedure could convert records a little more aggressive than those in nature. A published study proposed a procedure to estimate bias in projected structural response due to amplitude scaling of ground motions. It was alleged that earlier investigations may not have distinguished the scaling bias when records were scaled to match target spectrum at Sa (T_1) scaling.

The ground motions scaling was observed to have the unbiased median max inter-story drift ratios. In a study the selection parameter of seed ground motions for spectral matching were investigated. Spectral matching was described better ground motion modification technique over the scaling methods due to consideration of multiple vibration modes, which contribute remarkably towards seismic response of taller buildings. A Modal Push-over Based Scaling (MPS) technique was designed to scale records in order to implement LTHA for bridges and buildings. For the high-rise buildings of 19 as well as 52 story heights, bias (underestimation or overestimation) reached over 25% when related to the ASCE 7-05 scaling technique. It was observed that overestimation of bias by using ASCE scaling has increased with the increase of the building height. Another published study further applied MPS technique on steel high-rise revealed that the MPS technique was modified procedure over existing ASCE 7-05 procedure because of considering its higher mode effects and strength features of structure. A PEER report published in 2009 explored the effects of 16 out of 40 different scaling procedures with the goal of precisely estimating the median peak structure demand related to ground motions selection and modification. It was observed that when proper inelastic parameter or proper spectral shape were not considered in different scaling methods (e.g. ASCE SaT1 scaling, matching to a UHS), the peak inter story demand was consistently over predicted. A re- search gap was highlighted for an additional investigation to compare other EDPs like peak floor accelerations in the conclusions of this report. Another study investigated that the practicability of present fragility evaluations on the basis of scaled seismic ground-acceleration histories was uncertain, and scaling of ground motions need to be avoided.

An experiment was conducted to compare feasibility of amplitude scaling at fundamental structure period with the spectral matching. The results showed that spectral matching has greater stability in bias and dispersion of EDPs when com- pared with amplitude scaling. An investigation of four different ground motion modification procedures was done. These scaling methods were named as geomean scaling, spectral matching, first-mode-period scaling Sa (T_1) and spectral demand distribution scaling.

The first method of geomean scaling gave better preservation of uneven spectral plots of actual ground motions and little dispersion of EDPs. The second method of spectral matching showed underestimation of median displacement but the dispersions in the EDPs were smaller because the scattering of spectral peaks were eradicated through the matching process. The third method also produced greater dispersions as compared to geomean scaling for nonlinear systems. The last method named as distribution scaling, produced unbiased evaluations of median displacement responses. Conventionally, it estimated the scatterings in the displacement demand parameters. It was concluded that these all methods were investigated for first mode dominant structures with minute inelastic deformations. For higher mode dominant assemblies, these methods may be given a conservative EDPs and other methods needed to be investigated. The study was also performed to evaluate MPS for taller buildings using one component ground motions. The requirement of an additional step was proposed wherever seismic response was expected to occur as a result of higher vibration modes. ASCE 7-05 scaling was referred as fully deficient for predicting over estimation of EDPs. In another study precision of six different scaling methods for spectrum compatible records using soil structure interaction analysis was investigated.

It was found that choice of an appropriate scaling procedure for specific structural demand parameter vary from method to method and place to place. A further investigation was proposed by choosing diversified EDPs and scaling methods. The effects of spectrally matched ground motions were also investigated to assess consequence of bi-directional movements in plan-asymmetric systems. Spectral matching was performed by using seismo-matching software. The use of spectral matching was justified to be the best ground motion modification method for reducing number of required records. A consensus for practicality of spectral matching was developed. However, it was said to be still a conjecture as to what extent spectral matching is pragmatic.

A study was also conducted to reveal accurateness and effectiveness of spectral matching procedures. These values were compared to a benchmark and ASCE-7 scaling method. The use of spectrally matched records for LTHA was proven to be an accurate and precise method for high-rise buildings. It was claimed that at elastic modal periods of system, the spectral accelerations of ground motions are not essentially reliable ground motion intensity measures. Therefore, accurate number of spectrum-matched records were subjected to reduce the higher inelastic response.

A PEER report that was published in 2015 considered four to twenty story models to investigate competency of 14 ground motion selection and modification techniques. It was observed that the fundamental behavior does not change instantly for structures of other elevations. A peak inter story drift ratio was considered and other EDPs were supposed to be investigated in future. The use of two techniques for ground motion modifying were documented and the investigations of spectral matching method were intended to reproduce in the future. It was recommended to restrict the use of all these scaling methods up to 30 stories height and a further research gap was highlighted. Another experiment was performed by using matching sets of selected and modified records on the first mode, and one general matching set for spectral matching of records ground motions to evaluate the seis- mic demand of nonlinear and fundamental mode dominant systems to explicate the inconsistency in the intermediate structural response. It was disclosed that procedure of Spectral Matching was not mainly controlling the observed bias among Engineering demand parameters resulting from the two considered methods.

A study on the two approaches named weightage scaling (which was also named as amplitude scaling) and the spectral matching revealed that the existing consensus for choice between these two methods were still uncertain. Another article explained that the records selected outside the location of structures of interest needed to be matched with the target spectrum of that specific site using frequency domain or time domain spectral marching. The visual comparison of traces of acceleration, velocity, displacement, and possibly Arias Intensity were frequently used to assess spectrum-matched motions, before and after matching. Thus, a judgment is made whether the applied changes are significant or not after the adjustments were made.

Although there was a variety of researches on implementation and effectiveness of spectral matching methods. The Spectral matching was neither included in ASCE- 7-05 nor in ASCE-10. However, the choice between spectral matching and scaling method was allowed by other seismic regulating councils including PEER TBI (2009) and FEMA (2010). Among all, two of the spectral matching methods were considered utmost reliable at the time which were time domain and frequency domain spectral matching. In an investigation the FDSM and TDSM for seismic assessment of bridge structures were considered through two spectral matching software namely SYNTH for FDSM and RSPMATCH for TDSM. It was observed that both methods were capable of producing similar profiles for matched ground motions with minimum dispersions in seismic responses. The background and development of these procedures are described in the following sections.

2.8 Time Domain Spectral Matching

The Spectral matching in time domain was first adapted by Lilhanand and Tseng in 1988. They proposed an algorithm that modified the initial time histories by using reserve impulse wavelet function in a way that the targeted spectral becomes well-suited to a response spectral. This method has a fundamental assumption that adjustment of wavelet does not result in a change in peak response time. This assumption may not always be valid as the time of peak response may be shifted by addition of wavelet adjustments to acceleration time history. The time-domain ground motion Spectral matching does not change the character of a real ground motion, hence considered an excellent method of spectral matching. Spectral matching technique was described by highlighting the time domain approach. It permitted to use the real recordings from active regions, and was also eye-catching in the CEUS, although CEUS conditions were matched by enabling high frequency. There was no major issue in addition of high frequency motions into record because these were usually stochastic.

A data of CEUS ground motion record was developed by using this process in NUREG/CR 6728 report. A vital phase in evaluating the spectral-matched record is the comparison of initial and final history of displacement, acceleration and velocity, ensuring that they rationally represent the original time series (i.e. indicating the changes which were acceptable physically without unintentional time- domain characters). Perhaps, this was the most significant stage of spectral matching. The time-domain spectral matching algorithm comprises of repeated addition of sets of compact arrangements of wavelets (i.e. discrete length sinusoid-like functions) to acceleration histories.

The algorithm developed by N. A. Abrahamson in 1993 was modified for application to preserve mobile parts of initial ground motion at longer periods. It was applied in RSP-Match software with the modified cosine wavelet base, pre- serving the non-stationary ground motion characteristics. The consequences of these wavelets on spectral ordinates resulted in a linear system of equations to calculate the amplitudes for wavelet modification function. This technique provided a spectral-matched time history in distinct phase if the added wavelet had a direct consequence on sets of spectral ordinates. Different studies have revealed that when wavelet functions were added to acceleration histories, it has a non-linear consequence on spectral ordinates. These were the result of alteration in peak response time of single degree of freedom oscillators which were used to compute spectral ordinates. The peak response fluctuated in time or formerly smaller peaks became amplified to outstrip the original maximum due to addition of wavelet set adjustment function to acceleration record. Hence, Time Domain Spectral Matching Algorithms were frequentative likewise Newton algorithms or Modified Newton algorithms to anticipate non-linear behavior. After that the researchers used Brayden updating to investigate the time-domain spectral matching of earthquake ground-motions.

Improvements were first time made in the previous algorithm, to further discourse non-linearity related to the shifting time of ultimate response which included addition of supplementary compensating wavelet modifications or dropping those amplitudes which can cause problems in wavelet alteration function. An up- graded tapered cosine wavelet basis was produced to preserve an efficient form which have the ability to instantly fit in zero displacement and velocity, and need no baseline correction. Investigators have also explored further characteristics linked with usage of wavelets to Time Domain Spectral Matching. Spectral matching by different procedures is anticipated to associate wavelet analysis with neural networks. Various wavelet alterations and damage index were used to investigate inelastic spectral matching. The use of different mother wavelets in spectral-matching was explored such as adjusted tapering cosine wavelet were described and use of wavelet termed as an effective method that was also revealed by various scientists.

2.9 Frequency Domain Spectral Matching

The frequency domain spectral matching was reported in 1984 along with other spectral matching procedures at the times. This method was first commercialized by Silva and Lee by developing a software named RASCAL. This technique used Fourier transform to make the actual ground motion records compatible with target spectrum of site of interest. To do this, filtering of actual ground motions was done through the spectral ratio of the target response spectrum to the actual response spectrum of selected record. In primary iteration, the ratio of the target spectrum accelerogram of site to the spectral accelerogram of selected ground motion were calculated for the desired range of periods. These ratios were used to modify the frequency content and the amplitude of primary accelerogram so as the modified accelerogram was approximately compatible with the target spectrum. An average error and the misfit between the spectrally matched accelerogram and target spectrum's were calculated. If results are not satisfactory, further iterations are carried out and previously modified accelerograms are utilized. This procedure is iteratively repeated for getting spectral matching up to the desired level of acceptance and period range. The increased number of iterations are used to refine the compatibility of ground motions with the target spectrum.

