Portland State University

PDXScholar

Civil and Environmental Engineering Master's
Project Reports

Civil and Environmental Engineering

2014

Seismic Retrofitting of Existing Structures

Cetin Sahin
Portland State University

Follow this and additional works at: https://pdxscholar.library.pdx.edu/cengin_gradprojects

Part of the [Civil and Environmental Engineering Commons](#)

Let us know how access to this document benefits you.

Recommended Citation

Sahin, Cetin, "Seismic Retrofitting of Existing Structures" (2014). *Civil and Environmental Engineering Master's Project Reports*. 7.

<https://doi.org/10.15760/CEEMP.34>

This Project is brought to you for free and open access. It has been accepted for inclusion in Civil and Environmental Engineering Master's Project Reports by an authorized administrator of PDXScholar. Please contact us if we can make this document more accessible: pdxscholar@pdx.edu.

SEISMIC RETROFITTING OF EXISTING STRUCTURES

BY

CETIN SAHIN

A research project report submitted in partial fulfillment
of the requirement for the degree of

MASTER OF SCIENCE
IN
CIVIL AND ENVIRONMENTAL ENGINEERING

Project Advisor:
MIKE GORJI

Portland State University
©2014

ACKNOWLEDGMENTS

This study was conducted under the supervision of Dr. Mike Gorji. I would like to express my sincere appreciation for his inspiration, guidance, continuous support and valuable comments that he had provided me throughout this research project and my entire study at Portland State University.

I would also like to thank to Arlindo Lopez for his assistance and valuable suggestions in this study.

Special thanks go to my parents, my brother and my sister-in-law for their continuous support, encouragement and love which were vital to the completion of my goals.

Finally, I would like to thank my fiancé for her patience, understanding and encouragement through my entire BSc. and MSc. study.

ABSTRACT

This research project will give a brief presentation about earthquake resistant design and the methodology about seismic evaluation and rehabilitation of existing structures. It also provides certain aspects of computer software modeling against seismic loads and shows the necessity of seismic upgrading in a steel moment-frame building.

The seismic evaluation process consists of investigating if the structure meets the defined target structural performance levels. The main goal during earthquakes is to assure that building collapse doesn't occur and the risk of death or injury to people is minimized and beyond that to satisfy post-earthquake performance level for defined range of seismic hazards. Also seismic evaluation will determine which are the most vulnerable and weak components and deficiencies of a building during an expected earthquake. The seismic rehabilitation process aims to improve seismic performance and correct the deficiencies by increasing strength, stiffness or deformation capacity and improving connections. Thus, a proposed retrofit implementation can be said to be successful if it results an increase in strength and ductility capacity of the structure which is greater than the demands imposed by earthquakes.

Performance-based design aims to utilize performance objectives to determine acceptable levels of damage for a given earthquake hazard for new buildings or upgrade of existing buildings. These performance objectives can be such as limiting story drift, minimizing component damage etc. This study shows how to model a building in computer software and analyze its seismic resistance with linear methods and propose concentrically braced frame rehabilitation in order to increase the drift capacity. It also describes how the linear analysis may be followed by the pushover analysis in order to estimate the seismic resistance of retrofitted structure. One of the most significant advantages of nonlinear pushover analysis beyond the linear analyses is the opportunity to evaluate damage. The pushover analysis can give valuable information about performance of building in expected future seismic events.

Generally, the structural retrofit of concentrically braced frames improved the seismic resistance of the building and it can be considered in the retrofit of moment frame structures to prevent the risk of structural collapse under the design load with much more confidence.

CONTENTS

1. INTRODUCTION.....	1
1.1. Purpose and Objectives.....	1
1.2. An overview of Earthquake Resistant Design	1
1.3. Organization of Research.....	3
2. LITERATURE REVIEW	4
2.1. Choices for the Lateral Force-resisting Elements of Structural Systems.....	4
2.1.1 Shear Wall Seismic Resisting Systems.....	5
• Wood Shear Walls.....	5
• Reinforced Concrete Shear Walls	7
• Reinforced Masonry Shear Walls	8
• Steel Shear Walls	10
2.1.2 Braced-Frame Seismic Resistance Systems.....	12
• Wood Braced Frames.....	12
• Concentric Steel Braced Frames	14
• Eccentrically Braced Frames.....	16
• Buckling-Restrained Braced Frames (BRBFs)	17
2.1.3 Moment Frame Seismic Resisting Systems	19
• Reinforced Concrete Moment-Resisting Frames	19
• Steel Moment-Resisting Frames	20
2.1.4 Diaphragms	22
• Wood Diaphragms	22
• Reinforced Concrete Diaphragms	23
2.1.5 Seismic Isolation and Energy Dissipation Systems	24
• Seismic Isolation Systems.....	25
Elastomeric Bearings:	26
Sliding Systems:.....	28
• Passive Damping Devices	30
Viscoelastic Devices	30
Hysteretic Devices	32
2.2. Earthquake Design Philosophy	34
2.3. Building Codes & Standards.....	36
2.4. Seismic Hazard & Accepted Risk.....	39
2.5. Need for Seismic Retrofit	40

