DEDICATION

This thesis is dedicated to our family and friends

ACKNOWLEDGEMENTS

The authors would, first of all, like to thank the Almighty Allah, the most gracious and the most merciful, for blessing them with the qualities that were needed for the accomplishment of this project.

The authors of this document would like to express their utmost gratitude to our esteemed project supervisor, Dr. Shaukat Ali Khan, for his valuable guidance and support throughout the execution of this project.

The completion of this project would not have been possible without the assistance provided by Engr. Munawar Ghani and Engr. Arslan Mushtaq with respect to use of project relevant analysis software.

The authors would also like to acknowledge their family and friends for the constant encouragement and support provided by them during the execution of this project and their years at National University of Sciences And Technology.

LIST OF FIGURES

Figure 1 – Collapse of Margalla Towers 7

Figure 2 – Seismic Zonation Map of Pakistan (Arrow points to location of Islamabad) ... 9

Figure 3 – Work Plan 19

Figure 4 – West Elevation 20

Figure 5 – South Elevation 21

Figure 6 – Equations for computation of Site Adjusted Spectral Acceleration Parameters 23

Figure 7 Explanation of quantities used in Equation shown in Figure 6 24

Figure 8 – Checks in screening phase 25

Figure 9 – Equation for computation of pseudo lateral force 29

Figure 10 – Description of quantities shown in Figure 9 29

Figure 11 – Equation for calculation of Fundamental Period of building 30

Figure 12 – Offers a description of the quantities found in the equation shown in Figure 11 30

Figure 13 – Equations for vertical distribution of Pseudo Lateral Force 34

Figure 14 – Explanation of Quantities used in equations shown in Figure 13 34

Figure 15 – Characteristics for which application of a complex analysis method becomes necessary (Tier 3 evaluation) 37

Figure 16 Shear walls in a two-storey frame structure 42

Figure 17 – Different types of bracings 43

Figure 18 – The State Life Building modelled in PERFORM 3D.................................. 45

Figure 19 – State Life Building model in 3D view.. 45

Figure 20 – Hysteresis loop developed after analysis and relevant processing................ 50

Figure 21 – Backbone curve; Peak occurs at (3, 4945) 51

Figure 22 – Original 12" thick shear walls 53

Figure 23 – Shear wall, 9" th., placed in elevator shaft 54

Figure 24 – 12” th. shear walls placed on building perimeter 54

Figure 25 – 9" th. shear walls placed on building perimeter 55

Figure 26 – 9" th. shear walls placed on building perimeter and in part of the elevator shaft 55

Figure 27 – 6" th. shear walls placed on the building perimeter and in part of the elevator shaft 56

Figure 28 – Comparison of configurations in terms of backbone curves developed........ 57

Figure 29 – Failing columns 58

Figure A1 – Storey 1 plan 66

Figure A2 – Storey 2, 4, 5 plan 67

Figure A3 – Storey 3 plan 67

Figure A4 – Storey 6, 7, 8 plan 67

Figure A5 – Storey 9, 10, 11, 12, 13 plan 68

Figure A6 – Storey 14, 15, 16, 17, 18 plan 68

LIST OF TABLES

Table 1 – Seismic Zones with their corresponding PGHA ranges and Expected Damage Level 9

Table 2 – Comparison of PERFORM 3D vs SAP2000 18

Table 3 – Brief Description of Major Checks 26

Table 4 – Steps of the LSP 33

Table 5 – Steps in LDP 35

Table 6 – General Determinations 48

Table 7 – Checklists 48

Table 8 – Structural Checklist Results 49

Table 9 – Test configurations for shear walls 52

Table A1 – Properties of major materials used in building 66

Table A2 – Dimensions of important structure components 66

Table B1 – Time Period Computation (ASCE 31-03, Chapter 3, Section 3.5.2.4) 70

Table B2 – Spectral Acceleration Parameters (ASCE 31-03, chapter 3, section 3.5.2.3.1) 70

Table B3 – Seismic Weight Computation (ASCE 31-03, chapter 3, section 3.5.2.1) 70

Table B4 – Pseudo Lateral Force (PSL) computation (ASCE 31-03, chapter 3, section 3.5.2.1) 71

Table B5 – Story Shear Computations (ASCE 31-03, chapter 3, section 3.5.2.2) 71

Table B6 – Shear Stress Check Computations Part 1 (ASCE 31-03, chapter 3, section 3.5.3.2) 72

Table B7 – Shear Stress Check Computations Part 2 (ASCE 31-03, chapter 3, section 3.5.3.2) 73

Table B8 – Axial Stress Check (ASCE 31-03, chapter 3, section 3.5.3.6) 73

Table B9 - Adjacent building check computation 75

Table B10 – Mass irregularities check computations 75

TABLE OF CONTENTS

DEDICATION I

ACKNOWLEDGEMENTS II

LIST OF FIGURES III

LIST OF TABLES VI

CHAPTER 1: INTRODUCTION 6

1.1 What Is Seismic Retrofitting? 6

1.2 The Need For Seismic Retrofitting 6

1.2.1 General 6

1.2.2 The Need In Islamabad 6

1.3 Objectives Of Seismic Retrofitting 7

1.4 Seismic Zonation Of Pakistan 8

1.5 Objectives & Goals Of Project 10

1.5.1 Objectives 10

1.5.2 Goals 10

CHAPTER 2: LITERATURE REVIEW 11

2.1 General 11

2.2 Topical Handbooks 11

2.2.1 Handbook On Seismic Evaluation Of Existing Building – Federal Earthquake Management Authority (FEMA) 310, 1998 11

2.2.2 Handbook On Seismic Retrofit Of Buildings – Indian Building Congress (April 2007) 12

2.2.3 ASCE 31-03: Seismic Evaluation Of Existing Structures 12

2.3 Theses And Studies 13

2.3.1 Seismic Vulnerability Assessment Of A Building In Islamabad, Hamza Saeed Virk (2010) 13
2.3.2 Seismic Analysis Of Reinforced Concrete Structures In Pakistan, Ghazanfar Ali Anwar (2009) 13
2.3.3 Performance Based Seismic Design, Shahana Y. Janjua (2009) 14
2.3.4 Seismic Strengthening Of Rc Structures With Exterior Shear Walls, Prof. Humberto Varum Et Al. (2013) 15
2.3.5 Pushover Analysis Of A 19 Story Concrete Shear Wall Building, Rahul Rana, Limin Jin And Atila Zekioglu (2004) 15

2.4 Software Guides 16

2.4.1 CSIAmerica Perform 3d User Guide 16
2.4.2 Perform 3d Vs. Sap2000 17

2.5 Work Plan 19

2.6 State Life Building, Islamabad 19

CHAPTER 3: METHODOLOGY - UNDERSTANDING TIER 1 EVALUATION 22

3.1 General 22

3.2 Pre-Screening Requirements 22

3.2.1 Information Collection 22
3.2.2 Site Visit 23
3.2.3 Level Of Performance 23
3.2.4 Level Of Seismicity 23
3.2.5 Building Type 24
3.2.6 Benchmark Building 24
3.3.1 Checklists 25
3.3.2 Description Of Major Structural Checks 26
3.3.3 Computation Of Pseudo Lateral Force 29
3.3.4 Computation Of Fundamental Period Of Building 29

3.4 Screening Aftermath 30

CHAPTER 4: METHODOLOGY - UNDERSTANDING TIER 2 EVALUATION 32

4.1 General 32

4.2 Linear Static Procedure (LSP) 32

4.2.1 General 32

4.2.2 Distribution Of Pseudo Lateral Force Along Building Height 33

4.3 Linear Dynamic Procedure (LDP) 34

4.4 Tier 2 Aftermath 36

CHAPTER 5: METHODOLOGY - UNDERSTANDING TIER 3 EVALUATION 37

5.1 General 37

5.2 Nonlinear Static Or Pushover Analysis 38

5.2.1 General Description 38

5.2.1 Implementation Of Pushover Analysis 38

5.2.2 Further Description Of Pushover Analysis 39

5.2.3 Limitations Of Pushover Analysis 40

5.2.4 Why Pushover Analysis Over Nonlinear Dynamic Analysis 40

CHAPTER 6: METHODOLOGY - RETROFITTING 41

6.1 General 41

6.2 Common Seismic Strengthening Or Retrofitting Techniques Used In Pakistan…………………………………………………………………………………41

6.2.1 Shear Walls 41

6.2.2 Bracing 42

CHAPTER 7: METHODOLOGY - MODELLING AND ANALYSIS IN PERFORM-3D 44

7.1 Modelling 44

7.2 Pushover Analysis 46

CHAPTER 8: ANALYSIS AND RESULTS 48

8.1 Tier 1 Evaluation 48

8.2 Tier 2 Evaluation 50

8.3 Tier 3 Evaluation / Pushover Analysis 50

8.3.1 Hysteresis Loop 50

8.3.2 Backbone Curve 50

CHAPTER 9: FURTHER EVALUATIONS AND SUBSEQUENT RESULTS 52

9.1 Shear Wall Evaluation – Economic Aspect 52

9.2 Failures In Basement Columns 57

CHAPTER 10: CONCLUSIONS AND SUGGESTIONS FOR FURTHER RESEARCH 59

10.1 Conclusions 59

10.2 Suggestions for further Research .... **ERROR! BOOKMARK NOT DEFINED.**

BIBLIOGRAPHY 62

APPENDIX A – BUILDING DETAILS 65

APPENDIX B – TIER 1 COMPUTATIONS & DETERMINATIONS 69

THIS PAGE HAS BEEN LEFT BLANK INTENTIONALLY

CHAPTER 1

INTRODUCTION

1.1 What is seismic retrofitting?

Seismic retrofitting involves modifying an existing structure to make it more resistant to earthquakes or seismic activity. For seismic retrofitting to be performed, a seismic evaluation of the structure must be carried out first and that is one of the major objectives of this project.

1.2 The need for seismic retrofitting

1.2.1 General

Earthquakes are one of the most destructive phenomena on the planet. Apart from being a threat to human life, earthquakes can cause damage to the infrastructure of a nation and to the economy that the infrastructure supports. Seismic retrofitting is one of the ways in which we can protect the existing infrastructure against damage during earthquakes. Seismic retrofitting must be carried out for existing structures that are located in regions of medium to high seismic activity.

1.2.2 The need in Islamabad

The 2005 Kashmir earthquake claimed more than a 100,000 lives, displaced more than 3.5 million people, destroyed entire villages and towns and led to economic losses of approximately $ 2.5 billion. It was estimated that reconstruction and rehabilitation of damaged infrastructure would cost upwards of $ 2.65 billion. Most of the damage from the earthquake was contained to the areas of Khyber Pakhtunkhwa (KPK) and Kashmir. However, there were instances of significant damages across the capital of Pakistan, Islamabad. One instance, in particular, was the collapse of one half of an apartment complex known as Margalla towers (pictured in figure 1). The collapse caused the deaths of 73 people and injured more than a 100 people. The collapse of the Margalla Towers was a sign of the dire need for seismic strengthening of high-rise buildings in Islamabad.

The need for proper seismic design and retrofitting for structures in Islamabad should have been apparent, in spite of the Kashmir earthquake, as Islamabad is located in a zone of medium to high seismicity. The Building Code of Pakistan (BCP, 2007) provides fairly detailed description on procedures for seismic analysis and retrofitting. BCP 2007 was prepared in the aftermath of the Kashmir earthquake.

Figure 1 – Collapse of Margalla Towers

1.3 Objectives of Seismic Retrofitting

There are multiple ways and methods of retrofitting a structure. The objectives of a retrofit scheme or strategy can be a combination of any of the following:

1) Increasing the lateral strength and the lateral stiffness of the building.
2) Increasing the ductility of the structure as well as enhancing its ability to dissipate energy built up as a result of seismic activity

3) Enhancing unity of structures or providing unity where there isn't any.
4) Eliminating or reducing sources of weakness or those sources that produce undesirable concentration of stresses.
5) Enhancing redundancy in the number of lateral load resisting elements.
6) Implementing strengthening measures that are cost effective.

1.4 Seismic Zonation of Pakistan

Pakistan's area is spread over two converging tectonic plates: the Eurasian Plate and the Indian Plate. The Indian Plate is being subducted under the Eurasian Plate. The main fault line that divides Pakistan's area among the two plates is considered to be seismically active. The fault line mostly runs through KPK and Baluchistan. Settlements near the fault line are, therefore, considered vulnerable to seismic activity. The seismic zonation map of Pakistan is shown in figure 3 (lifted from BCP 2007). There are five zones in total. Table 1 shows the peak ground acceleration range in each zone and the expected damage from significant seismic activity.

In Pakistan, almost all the structures are designed for the action of gravity loads and lateral loads (such as wind, earthquake etc.) are mostly neglected. The structures are, therefore, often unable to resist the action of lateral forces. The lateral forces cause the decrease of stiffness of the structural members and as a result, the structure can experience crippling deformations. There has been little work done on the assessment and evaluation of seismic vulnerability of existing buildings in Pakistan (Virk, 2010). The Building Code of Pakistan (BCP, 2007) provides provisions for the seismic design of new buildings but does not provide details regarding seismic retrofitting of existing structures. There is a need to develop guidelines that can facilitate seismic retrofitting of structures found in the moderate to high seismicity zones of Pakistan. There is also a need for enhancement of expertise in the areas of seismic design and strengthening, of the relevant segments of the engineering workforce.

As can be seen in figure 3, Islamabad lies approximately on the boundary between zone 3 and zone 2B. Zone 3 is a region of high seismicity while zone 2B is a region of medium seismicity. In any case, structures to be constructed in Islamabad must be designed to withstand seismic events.

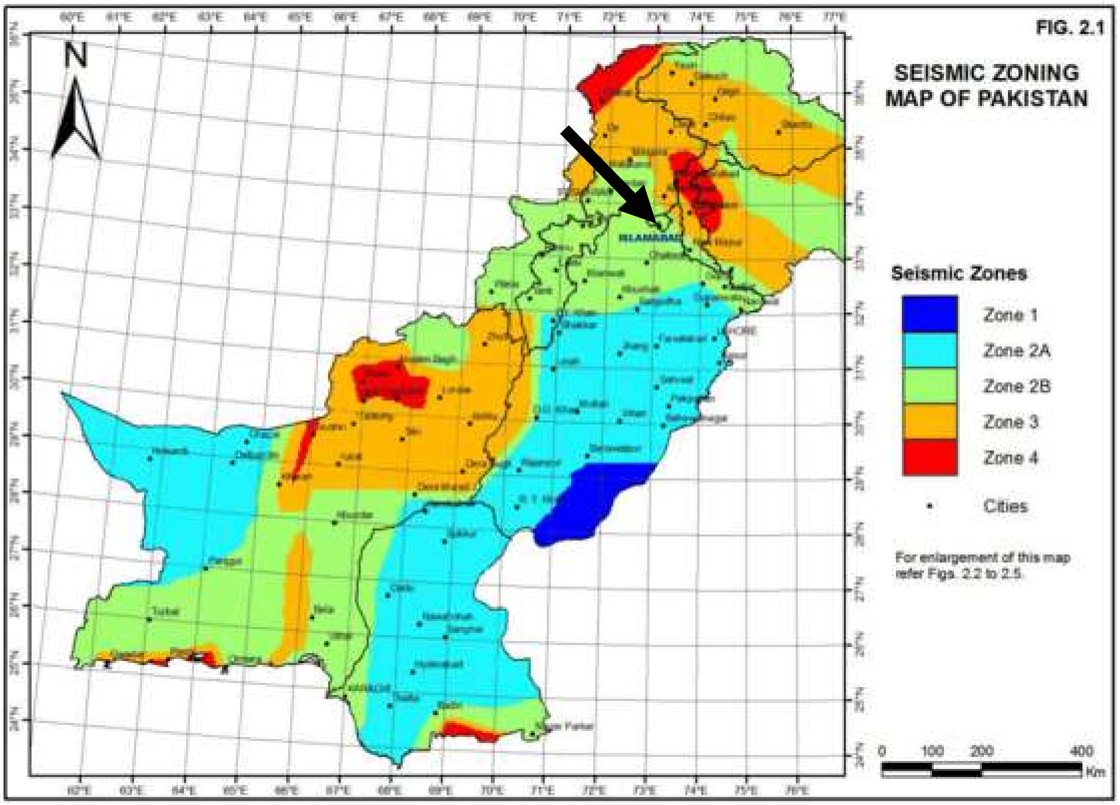

Figure 2 – Seismic Zonation Map of Pakistan (Arrow points to location of Islamabad)

Table 1 – Seismic Zones with their corresponding PGHA ranges and Expected Damage Level

Seismic Zone	Peak Ground Horizontal Acceleration (PGHA)	Expected Damage
1	0.05g – 0.08g	Very Low
2A	0.08g – 0.16g	Low
2B	0.16g – 0.24g	Moderate
3	0.24g – 0.32g	High
4	> 0.32g	Very High

1.5 Objectives & Goals of Project

1.5.1 Objectives

- Carry out a seismic evaluation for an existing RC structure in Islamabad
- Devise a suitable retrofit strategy for the building, if applicable, or evaluate suitability of retrofitting system already in place.