This technique only modifies Fourier spectral amplitudes of input ground motions and keeps the Fourier phases (sinusoids) of original record constant. The preservation of ground motion phase characteristics is significant as the nonlinear analysis ignites as a result of phasing. To preserve the Fourier phase, a zero only imaginary component transfer function was applied to the signal amplitudes and re-scaled. The FDSM has been considered very simple and straightforward process but some downsides of this methods are also reported in literature. In 1995, it was investigated that this method expressively modifies nonstationary characteristics of original ground motions and has tendency to enhance its overall energy. The two main downsides of this method were also reported. Firstly, the produced acceleration time histories do not have convergence properties. Secondly, the drift was also produced in the resultant displacement and velocity time series.

A modification in FDSM using random vibration theory was proposed to adjust the Fourier Amplitude Spectrum. In this technique, power spectral density functions were computed by using sinusoidal signals and smoothened response spectrum alongside random amplitudes and phase angles. These functions were practiced repetitively to develop distinct matching levels with recorded acceleration response and target response spectrum. By using this technique, results were obtained through considering velocity and acceleration time history records only. Even if various base line correction methods were followed, the characteristics of displacement time series was changed. FEMA chapter three section 3.3.1.4 allowed the transformation of the time-acceleration spectra using fast Fourier transform using Frequency Domain Spectral Matching (FDSM).

In order to get precise match of the target spectrum, amplitude modifications at particular frequencies were done and then transformed back into the time domain. This process interrupted frequency content, amplitude and phasing of the ground motions which may lead to enhance the total input energy of the ground motions. This technique was designated effective for estimating mean structural response with lesser number of ground motions. However, it was slightly doubting the potential inconsistency of that response. The application of this technique has been allowed by the seismic codes, but reduction of number of records as used for time-domain scaling is not yet allowed. It was also investigated that spectral matching in the frequency domain produce unexpected interruption in velocity and displacement after the matching process. This interruption produced a drift at the end of the velocity time histories and constantly enhancing or reducing displacement time histories in matched ground motions. In order to overcome this interruption a baseline modification was proposed A step by step procedure of spectral matching in frequency domain is summarized in figure below

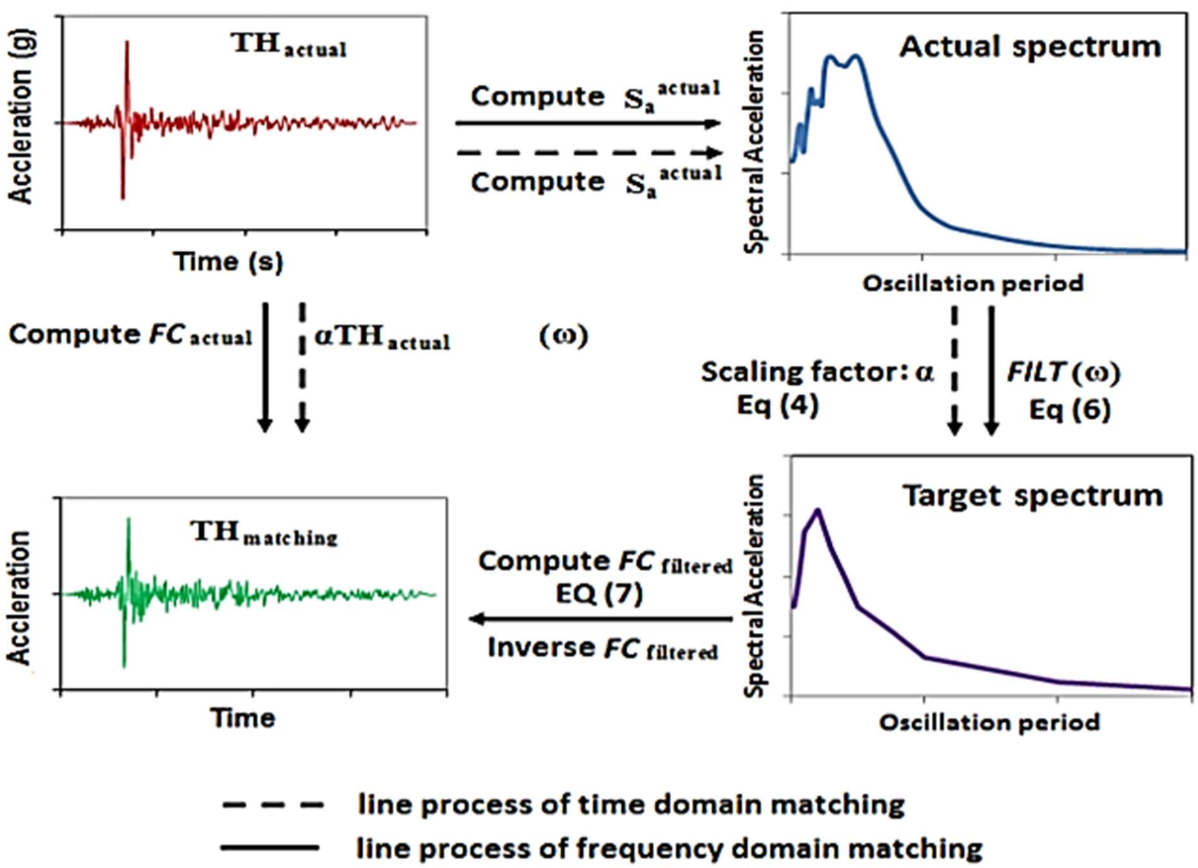


Figure 2-3: Stepwise procedure for spectral matching (Makrup 2017)

2.10 Summary

Different techniques have been used to design the high-rise buildings, the Linear Time History Analysis (LTHA) has been accepted as one of the finest among ELF, RSA and LTHA. The LTHA utilizes artificial generated and real recorded ground motions to estimate seismic response of buildings at representative of specific site locations. The real recorded ground motions are preferred nowadays because of ease of global access to ground motion databanks and for comprising original ground motion characteristics (COSMOS, USGS, NGA PEER database etc.). These real recorded ground motions, selected from seismic databanks, require prior modification before using at structures representative site of interest

There are mainly two types of these ground motion modifications, the spectral matching and the amplitude or magnitude scaling. A single factor is multiplied in typical amplitude scaling to linearly scale up or down the ground motion records with the target spectrum which provides unchanged frequency content of scaled ground motions. While the spectral matching involves the modification of time histories over the range of interest of time periods which may yield a little change in the frequency content and amplitude of ground motions. However, the spectral matching has been proved to give lesser dispersion's in EDPs as compared to amplitude scaling and henceforth preferred for high-rise buildings.

In previous explorations, the spectral matching was done using external source softwares and modeling in same softwares (i.e. ETABS) was not done for spectral matching comparison. The spectral matching is mainly divided into two categories, the Time Domain Spectral Matching (DSM) and Frequency Domain Spectral Matching (FDSM). The TDSM is considered a better spectral matching approach that utilizes the addition of wavelets in initial time histories by using latest softwares. The FDSM is also a commercially available technique in latest software's which uses Fourier transform but it upsurges the frequency content. As both spectral matchings are available in numerous softwares including but not limited to RSPMatch09, Seismosoft TARSCTHS or SIMQKE and ETABS17 practicability of spectral matching and choice of matching software turn out to be a fundamental question for designers. Hence, there is a need to investigate the behavior of high-rise buildings against these spectral matching techniques.

CHAPTER 3

RESEARCH METHODOLOGY

3.1 Introduction

In this study, six existing reinforced concrete frame structure buildings were selected as a case study. The building varied in height form 4 stories to 20 stories. All the buildings resisted the gravity load through the column beam framework while the lateral load was resisted by the RCC walls and cores. Masonry infill walls were extensively used in all the selected buildings, All the buildings have some sort of structural irregularity present. All the Buildings were in similar seismic region. The material properties for different structural elements were also similar across the selected buildings. Finite element models of these buildings were generated in ETABS version 16. The buildings were first analyzed by using Equivalent Static Analysis (ESA) and Response Spectrum Analysis (RSA) procedures using the site-specific parameters from BCP-07. The same buildings were then analyzed according to Linear dynamic analysis (LDA). Site-specific ground motions were taken from PEER database by using building code of Pakistan faulting map as reference. These ground motions were imported in CSI EATBS V16 and spectral matching was separately done in time domain and frequency domain methods. Seismic behavior of the selected buildings was then compared in terms of base shear, story drift, story shear, story moment and inter-story drift ratio. It was expected that all results for a building would show similar trend for different procedures.

3.2 Description of Case Study Buildings

The features of selected existing buildings are described below.

3.2.1 Building 1

The structure is an existing building located in Peshawar, Pakistan. The building has six stories with typical story height 14.ft. Height of the building is 85ft. The building has a reinforced concrete frame, elevator shaft and one-way or two-way slab systems at different floor levels. From the available design data, the strength of concrete is 3,000 psi and reinforcement is 60,000 psi. The slab load composed of self-weight and superimposed load (D) and the live load (L) is applied as per the code requirement. The center of the mass of the building is calculated based on mass distribution at each node. The design seismic load is calculated based on the UBC-97 code. The building is located in seismic zone 2B, with soil type SD and building importance coefficient (I) equal to 1. ETABS V16.0.2 finite element software is utilized for three dimensional modeling and analyses of the example building. Three dimensional physical and analytical model is shown in Fig 3-1 below.