3. SEISMIC EVALUATION AND REHABILITATION OF EXISTING STRUCTURES	41
3.1. Introduction.....	41
3.2. Code Basics.....	42
3.2.1 ASCE 31-03	43
3.2.2 ASCE 41-06	45
3.3. Categories of Seismic Deficiencies.....	50
3.4. Techniques For Seismic Strengthening of Existing Structures.....	56
3.4.1 Add Elements.....	57
3.4.2 Enhance Performance of Existing Elements	57
3.4.3 Improve Connections between Components.....	57
3.4.4 Reduce Demand	58
3.4.5 Remove Selected Components.....	58
3.5. Performance Based Seismic Analysis Methods.....	58
3.5.1 Introduction.....	58
3.5.2 Analysis Methods.....	60
• Linear Static Procedure	60
• Linear Dynamic Procedure	60
3.5.3 Nonlinear Static Procedure	61
3.5.4 Nonlinear Dynamic Procedure.....	63
4. COMPUTER MODELING OF BUILDING DESIGN.....	65
4.1. Building Modeling	65
4.1.1 Description of Structure	65
4.1.2 Lateral Force-Resisting System	67
4.1.3 Diaphragm Modeling	68
4.1.4 Foundation	69
4.1.5 Gravity Loads.....	69
4.1.6 Response Spectra per Site Location.....	69
4.1.7 Dynamic Properties.....	70
• Mass	70
• Period of Vibration and Base Shear	70
• Damping.....	71
4.1.8 Vertical Distribution of Seismic Loads per Equivalent Static Method.....	71
4.1.9 Load Combinations.....	73
4.2. Objective	73
4.2.1 Evaluation Of Existing Structure Against Seismic Loads	73

4.2.2 The Need For Seismic Retrofit	74
4.3. Analysis of Existing Structure	74
4.3.1 Equivalent Static Method.....	74
4.3.2 Modal Response Spectrum Method	76
4.3.3 Modal Time History Analysis.....	78
4.3.4 Comparison of Results from Various Methods of Analysis	85
4.4. Seismic Retrofitting	88
4.5. Nonlinear Static Method (Pushover Analysis).....	88
• Basis of Procedure.....	89
• Modeling	90
• Pushover Analysis.....	91
• Results	92
5. CONCLUSIONS	95
5.1. Final Conclusions.....	95
5.2. Recommendations	96
6. REFERENCES.....	97

LIST OF TABLES

<i>Table 3.1- Target building performance and levels from ASCE 41-06 Table C1-8.....</i>	46
<i>Table 3.2- Rehabilitation objectives from ASCE 41-06 Table C1-1.....</i>	49
<i>Table 3.3- Earthquake hazard levels from ASCE 41-06.....</i>	50
<i>Table 4.1- Building member sizes</i>	66
<i>Table 4.2 - Seismic parameters.....</i>	72
<i>Table 4.3- Seismic forces according to equivalent lateral load procedure.....</i>	72
<i>Table 4.4 - The story drifts from equivalent lateral force analysis</i>	75
<i>Table 4.5 - Modal analysis results</i>	76
<i>Table 4.6 - Maximum member forces from SAP 2000.....</i>	76
<i>Table 4.7- Modal analysis results</i>	77
<i>Table 4.8- The story drifts in X-direction.....</i>	78
<i>Table 4.9- The story drifts in Y-direction.....</i>	78
<i>Table 4.10 - Maximum member forces from SAP 2000.....</i>	78
<i>Table 4.11- Ground Motion Parameters (Unscaled).....</i>	79
<i>Table 4.12 - Modal analysis results</i>	84
<i>Table 4.13 - The story drifts in X-direction</i>	84
<i>Table 4.14 - The story drifts in Y-direction.....</i>	85
<i>Table 4.15- Target lateral roof displacement</i>	91