1.5.2 Goals

- To use the knowledge, lessons and experience gained from this project in similar projects and in one's chosen field of work.

CHAPTER 2

LITERATURE REVIEW

2.1 General

To gain a better understanding of the project topic i.e. "Seismic retrofitting of State Life Building (Islamabad) using shear walls", literature relevant to the aforementioned topic had to be reviewed. Four types of literature were studied – theses & studies, Building codes, Software guides and topical handbooks. Description and details of the literature review are provided below.

2.2 Topical Handbooks

2.2.1 Handbook on Seismic Evaluation of Existing Building – Federal Earthquake Management Authority (FEMA) 310, 1998

FEMA 310 has been created by the American Society Of Civil Engineers (ASCE) and provides fairly detailed information regarding the assessment of the seismic strength of buildings and the process of retrofitting of seismically vulnerable structures. FEMA 310 prescribes the use of either the Life Safety (LS) or Immediate Occupancy (IO) level of performance for seismic design and retrofitting of buildings. According to FEMA 310, the seismic evaluation of a building can be carried out in three main stages: the screening phase, the evaluation phase and the detailed evaluation phase.

The handbook states that a building may become vulnerable to earthquakes due to structural and/or non-structural deficiencies and that it is up to the professional using the handbook to decide the method through which any identified deficiencies will be overcome.

The handbook also covers in detail the methods such as the Linear Static Procedure, the Linear Dynamic procedure etc., that can be used to determine the extent of a building's vulnerability to seismic loads.

2.2.2 Handbook on Seismic Retrofit of Buildings – Indian Building Congress (April 2007)

As its name suggests, this handbook has been created by the Indian Building Congress (IBC). The Indian Building Congress has developed the handbook in collaboration with the Indian Institute of Technology Madras (IITM) and the Central Public Works Department which is a part of the Indian government.

Although the handbook is strictly meant to provide guidelines for carrying out retrofitting of buildings, it also throws light on concepts of Structural Dynamics. It provides case studies to aid professionals in understanding the different phases involved in the seismic evaluation of buildings. It also contains information regarding proper seismic design for structural designers. The handbook also provides a comparison between the conventional and novel methods of seismic design and evaluation.

A major part of the handbook is devoted to the description of multiple retrofitting techniques and measures that can be employed to protect buildings against earthquakes.

2.2.3 ASCE 31-03: Seismic Evaluation of Existing Structures

The main text followed in the execution of this project, ASCE 31-03, is used extensively in the United States of America for seismic evaluation of buildings and related infrastructure. It is similar to FEMA 310 in that describes the main processes involved in the process of the seismic evaluation of a building in a good amount of detail. It is, however, a little easier to comprehend and thus easier to follow.

ASCE 31-03, like FEMA 310, provides a three-tiered process for the seismic evaluation of the buildings. In Tier 1, the structural and non-structural deficiencies in the building must be identified through visual screening and certain mathematical computations. ASCE 31-03 contains detailed checklists that can be used in this stage of the evaluation process.

In Tier 2, fairly detailed descriptions of analysis methods (mostly linear analysis) used for determination of the seismic vulnerability or seismic strength of a structure are provided. The professional using the standard has the option of stopping the evaluation process at

Tier 2 and addressing any identified weaknesses in the structure. If the professional using the standard is not satisfied with results from Tier 2, he or she can move onto Tier 3.

In Tier 3, descriptions of the non-linear analysis methods used for assessing the seismic vulnerability of a structure are provided.

2.3 Theses and Studies

2.3.1 Seismic vulnerability assessment of a building in Islamabad, Hamza Saeed Virk (2010)

In this thesis, a description of the seismic vulnerability assessment of a multi-storey building with a Reinforced Concrete frame structure containing a shear wall is provided. It was stated in the thesis that the building had been constructed in Islamabad in 1991 and had been designed in accordance with the 1985 Building Code of Pakistan (BCP). The assessment for the building was carried out by the author in three stages mostly in accordance with ASCE 31-03 and FEMA 310 guidelines.

Before beginning the assessment process, the author had reviewed seismic evaluation standards adopted by the European Union, New Zealand and America. After the review, he concluded that ASCE 31-03 and FEMA 310 were best suited for the seismic vulnerability assessment process of his chosen building and given his expertise and the resources available to him.

2.3.2 Seismic Analysis of Reinforced Concrete Structures in Pakistan, Ghazanfar Ali Anwar (2009)

This study contained a description of the behavior of Reinforced Concrete (RC) frame structures under seismic loads and the contribution of the confinement of concrete near joints (beam-column) to that behavior. Modelling and analysis of a hypothetically created building was carried out in software such as STAAD pro and PERFORM 3D. The standards used to aid in the analysis and related computations included the 2007 Building Code of Pakistan (BCP 2007), ASCE 7-05 and ACI 318-08.

An important aspect of this study was the creation of a risk evaluation guideline for Kashmir (Pakistan) and the district of Mansehra by the author. The evaluation guideline developed allowed computation of damage due to seismic activity in buildings in these areas in terms of a percentage (number of retrofitted components in the building to that number of components that had not undergone retrofitting). This percentage was then linked to the monetary losses and deaths resulting from the application of the seismic loads in graphical terms.

Based on the research and analysis performed, the author concluded that:

1) Under great PGAs, Retrofitted buildings sustain less damage than un-retrofitted buildings.
2) Nearly 50 % of the deaths and injuries resulting from an earthquake in the region could be avoided and more than 50 % of the structures in the area could be strengthened to withstand earthquakes.
3) Retrofitting should be adopted for buildings located in areas where the probability of occurrence of earthquakes is relatively higher. For areas situated in low seismicity zones, retrofitting should not be pursued.

2.3.3 Performance Based Seismic Design, Shahana Y. Janjua (2009)

In this thesis, the author performed seismic evaluation of three multi-storey buildings located in Islamabad. The performance level selected for all three buildings was Life Safety (LS). The buildings and the gravity loads acting on them were first modelled in SAP2000. The buildings were then subjected to a Pushover analysis which is a non-linear analysis technique. The basic aim of this technique is to apply loads on the building in increments until it reaches the desired level of performance (in this case LS). The pushover analysis is an iterative process and one that can take a large amount of time to complete depending upon the complexity of the structure being tested and the configuration of the system on which the analysis is being carried out.

The evaluation is started by choosing a performance level and setting performance objectives with respect to the amount of damage that can be allowed to occur due to a certain level of seismic activity. The second step is to create a model of the building and the third step is to analyze it. The building's evaluation or design process continues in a cycle until the required performance level is achieved.

2.3.4 Seismic Strengthening of RC Structures with Exterior Shear walls, Prof. Humberto Varum et al. (2013)

The paper elaborates on the lack of proper Seismic provisions under the old European building codes resulting in the presence of a large amount of seismically vulnerable buildings in European countries. The reinforced concrete buildings of the late 1970's are discussed and the use of masonry infill panels and smooth reinforcement bars are mainly focused upon. Smooth reinforcement bars result in a sudden loss of concrete-steel bond resulting in the brittle failure of Reinforced concrete elements. Another reason for the inadequate response towards seismic activity is described under the behavior of axially loaded reinforced concrete members under biaxial bending moment. More reasons associated with inadequate response are: stirrups/hoops, confinement and ductility; bond, anchorage, lap-splices and bond splitting; inadequate shear capacity and failure; inadequate flexural capacity and failure; inadequate shear strength of the joints; influence of infill masonry; vertical and horizontal irregularities; higher modes effect; strong-beam weak-column mechanism, and, structural deficiencies due to architectural requirements. The paper describes the combination of these factors as the prime reason for the inadequate seismic response. The importance of experimental studies on full scale buildings is mentioned. Pseudo-dynamic tests on two full scale four storey reinforced concrete buildings are carried out. Each building has three bays, two of 5 m span and one of 2.5m span. The inter-storey height is 2.7m and the slab thickness is 0.15m. These tests show the extent of vulnerability of the structures and the presence of high risk to human life. The retrofitting techniques proposed show substantial decrease in the vulnerability.

2.3.5 Pushover Analysis Of A 19 Story Concrete Shear Wall Building, Rahul Rana, Limin Jin And Atila Zekioglu (2004)

In this thesis the author performed pushover analysis on a high rise building located in San Francisco. The building analyzed is a nineteen story, 240 feet tall slender concrete tower with an area of 430,000 square feet. The building was located in Earthquake zone 4 with only shear walls as lateral force resisting system, and was designed based on 1997 Uniform Building Code.

Due to the unique shape of the floor plan, 8 separate Pushover analysis were performed on it. The building was analyzed using SAP 2000 and ETABS. The performance level selected for the building was Life Safety. The pushover analysis is an iterative process, one that takes a lot of time to complete depending upon the complexity of the building.

The evaluation was started by selecting the performance objective, then modeling the building. The third step is to analyze the building. The building's evaluation or design process continues in a cycle until the required performance level is achieved.

2.4 Software Guides

2.4.1 CSI America PERFORM 3D User Guide

Multiple software have been used in the execution of this project such as SAP2000, ETABS and PERFORM 3D. The most notable and obscure among these software is PERFORM 3D.

In the area of Seismic evaluation of structures, PERFORM 3D is mainly used to carry out non-linear analysis of the seismic strength of structures. It is mostly used in the Tier 3 analysis stage. According to ASCE 31-03, a Tier 3 evaluation requires a more accurate and more complex analysis of a structure's seismic capacity. In the realm of seismic design and related analysis, non-linear inelastic methods of analysis are considered to be more complex and more accurate compared to linear elastic methods of analysis. Compared to SAP2000 and ETABS, PERFORM 3D can deliver better results for a non-linear inelastic or elastic methods of analysis.

PERFORM 3D uses many standards as the basis for its analysis functions. One of those standards is the ASCE 41-13 which is a pioneer of the relatively novel deformation based design techniques.

To understand the functionality and features of PERFORM 3D, a user guide provided by PERFORM 3D's developers at Computers and Structures Incorporated (CSI) America was thoroughly studied

2.4.2 PERFORM 3D vs. SAP2000

A comparison of the PERFORM 3D and SAP2000 is given below (Table 2) to provide an insight into why PERFORM 3D was intended for use as part of this project.

P.T.O

Table 2 – Comparison of PERFORM 3D vs SAP2000

Sr.No	PERFORM 3D	SAP2000
1.	Ideal for nonlinear performance based analysis and Design.	2D & 3D linear analysis and design of any structural system.
2.	It offers some nonlinear features which are not being currently offered by SAP2000,like Shear wall, infill panel, etc.	It deals with linear features only and doesn't include non-linear elements.
3.	It enables various limit states and output Demand/Capacity ratio to be defined for object groups.	Basic and advanced systems, ranging from 2D to 3D, of simple geometry to complex, may be modeled, analyzed, designed.
4.	Color coordinated animations depict dynamic response of enabled limit states.	Majorly used for analysis of linear elements like beam, column, slab etc.
5.	Its advanced modelling tools enable a sophisticated simulation of structural behavior.	Modelling tools enable user to work with them more conveniently.
6.	Nonlinear analysis strategies are very reliable.	Linear analysis strategies are very reliable.

7.	Analysis Types: 1. Static Push over analysis 2. Gravity loads 3. Response history for earthquake ground motion 4. Response history for dynamic force	Analysis Types: 1. Static Analysis 2. Dynamic Analysis 3. Buckling 4. Push over 5. P-Delta 6. Steady state analysis

2.5 Work Plan

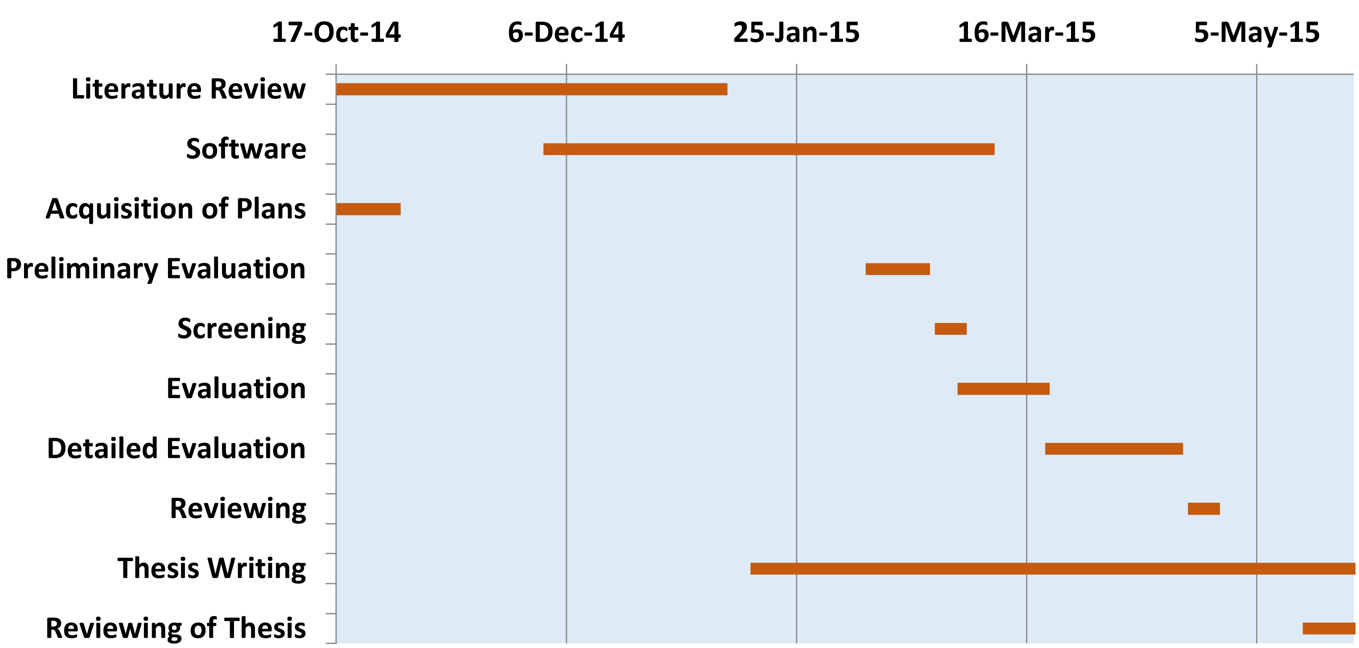

Figure 3 – Work Plan

2.6 State Life Building, Islamabad

The State Life building was chosen as the structure for which retrofitting would be carried out. It's located in Islamabad's central business district, Blue Area. It consists of a total of

18 storeys above ground with 3 additional storeys below ground. The above ground height of the building is approximately 285’. The building is mainly a Reinforced Concrete (RC) frame structure with notable architectural features.