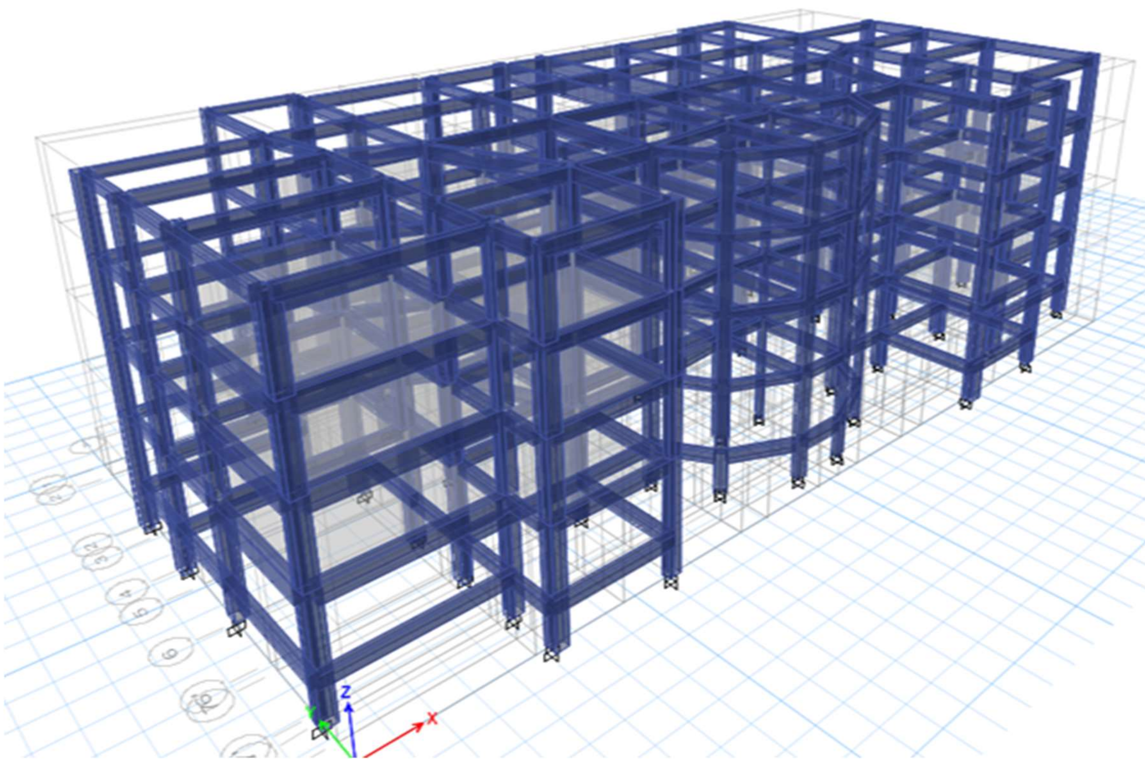


Figure 3-1: Isometric view of building 1.

3.2.2 Building 2

The structure is an existing building located in Peshawar, Pakistan. The building has four stories with typical story height 14.75 ft. Building has a height of 55 ft. The building has a reinforced concrete frame, elevator shaft and one-way or two-way slab systems at different floor levels. From the available design data, the strength of concrete is 3,000 psi and reinforcement is 60,000 psi. The slab load composed of self-weight and superimposed load (D) and the live load (L) is applied as per the code requirement. The center of the mass of the building is calculated based on mass distribution at each node. The design seismic load is calculated based on the UBC-97 code. The building is located in seismic zone 2B, with soil type SD and building importance coefficient (I) equal to 1. ETABS V16.0.2 finite element software is utilized for three dimensional modeling and analyses of the example building. Possess irregular features i.e.. Lift core wall in one corner. Three dimensional physical and analytical model is shown in Fig 3-2 below.

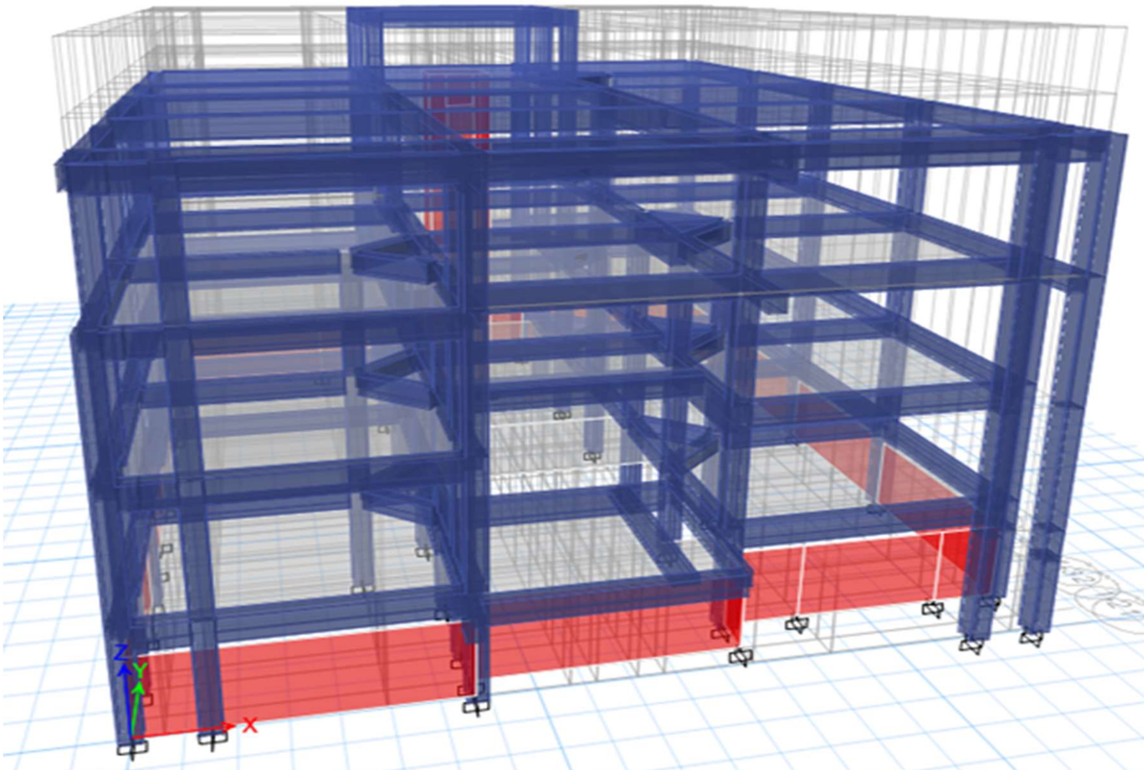


Figure 3-2: Isometric view of building 2.

3.2.3 Building 3

The structure is an existing building located in Kamra, Pakistan. The building has six stories with typical story height 14 ft. Building has a height of 85 ft. The building has a reinforced concrete frame, elevator shaft and one-way or two-way slab systems at different floor levels. From the available design data, the strength of concrete is 3,000 psi and reinforcement is 60,000 psi. The slab load composed of self-weight and superimposed load (D) and the live load (L) is applied as per the code requirement. The center of the mass of the building is calculated based on mass distribution at each node. The design seismic load is calculated based on the UBC-97 code. The building is located in seismic zone 2B, with soil type SD and building importance coefficient (I) equal to 1. ETABS V16.0.2 finite element software is utilized for three dimensional modeling and analyses of the example building. Three dimensional physical and analytical model is shown in Fig 3-3 below.

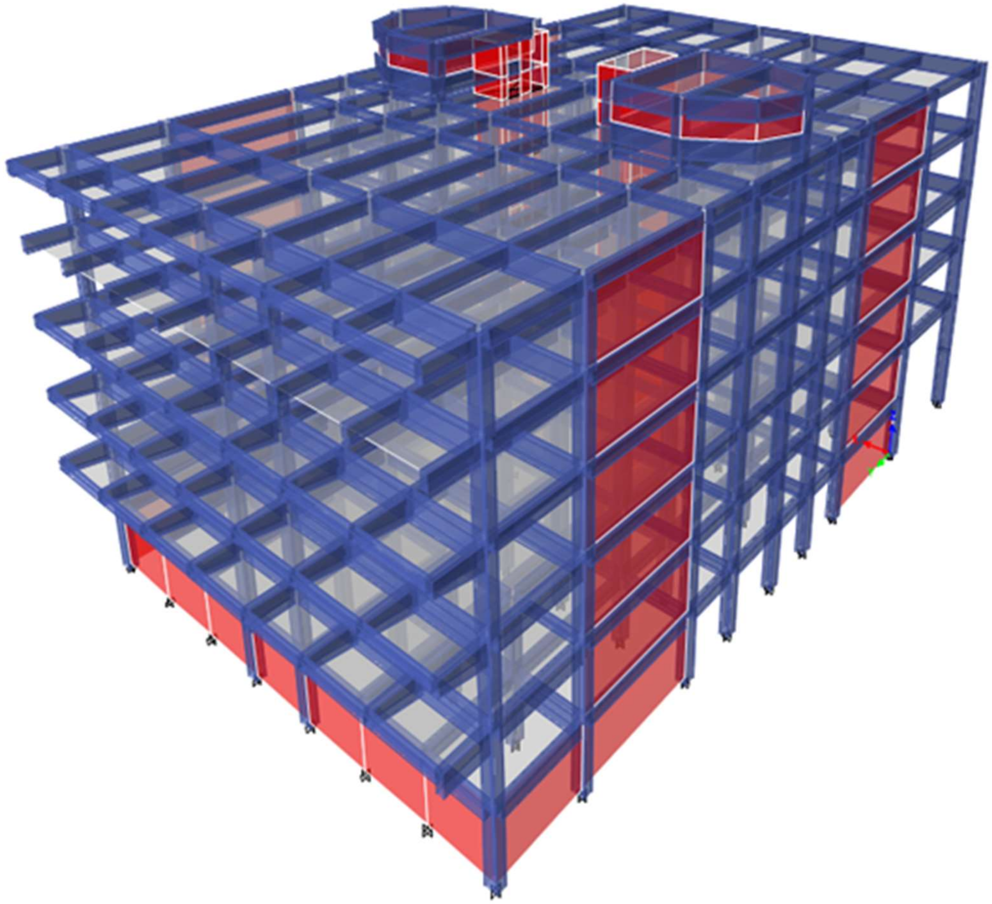


Figure 3-3: Isometric view of building 3.