LIST OF FIGURES

Figure 2.1 - Shear wall (1.Wood, 2.Reinforced Concrete, 3.Reinforced Masonry, 4.Steel).....	5
Figure 2.2 - Braced frame (1.Wood, 2.Steel).....	5
Figure 2.3 - Moment frame (1.Reinforced Concrete, 2.Steel)	5
Figure 2.4 - Diaphragms (1.Wood 2.Reinforced Concrete).....	5
Figure 2.5 - Seismic forces on a building	6
Figure 2.6 - Reinforced concrete shear walls in buildings.....	7
Figure 2.7 - Reinforced masonry	10
Figure 2.8 - Stiffened and un-stiffened shear wall.....	11
Figure 2.9 - Knee brace for wood frames	13
Figure 2.10 - Common diagonal braces in wood frames	14
Figure 2.11 - Typical Configurations of Concentric Braced Frames.....	15
Figure 2.12 - Typical Configurations of Eccentric Braced Frames	16
Figure 2.13 - Typical link inelastic behavior	17
Figure 2.14 - Buckling restraint brace schematic.....	18
Figure 2.15 - Balanced Hysteresis of BRB's	18
Figure 2.16 - Formation of single story frame mechanism and idealized sidesway mechanism	22
Figure 2.17 - a) Conventional design of seismic resistant building.....	25
Figure 2.18 - Example of lead rubber bearings.....	27
Figure 2.19 - Example of friction pendulum sliding isolation system.....	30
Figure 2.20 - Fluid viscoelastic device	31
Figure 2.21 - Solid viscoelastic device	32
Figure 2.22 - Metallic damper.....	33
Figure 2.23 - Friction damper	34
Figure 2.24 - Schematic system behavior for low, moderate and high seismic demands (Hines & Fahnestock, 2010)	35
Figure 2.25 - Relationship between expected damage to building and earthquake levels.....	40
Figure 3.1 - Target Building performance levels from ASCE 41-06 Figure C1-2	47
Figure 3.2 - Expected post-earthquake damage states from ASCE 41-06 (Courtesy of R.Hamburger)	48
Figure 3.3 - Performance and structural deformation demand for ductile systems (FEMA 274 Figure C2-3)	48
Figure 4.1 - Proposed actual building to be analyzed.....	65
Figure 4.2 - Moment frame elevation	66
Figure 4.3 - Model layout in SAP2000.....	67
Figure 4.4 - Frame plan view and member sizes for the first floor.....	67
Figure 4.5 - Frame member sizes for outer frame on X-Z plane	68

Figure 4.6 - Assigning frame releases and panel zone in SAP 2000	68
Figure 4.7 - Design and MCE response spectrum	70
Figure 4.8 - Deformed shape of structure with vertical and lateral loads	75
Figure 4.9 - Time histories and response spectra for record NGA0952	80
Figure 4.10 - Time histories and response spectra for record NGA0960	81
Figure 4.11 - Time histories and response spectra for record NGA0963	82
Figure 4.12 - Unscaled SRSS spectra of the ground motion pairs with design spectrum.....	83
Figure 4.13 - Ratio of average scaled SRSS spectrum to design spectrum	83
Figure 4.14 - Comparison of inter-storey drifts of analysis methods.....	85
Figure 4.15 - BSE-1 level earthquake DCR for columns per ASCE 41-06 Acceptance Criteria	87
Figure 4.16 - Retrofitted section	88
Figure 4.17 - Force vs. displacement properties of hinges elements	89
Figure 4.18 - Locations of plastic hinges in the frame elements	90
Figure 4.19 - 2D model of the retrofitted frame.....	91
Figure 4.20 - Component or element acceptance criteria from FEMA 356/2000.....	92
Figure 4.21 - Pushover curve from SAP 2000 (k-in)	92
Figure 4.22 - Capacity-demand curve for the building (FEMA 440 Equivalent Linearization Method)	93
Figure 4.23 - Deformed shape and plastic hinge formation for envelope analysis	93

1. INTRODUCTION

1.1. Purpose and Objectives

The main purpose of this study is to increase knowledge and proficiency in earthquake resistant design and seismic rehabilitation of existing structures and to gain familiarity with modeling and analyzing buildings against seismic loads by using computer software. The objectives of this research are: (I) to investigate the effects of earthquake forces on buildings and literature search on earthquake resistant design (II) to evaluate the feasibility of seismic evaluation of buildings and advantages of applying the retrofit measures developed for strengthening (III) to analyze performance based design and compare different seismic analysis method (IV) to model a real building with a structural analysis software and investigate the earthquake effects with different analysis methods prescribed in codes & standards and propose appropriate rehabilitation methods in terms of the performance.