Only the buildings main structure with additional architectural components stand today. The building’s interior and exterior finishing remains to be carried out. The building’s floor areas decrease as we move upwards. Figures 3 and 4 provide an outline of the building and its exterior features.

A noticeable feature of the building is the presence of lateral force resisting systems such as shear walls (12” Thick) and bracing beams (RCC).

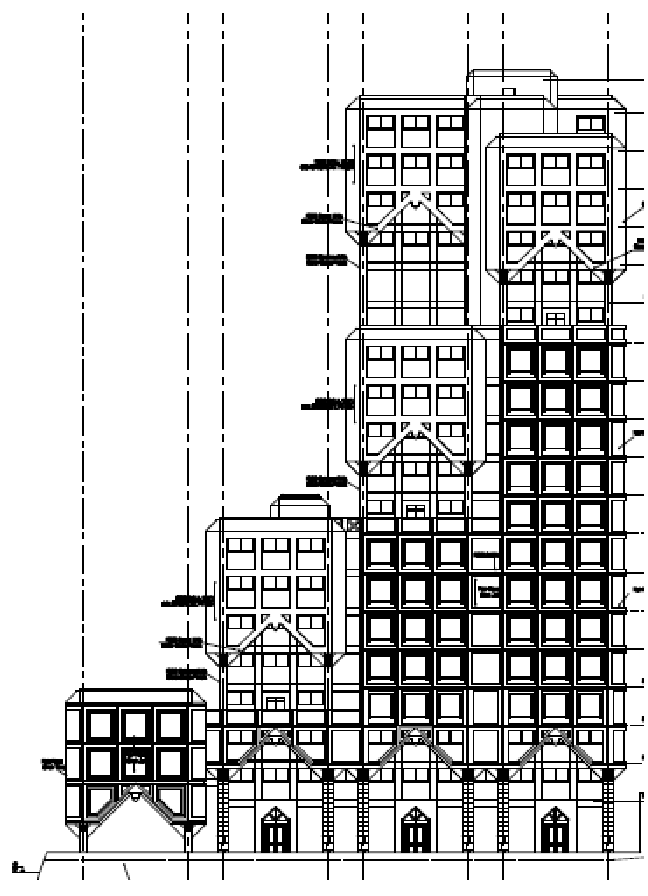

Figure 4 – West Elevation

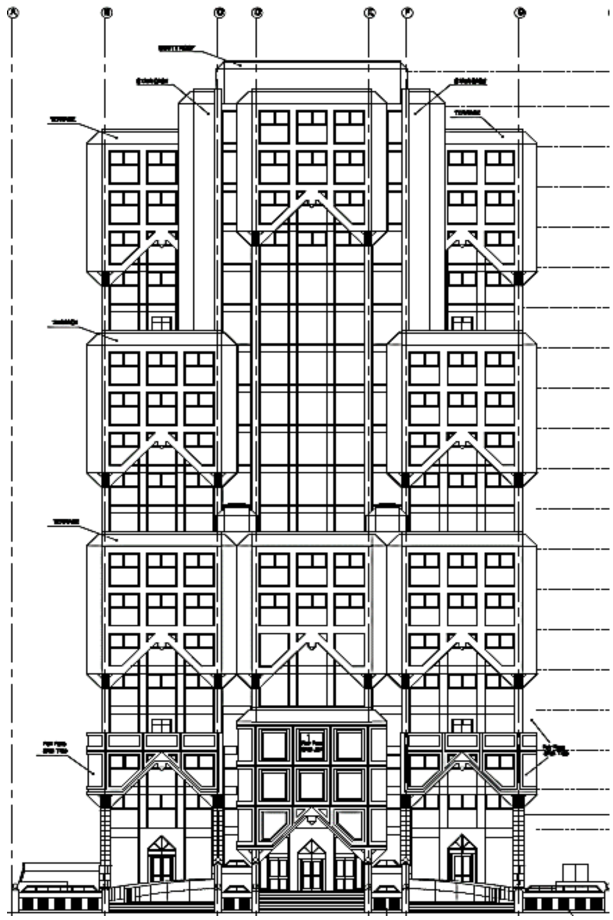

Figure 5 – South Elevation

CHAPTER 3

METHODOLOGY - UNDERSTANDING TIER 1 EVALUATION

3.1 General

The procedures laid down by ASCE 31-03 have been chosen to carry out seismic evaluation of the chosen building i.e. State Life Building (Islamabad), because , as mentioned earlier, the standard is easier to comprehend and contains clearer guidelines compared to other standards reviewed such as FEMA 310 (which is an earlier version of the ASCE 31-03).

According to ASCE 31-03, the first stage in seismic evaluation of any building mostly involves visual screening of the structure to be evaluated for deficiencies. The deficiencies can either be structural or non-structural or in the form of site specific hazards.

3.2 Pre-screening Requirements

3.2.1 Information Collection

Before a proper screening is done, detailed information regarding the building must be gathered. The information can include records of the building's construction, structural or architectural drawings and any other supporting drawings, material records, site investigation reports, quality control records etcetera. Additionally, destructive and non-destructive tests on components of the building can be performed where possible. Any information gathered in the earliest stages of the evaluation process will aid in the execution of latter stages.

In case there is a dearth of information regarding the building available, ASCE 31-03 allows for assumptions to be made of the characteristics of the building's components e.g. the strength of concrete must be assumed as 2000 psi in absence of any information regarding the same.

3.2.2 Site Visit

For the visual screening process, a site visit must be carried out. The site visit can occur any number of times. Along with visual screening of the building, the purpose of the site visit is to collect data or information regarding the building and its location as well as to verify already collected data or information. In ASCE 31-03, topic numbered 2.3 provides a brief description of what the site visit entails.

3.2.3 Level of Performance

Before starting the evaluation process using ASCE 31-03, one is required to set the desired level of performance for the building. ASCE 31-03 has designated Life Safety (LS) and Immediate Occupancy (IO) as the two levels of performance from which a selection must be made.

3.2.4 Level of Seismicity

The level of seismicity of the site in which the building is located must be identified. ASCE 31-03 provides definition for levels of seismicity based on spectral acceleration and site amplification factors in its table 2-1. Three levels of seismicity are defined in ASCE 31-03, namely, low, moderate and high. The distinguished by the range of S_{DS} and S_{D1} values which are parameters of the adjusted design spectral acceleration (spectral acceleration produced by Design Earthquake or Maximum Considered Earthquake, usually, an earthquake with a 2% probability of occurrence in 50 years, adjusted to the site under investigation). Figure 6 shows the equations for calculation of the aforementioned parameters whereas Figure 7 explains the quantities mentioned in the equation. Both figures have been reproduced from ASCE 31-03.

$$S_{D1} = \frac{2}{3} F_v S_1$$

$$S_{DS} = \frac{2}{3} F_a S_s$$

Figure 6 – Equations for computation of Site Adjusted Spectral Acceleration Parameters

T	=	Fundamental period of vibration of the building, calculated in accordance with Section 3.5.2.4
S_s and S_1	=	Short period response acceleration and spectral response acceleration at a one-second period, respectively, for the Maximum Considered Earthquake (MCE) obtained (ASCE 7-02)
F_v and F_a	=	Site coefficients determined from Tables 3-5 and 3-6, respectively, based on the site class and the values of the response acceleration parameters S_s and S_1. The site class of the building shall be defined as one of the following:

- **Class A:** Hard rock with measured shear wave velocity, $\bar{v}_s > 5{,}000$ ft/sec
- **Class B:** Rock with 2,500 ft/sec $< \bar{v}_s <$ 5,000 ft/sec
- **Class C:** Very dense soil and soft rock with 1,200 ft/sec $< \bar{v}_s <$ 2,500 ft/sec or with either standard blow count $\bar{N} > 50$ or undrained shear strength $\bar{s}_u >$ 2,000 psf
- **Class D:** Stiff soil with 600 ft/sec $< \bar{v}_s <$ 1,200 ft/sec or with $15 < \bar{N} < 50$ or 1,000 psf $< \bar{s}_u <$ 2,000 psf
- **Class E:** Any profile with more than 10 feet of soft clay defined as soil with plasticity index $PI > 20$, or water content $w > 40$ percent, and $\bar{s}_u <$ 500 psf or a soil profile with $\bar{v}_s <$ 600 ft/sec
- **Class F:** Soils requiring a site-specific geotechnical investigation and dynamic site response analyses:
  - Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils; quick, highly sensitive clays; collapsible, weakly cemented soils
  - Peats and/or highly organic clays ($H >$ 10 feet of peat and/or highly organic clay; where H = thickness of soil)
  - Very high plasticity clays ($H >$ 25 feet with $PI >$ 75 percent)
  - Very thick soft/medium stiff clays ($H >$ 120 feet)

Figure 7 Explanation of quantities used in Equation shown in Figure 6

3.2.5 Building Type

ASCE 31-03 has defined certain building types for which screening checklists have been specifically developed. The building types are outlined in Table 2-2 of ASCE 31-03. The screening process will become easier if the building to be evaluated can conform to any of the types described by ASCE 31-03.

3.2.6 Benchmark Building

ASCE 31-03 states that for buildings constructed under provisions of codes listed in Table 3-1 of its chapter 3.0, a structural seismic evaluation is not necessary. Buildings conforming to the criteria given in Table 3-1 are called as benchmark buildings and while their structural seismic evaluation may not be necessary, the effects of the building's non-structural components and its foundation on its seismic strength cannot be ignored and an evaluation of the same will thus be required.

3.3 Screening

3.3.1 Checklists

ASCE 31-03 provides checklists and relevant criteria for screening of the building in its section 3.3. These checklists have been prepared keeping in view the level of seismicity, level of performance and building type. Along with the checklists, criteria for the level of evaluation required after Tier 1 for a given building type and characteristics are also given in Table 3-3. The checklists are provided for identification of structural, non-structural and site specific deficiencies and hazards. The most important of these is the structural checklist and is mostly where deficiencies emerge. The following figure shows a breakdown of the elements or aspects that have to be usually evaluated or checked in the screening phase as part of the structural checklist:

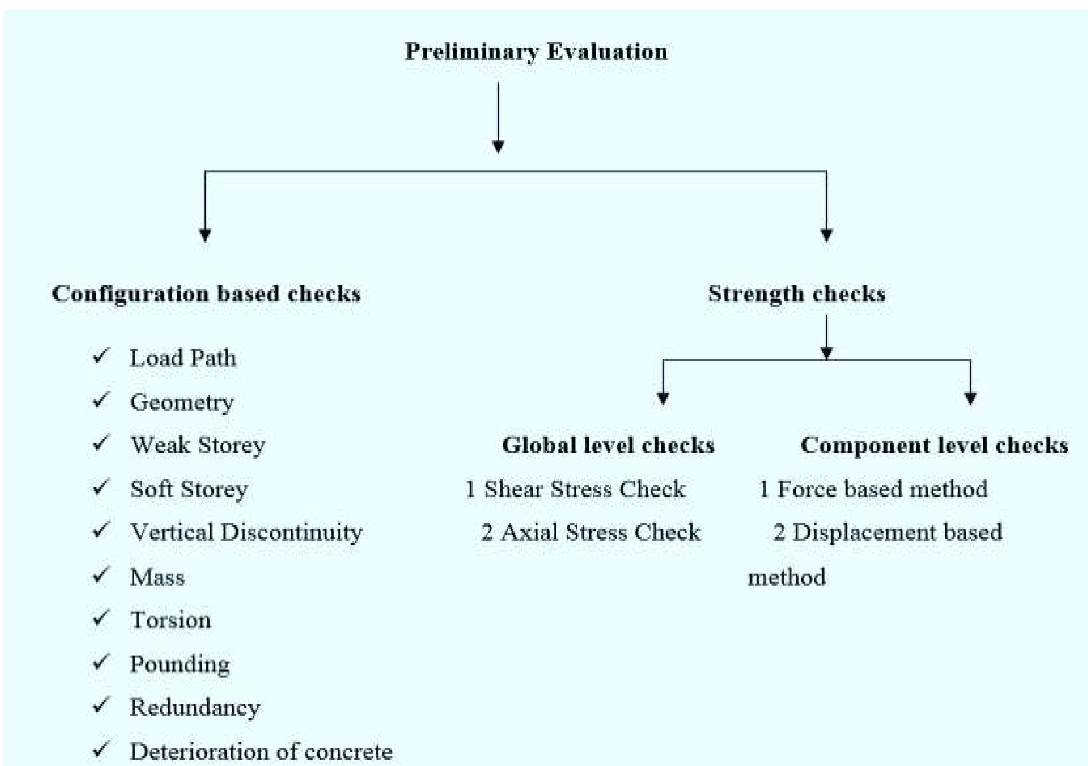

Figure 8 – Checks in screening phase

3.3.2 Description of Major Structural Checks

As shown in Figure 6, the screening checks can be divided broadly into two categories: Configuration-based (CB) checks and Strength (S) Checks. The configuration-based checks deal with the functional efficiency of a building with regards to the arrangements of components in its framework. The strength checks, as their name suggests, are checks that ensure that the forces in the components do not exceed a certain level and within the strength capability of the framework. The strength checks are further divided into global and component or local level checks. A brief description of the checks is provided in Table - 3 below. Details of computations related to some of the checks are given in ASCE 31-03's chapter 3.0.

Table 3 – Brief Description of Major Checks

#	Check	Type	Description/Compliance Criteria from ASCE 31-03
1	Load Path	CB	A building must have at least one complete and continuous load path for the transmission of seismic forces caused by acceleration of building components. A load path is deemed to be complete if it is able to transfer forces from the building's main resisting systems or other main components to the building foundation.
2	Geometry	CB	The change in horizontal dimension of the Lateral force resisting system between adjacent storeys must not be more than 30%
3	Weak Storey	CB	The strength of any lateral force resisting system in any storey must be at least 80% that of both of its adjacent storeys. If this is not the case for a storey, then it is a weak storey. Weak storeys can experience a concentration of

			stresses and as a result can undergo a partial or total collapse.
4	Mass Irregularities	CB	A difference between storey weights or effective masses is called a mass irregularity. The mass irregularity between adjacent stories must not exceed 50%.
5	Torsion	CB	Torsion in a storey is caused by the difference between its centre of mass and centre of rigidity. The difference between the centre of mass and centre of rigidity of any storey within the building must not exceed 20% of the storey's width in either plan dimensions.
6	Deterioration of Concrete or Steel reinforcement	CB	There should be no deterioration observed in the concrete or steel reinforcement in any component or member of any lateral or vertical force resisting system. (This check is for RC frame buildings)
7	Soft Storey	CB	A soft storey is marked by a significantly low stiffness of its lateral force resisting system relative to that of the lateral force resisting system of any of its adjacent storeys. A storey is called a soft storey when its lateral force resisting system stiffness is less than 70% of that of any adjacent storey or 80% of the average stiffness for any three adjacent storeys.
8	Pounding	CB	Two buildings can pound or strike each other during an earthquake due to their proximity to each other. For a building to pass this check, there must be sufficient distance between it and

			adjacent buildings. According to ASCE 31-03, that sufficient distance (clear) is equal to 4% the height of the shorter building among the two adjacent buildings.
9	Redundancy	CB	Check differs for building types. Refer to ASCE 31-03 for detail.
10	Vertical Discontinuity	CB	Vertical components in the Lateral force resisting system must be continuous till the foundation.
11	Shear Stress	S (Global)	The shear stress in columns of concrete must not exceed the larger of 100 psi or 2(f'_c) ^0.5.
12	Axial Stress	S (Global)	Sometimes columns are already carrying an excessive amount of axial forces that they cannot bear additional loading from seismic forces. The Axial Stress check states that the: • The axial stress in columns brought on by gravity loads and overturning forces must not exceed 0.10 f'_c ; **or** • The axial stress brought on by overturning forces alone must not exceed 0.30 f'_c.
13	Force Based Method and Deformation Based	S (Component)	These checks are applied to determine whether the components of a building's structural framework have the strength to withstand the gravity and earthquake forces and the resulting deformations and subsequent changes in force actions.