3.2.4 Building 4

The structure is an existing building located in Kamra, Pakistan. The building has six stories with typical story height 14 ft. Building has a height of 85 ft. The building has a reinforced concrete frame, elevator shaft and one-way or two-way slab systems at different floor levels. From the available design data, the strength of concrete is 3,000 psi and reinforcement is 60,000 psi. The slab load composed of self-weight and superimposed load (D) and the live load (L) is applied as per the code requirement. The center of the mass of the building is calculated based on mass distribution at each node. The design seismic load is calculated based on the UBC-97 code. The building is located in seismic zone 2B, with soil type SD and building importance coefficient (I) equal to 1. ETABS V16.0.2 finite element software is utilized for three-dimensional modeling and analyses of the example building. Possess irregular features i.e.. podium and non-symmetrical arrangement of core walls. Three-dimensional physical and analytical model is shown in Fig 3-4 below.

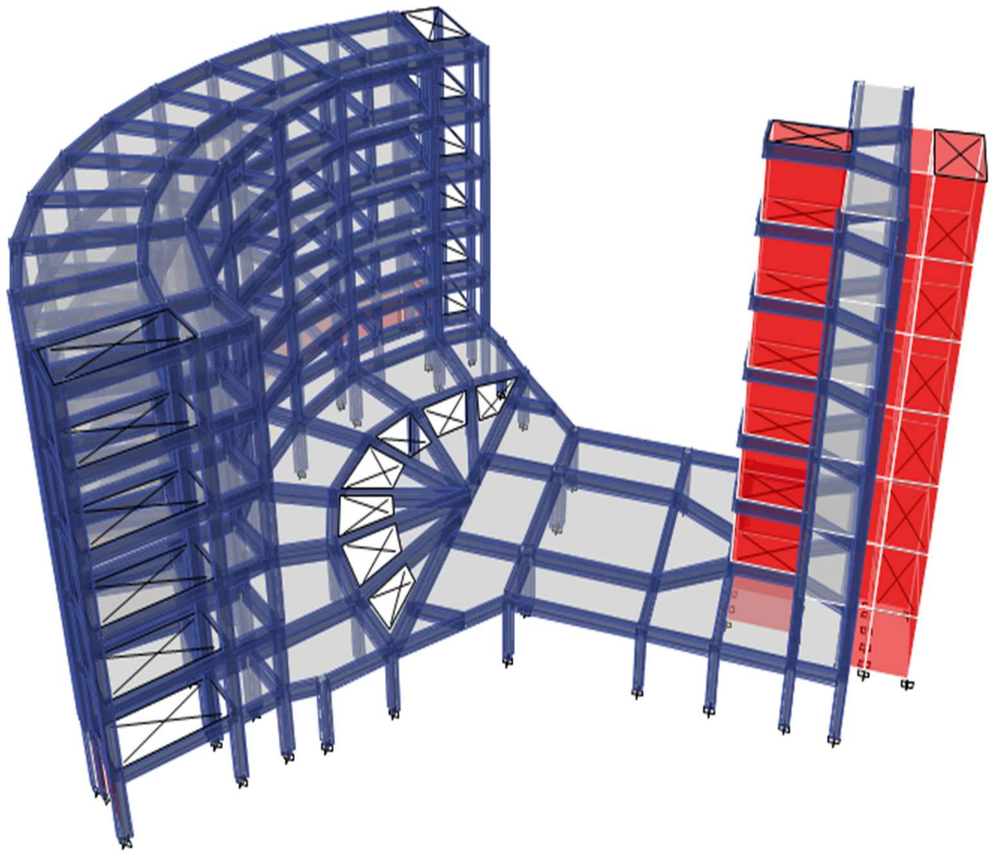


Figure 3-4: Isometric view of building 4.

3.2.5 Building 5

The structure is an existing building located in Islamabad, Pakistan. The building has fifteen stories with typical story height 10.75 ft. Building has a height of 163 ft. The building has a reinforced concrete frame, elevator shaft and one-way or two-way slab systems at different floor levels. From the available design data, the strength of concrete is 3,000 psi and reinforcement is 60,000 psi. The slab load composed of self-weight and superimposed load (D) and the live load (L) is applied as per the code requirement. The center of the mass of the building is calculated based on mass distribution at each node. The design seismic load is calculated based on the UBC-97 code. The building is located in seismic zone 2B, with soil type SD and building importance coefficient (I) equal to 1. ETABS V16.0.2 finite element software is utilized for three dimensional modeling and analyses of the example building. Possess irregular features i.e. Non-symmetrical openings at different floors. Three dimensional physical and analytical model is shown in Fig 3-5 below.

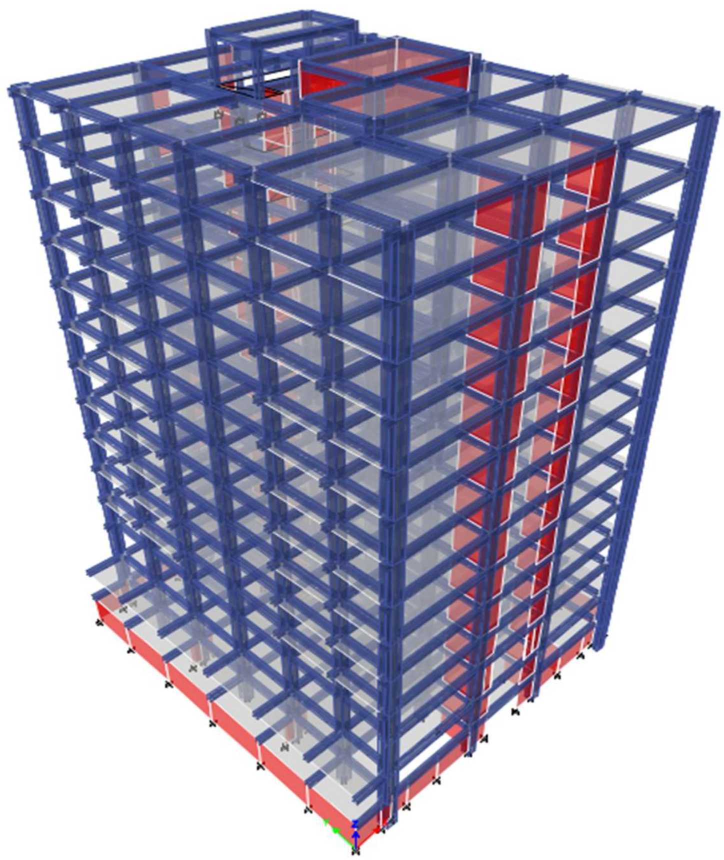


Figure 3-5: Isometric view of building 5.

3.2.6 Building 6

The structure is an existing building located in Peshawar, Pakistan. The building has twenty stories with typical story height 12.5ft. Building has a height of 250 ft. The building has a reinforced concrete frame, elevator shaft and one-way or two-way slab systems at different floor levels. From the available design data, the strength of concrete is 3,000 psi and reinforcement is 60,000 psi. The slab load composed of self-weight and superimposed load (D) and the live load (L) is applied as per the code requirement. The center of the mass of the building is calculated based on mass distribution at each node. The design seismic load is calculated based on the UBC-97 code. The building is located in seismic zone 2B, with soil type SD and building importance coefficient (I) equal to 1. ETABS V16.0.2 finite element software is utilized for three dimensional modeling and analyses of the example building. Possess irregular features i.e. Plan irregularity and non-symmetrical arrangement of RC walls .Three dimensional physical and analytical model is shown in Fig 3-6 below.

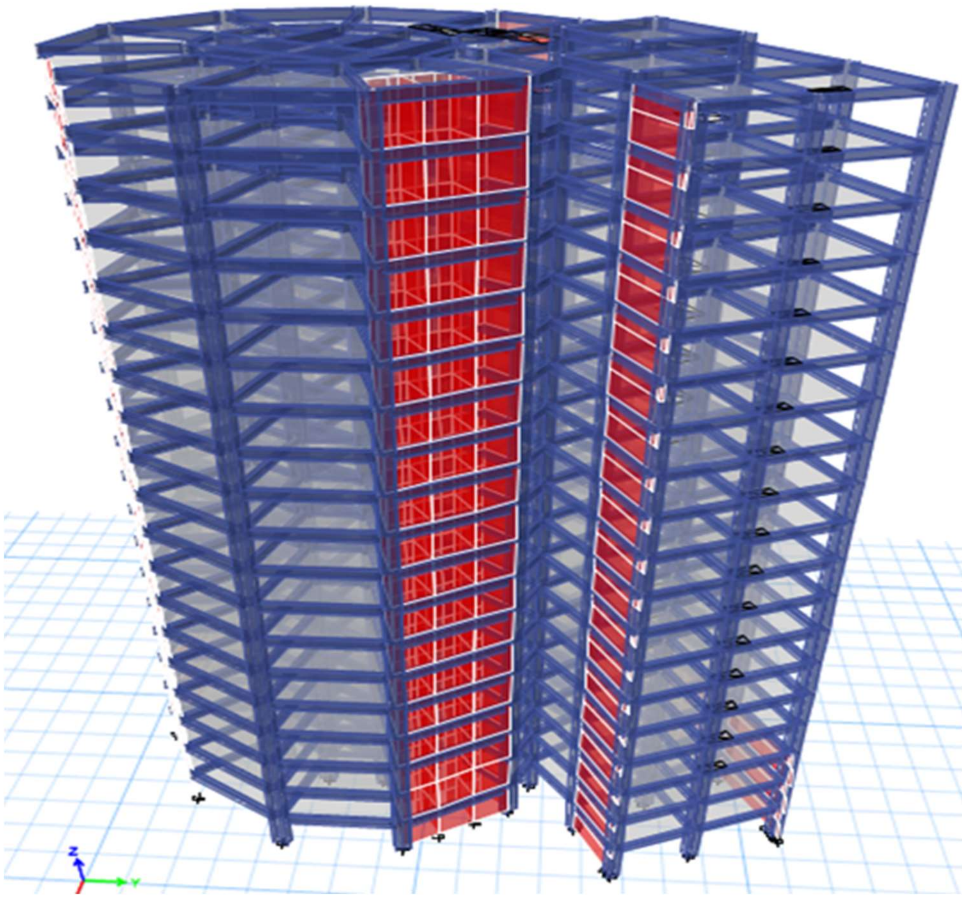


Figure 3-6: Isometric view of building 6.