1.2. An overview of Earthquake Resistant Design

Most earthquakes occur through the sudden movement of earth crust in faults zones. The sudden movement releases strain energy and causes seismic waves through the crust around the fault. These seismic waves cause the ground surface to shake and this ground shaking is the principal concern of structural engineering to resist earthquakes among many other effects. Historical records and geological records of the earthquakes are the main data sources in estimating the possibility of ground shaking or seismicity at a certain location. Both data sets have been taken into account to develop the seismic hazard maps.

The initial difference of structural response to an earthquake compared to most other loadings is that the earthquake response is dynamic, not static. With the ground shaking, the portion of the structure above ground is not subjected to any applied force. The earthquake forces are generated by the inertia of buildings as they respond to earthquake induced ground shaking. In design, the response of a structure to an earthquake is predicted from a design spectrum such as specified in ASCE-7. To create a design response spectrum, the first step is to determine the maximum response of the structure to a specific ground motion. This is generally prepared by seismologists and geotechnical engineers by presenting a response function (acceleration, velocity or displacement) against response period. The theory behind is based on the response of a single degree of freedom system such one story frame with mass concentrated at the roof. By recalculating the record response by time to a specific

ground motion for a wide range of frequencies and a common amount of damping, the response spectra for one ground motion may be determined (see Section 4.3.3). The principles of dynamic modal analysis, on the other hand, allow a reasonable approximation of the maximum response of multi degree system. The procedure involves determining the maximum response of each mode from a single degree of freedom response spectrum and then estimating the maximum total response by summing the responses of the individual modes. Due to the contribution of the higher modes to total response is relatively minor; it is not required to consider all possible modes of vibration.

The primary objective of earthquake resistant design is to prevent building collapse during earthquakes to minimize the risk of death or injury to people. Severe earthquakes have extremely low probability of occurrence during the life time of a structure. In the traditional structural design against most type of loads, stresses and strains are not permitted to approach the elastic limit. However in earthquake design, structures are permitted to strain beyond elastic limit in response to ground motion. If a structure has to resist such earthquakes elastically, it would require an expensive lateral load resisting system. During a severe earthquake, the structure is likely to undergo inelastic deformation and has to rely on its ductility and hysteric energy dissipation capacity to avoid collapse. Modern buildings can be designed to be safe under severe ground shaking by avoiding collapse. An effective earthquake engineering design requires that the designer controls the building's response. This can be achieved by selecting a preferred response mode, adopting inelastic deformations to acceptable zones by providing necessary detailing and preventing the development of the undesirable response modes which could lead to building collapse.

Many existing structures that were built according to past design codes & standards are often found vulnerable to earthquake damage due to inadequate detailing, underestimated earthquake loads or material deterioration by time, etc. The high cost of new construction and historical importance of older buildings has led building owners to renovate rather than replace the existing structures. This has caused governmental institutions to implement mandatory seismic strengthening regulations. In this way, seismic rehabilitation techniques for buildings need to include both the evaluation of the existing lateral-force-resisting elements of the building and the addition of new elements where necessary.

Most retrofitting techniques will result an increase in stiffness and slightly increase in mass which causes in return a shorter period. Shortening in period of vibration often results an increase in strength and ductility of retrofitted structure. Thus, a proposed retrofit scheme can be said to be successful if it results an increase in strength and ductility capacity of the structure which is greater than the demands imposed by earthquakes.

1.3. Organization of Research

This research is organized in five chapters. The main chapters of this work are those outlined below:

First chapter: Brief and general discussion of the appropriate research methodology is carried out.

Second chapter: An overview of the most important literature relevant to the seismic resistant structure is investigated. The discussion is based on the seismic force resisting systems in buildings and the general design methodology against lateral loads.

Third chapter: Seismic evaluation of buildings and seismic rehabilitation strategies to increase the resistance against earthquake loads are investigated. Performance based design tools were discussed and compared.