3.3.3 Computation of Pseudo Lateral Force

The pseudo lateral force needs to be computed as part of Tier 1 evaluation to be used in calculations related to the aforementioned checks. The pseudo lateral force is imagined as the force that is applied to a building during the design earthquake and will be used in Tier 2 evaluation to determine the subsequent "actual" or design displacements of building. The pseudo lateral force is computed through the equation shown in Figure 9. Additionally a description of the quantities used in the equation is shown in Figure 10. Both figures are reproductions from ASCE 31-03. Further details of the equation and its usage are provided in ASCE 31-03's chapter 3.0.

$$\boxed{V = CS_aW}$$

Figure 9 – Equation for computation of pseudo lateral force

V = Pseudo lateral force.
C = Modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response; C shall be taken from Table 3-4.
S_a = Response spectral acceleration at the fundamental period of the building in the direction under consideration. The value of S_a shall be calculated in accordance with the procedures in Section 3.5.2.3.
W = Effective seismic weight of the building including the total dead load and applicable portions of other gravity loads listed below:

1. In areas used for storage, a minimum of 25 percent of the floor live load shall be applicable. The live load shall be permitted to be reduced for tributary area as approved by the code official. Floor live load in public garages and open parking structures need not be considered.
2. Where an allowance for partition load is included in the floor load design, the actual partition weight or a minimum weight of 10 psf of floor area, whichever is greater, shall be applied.
3. Total operating weight of permanent equipment.
4. Where the design flat roof snow load calculated in accordance with ASCE 7-02 exceeds 30 psf, the effective snow load shall be taken as 20 percent of the design snow load. Where the design flat roof snow load is 30 psf or less, the effective snow load shall be permitted to be zero.

Figure 10 – Description of quantities shown in Figure 9

3.3.4 Computation of Fundamental Period of building

The Fundamental period of a building is the time it takes for the building to complete one cycle of free vibration under the action of forces such as seismic forces. The fundamental time period of the building can be calculated in accordance with the equation shown in Figure 11 along with additional relevant description provided in Figure 12.

$$T = C_t h_n^{\beta}$$

Figure 11 – Equation for calculation of Fundamental Period of building

T	=	Fundamental period (in seconds) in the direction under consideration
C_t	=	0.060 for wood buildings (Building Types W1, W1A, and W2)
	=	0.035 for moment-resisting frame systems of steel (Building Types S1 and S1A)
	=	0.030 for moment-resisting frames of reinforced concrete (Building Type C1)
	=	0.030 for eccentrically braced steel frames (Building Types S2 and S2A)
	=	0.020 for all other framing systems
h_n	=	height (in feet) above the base to the roof level
β	=	0.80 for moment-resisting frame systems of steel (Building Types S1 and S1A)
	=	0.90 for moment-resisting frame systems of reinforced concrete (Building Type C1)
	=	0.75 for all other framing systems

Figure 12 – Offers a description of the quantities found in the equation shown in Figure 11

The fundamental time period of a building can also be determined through an eigenvalue or dynamic analysis of the mathematical model of the building. The fundamental time period will be used in further computations and analysis for seismic evaluations.

3.4 Screening Aftermath

Any deficiencies identified in the screening phase will be noted down. The presence of deficiencies means the building is seismically vulnerable but this cannot be said with certainty. The next step, if one chooses to proceed with it, will be a Tier 2 evaluation in which a fairly detailed qualitative and quantitative analysis of the building's seismic strength will be carried. A Tier 2 evaluation can be limited to the deficient components identified in the screening or can be done for the whole building. Alternatively, one can choose to address the identified deficiencies without performing any further analysis.

ASCE 31-03 provides criteria for buildings for which a Tier 2 evaluation is strongly recommended in its chapter 3.0. Tier 2 evaluation is explained in the next chapter.

Another option available at this stage is application of a Tier 3 evaluation. There are significant differences in the techniques used in a Tier 2 and Tier 3 evaluation, but the basic purpose of both evaluation phases is the same i.e. to verify whether a building is strong enough to resist a considerable level of seismic activity as indicated by the results of a Tier 1 evaluation.

CHAPTER 4

METHODOLOGY - UNDERSTANDING TIER 2 EVALUATION

4.1 General

Compared to Tier 1 evaluation, a Tier 2 evaluation is a detailed and mostly quantitative analysis of the building's Seismic Strength. Data collected as part of screening phase will be used in the Tier 2 evaluation extensively.

According to ASCE 31-03, a Tier 2 evaluation will involve the use of a linear analysis method to determine the seismic strength of the building. ASCE 31-03 lists the following procedures for a Tier 2 evaluation:

- Linear Static Procedure (LSP)
- Linear Dynamic Procedure (LDP)
- Special Procedure
- Procedures for non-structural components

For a Tier 2 evaluation in most buildings, either the LSP or the LDP is followed. The LSP can be applied to any building but for buildings having a height exceeding 100 ft and major mass, stiffness and geometric irregularities (identified in Tier 1), the LDP is recommended.

The special procedure applies to buildings constructed with unreinforced masonry and containing flexible diaphragms.

ASCE 31-03 lists down procedures for evaluating contribution of non-structural building elements or components to the seismic vulnerability of a building in section 4.8 of its chapter 4.0.

4.2 Linear Static Procedure (LSP)

4.2.1 General

The LSP is applicable to most types of buildings but is likely to give inaccurate results for buildings with heights greater than 100 ft or with major structural deficiencies. In application of the LSP, it is assumed that a building responds elastically to the application of seismic forces (design earthquake; 10% probability in 50 years). This, however, is not the case in reality as buildings tend to respond inelastically to seismic forces. Due to this assumption, LSP provides overestimations of the forces that will develop in the building's framework as a result of the application of seismic forces. In LSP, the loads are applied gradually to the building until they reach their peak and then do not vary with time. There are no considerations for damping or inertial forces in LSP. Table 4 provides an outline of the steps involved in the LSP.

Table 4 – Steps of the LSP

Step	Description	ASCE 31-03 Reference
1	Development of a 2D or 3D mathematical model of the building	Section 4.2.3
2	Calculation of Pseudo Lateral Force (Tier 1)	Section 4.2.2.1.1
3	Calculation of lateral forces (usually pseudo lateral force) to be distributed along the building's height	Section 4.2.2.1.2
4	Use of a Linear elastic method for calculation of the forces (component forces) developed in the building's framework	-
5	If required, compute the diaphragm forces	Section 4.2.2.1.4
6	Compare component Nominal and Expected strength to applied loads	Section 4.2.4.5

4.2.2 Distribution of Pseudo Lateral Force along building height

As part of the LSP, the Pseudo Lateral Force must be distributed along the height of the building being evaluated. The equations shown in Figure 13, along with supporting information shown in Figure 14, must be used to accomplish this. This step must be carried out for use in linear elastic method of calculation for component forces.

$$F_x = C_{vx} V$$

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

Figure 13 – Equations for vertical distribution of Pseudo Lateral Force

where:

k	=	1.0 for $T < 0.5$ second
	=	2.0 for $T > 2.5$ seconds (linear interpolation shall be used for intermediate values of k)
C_{vx}	=	Vertical distribution factor at floor level x
V	=	Pseudo lateral force (Section 4.2.2.1.1)
w_i	=	Portion of the total building weight W located on or assigned to floor level i
w_x	=	Portion of the total building weight W located on or assigned to floor level x
h_i	=	Height (ft) from the base to floor level i
h_x	=	Height (ft) from the base to floor level x

Figure 14 – Explanation of Quantities used in equations shown in Figure 13

4.3 Linear Dynamic Procedure (LDP)

As stated above, the Linear Dynamic Procedure or LDP is applicable when:

1- Building height exceeds 100 ft, or
2- Building has significant irregularities in mass, geometry and stiffness

Unlike LSP, the applied loads on the building in LDP vary with time and thus are time dependent. The inertial and damping forces are given significant consideration in the LDP.

Like the LSP, it is assumed in the LDP that the building behaves elastically in response to an earthquake.

The following table provides an outline of the steps involved in the LDP.

Table 5 – Steps in LDP

Step	Description	ASCE 31-03 Reference
1	Develop a 2D or 3D mathematical model of the building	Section 4.2.3
2	Develop a site specific response spectrum (A plot of the steady state response of a series of oscillators with changing natural frequency that have been excited by the same base vibration).	Section 4.2.2.2.2
3	Execute a response spectrum analysis (a linear dynamic statistical analysis method that measures the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an essentially elastic structure for the building	-
4	Modify deformations and actions	Section 4.2.2.2.3
5	If required, compute the diaphragm forces	Section 4.2.2.4

6	Compute component forces or actions	Section 4.2.4.3
7	Compare component forces to component demand	Section 4.2.4.5

4.4 Tier 2 Aftermath

If the building is found to be deficient in seismic strength as a result of a Tier 2 evaluation, the building must be retrofitted using a suitable retrofit strategy. If one is not satisfied with the accuracy of results of Tier 2 and finds the results to be conservative, a Tier 3 evaluation, which is more detailed and complex and will likely allow a retrofit strategy to be determined more easily, can be performed. Explanation of Tier 3 evaluation is given in the next chapter.

CHAPTER 5

METHODOLOGY - UNDERSTANDING TIER 3 EVALUATION

5.1 General

After completing a Tier 1 evaluation, one can opt for a number of quantitative analysis methods for determination of seismic strength or vulnerability of a building. Both Tier 2 and Tier 3 evaluations involve the use of those methods. A Tier 3 evaluation involves the use of complex analysis methods (compared to those in Tier 2) to determine seismic strength of a building.

A Tier 3 evaluation must be carried out with caution since it is very detailed and requires the consideration of multiple provisions and codes developed for seismic design. A Tier 3 evaluation is usually performed using non-linear static or linear dynamic methods. For more accuracy, one can opt for non-linear dynamic analysis methods as well.

In Tier 3 evaluation, a building must be evaluated by a linear dynamic, non-linear static or non-linear dynamic analysis method if it has one or more of the characteristics described in Figure 15.

- The fundamental period of the building, T, is greater than or equal to 3.5 times S_{D1}/S_{DS}.
- The ratio of the horizontal dimension at any story to the corresponding dimension at an adjacent story exceeds 1.4 (excluding penthouses and mezzanines).
- The building has a torsional stiffness irregularity in any story. A torsional stiffness irregularity exists in a story if the diaphragm above the story under consideration is not flexible and the results of the analysis indicate that the drift along any side of the structure is more than 150 percent of the average story drift.
- The building has a vertical mass or stiffness irregularity. A vertical mass or stiffness irregularity exists when the average drift in any story (except penthouses) exceeds that of the story above or below by more than 150 percent.
- The building has a non-orthogonal lateral-force-resisting system.

Figure 15 – Characteristics for which application of a complex analysis method becomes necessary (Tier 3 evaluation)

Considering the scope of this project and the relevant capacity of the authors, a Tier 3 evaluation carried out will involve the use of a common analysis method. The most commonly used method of analysis in a Tier 3 evaluation is the Non-linear Static or Pushover analysis method. A description of the method is provided in the next few sections.

5.2 Nonlinear Static or Pushover Analysis

5.2.1 General Description

The nonlinear static analysis (Pushover analysis) in the recent years is becoming a popular method of predicting seismic forces and deformation demands for the purpose of performance evaluation of existing and new structures. The nonlinear analysis of a structure is an iterative procedure. It depends on the final displacement, as the effective damping depends on the hysteretic energy loss due to inelastic deformations, which in turn depends on the final displacement. This makes the analysis procedure iterative.

Pushover is a static-nonlinear analysis method where a structure is subjected to gravity loading and a monotonic displacement-controlled lateral load pattern which continuously increases through elastic and inelastic behavior until an ultimate condition is reached. Lateral load may represent the range of base shear induced by earthquake loading, and its configuration may be proportional to the distribution of mass along building height, mode shapes, or another practical means.

Pushover analysis is nonlinear static analysis which provides 'capacity curve' of the structure, it is a plot of total base force vs. roof displacement. The analysis is carried out up to failure, it helps determination of collapse load and ductility capacity of the structure. The pushover analysis is a method to observe the successive damage state of the building.

Pushover analysis is a useful tool of Performance Based Seismic Engineering to study post-yield behavior of a structure. It is more complex than traditional linear analysis, but it requires less effort and deals with much less amount of data than a non-linear response history analysis. Pushover analysis results in set of values of base or storey shear and corresponding roof displacement or drift.

5.2.1 Implementation of Pushover Analysis

The Pushover process is to construct an analytical model, apply gravity loads, lateral loads and push the structure under these load patterns to targeted displacements. These deformations and forces at the target displacements are used to determine the strength and deformation demands.

Alternatively, development of a backbone curve and a comparison of the storey shear values computed against the design or maximum considered earthquake can give an insight into the adequacy of a building's strength under significant seismic loading.

Some points that should be emphasized in performance evaluation are as follows:

- A proper load path exists Load path is sound even at deformations.
- Individual elements are not overloaded
- Localized failures should not pose a safety hazard

5.2.2 Further Description of Pushover Analysis

Pushover analysis can be performed as force-controlled or displacement-controlled. Force-controlled is used when the load is known (such as gravity loading). In displacement-controlled procedure is used where the magnitude of applied load is not known in advance. The load is increased until the control displacement reaches a specified value. Generally, roof displacement at the center of mass of structure is chosen as the control displacement. The internal forces and deformations at the target displacement give inelastic strength and deformation demands which is compared with available capacities to find a performance point.

Available simplified conventional nonlinear static procedures are as follow:

- **Capacity Spectrum Method**, that uses intersection of capacity (pushover) curve and a reduced response spectrum in spectral coordinates (Acceleration Displacement Response Spectrum Format) to find a performance point. The specifications of this method are covered in ATC40.
- **Displacement Coefficient Method** described in FEMA-356 is a non-iterative approximate procedure based on displacement modification factors. The expected maximum inelastic displacement of nonlinear MDOF system is obtained by modifying the elastic spectral displacement of an equivalent SDOF system with a series of coefficients. The procedure proposed by Newmark and Hall is based on the

estimation of inelastic response spectra from elastic response spectra while displacement modification factor varies depending on the spectral region.

5.2.3 Limitations of Pushover analysis

For the structures vibrating in fundamental mode, pushover analysis provides good estimate of local and global inelastic deformation demands. With all the advantages of the pushover analysis there are some inherent limitations of the procedure, which are:

- It is approximate analysis and is based on static loading, therefore cannot represent dynamic phenomena accurately. It detects the fundamental mode and not all the modes resulting by seismic activity.
- Certain deformations are favored by selecting a load pattern which results in some other modes being neglected thus good judgment is required in selecting load patterns and in interpreting the obtained results.

5.2.4 Why Pushover Analysis over Nonlinear Dynamic Analysis

Some of the reasons why Pushover analysis should be preferred to a full scale Non-linear dynamic analysis are as follows:

- A Non-linear dynamic analysis takes a long time to run even for a simple structure, whereas the Pushover analysis can give accurate results within a fraction of time it would take to perform a Non-linear dynamic analysis.
- For obtaining accurate results via performing a Non-linear dynamic analysis a series of earthquake cases should be used, whereas the Pushover Analysis naturally accounts for all earthquakes with the same probability of exceedance by predicting the maximum displacement that can be expected in the form of the Target Displacement.