3.3 Equivalent Lateral Force Parameters

ELF is a simplified procedure extensively used in seismic design businesses. This procedure is considered a better approach to design first mode dominant building. Due to higher modes contribution in midrise and high-rise buildings, this approach is not thought of as a better technique but still provide a design basis for other seismic design procedures like RSA. An equivalent static analysis and modal analysis were first executed to estimate the mode shapes, modal mass contribution coefficients and natural periods for all the governing translational modes in both primary horizontal directions (X, Y). In this study both the equivalent static and Response spectrum analysis were performed by using CSI ETABS, version 16. All seismic design practices given in BCP-07 and UBC-97 were followed

Following Parameters were used for the ELF procedure.

- Seismic zone factor: Zone 2B, 0.2g
- Soil profile type: SD
- Seismic Coefficient, Ca: 0.28
- Seismic Coefficient, Cv: 0.4
- Numerical Coefficient, Ct: 0.03
- Over strength factor, R: 5.5
- Importance Factor: 1
- Eccentricity Ratio (All diaphragm): 0.05

3.4 Response Spectrum Parameters

RSA is a linear-dynamic statistical analysis procedure that capture the involvement of all respective natural vibrating mode to designate the peak seismic response of a building. The RSA technique as per UBC-97 was implemented in this investigation to estimate initial response of structure. RSA was helpful to make design basis because it reflects structural element choice against dynamic reciprocation. The short period structures get a larger acceleration, while long period structures get larger displacements. The mass and stiffness dispersion of structures regulates the seismic response of structures. A response spectrum is mainly a graph for the steady state or ultimate response (accelerations, velocities or displacements) of a succession of oscillators of fluctuating natural frequencies, which are carried in the form of waves by the same base shaking or tremor. It was necessary to get more than or equal to 98 percent of the modal mass contribution of the structure in both respective planes. It was restricted to only X plane that was adequate for the intention of this investigation. The Response Combinations in accordance to UBC-97 were used for both analysis . The design spectrum considered in this RSA process was the elastic response spectrum at 5% damping ratio (ξ). Using this process, elastic responses of all dominant vibration modes were calculated from the design spectrum at first, followed by calculation of total responses, and then decreased to the seismic demands for designing through the response modification factor.

Following parameters were used for RSA:

- Seismic Coefficient, C_a:0.28
- Seismic Coefficient, C_v: 0.4
- Damping Ratio: 0.05

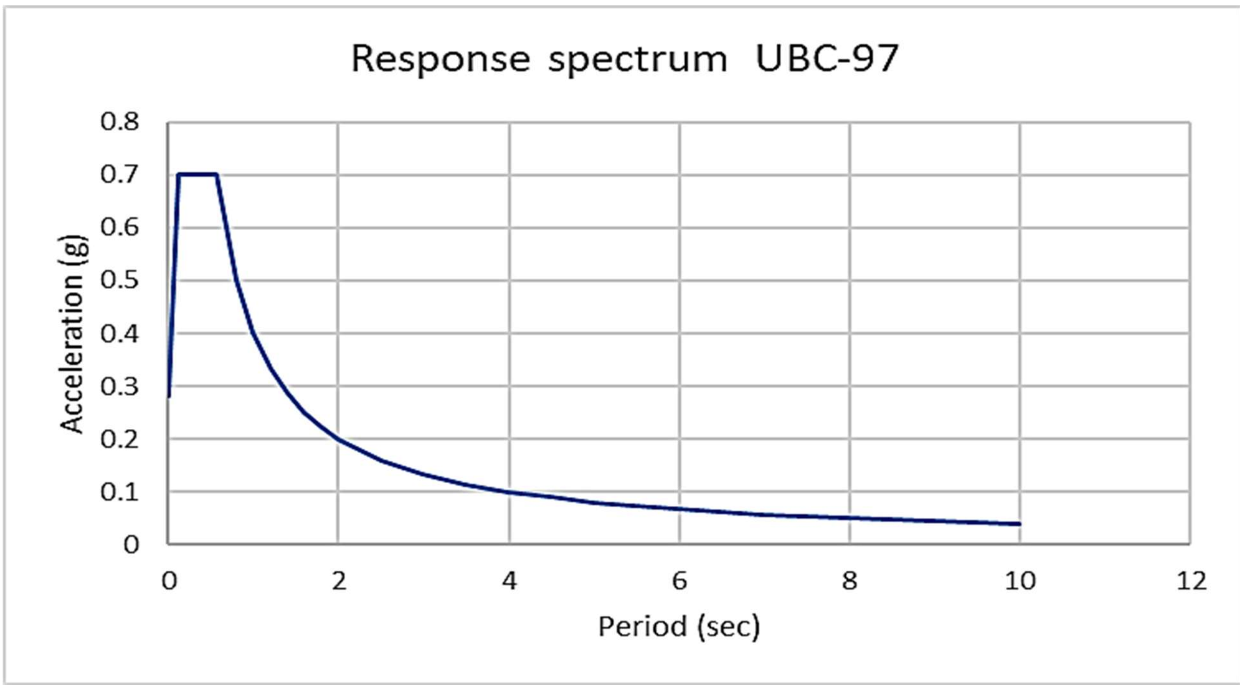


Figure 3-7: Response Spectrum as prescribed in UBC-97.

3.5 Linear Time History Parameters

Linear Time History analysis subjects the models to real ground-motion records. It captures the dynamic and time dependent response of a structure. It is the most accurate method available for representing seismic loading. Three time histories analyses were performed, then the maximum response of the parameter of interest was used for obtaining results.

Three-time histories were downloaded from PEER database using the following inputs.

- Fault Type: Reverse/Oblique
- Magnitude: 6.5-7.8

- R (km): 50-150
- Vs30 (m/s): 180-360
- D5-95: 15-60

Selected Ground Motion 1:

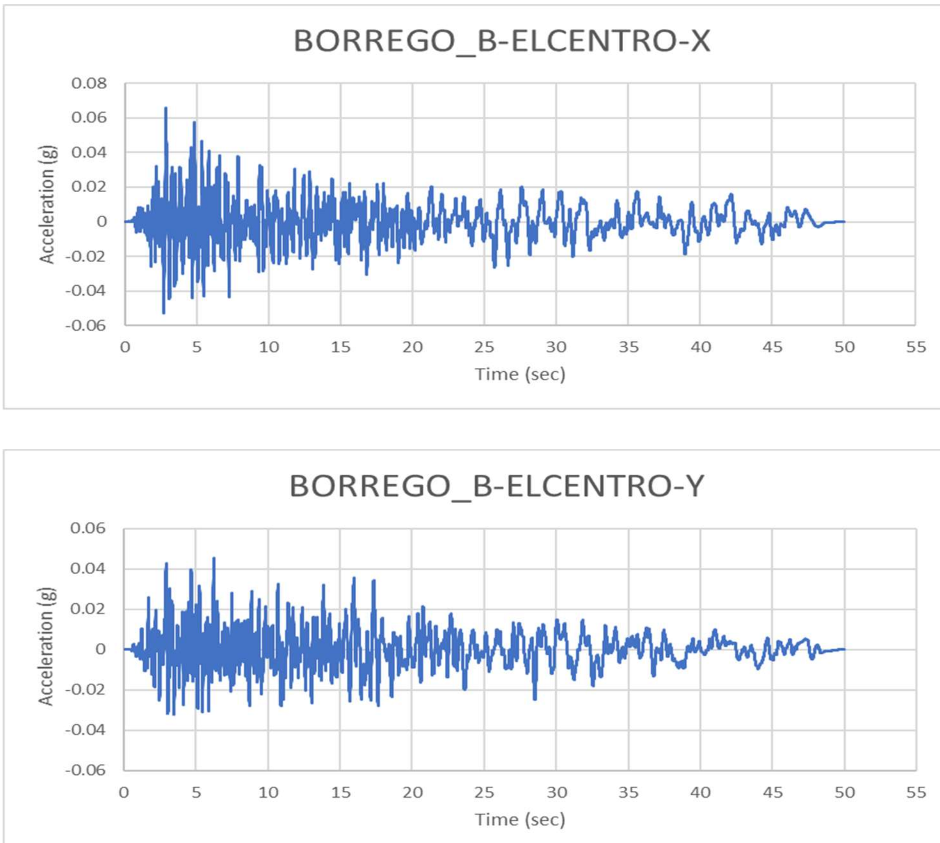
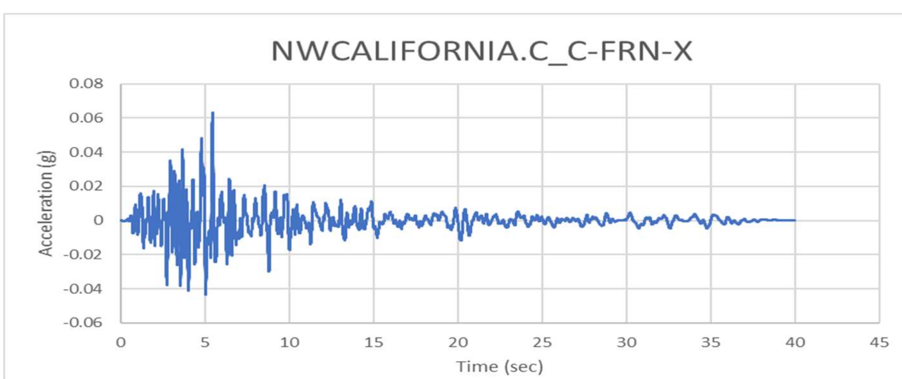


Figure 3-8: Elcentro Time History.