Fourth chapter: A seven-storey steel moment frame structure is modeled in SAP2000 computer software and was analyzed against earthquake loads per different analysis methods. The results were compared and the performance of the structure under the designed loads is discussed.

Fifth chapter: The most important aspects of research are summarized. A thorough presentation and conclusion of the main findings and the identification of the possible project for future research are also outlined.

2. LITERATURE REVIEW

There are three key concepts in seismic design that were fully developed by researchers and engineers. First, earthquake ground motions generate inertial loads that rapidly change with time. Thus, it is common that calculations include a term labeled with a unit of time (usually seconds) and these terms include periods of vibration or their inverse, frequencies; accelerations and velocities. In many other structural engineering problems such as calculations of gravity loads, no unit of time is used.

Second, due to the large uncertainty associated with the forces and structural responses, probabilistic analysis is central to earthquake engineering (Benjamin & Cornell, 1970). The earthquake occurrence time, its magnitude, rupture surface features and dynamic response behavior of the structure cannot be predicted with certainty. Methods of probability and statistics are required to include these uncertainties and their effects on the structural performance evaluation and design.

The third fundamental earthquake engineering concept that distinguishes this field is that the earthquake loading can be so severe that the materials must often be designed to behave inelastically. Within the domain of Hooke's Law, stress is proportional to strain, but beyond that point, behavior becomes complex. Most of the analytical and experimental work investigating inelastic behavior began approximately in the 1960s (Newmark & Veletsos, 1960).

2.1. Choices for the Lateral Force-resisting Elements of Structural Systems

The choices of lateral-force-resisting systems in buildings are limited to the following, with very few exceptions: braced frames (vertically-oriented truss elements), moment-resistant frames, shear walls, diaphragms and response modification techniques that change the seismic demand on the lateral-force-resisting elements. Such techniques focus on changing the forces in the structure due to ground motion (e.g. seismic isolation) or changing the displacement within the structure due to the ground motion (e.g. damping devices). The materials, of which these elements can be made, with very few exceptions, are limited to: steel (including aluminum or other metals), reinforced masonry, reinforced concrete and wood. For purposes of assigning the design coefficients, such as the R or response modification factor, the ASCE 7-10 (Chapter 12) tabulates 85 combinations with a mixture of

these elements and materials. The lateral-force-resisting systems in the buildings are illustrated below on Figure 2.1-

Figure 2.1 - Shear wall (1.Wood, 2.Reinforced Concrete, 3.Reinforced Masonry, 4.Steel)

Figure 2.2 - Braced frame (1.Wood, 2.Steel)

Figure 2.3 - Moment frame (1.Reinforced Concrete, 2.Steel)

Figure 2.4 - Diaphragms (1.Wood 2.Reinforced Concrete)

2.1.1 Shear Wall Seismic Resisting Systems

• Wood Shear Walls

Wood is an economical choice building material compared to the higher priced steel and concrete structural components. The general range of the usage of wood structures to total structures is assumed to be between 80% and 90% in all regions of the US and the majority relies on wood shear walls which makes it the most common of the elements discussed here (Malik 1995). Therefore, their ability to adequately resist random and cyclic lateral forces is critical to the safety of the inhabitants and to the soundness of our residential infrastructure. When properly constructed, its performance in past earthquakes has typically been reliable.

Shear walls are vertical elements of the horizontal force resisting system. They are typically wood frame stud walls covered with a structural sheathing material like plywood. When the sheathing is properly fastened to the stud wall framing, the shear wall can resist forces directed along the length of the wall. When shear walls are designed and constructed properly, they will have the strength and stiffness to resist the horizontal forces to prevent the roof or floor above from excessive side-sway and they will transfer these horizontal forces to the next element in the load path below them. These other components in the load path may be other shear walls, floors, foundation walls, slabs or footings. The strength of the shear wall depends on the combined strengths of its three components: lumber, sheathing and fasteners. When all of the components are properly in place, the shear wall can provide its intended strength (McCormick, 2009).