CHAPTER 6

METHODOLOGY - RETROFITTING

6.1 General

If building is found to be deficient in either Tier 2 or Tier 3, it must be retrofitted to protect it against the kind of seismic activity against which the evaluation was carried out. Retrofitting can be done in many ways but a good retrofit strategy or technique must have the following characteristics:

- Must be suitable to the building characteristics and seismic demands
- Must be economical with respect to implementation

6.2 Common seismic strengthening or retrofitting techniques used in Pakistan

6.2.1 Shear walls

Shear wall is a structural system comprised of braced panels which are also known as shear panels. They are like vertically-oriented wide beams that carry the wind and earthquake loads downwards to the foundation. They resist the lateral in-plane loading along the height of a structure caused by wind and seismic lateral forces. They generally start from the foundation and extend till the top of the structure/building. According to International building code and Uniform Building code, all exterior walls of a structure made from wood or steel frame construction must be braced. Shear walls are designed to stop the lateral sway of a building and to carry lateral loads while columns are used to carry the gravity loads. They provide large strength and stiffness to the building in the direction of their orientation. The placement of shear walls is done in both the length and width of the building. It may be constructed externally and/or internally. Alternatively, a shear core can also be constructed to resist the lateral forces developed by wind and seismic activity. A shear core is the encasing of shear walls around an elevator shaft or stairwell. Shear walls are mostly connected to the foundations of the buildings but in cases where the lateral loads are less and the dynamic effects are not appreciable, they can be supported on columns, connected by a transfer beam to provide clear space. Shear walls are placed in a building keeping in mind the symmetry in the building's plan (if any exists), the centre of mass of the building and its centre of rigidity.

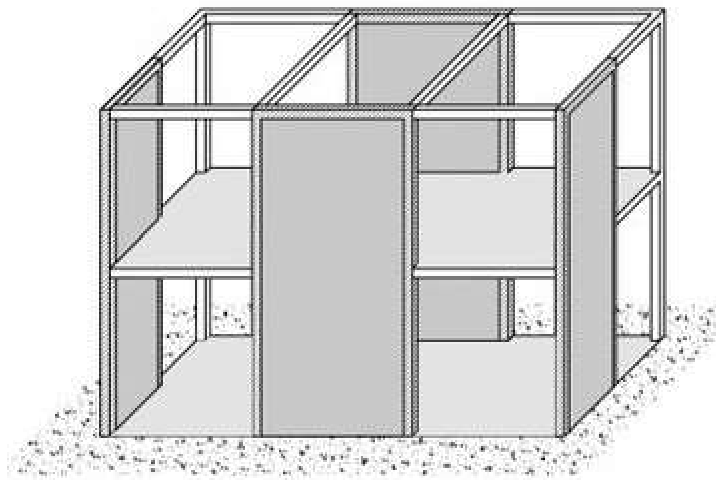

Figure 16 Shear walls in a two-storey frame structure

6.2.2 Bracing

Bracing is one of the most common ways of enhancing the global stiffness of a building. It is mostly applicable in buildings with a frame structure. Bracings can increase the absorption of energy that is transmitted to the structure under the action of seismic loads. Bracings can be applied in different orientations and manners and can be made of wood, concrete, steel or other similar materials. They are very costly to implement.

P.T.O

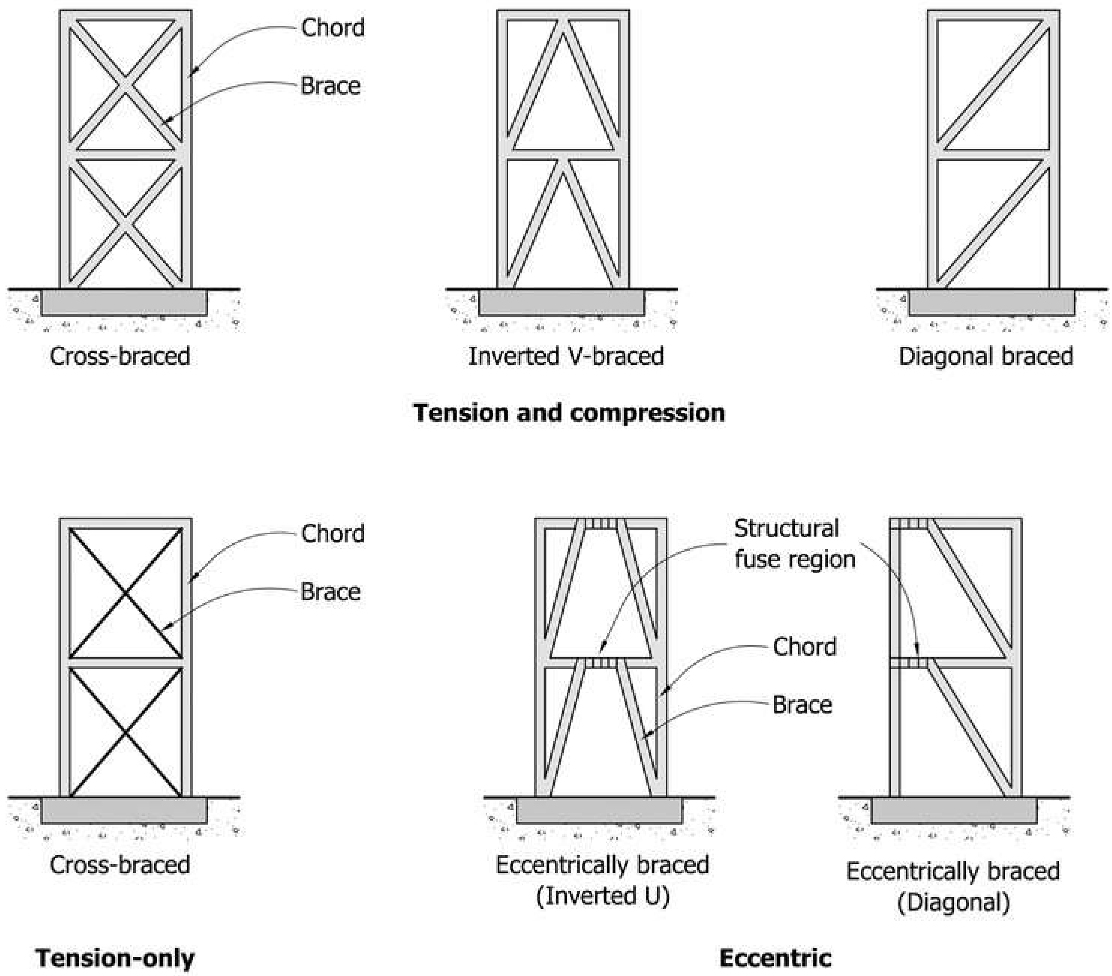

Figure 17 – Different types of bracings

CHAPTER 7

Methodology - Modelling & Analysis in PERFORM 3D

7.1 Modelling

Compared to design software such as SAP2000 and ETABS, modeling in PERFORM 3D is very difficult as its user interface is relatively less user-friendly and takes a lot of time to comprehend.

The main steps in modeling on PERFORM 3D are:

- Set to modeling phase
- Create elements and nodes (these are joints between the elements)
- Define restraint conditions. (hinges must be defined for Pushover analysis)
- Define component and material properties
- Assign component properties to the elements

Figure 18 shows an elevation of the model developed in PERFORM 3D.

P.T.O

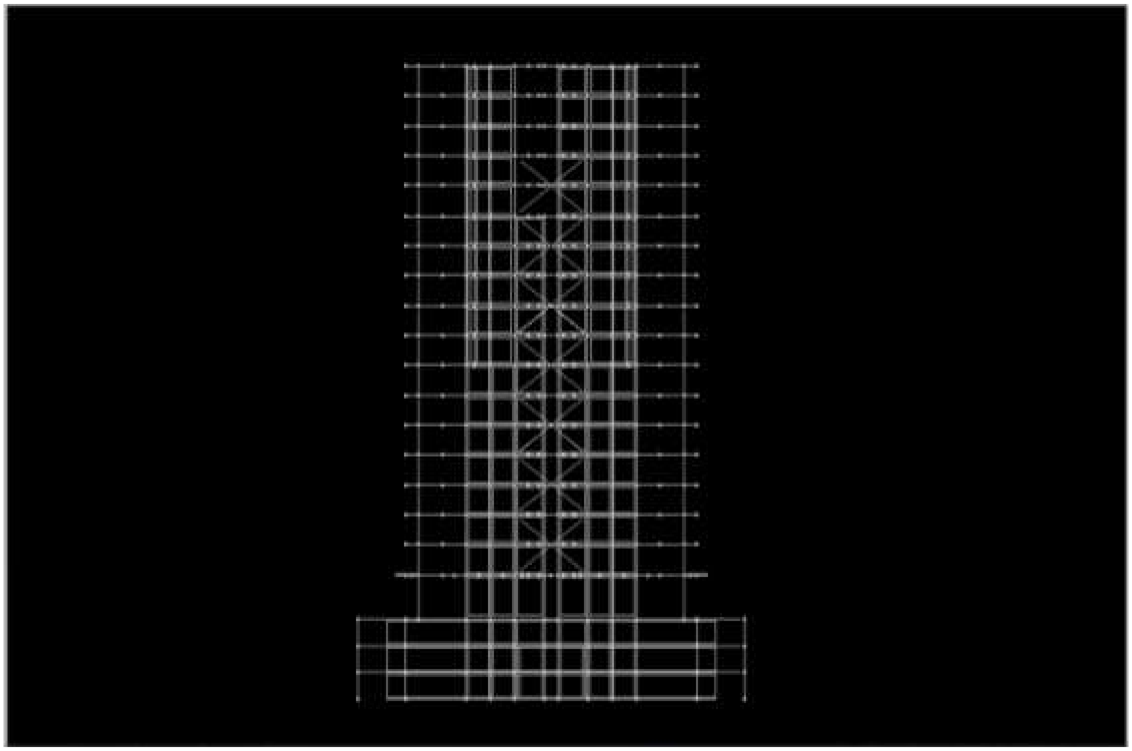

Figure 18 – The State Life Building modelled in PERFORM 3D

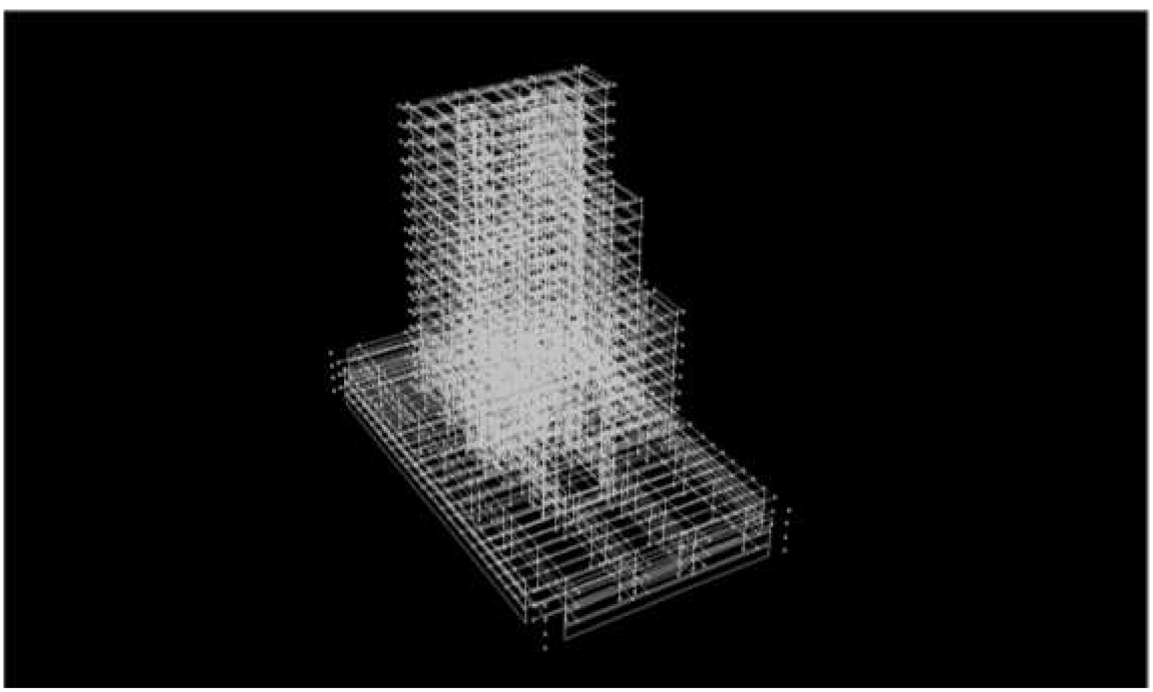

Figure 19 – State Life Building model in 3D view

7.2 Pushover Analysis

For running any analysis, load cases and relevant conditions must be defined with respect to the analysis type after adjusting the window to analysis phase.

The steps involved in Pushover Analysis using PERFORM 3D are:

- Define reference drift
- Define Gravity Load case as a non-linear case.
- Define Pushover cases. The first pushover case defined will be set to accelerate building in the positive direction for a target or maximum displacement of 1%. The second pushover case will be set to accelerate the building in the opposite direction to the first and for a maximum displacement of 1.5%. The third case will be similar to the first but with a maximum displacement of 2%. All subsequent cases will be developed in this pattern of changing directions and displacements ending with a case for a maximum displacement of 10% (max. allowed in PERFORM 3D) being defined. The definition of pushover cases in this way is done to simulate the vibratory motion of the building during an earthquake.
- Shift to the analysis phase and define the sequence in which the load cases defined above will be run (Gravity loads followed by the pushover cases with increasing displacement and changing directions). Run the analysis once the application sequence of load cases is defined.
- After analysis is complete, save roof drift plot data under time histories panel to your computer and then import the plot data into Microsoft Excel (MS-Excel).
- Use MS-Excel or any dedicated program to create a hysteresis loop (a combined plot of the response (force vs drift) of the structure to each pushover case) and plot the backbone or capacity curve for the structure by joining the positive peak points of the hysteresis loop.
- Compare maximum story shear computed in Tier 1 for expected maximum PGA or Maximum considered earthquake (Appendix B) with the story shear at failure shown in the capacity curve. The peak of the backbone curve represents the point at which the maximum story shear occurs and the point after which a global

structural failure in the building is set to occur. If the computed story shear is more than the maximum curve story shear then the structure is not likely to withstand the maximum considered ground motion or design earthquake in Tier 1 and retrofitting will be required.

CHAPTER 8

Analysis & Results

8.1 Tier 1 Evaluation

Table 6 – General Determinations

General Considerations	
Level Of Performance	Life Safety
Level Of Seismicity	Moderate
Building Type	C1 – Concrete Moment Frame
Benchmark Building	Not Compliant
Design Earthquake	Probability Of Occurrence 10 % In 50 Years

Refer to Appendix B for computations related to Level of Seismicity.

Table 7 – Checklists

Checklists	
Structural	Refer To Table
Non-Structural	Mostly not applicable, but compliant where applicable
Site Specific Standards And Hazards	All Compliant

P.T.O

Table 8 – Structural Checklist Results

S-No	Property	COMPLIANT (C) / NON-COMPLIANT (NC) / NOT APPLICABLE (NA)
1	Load Path	C
2	Adjacent Buildings	C
3	Mezzanines	C
4	Weak Story	C (visual check)
5	Soft Story	NC (visual check)
6	Geometry	C
7	Vertical Discontinuities	C
8	Mass Discontinuities	C
9	Torsion	NC (model analysis showed that building was undergoing slight torsion)
10	Deterioration Of Concrete	C
11	Post Tensioning Anchor	C
Lateral Force Resisting System		
1	Redundancy	C
2	Interfering Walls	C
3	Shear Stress Check	NC
4	Axial Stress Check	NC
5	Concrete Columns Connections	C

For computations relevant to properties in table 8 refer to Appendix B and ASCE 31-03, chapter 4, section 4.3.

8.2 Tier 2 Evaluation

Considering both Tier 2 and Tier 3 evaluations have similar goals and Tier 2 evaluation produces more conservative results than Tier 3, a Tier 3 evaluation is recommended.