Selected Ground Motion 2:



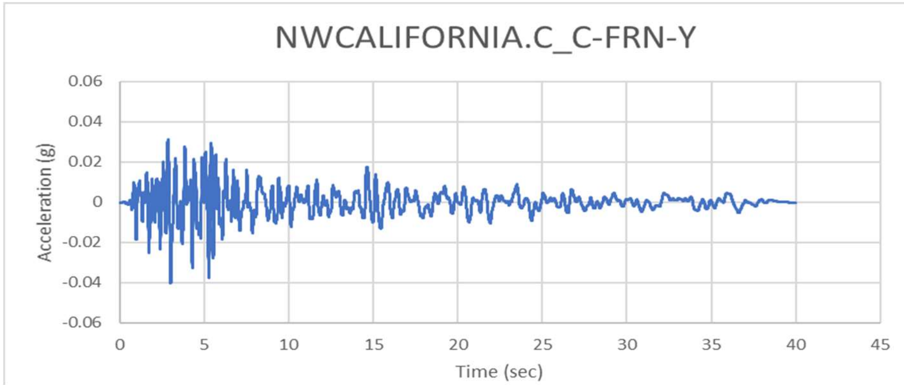


Figure 3-9: North- West California Time History.

Selected Ground Motion 3:

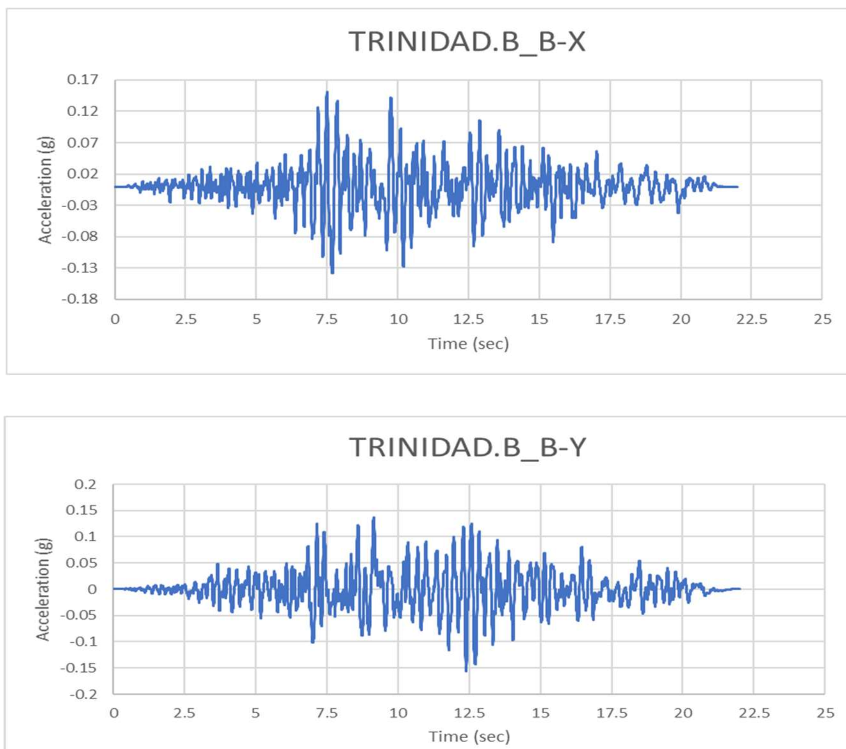


Figure 3-10: Trinidad Time History.

CHAPTER 4

RESULTS AND DISCUSSION

4.1 Background

In previous chapters, Equivalent Lateral Force (ELF), Response Spectrum Analysis (RSA) and Linear Response History Analysis (LRHA) procedures has been discussed. These linear and non-linear procedures have been applied on the six buildings and the results has been compared on five different engineering demand parameters (EDPs). Seismic response of selected building against each matching technique was further required to compare the superiority or equivalency of each matching technique. The following parameters have been considered for this investigation.

- Story drift ratio
- Story displacement
- Story shear
- Story moment
- Story acceleration

4.2 Modal Analysis Results

Tables 4-1 and 4-2 are showing the modal analysis results of case study buildings.

Table 4-1: Modal Analysis for Building 1-3

	Building 1	Building 2	Building 3
X- Direction	T_1= 0.23 sec, MPF = 0.600	T_1= 0.51 sec, MPF = 0.291	T_1= 0.11 sec, MPF = 0.369
	T_2= 0.87 sec, MPF = 0.361	T_2= 0.05 sec, MPF = 0.233	T_2= 0.42 sec, MPF = 0.316
	T_3= 0.08 sec, MPF = 0.012	T_3= 0.14 sec, MPF = 0.150	T_3= 0.07 sec, MPF = 0.077
Y- Direction	T_1= 0.25 sec, MPF = 0.580	T_1= 0.58 sec, MPF = 0.236	T_1= 1.19 sec, MPF = 0.327
	T_2= 0.91 sec, MPF = 0.330	T_2= 0.04 sec, MPF = 0.201	T_2= 0.42 sec, MPF = 0.315
	T_3= 0.12 sec, MPF = 0.030	T_3= 0.48 sec, MPF = 0.110	T_3= 0.14 sec, MPF = 0.070
Torsion	T_1= 0.80 sec, MPF = 0.827	T_1= 0.48 sec, MPF = 0.350	T_1= 0.49 sec, MPF = 0.715
	T_2= 0.22 sec, MPF = 0.093	T_2= 0.58 sec, MPF = 0.160	T_2= 0.15 sec, MPF = 0.140
	T_3= 0.92 sec, MPF = 0.04	T_3= 0.51 sec, MPF = 0.100	T_3= 0.05 sec, MPF = 0.015

Table 4-2: Modal Analyisis for Building 4-6

	Building 4	Building 5	Building 6
X- Direction	T_1= 1.06 sec, MPF = 0.299	T_1= 2.48 sec, MPF = 0.326	T_1= 2.07 sec, MPF = 0.375
	T_2= 0.08 sec, MPF = 0.190	T_2= 0.68 sec, MPF = 0.260	T_2= 0.49 sec, MPF = 0.200
	T_3= 0.33 sec, MPF = 0.143	T_3= 0.02 sec, MPF = 0.070	T_3= 0.23 sec, MPF = 0.071
Y- Direction	T_1= 1.00 sec, MPF = 0.243	T_1= 2.80 sec, MPF = 0.262	T_1= 2.31 sec, MPF = 0.368
	T_2= 0.33 sec, MPF = 0.161	T_2= 0.53 sec, MPF = 0.130	T_2= 0.56 sec, MPF = 0.149
	T_3= 0.03 sec, MPF = 0.09	T_3= 0.69 sec, MPF = 0.110	T_3= 0.26 sec, MPF = 0.050
Torsion	T_1= 0.93 sec, MPF = 0.365	T_1= 2.23 sec, MPF = 0.530	T_1= 1.68 sec, MPF = 0.591
	T_2= 0.24 sec, MPF = 0.215	T_2= 2.81 sec, MPF = 0.140	T_2= 0.43 sec, MPF = 0.121
	T_3= 0.28 sec, MPF = 0.051	T_3= 0.69 sec, MPF = 0.060	T_3= 0.21 sec, MPF = 0.023

4.3 Story Drift Ratio

Story drift ratio is one of the most important EDPs in structural engineering and is defined as the difference of displacements between the two successive stories divided by the in-between height of stories in consideration. To compare the maximum story drift ratios for each building, maximum story drift ratios were plotted for ELF, RSA and LRHA which is then converted into a single graph for average response as shown in figure below.

In next three beams specimen Ref-B has highest value of load at Peak, Yield and Ultimate points, while showing Least value of deflection as shown in Figure 17. Specimens without reinforcement have shown bilinear response. They showed a large deflection at mid-span exceeding 1/20 span at less than 3 percent reduction of Peak load P_o. The specimens were able to take more load after a sudden drop, but the test was stopped at 20 percent reduction in Peak load Po due to limitation of slippage at supports. The specimens showed a small decrease in flexural strength with large deformation. exceed 0.025 times the story height. Thus, for this structure the calculated value is 0.0025 times the story height i.e. 0.0025 x 10 x 100 = 2.5% for all stories except the ground floor. For ground floor the allowable story drift is found to be 0.0025 x 20 x 100 = 5%. Hence, the story drift for all the floors has been observed within the allowable code restrictions. While comparing the story drift ratios of two spectral matching in consideration, it was observed that story drift ratios have similar pattern and alike values for all individual responses with minute differences.