For shear wall sheathing, the use of gypsum wallboard, cement plaster, fiberboard, wood particleboard, plywood and oriented strand board are common practices. Plywood, which became mass produced and inexpensive after 1960's provided a much stronger and stiffer sheathing material than board siding, which was a finish material used since the early years of the development of the United States. After 1960's oriented strand board (OSB) appeared on the scene and competes in popularity with plywood for shear wall sheathing for wood frame ("two-by-four") construction. The nailing around the edges of a sheathing panel is designed to have a closer spacing than over the intermediate studs, and when a wall is loaded laterally, deformation is absorbed mostly by the nails around the perimeter of each panel (Breyer, Fridley & Cobeen, 1999).

Fasteners for shear wall construction may be staples, screws or nails. Sheathing connections resist shear forces, whereas anchors resist uplift forces. The choice of the anchorage method affects the shear wall stiffness, capacity, and failure mode as well as ductility and seismic resistance of the shear wall. The inertial forces generated by the ground movement of the earthquake concentrate lateral forces in the roof and floors where the mass of the building is greatest. The forces in the roof and floors must be adequately connected to the foundation shown in Figure 2.5 below.

Figure 2.5 - Seismic forces on a building

In structural engineering design, an adequate lateral load path is established and each element of the load path is designed and detailed to resist the calculated earthquake force. Roofs and floors are designed as diaphragms and some of the walls in the building are designed as shear walls. In order to enhance wood shear wall performance in resisting racking, sliding and overturning forces of the earthquakes, some construction practices has to be followed such as (Canadian Wood Council, 2001);

- Structural wood sheathing (OSB or plywood) is thick enough to resist the calculated forces
- Nailing is adequate to transfer the shear forces in the sheathing to the roof, floor or wall framing,
- Blocking is specified at the edges of the structural sheathing in the diaphragms and shearwalls
- Framing members around the perimeter of the diaphragms and shearwalls are strong enough and properly splices to resist the calculated tension and compression forces.
- Enhancing strength and stiffness by increasing the thickness of the structural panels and number or size of the nails.

• **Reinforced Concrete Shear Walls**

The reinforced concrete shear walls generally start at foundation level and are continuous throughout the building height. Shear walls are usually provided along both length and width of buildings as shown in Figure 2.6. Shear walls are like vertically-oriented wide beams that carry earthquake loads downwards to the foundation.

Figure 2.6 - Reinforced concrete shear walls in buildings

Shear wall buildings are a popular choice for reinforced concrete buildings in many earthquake zone countries, like USA, Canada, Chile and New Zealand. Shear walls are easy to construct, because reinforcement detailing of walls is relatively straight-forward and therefore easily implemented at site. Shear walls are efficient, both in terms of construction cost and effectiveness in minimizing earthquake damage in structural and non-structural elements. Properly designed and detailed buildings with shear walls have shown very good performance in past earthquakes. In many cases the “failure” mode of adequately reinforced concrete walls in severe earthquakes has been the formation of horizontal or diagonal cracks which are easily repaired after the earthquake (Paulay & Priestley, 1992).

In earthquakes of the first half of the twentieth century in the USA, reinforced concrete shear wall buildings had generally good performance, with exception of collapse from the 1925 Santa Barbara Earthquake (San Marcos Building), 1940 El Centro (Dunlack Hotel), and 1952 Kern County Earthquakes (Cummings Valley School). Severe damage was usually due to inadequate width of walls and low quality construction (Steinbrugge, 1970). In the 1964 Alaska Earthquake, core walls failed in the Four Seasons Building and the six-story building collapsed, probably because of inadequate lap splices (Steinbrugge, et al., 1967). In the 1994 Northridge Earthquake, large concrete parking garages collapsed, but the main factors had to do with diaphragms, gravity-load-resisting columns with inadequate tolerance for imposed deformations, and pre-cast elements, rather than traditional reinforced concrete shear walls.

Most of the reinforced concrete buildings with shear walls also have columns and these columns primarily carry gravity loads. Shear walls provide large strength and stiffness to buildings in the direction of their orientation, which significantly reduces lateral side-sway of the building. Due to shear walls carry large horizontal earthquake forces, the overturning effects on them are large. Thus, design of their foundations requires special attention. For higher overturning demands, pile foundations, possibly including tension tie-down capacity, can be used.

Walls should be well distributed within the building plan providing resistance to story shears in each principal direction and also they should be positioned such that their center of resistance is close to the center of mass to avoid induced torsion. Walls located near the perimeter may be preferred because they maximize torsional resistance. Furthermore, good connections between diaphragms and structural walls are essential to the seismic force path. Selection of special reinforced concrete shear walls as primary seismic force-resisting

elements is influenced by considerations of seismic performance, functionality, constructability, and cost. For low-to mid-rise buildings, shear walls typically are more cost-effective than other systems such as concrete moment frames (Moehle et al., 2012).