8.3 Tier 3 Evaluation / Pushover Analysis

8.3.1 Hysteresis Loop

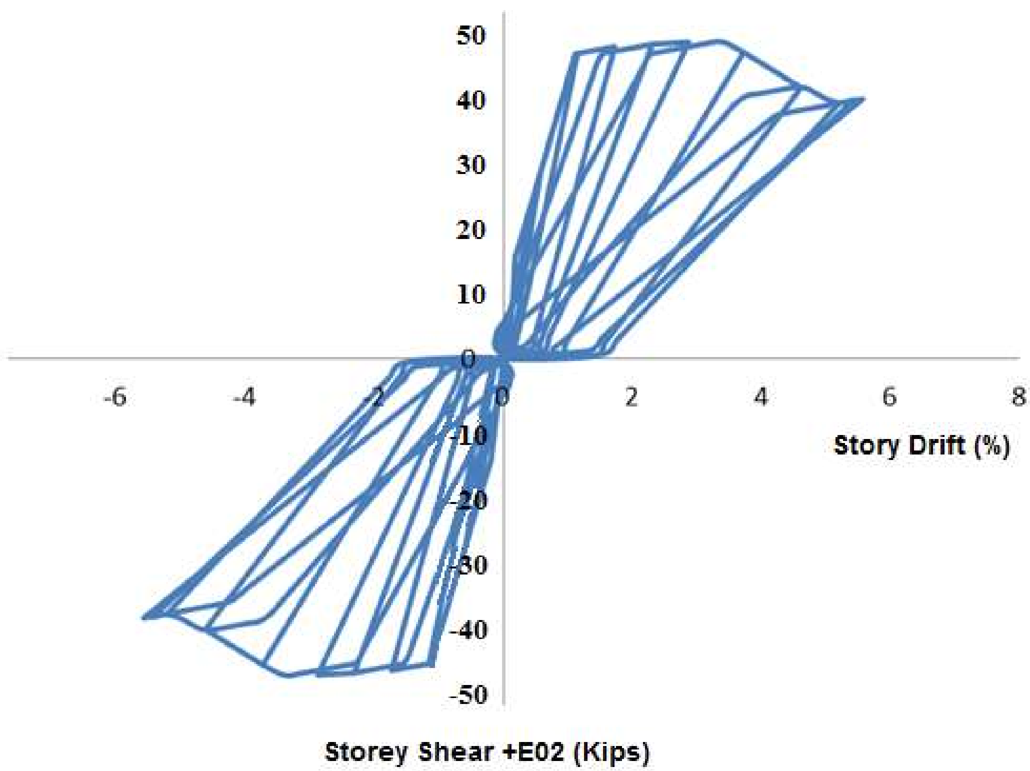

Figure 20 – Hysteresis loop developed after analysis and relevant processing

Once the Hysteresis loop is created, the positive peaks of the loop are combined to create a backbone or capacity curve.

8.3.2 Backbone curve

The backbone curve developed from joining positive peaks of the hysteresis loop is shown in Figure 21.

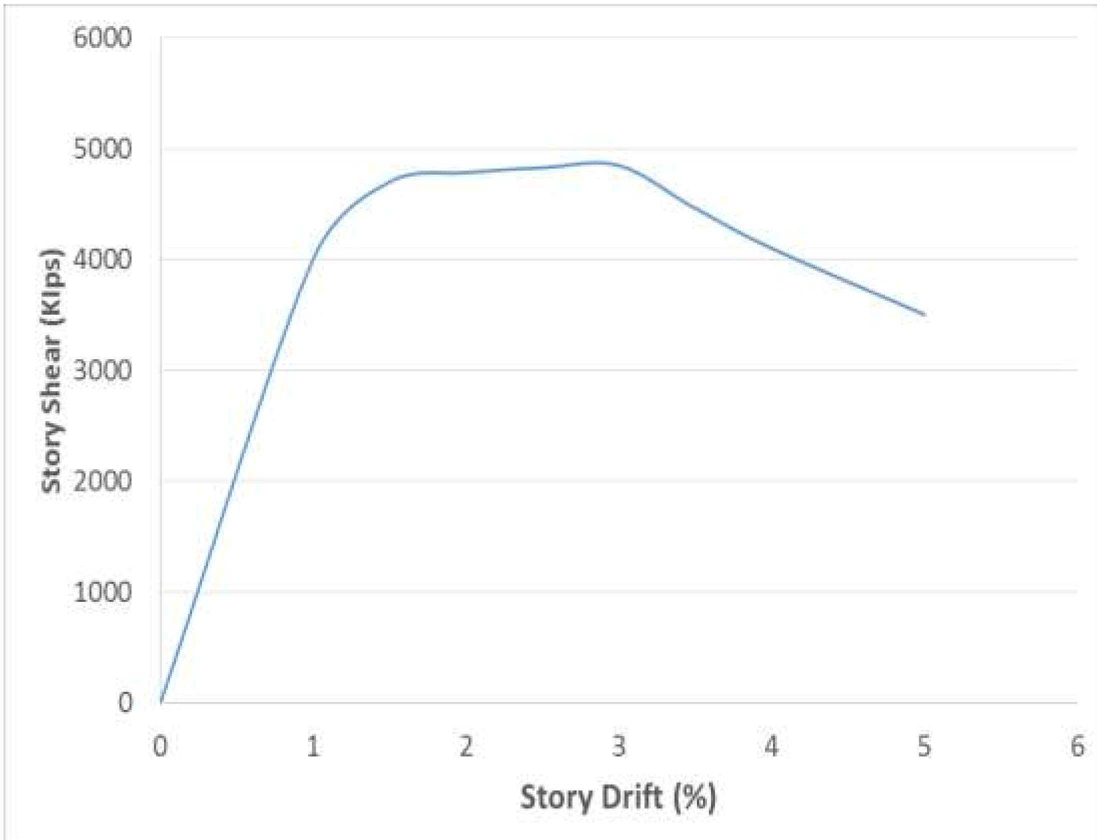

Figure 21 – Backbone curve; Peak occurs at (3, 4945)

The peak point of the backbone curve developed (Figure 21), shows that a decrease in the capacity of the building occurs after the story drift (ratio of roof drift to the drift of a collinear point on the base of the structure in this case) reaches 3% and the story shear reaches a value of nearly 5000 kips.

The maximum story shear expected for our considered PGA (Refer to Appendix B – Tier 1 Computations) is close to 4500 Kips. Considering the building is failing at a story shear of 5000 Kips which is higher than our expected story shear of 4500 Kips, the building will be able to withstand the PGA considered.

It must be noted that the building is unusually strong as it takes a large amount of story shear to produce even 1% story drift in it.

CHAPTER 9

FURTHER EVALUATIONS & SUBSEQUENT RESULTS

9.1 Shear wall evaluation – economic aspect

It has been determined in the previous chapter that the building can withstand a significant level of seismic activity and thus requires no retrofitting. It can be logically construed that the lateral force resisting systems employed in the building such as shear walls and bracing beams are responsible for preventing the building's failure under the PGA considered in Tier 1.

After obtaining the results shown in the previous chapter, a decision was made to evaluate the shear walls used in the building with respect to the economy in their usage.

The placement and the thickness of the shear walls was altered in the building's model and a pushover analysis performed using the previously defined pushover cases. A backbone curve was determined for each configuration of shear walls and compared to the one shown in Chapter 8.

Due to the complexity of the task, only 5 configurations were defined and tested. The total length of the shear walls in each case was kept constant and equal to the total length of the 12" thick shear walls used in the building.

Table 9 – Test configurations for shear walls

Configurations			
S No.	**Thickness**	**Placement (compared to that of original 12" th. Shear walls)**	**Reference Figure**
1	9"	Same	23
2	12"	Different; Exterior	24

3	9”	Different; Exterior	25
4	9”	Different; Interior + Exterior	26
5	6”	Different; Interior + Exterior	27

While defining the configurations, efforts were made to keep the placement of the walls symmetrical with respect to the building’s floor plans. Also, the walls were placed in a manner so as to be easily extended from the lowest storey to the topmost storey. Since the building was shown to undergo slight torsion during the analysis stages, the placement of the walls was made to lessen the effects of it.

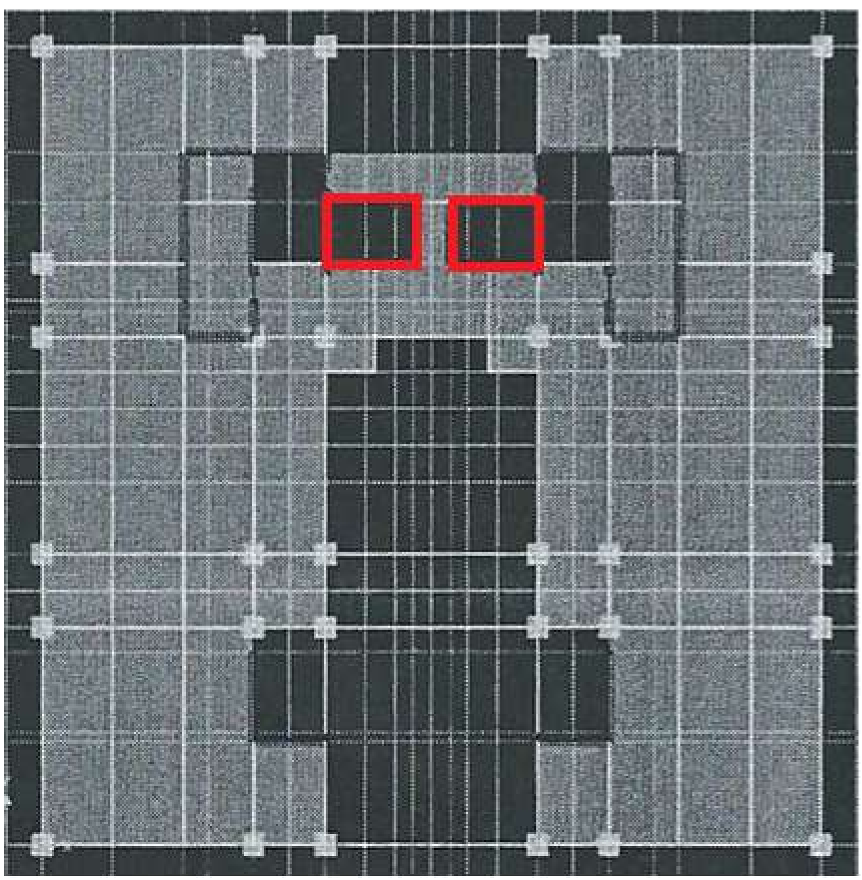

Figure 22 – Original 12" thick shear walls

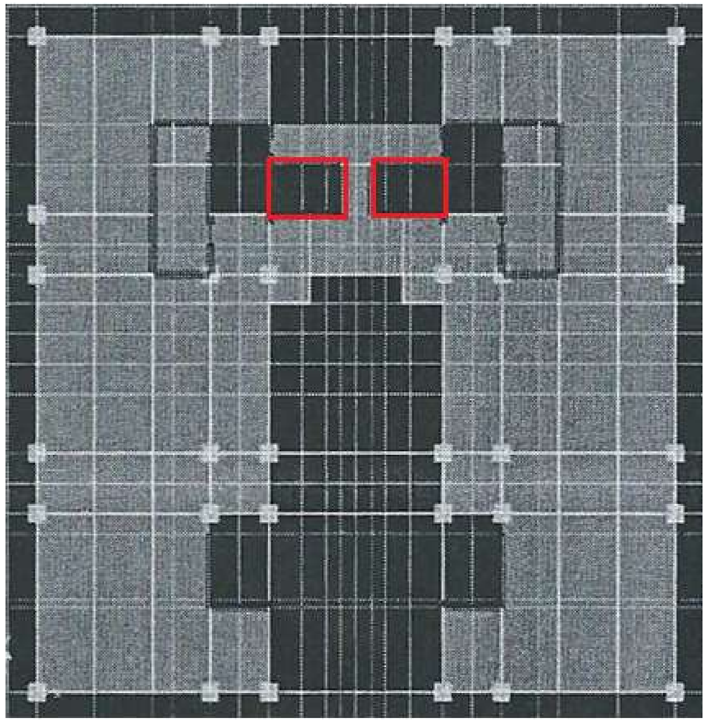

Figure 23 – Shear wall, 9" th., placed in elevator shaft

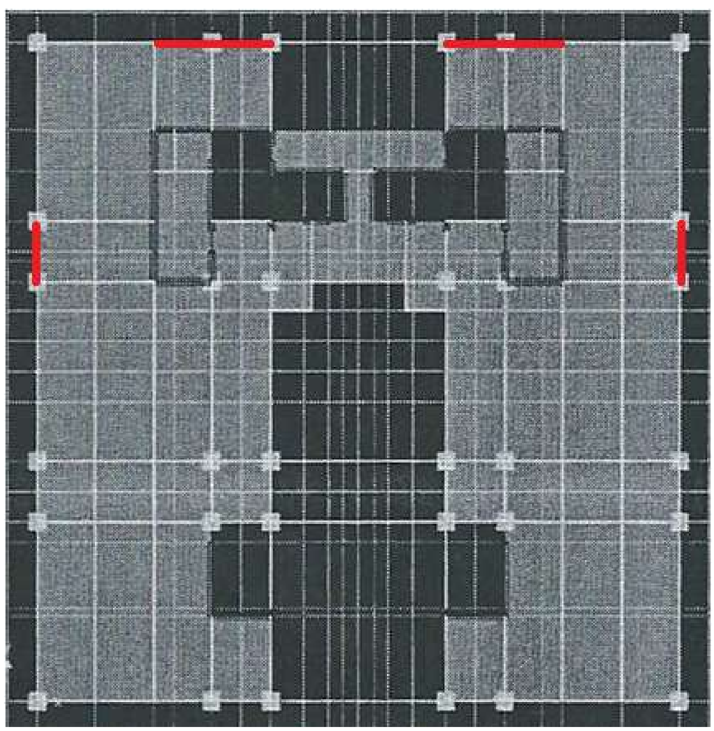

Figure 24 – 12" th. shear walls placed on building perimeter

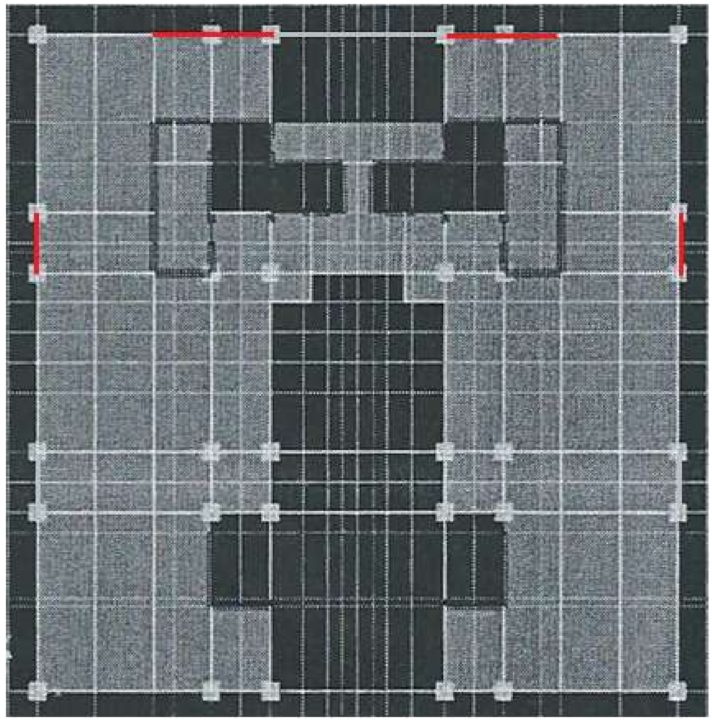

Figure 25 – 9″ th. shear walls placed on building perimeter

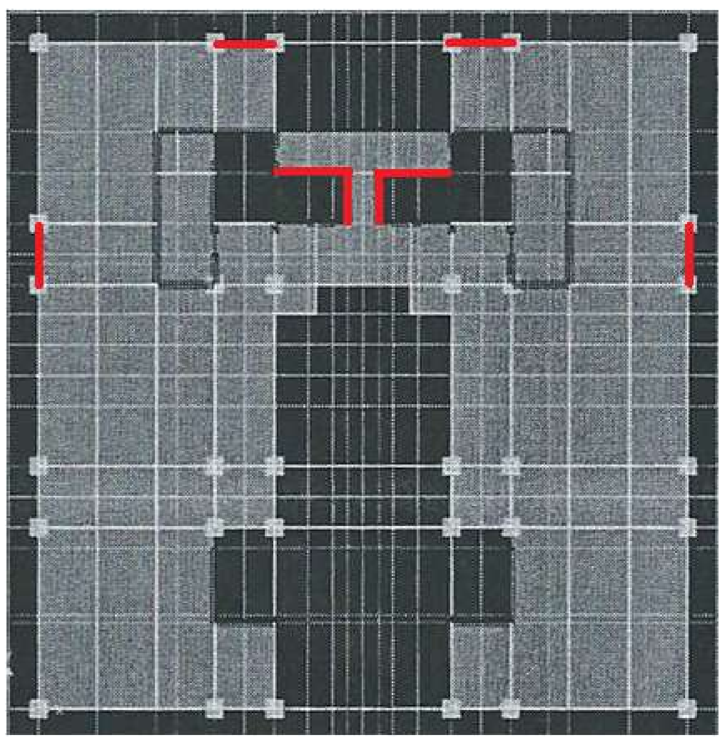

Figure 26 – 9″ th. shear walls placed on building perimeter and in part of the elevator shaft