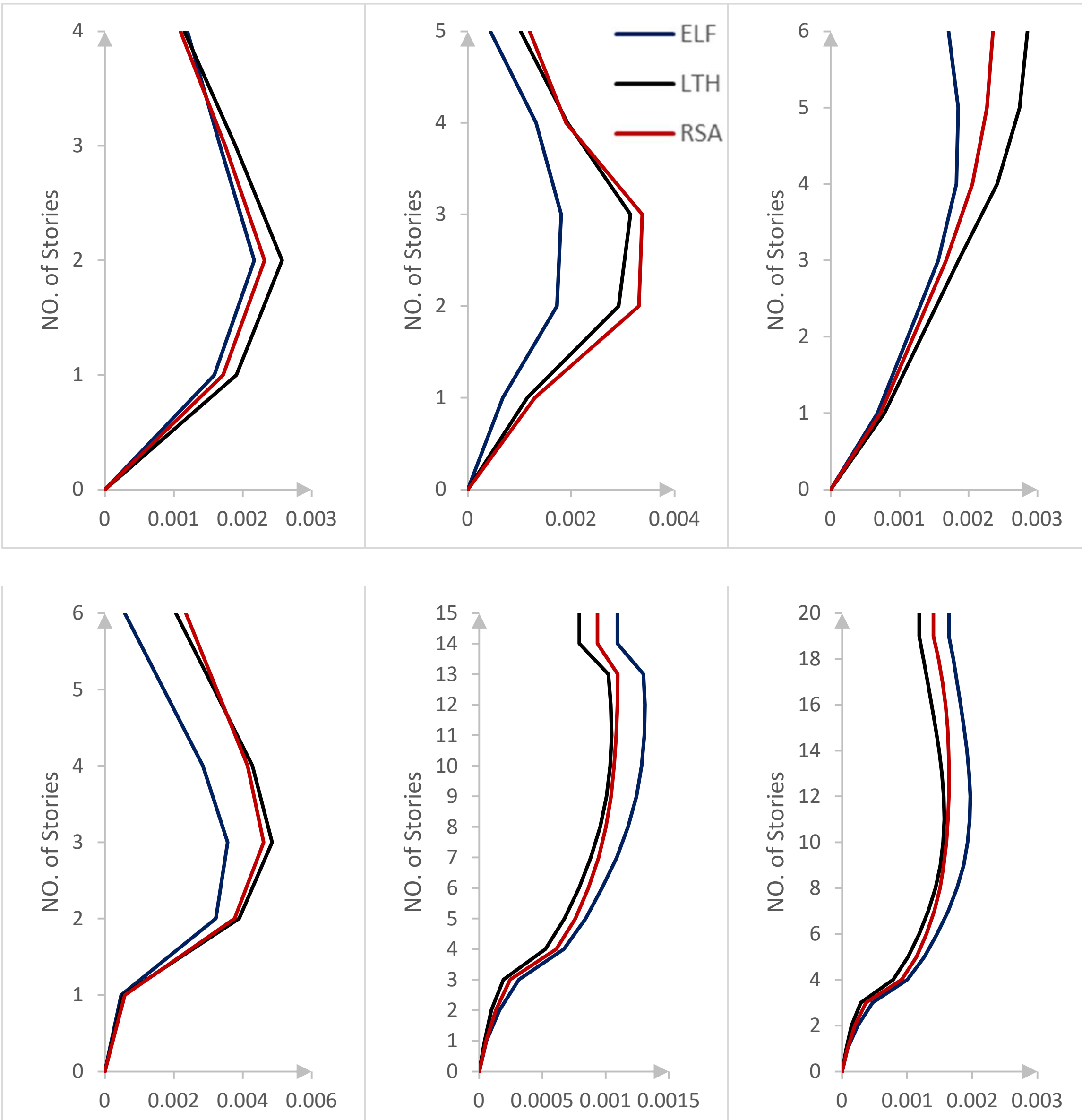


Figure 4-1: Inter-story drift ratios of case study buildings.

4.4 Story Displacement Plots

The absolute story displacement because of the lateral forces is called the story dis- placement. Story displacement is global parameter which refers to the lateral displacement of the roof of the structure with respect to its base. The roof displacement is a parameter of measure of lateral displacement response of story against lateral loading relative to the base. The story displacements against all individual and average spectrally matched ground motions were plotted as shown in figure below. It was observed that the story displacement has linear increase from bottom to top against both spectral matched ground motions. The displacement demands have almost similar behavior as of story drift ratios.

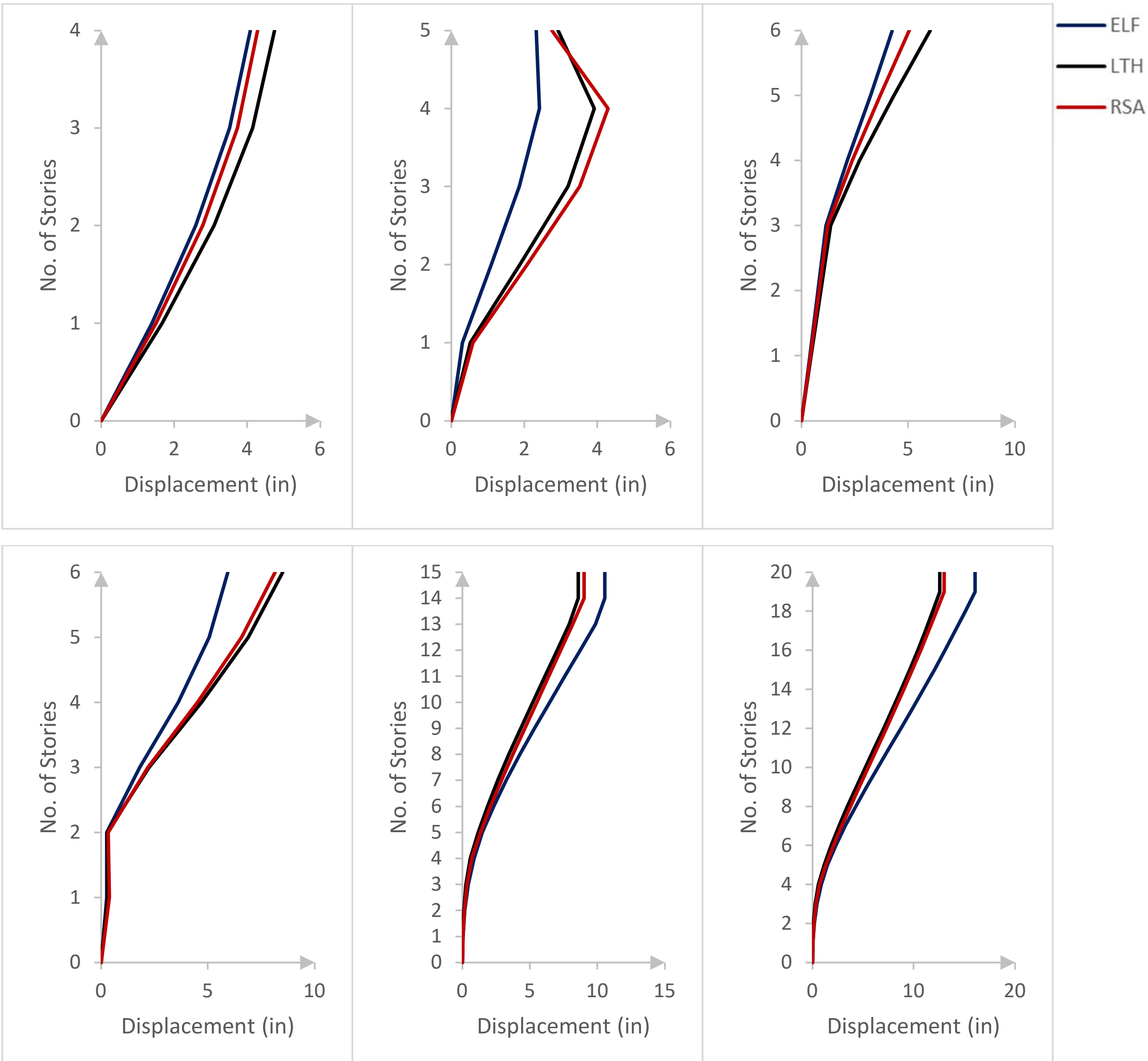
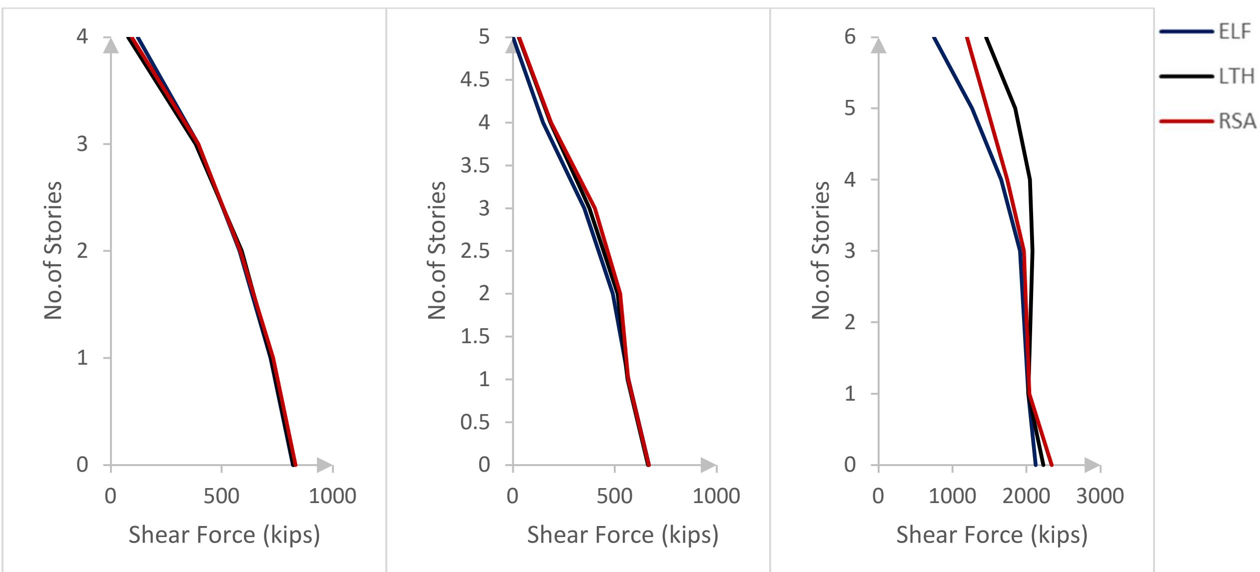


Figure 4-2: Story displacement of case study buildings.