- **Reinforced Masonry Shear Walls**

Masonry is one of the oldest construction materials and most of the buildings studied in architectural history are made of masonry: Egyptian and Greek temples, Roman coliseum, Persian palaces, Byzantine domes, Islamic minarets, Gothic cathedrals and the list goes on. This is because; (1) masonry was chosen for its high quality; (2) it aesthetically complemented many different styles; and (3) the structures have been standing long.

Due to its constructability and substantial durability, construction using unreinforced masonry (URM) was widespread throughout the 19th Century in the United States. However, a series of earthquakes, including the 1906 San Francisco, 1925 Santa Barbara, and the 1933 Long Beach earthquakes, clearly illustrated the seismic vulnerability of URM structures. Prior to the 1933 Long Beach Earthquake, reinforced masonry was so rare that the “unreinforced” adjective wasn’t necessary when referring to masonry walls without benefit steel reinforcing embedded in grout. (Cowan, 1977).

These vulnerabilities of URM structures have led engineers and builders to seek a more ductile, earthquake resistant form of masonry construction for high seismic regions. This movement resulted in the development of reinforced masonry, which typically combines high strength manufactured concrete and clay masonry units along with grouting and reinforcing steel as shown in Figure 2.7 below to more efficiently resist tensile stresses and provide a more ductile and reliable system. Throughout the last 70 years, this type of construction has been used extensively in high seismic regions throughout the United States and based on past performance, it is widely considered as one of the most earthquake resistant structural systems. Codes still allow unreinforced masonry buildings in low seismic zones in the USA today; the design philosophy is to provide enough strength for the low level of loading to keep the structure elastic.

Figure 2.7 - Reinforced masonry

Although, in current practice most seismically designed masonry buildings in the USA in high seismic zones are made of concrete block (also called concrete masonry unit or CMU) is more common, clay masonry construction have been widely used in the past along with concrete masonry. Depending on the distribution of vertical and horizontal reinforcement, the MSJC code (2005) classifies reinforced masonry into the following three categories: Ordinary, intermediate and special reinforced masonry shear walls. Besides the distribution of reinforcing steel, reinforced masonry walls can also be distinguished based on the grouting; partially grouted masonry walls typically only have grout placed where reinforcement is located whereas fully grouted masonry walls have grout placed in every cell. Although in terms of construction practice and economy, partially grouted masonry is widely used, fully grouted walls are superior in terms of structural behavior since the tensile and shear strength are better (Drysdale, et al., 1999).

• **Steel Shear Walls**

A tremendous lateral load capacity can result with a solid wall rather than frame from the strongest of the structural materials. Steel shear walls, which more common in ship design, are only rarely used in buildings and relatively new to structural engineering. This type of element offers the designer an option that can concentrate massive amounts of earthquake resistance into compact spaces where other types of elements would not be strong enough. As a result, steel shear walls can be very efficient and economical lateral load resisting systems and due to having high initial stiffness it is very effective in limiting the drift. The twenty-story Nippon Steel Building in Tokyo, built in 1970, was the first seismically designed structure to use steel shear walls (Thorburn, et al., 1983).

In general, steel plate shear wall system consists of a steel plate wall, boundary columns and horizontal floor beams. Together, the steel plate wall and boundary columns act as a vertical

plate girder. The columns act as flanges of the vertical plate girder and the steel plate wall acts as its web. The horizontal floor beams act, more or less, as transverse stiffeners in a plate girder (Astaneh, 2001).

There are three different steel shear wall systems: (1) unstiffened, thin steel shear wall (2) stiffened steel shear wall, (3) composite concrete steel shear wall. The most common application in North America is the un-stiffened thin, while In Japan, the stiffened steel shear wall system is more common as shown in Figure 2.8 below. Prior to 1980s, the design approach used by the Japanese and Americans concentrated on preventing the steel plate shear panels from buckling prior to the attainment of shear yield. In Japan, this was achieved by reinforcing the thin panels with a relatively large number of longitudinal and transverse stiffeners and in the United States; this was achieved by using the thick steel plate shear walls. However, several experimental and analytical studies using dynamic loading showed that the post buckling strength and ductility of thin un-stiffened steel plate shear walls can be substantial. As a result, thin un-stiffened steel plate shear wall is rapidly gaining popularity as a very effective lateral load resisting system in highly seismic areas.