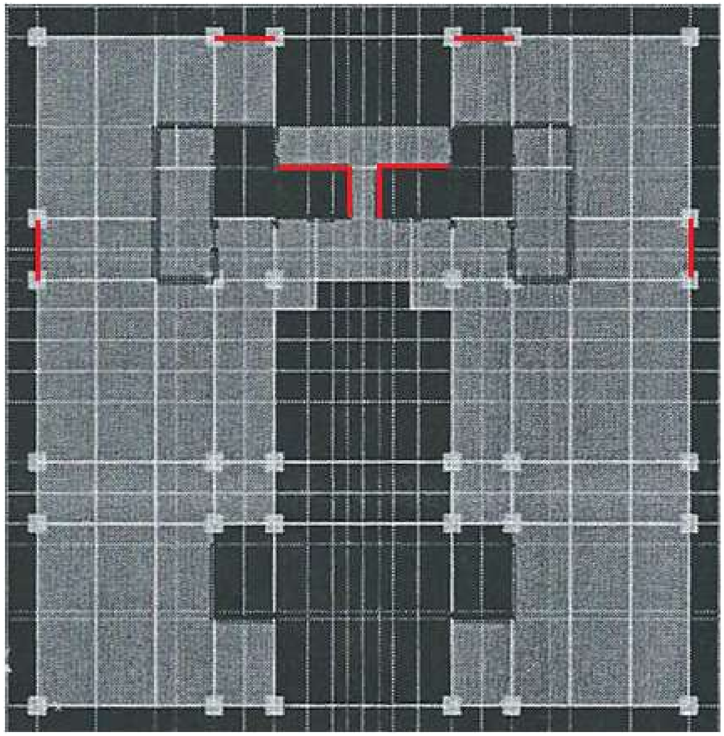

Figure 27 – 6" th. shear walls placed on the building perimeter and in part of the elevator shaft

The backbone curves developed after analysis of each of the configurations are shown in Figure 28 along with the backbone curve for the original configuration. From the comparison of the curves, it is seen that with configurations 1 and 4 (refer to Table 10) the building has the same capacity as the original configuration and so would have been more economical to use instead of the original configuration.

Although the effect of variations in story shear demands due to variations in configurations cannot be discounted, it can be logically assumed that the story shears would decrease with decrease in shear wall thickness (story weights altered).

P.T.O

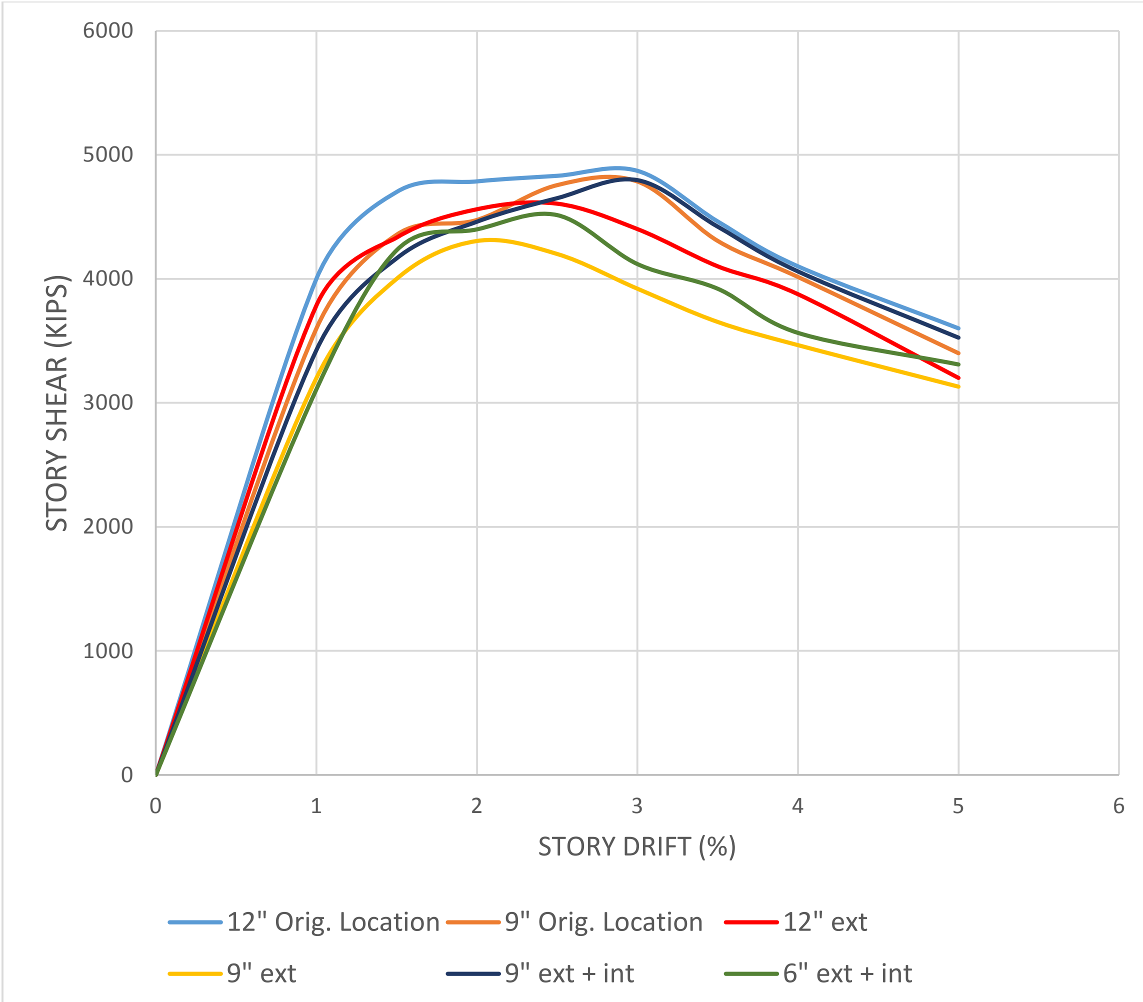

Figure 28 – Comparison of configurations in terms of backbone curves developed

9.2 Failures in basement columns

An examination of the limit states of the building components at 1.5% story drift (corresponding story shear is closest to expected maximum story shear from Tier 1 evaluation) showed that some of the interior columns in the basement had exceeded the usage ratio limit of 1 and were not also not compliant with the performance level of life safety.

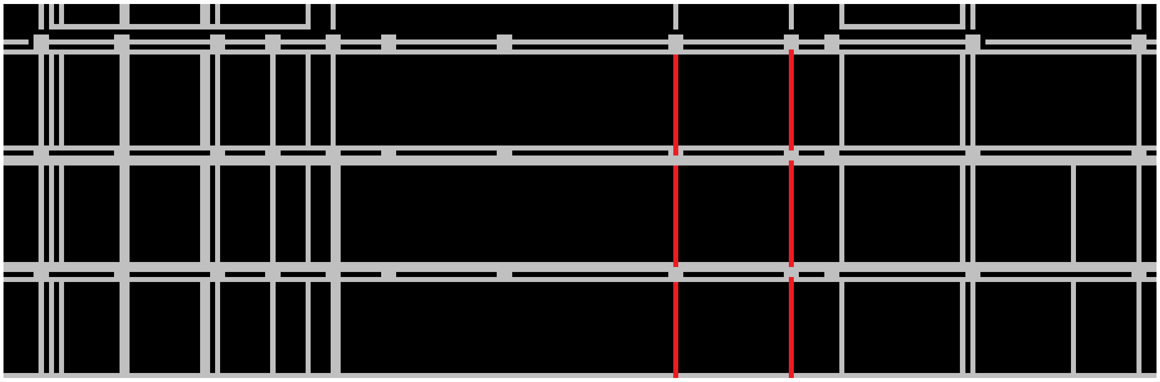

Figure 29 - Failing columns

Although it is a local failure, it may cause global failure of the building to occur much more quickly. So in order to remove this deficiency in the columns, column jacketing can be done.

CHAPTER 10

CONCLUSIONS AND SUGGESTIONS FOR FUTHER RESEARCH

10.1 Conclusions

- Although seismic evaluation seems a straight-forward process on paper, it is very difficult to execute and requires an intimate knowledge of earthquakes and earthquake engineering concepts.
- Use of SAP2000 and ETABS for a parallel evaluation alongside PERFORM 3D of the building can help in verifying and understanding the results.
- A combination of shear walls placed on the perimeter of the building and inside the building is likely to enhance seismic strength of the building more so than a sole exterior or interior placement of shear walls.
- Localized failures may not be prevented as a result of applying a global retrofit strategy or seismic strengthening design measure.
- Economy should play a major part in the selection of a retrofit strategy.
- Multiple variations of a retrofit strategy should be tested and the one that delivers the best in terms of economy and efficiency of purpose must be selected.
- Comparison of storey shears calculated in Tier 1 with those determined in the pushover analysis stage can provide a good overview of a building's performance (globally not locally) under seismic loads.

10.2 Suggestions for further research

- Extent of the contribution of localized failures to globalized failures in a building is an area of further research in the field of seismic evaluation and retrofitting.
- There is a need for more studies of a nature similar to this one using purely analytical software such as PERFORM 3D and nonlinear analysis techniques.
- Torsional effects created from the various placements of shear walls can be further studied and strategies to remove them should be devised.

THIS PAGE HAS BEEN LEFT BLANK INTENTIONALLY

BIBLIOGRAPHY

BIBLIOGRAPHY

1. *ASCE/SEI 31-03 Seismic Evaluation of Existing Buildings*. (2003). Reston, Virginia.
2. *FEMA-310 Handbook for the Seismic Evaluation of Buildings*. (1998).
3. Chakrabarti, A., Menon, D., & Sengupta, A. (2008). *Handbook on seismic retrofit of buildings*. New Delhi: Narosa Publishing House.
4. Yousaf, S. (2009). *Performance Based Seismic Design*. National University Of Sciences and Technology.
5. Virk, H. (2010). *Seismic vulnerability assessment of a building in Islamabad*.
6. Anwar, G. (2009). *Seismic Analysis of Reinforced Concrete Structures in Pakistan*.
7. Varum, H. (2013). *Seismic Strengthening of RC Structures with Exterior Shear walls*.
8. *PERFORM-3D Nonlinear Analysis and Performance Assessment for 3D Structures USER GUIDE*. (2006) (4th ed.). Berkeley, California.
9. Geohazards.usgs.gov,. (2015). *Worldwide Seismic Design Tool (Beta)*. Retrieved 15 May 2015, from http://geohazards.usgs.gov/designmaps/ww/
10. Rana, R., Jin, L., & Zekioglu, A. (2004). *Pushover Analysis Of A 19 Story Concrete Shear Wall Building*. Indian Institute of Technology Kanpur.
11. Wikipedia,. (2015). *Seismic retrofit*. Retrieved 17 October 2014, from http://en.wikipedia.org/wiki/Seismic_retrofit
12. Menjivar, M. (2003). *3D Pushover of Irregular Reinforced Concrete Structures* (Post-graduate). Universita degli Studi di Pavia.
13. Chopra, A. (2007). *Dynamics of structures*. Upper Saddle River, N.J.: Pearson/Prentice Hall.
14. Shinde, D., Veena, N., Yojana, P. (2014). *Pushover Analysis Of Multi-Story Building*. *IJRET*, *03*(15), 691-693. doi:10.15623/ijret.2014.0315129
15. Powell, G. (2007). *Performance based design using nonlinear analysis*. Berkeley, CA: Computers & Structures, Inc.
16. Commend, S. (2010). *Motorway Exit Bridge 3D Pushover Analysis*. Presentation, Lausanne.

17. The Structural Engineering: An Art,. (2010). *Earthquake Load Calculation (base Shear method)*. Retrieved 18 February 2015, from https://ztwocents.wordpress.com/2010/04/07/earthquake-load-calculation-base-shear-method/
18. Comartin, C., & Niewiarowski, R. (1996). *Seismic evaluation and retrofit of concrete buildings*. Redwood City, Calif.: Applied Technology Council.
19. Liao, W. (2010). *Performance-Based Plastic Design Of Earthquake Resistant Reinforced Concrete Moment Frames* (PhD.). University of Michigan.
20. Wiki.csiamerica.com,. (2013). *Static pushover methods - explanation, comparison and implementation - Perform-3D - CSI*. Retrieved 16 May 2015, from https://wiki.csiamerica.com/display/perform/Static+pushover+methods++explanation,+comparison+and+implementation
21. *Step-by-Step Procedure For Performance-based Design of 40 Storey Reinforced Concrete Building*. (2014). Asian Institute of Technology.
22. Rafi, Z., Leghari, A., Lindholm, C., Bungum, H., & Lang, D. (2007). *Seismic Hazard Analysis and Zonation for Pakistan, Azad Jammu and Kashmir*. Islamabad, Pakistan: Pakistan Meteorological Department in collaboration with NORSAR.
23. Rajan, T. *Non-Linear Static Push-over Analysis*. *Architectjaved.com*. Retrieved 8 May 2015, from http://www.architectjaved.com/nonlinear-static-pushover-analysis/
24. *Building Code of Pakistan*. (2007). Islamabad, Pakistan.

APPENDICES

APPENDIX A – BUILDING DETAILS

Table A1 – Properties of major materials used in building

Building Material Properties	
Structure Type	Reinforced Concrete Moment Frame
Concrete Strength in Columns	4000 psi
Concrete Strength in Shear Walls/Roof/Slab	3000 psi
Concrete Strength Beams	3000 – 4000 psi
Steel Strength	60 ksi

Table A2 – Dimensions of important structure components

Component Dimensions	
Columns	Range from 36” x 36” in the basement levels to 12” x 12” in the topmost stories
Beams	Beams mostly rectangular; depths between 30” & 21” and widths between 18” & 12”
Slabs	6” thick slab on upper stories and 18” thick in basement levels
Shear Walls	12” thick running constructed in the elevator shafts, Non-load bearing
Bracing Beams	18” x 18” & 21” x 18”, cross bracing.