4.5 Maximum Story Shear

The story shear is the graph to present lateral seismic forces acting on each story level. Story shear is the lateral force generated at each level of the building in case of a seismic event. Story shear is calculated at each story as it varies from story to story across the height depending on masses and stiffness. It varies from maximum at the bottom to minimum at the top of the building. The maximum lateral force that the structure experience at the base of a structure due to seismic forces is equal to base shear. Base shear is also a global response parameter which narrate the lateral reaction at the base of the structure. It primarily depends on the mass of the structure, lateral load magnitude and lateral resistance offered by the structure. The comparison of story shear for frequency domain spectrally matched and time domain spectrally matched ground motions are shown in figure below.



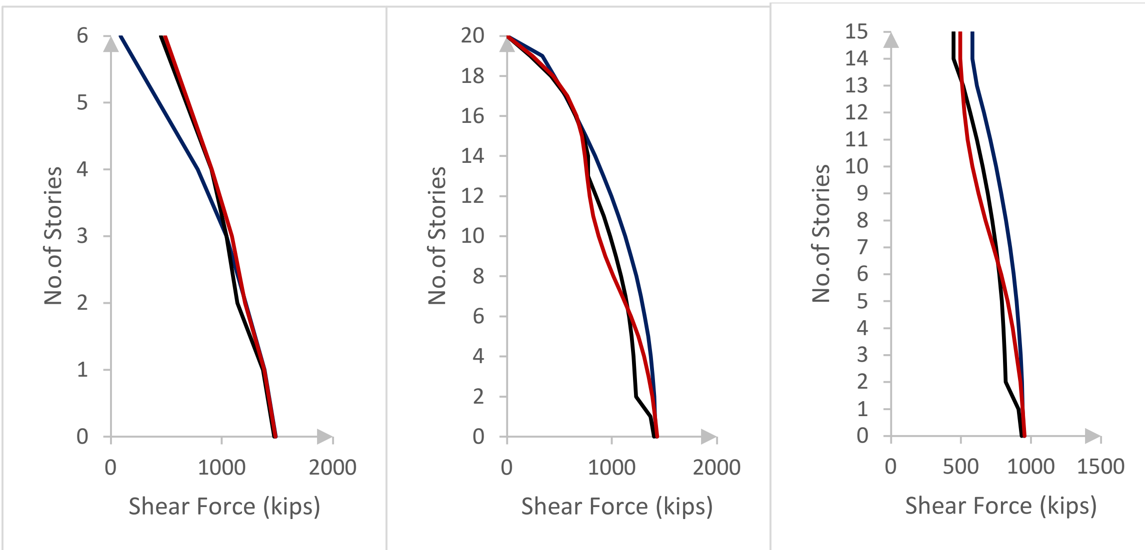
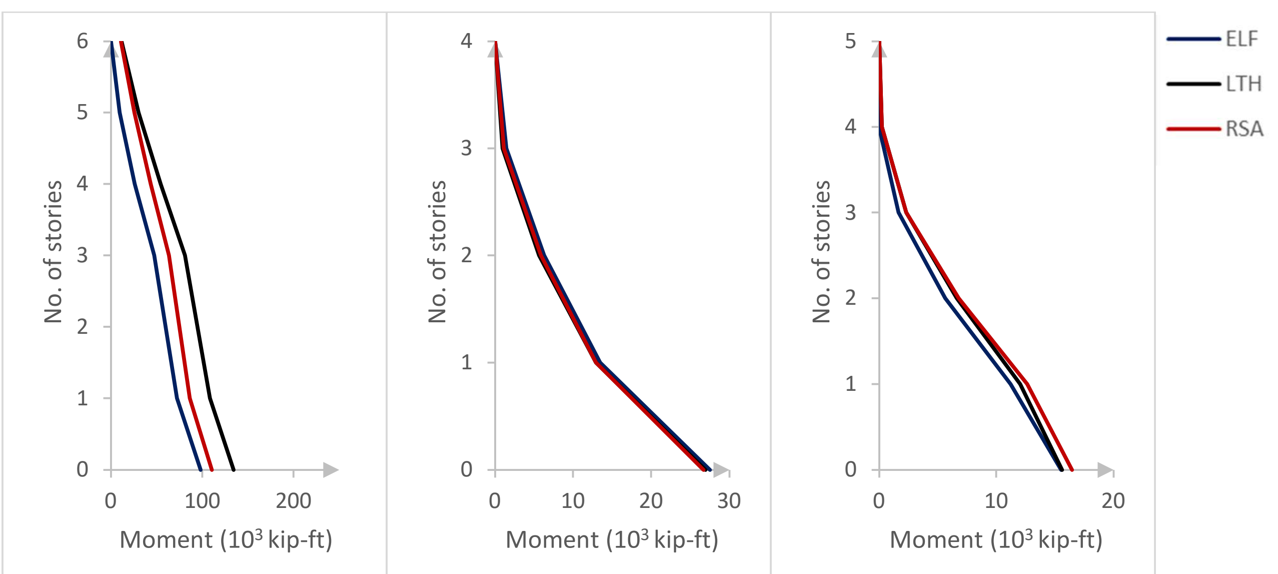


Figure 4-3: Story shears of case study buildings.

4.6 Story Overturning Moments

The story moment responses against FDSM and TDSM are plotted in figure below. The story moment at ground floor was observed to be maximum as predicted due to plastic hinge development. Overturning moment of story is the torque due to the resulting applied lateral forces about the points of contact with the ground or base. Overturning moment of a story is defined as the cumulative product of lateral forces and moment arm up to that story level.



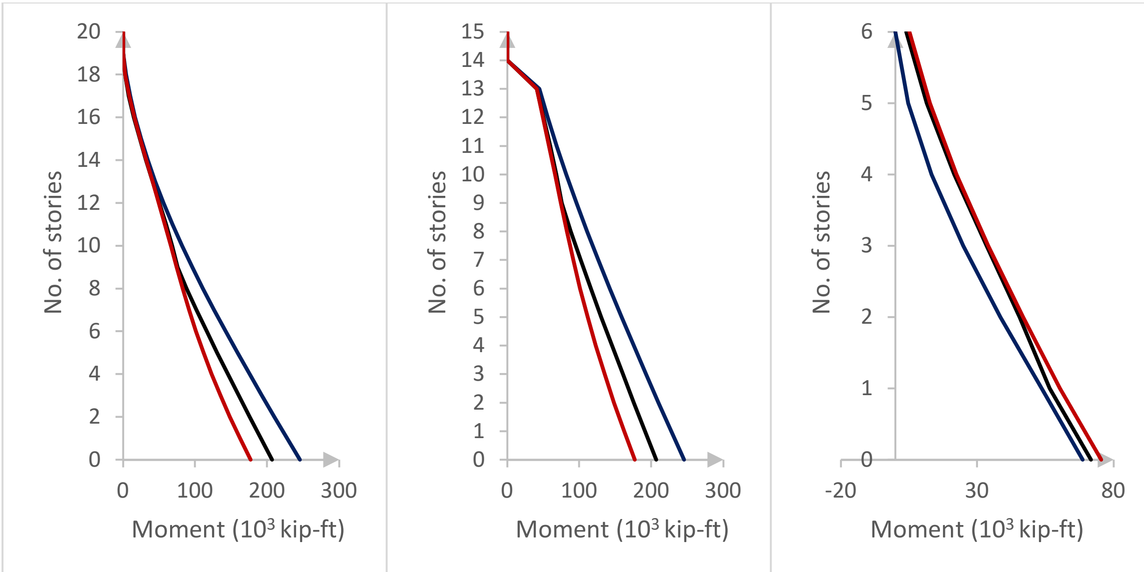


Figure 4-4: Story moments of case study buildings.

4.7 Summary

In this chapter results obtained from the seismic performance assessment of all six-building analyzed Equivalent Lateral Force (ELF), Response Spectrum Analysis (RSA) and Linear Response History Analysis (LRHA) procedures has been. For this purpose, an analytical seismic framework was developed for analyzing the seismic response of structures using engineering demand parameters (EDPs).

The next chapter presents the conclusions and recommendations for the further research.

CHAPTER 5

CONCLUSIONS AND RECOMMENDATIONS

5.1. Conclusions

The following conclusions can be drawn from the obtained results and discussion presented in Chapter 4.

- For low-rise buildings (4, 5 stories), the ELF procedure underestimates the displacement and drift demands. However, the prediction of force demands (shears and moments) for all three seismic analysis procedures (ELF, RSA and LRHA) are almost equal
- Only first vibration mode governs the dynamic response, as evident from modal analysis that modal mass participation factor for first mode is greater, and no signaficant contribution from higher vibration modes.
- ELF is empirical based without consideration of dynamics of structures. Displacement and drifts values are not reliable until and unless dynamic analysis RSA and LTHA are not performed
- For mid-rise buildings (6 stories), the ELF procedure is underestimating the true dynamic seismic demands (forces and displacements). The RSA procedure is also underestimating the demands
- RSA considers all significant vibration modes but unable to capture the dynamic amplification.
- Significant dynamic amplification of demands is expected in high-rise buildings**.** For high-rise buildings (14, 20 stories), the RSA procedure is again underestimating the true dynamic seismic demands (especially forces) for the same reason.
- For high-rise buildings (14, 20 stories), the ELF procedure is overestimating the true dynamic seismic demands (especially forces)
- Equivalent lateral forces are empirically determined with no hazard scaling compared to RSA/LTHA.

5.2 Recommendations

Further research is recommended by using non-linear seismic analysis procedures for critical review of initial code-based design. Some specific recommendations are as follows.

- It is recommended to study more building with similar configuration and different seismic zones in order to refine these research findings.
- The cost comparison studies can also be performed for having detail idea about economic aspects.
- The performance-based evaluation of Existing building in Pakistan should be carried out to find the seismic vulnerability of the existing stock of RCC building in Pakistan.

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