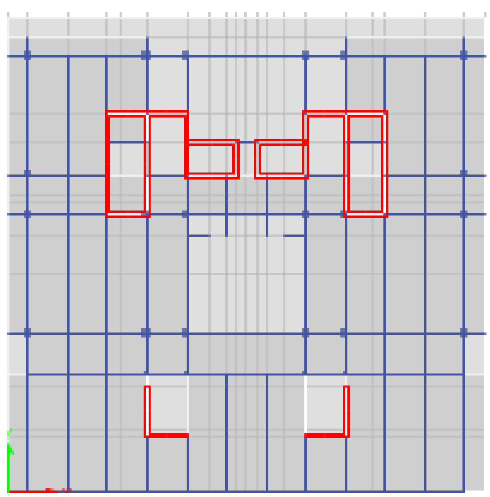

Figure A1 – Storey 1 plan

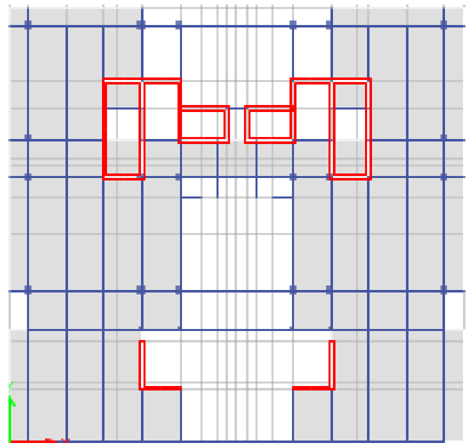

Figure A2 – Storey 2, 4, 5 plan

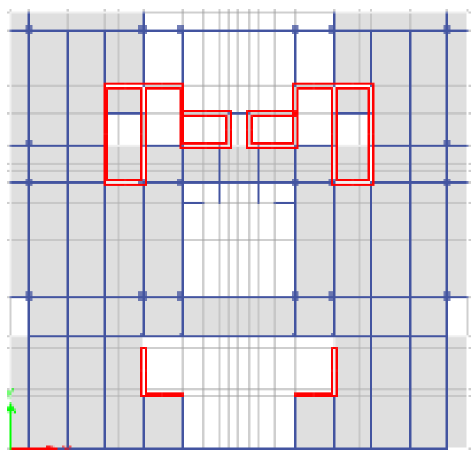

Figure A3 – Storey 3 plan

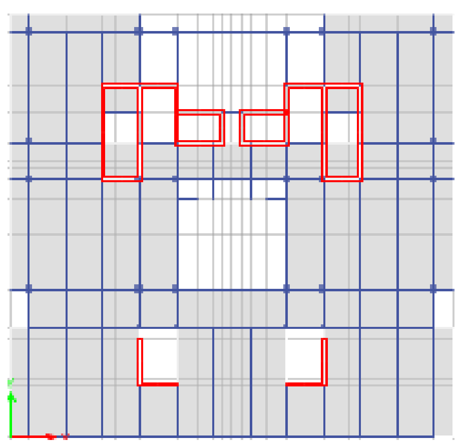

Figure A4 -Storey 6, 7, 8 plan

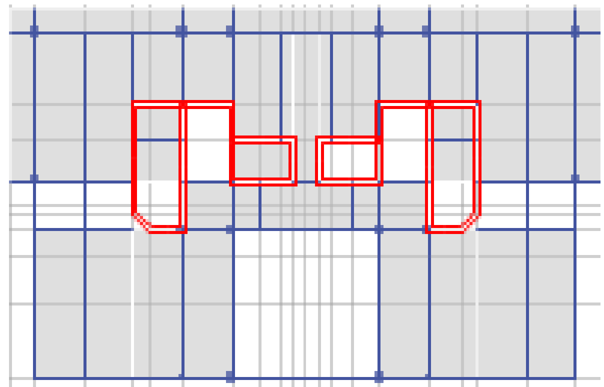

Figure A5 – Storey 9, 10, 11, 12, 13 plan

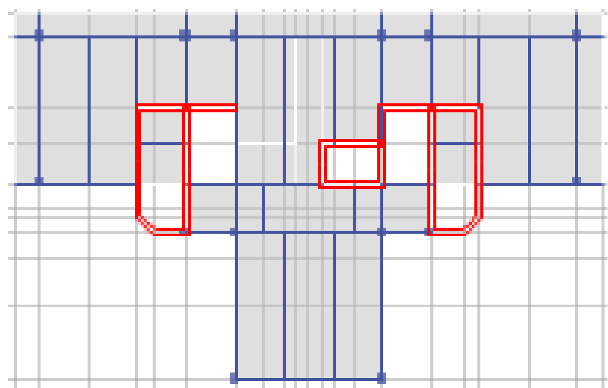

Figure A6 – Storey 14, 15, 16, 17, 18 plan

APPENDIX B – TIER 1 COMPUTATIONS & DETERMINATIONS

Table B1 – Time Period Computation (ASCE 31-03, Chapter 3, Section 3.5.2.4)

Ct	Hn	B	Time Period (s)
0.03	246	0.9	4.255620097

Table B2 – Spectral Acceleration Parameters (ASCE 31-03, chapter 3, section 3.5.2.3.1)

Building Type	PGA	Fv	S_1	S_{D1}	Sa	Fa	S_s	S_{DS}	Is $S_{DS} > S_{D1}$
C1	0.45g	1.5	0.75	0.75	0.176	1	1.68	1.12	Yes! OK!

Table B3 – Seismic Weight Computation (ASCE 31-03, chapter 3, section 3.5.2.1)

Fl #	Floor Area	Slab Thickness	Unit Weight	Dead Load (including finishes)	Partition Load	Partition Load	Live Load	Live Load	Seismic Loads
	ft^2	ft	lb/ft^3	kips	psf	kips	psf	kips	kips
1	8855.22	0.5	150	664.1	20	177.10	65	575.59	1771.04
2	8855.22	0.5	150	664.1	20	177.10	65	575.59	1771.04
3	8860.31	0.5	150	664.5	20	177.21	65	575.92	1772.06
4	8860.31	0.5	150	664.5	20	177.21	65	575.92	1772.06
5	8860.31	0.5	150	664.5	20	177.21	65	575.92	1772.06
6	8860.31	0.5	150	664.5	20	177.21	65	575.92	1772.06
7	8860.31	0.5	150	664.5	20	177.21	65	575.92	1772.06
8	9854.69	0.5	150	739.1	20	197.09	65	640.55	1970.94
9	9854.69	0.5	150	739.1	20	197.09	65	640.55	1970.94
10	9854.69	0.5	150	739.1	20	197.09	65	640.55	1970.94
11	9854.69	0.5	150	739.1	20	197.09	65	640.55	1970.94
12	9854.69	0.5	150	739.1	20	197.09	65	640.55	1970.94
13	7886.83	0.5	150	591.5	20	157.74	65	512.64	1577.37
14	4558.56	0.5	150	341.9	20	91.17	65	296.31	911.71
15	4558.56	0.5	150	341.9	20	91.17	65	296.31	911.71
16	4558.56	0.5	150	341.9	20	91.17	65	296.31	911.71
17	5775.48	0.5	150	433.2	20	115.51	65	375.41	1155.10
18	5850.31	0.5	150	438.8	20	117.01	65	380.27	1170.06
Seismic Weight (Kips)									28894.75

Table B4 – Pseudo Lateral Force (PSL) computation (ASCE 31-03, chapter 3, section 3.5.2.1)

C	S_a	Seismic Weight (Kips)	PSL (Kips)
1	0.15437595	28894.78	5092.339194

Table B5 – Story Shear Computations (ASCE 31-03, chapter 3, section 3.5.2.2)

Fl. #	Wx	Hx	Hx^{k} (K=2)	$Wx*Hx^{k}$ (X)	X/Y	PSL (V)	Fx	Vx
	kips	ft	ft	k-ft		kips	kips	kips
18	1170.1	246	60516	70807472.0	0.1	4460.7	560.1	4460.7
17	1155.1	233	54289	62709006.7	0.1	4460.7	496.0	3900.6
16	911.7	220	48400	44126860.8	0.1	4460.7	349.0	3404.5
15	911.7	207	42849	39065947.5	0.1	4460.7	309.0	3055.5
14	911.7	194	37636	34313192.8	0.1	4460.7	271.4	2746.5
13	1577.4	181	32761	51676087.5	0.1	4460.7	408.8	2475.1
12	1970.9	168	28224	55627754.1	0.1	4460.7	440.0	2066.3
11	1970.9	155	24025	47351785.5	0.1	4460.7	374.5	1626.3
10	1970.9	142	20164	39741993.8	0.1	4460.7	314.4	1251.8
9	1970.9	129	16641	32798379.3	0.1	4460.7	259.4	937.4
8	1970.9	116	13456	26520941.7	0.0	4460.7	209.8	678.0
7	1772.1	103	10609	18799805.8	0.0	4460.7	148.7	468.2
6	1772.1	90	8100	14353702.2	0.0	4460.7	113.5	319.5
5	1772.1	77	5929	10506555.6	0.0	4460.7	83.1	206.0
4	1772.1	64	4096	7258366.0	0.0	4460.7	57.4	122.9
3	1772.1	51	2601	4609133.3	0.0	4460.7	36.5	65.4
2	1771.0	38	1444	2557387.5	0.0	4460.7	20.2	29.0
1	1771.0	25	625	1106902.5	0.0	4460.7	8.8	8.8
			$\sum W_T H_T^K$ (Y)	563931274.6				

P.T.O

Table B6 – Shear Stress Check Computations Part 1 (ASCE 31-03, chapter 3, section 3.5.3.2)

Floor #	Exterior Column	Area (Exterior)	Interior Column	Area (Interior)	Total Columns	Total Area	Frames
1	20	7.11	16	4	36	206.20	6
2	20	7.11	16	4	36	206.20	6
3	20	7.11	16	4	36	206.20	6
4	20	7.11	16	4	36	206.20	6
5	20	7.11	16	4	36	206.20	6
6	20	7.11	16	4	36	206.20	6
7	20	7.11	16	4	36	206.20	6
8	20	7.11	16	4	36	206.20	6
9	16	7.11	8	4	24	145.76	6
10	16	7.11	8	4	24	145.76	6
11	16	7.11	8	4	24	145.76	6
12	16	7.11	8	4	24	145.76	6
13	16	7.11	8	4	24	145.76	6
14	14	7.11	4	4	18	115.54	6
15	14	7.11	4	4	18	115.54	6
16	14	7.11	4	4	18	115.54	6
17	14	7.11	4	4	18	115.54	6
18	14	7.11	4	4	18	115.54	6

Table B7 – Shear Stress Check Computations Part 2 (ASCE 31-03, chapter 3, section 3.5.3.2)

Storey Shear	Vx / At	1/m	(Nc/(Nc-Nf))	Vj (AVG)	Comparison	Status
Kips				psi		
8.76	0.04	0.50	1.20	25.48	**<100 psi**	**C**
28.98	0.14	0.50	1.20	84.34	**<100 psi**	**C**
65.44	0.32	0.50	1.20	190.42	**>100 psi**	**NC**
122.86	0.60	0.50	1.20	357.48	**>100 psi**	**NC**
205.96	1.00	0.50	1.20	599.31	**>100 psi**	**NC**
319.50	1.55	0.50	1.20	929.67	**>100 psi**	**NC**
468.20	2.27	0.50	1.20	1362.38	**>100 psi**	**NC**
677.98	3.29	0.50	1.20	1972.79	**>100 psi**	**NC**
937.41	6.43	0.50	1.33	4287.48	**>100 psi**	**NC**
1251.77	8.59	0.50	1.33	5725.26	**>100 psi**	**NC**
1626.32	11.16	0.50	1.33	7438.35	**>100 psi**	**NC**
2066.33	14.18	0.50	1.33	9450.84	**>100 psi**	**NC**
2475.09	16.98	0.50	1.33	11320.37	**>100 psi**	**NC**
2746.50	23.77	0.50	1.50	17828.24	**>100 psi**	**NC**
3055.51	26.45	0.50	1.50	19834.10	**>100 psi**	**NC**
3404.55	29.47	0.50	1.50	22099.81	**>100 psi**	**NC**
3900.57	33.76	0.50	1.50	25319.62	**>100 psi**	**NC**
4460.65	38.61	0.50	1.50	28955.26	**>100 psi**	**NC**

Table B8 – Axial Stress Check (ASCE 31-03, chapter 3, section 3.5.3.6)

Fl #	1/m	V	hn	Beam Length (ft)	Col. Length (ft)	Total Length of Frame (ft)	nf	A col	P *ot (psi)	0.30fc ' psi	Is P*ot < 0.30f'c
1	0.5	4461	25	110.48	20.3	241.26	6	4	6419.8	1200	**NC**
2	0.5	4461	38	110.48	11.5	232.46	6	4	10127.5	1200	**NC**

3	0.5	4461	51	110.48	11.5	232.46	6	4	13592.1	1200	**NC**
4	0.5	4461	64	110.48	11.5	232.46	6	4	17056.8	1200	**NC**
5	0.5	4461	77	110.48	11.5	232.46	6	4	20521.5	1200	**NC**
6	0.5	4461	90	110.48	11.5	232.46	6	4	23986.1	1200	**NC**
7	0.5	4461	103	110.48	11.5	232.46	6	4	27450.8	1200	**NC**
8	0.5	4461	116	71.24	11.5	153.98	6	4	46672.2	1200	**NC**
9	0.5	4461	129	71.24	11.5	153.98	6	4	51902.7	1200	**NC**
10	0.5	4461	142	71.24	11.5	153.98	6	4	57133.3	1200	**NC**
11	0.5	4461	155	71.24	11.5	153.98	6	4	62363.8	1200	**NC**
12	0.5	4461	168	30.00	11.5	71.50	6	4	145568.3	1200	**NC**
13	0.5	4461	181	30.00	11.5	71.50	6	4	156832.5	1200	**NC**
14	0.5	4461	194	30.00	11.5	71.50	6	4	168096.7	1200	**NC**
15	0.5	4461	207	30.00	11.5	71.50	6	4	179361.0	1200	**NC**
16	0.5	4461	220	30.00	11.5	71.50	6	4	190625.2	1200	**NC**
17	0.5	4461	233	30.00	11.5	71.50	6	4	201889.4	1200	**NC**

18	0.5	4461	246	30.00	11.5	71.50	6	4	21315 3.6	1200	**NC**

Table B9 - Adjacent building check computation

Height of State Life	**Height of Saudi Pak**	**Distance**	**4% of Saudi Pak**
ft	**ft**	**ft**	**ft**
285	236	150	9.44

Table B10 – Mass irregularities check computations

Story No	**Floor Area**	**Slab Thickness**	**Unit Weight**	**Dead Load**	**Partition Load**	**Partition Load**	**Mass**	**Diff**	**%**	**< 50 %?**
	ft^2	**ft**	**lb/ft^3**	**kips**	**psf**	**kips**	**kips**			
1	8855	1	150	1328.28	20	132.8	1461.1			
2	8855	1	150	1328.28	20	132.8	1461.1	0.0	0.0	Yes
3	8860	1	150	1329.05	20	132.9	1462.0	0.8	0.1	Yes
4	8860	1	150	1329.05	20	132.9	1462.0	0.0	0.0	Yes
5	8860	1	150	1329.05	20	132.9	1462.0	0.0	0.0	Yes
6	8860	1	150	1329.05	20	132.9	1462.0	0.0	0.0	Yes
7	8860	1	150	1329.05	20	132.9	1462.0	0.0	0.0	Yes
8	9855	1	150	1478.20	20	147.8	1626.0	164.1	11.2	Yes
9	9855	1	150	1478.20	20	147.8	1626.0	0.0	0.0	Yes
10	9855	1	150	1478.20	20	147.8	1626.0	0.0	0.0	Yes
11	9855	1	150	1478.20	20	147.8	1626.0	0.0	0.0	Yes
12	9855	1	150	1478.20	20	147.8	1626.0	0.0	0.0	Yes
13	7887	1	150	1183.02	20	118.3	1301.3	324.7	20.0	Yes
14	4559	1	150	683.78	20	68.4	752.2	549.2	42.2	Yes
15	4559	1	150	683.78	20	68.4	752.2	0.0	0.0	Yes
16	4559	1	150	683.78	20	68.4	752.2	0.0	0.0	Yes
17	5775	1	150	866.32	20	86.6	953.0	200.8	26.7	Yes
18	5850	1	150	877.55	20	87.8	965.3	12.3	1.3	Yes

19	5850	1	150	877.55	20	87.8	965.3	0.0	0.0	Yes
20	5850	1	150	877.55	20	87.8	965.3	0.0	0.0	Yes
21	5850	1	150	877.55	20	87.8	965.3	0.0	0.0	Yes