Figure 2.8 - Stiffened and un-stiffened shear wall

Buildings with steel plate shear walls have been subjected to actual earthquakes. One of the most important buildings with steel plate shear wall, subjected to a relatively strong earthquake, was the 35 story high-rise in Kobe, which was subjected to the 1995 Kobe earthquake. Researchers in Japan, (Fujitani et al., 1996) have studied seismic performance of this building. The study indicated that the damage was minor and consisted of local buckling of stiffened steel plate shear walls on the 26th story and a permanent roof drift. The results of inelastic analyses of this structure indicate that soft stories may have formed at floors between 24th and 28th level of the building.

Steel plate shear walls allow for less structural wall thickness in comparison to the thickness of concrete shear walls and result in a lesser building weight in comparison to buildings that use concrete shear walls which results in a reduction of foundation loads due to gravity and overall building seismic loads. Also the use of a steel plate shear walls system reduces construction time. Not only is it fast to erect, but there also is no curing period. As mentioned before, relatively thin steel plate has excellent post-buckling capacity. Due to relatively small thickness of steel plate shear walls compared to reinforced concrete shear walls, from architectural point of view, steel plate shear walls occupy much less space than the equivalent reinforced concrete shear walls. The system also has been tested since the 1970s and has been recognized by the American Institute of Steel Construction (AISC) Seismic Provisions in 2005 (Seilie & Hooper, 2005).

However, steel plate shear wall systems are usually more costly in comparison to concrete shear walls, primarily due to their flexural flexibility. Therefore, when using steel plate shear walls in tall buildings, the engineer must provide additional flexural stiffness. Another disadvantage is excessive initial compressive force in the steel plate panel may delay the development of the tension-field action. It is important that the construction sequence be designed to avoid excessive compression in the panel.

2.1.2 Braced-Frame Seismic Resistance Systems

• Wood Braced Frames

Wood frame construction is used throughout America widely for residential housing. Wood is an economical choice building material compared to the higher priced steel and concrete structural components. In terms of visual aspects, large timbers have great advantage. Thus, the wooden braced frames have the unique advantage of showing off a structural material that is also aesthetically appreciated. Besides visual aspects, the lightweight and high energy absorbing capabilities of wood-framing are important characteristics that make wood-framing a preferred building system in earthquake regions. Surveys have shown that wood-frame buildings meet the basic requirements for wall bracing, connectivity and anchorage, provide safety to their occupants during earthquakes (Canadian Wood Council, 2001). Generally, wood posts and beams cannot be feasibly joined together rigidly enough to create a joint that can resist rotation. Thus, the two common options for vertical timber elements for resisting lateral forces are shear walls (separately discussed) and braced frames.

The wood members in braced frames are often large in cross section to handle their structural loads and/or because fire provisions in building codes. The knee brace shown in Figure 2.9 which is usually diagonal timbers that typically connect a timber post to a beam above, is used in light mass timber structures where a large amount of flexibility does not cause drift-induced damage to the nonstructural components. Generally, the intent of knee bracing is to supplement the resistance of post frames under lateral loads. They can reduce the unsupported length of the columns and they make it less vulnerable to buckle.

Figure 2.9 - Knee brace for wood frames

The effectiveness of a knee brace is mostly dependent on the stiffness of the connections to the post and the truss. If the brace connections are made very stiff (by installing many nails or bolts) the brace could effectively resist lateral loading, but overload the truss. If the brace connections are made very flexible (by installing not many nails or bolts) the brace could be ineffective.

It is difficult to make the connections in a wood braced frame as strong as the members. It is desirable for connections to develop the full strength of the members so that a brittle failure at a joint cannot suddenly occur. Because of their inherent stiffness and low ductility, braced frames are particularly vulnerable in severe earthquakes. The only elements that can provide any significant amount of energy absorption are the connections, and therefore special attention needs to be paid to connection detailing to assure ductile behavior under large deformations (Prion et al., 1999).

Light diagonal braces built into the stud wall framing act essentially as small braced frames, but they tend to quickly lose capacity in the inelastic range. Cut-in braces use pieces of wood the same depth as the wall studs and are inserted between them so that the segments line up to form a diagonal. Let-in braces use one continuous piece of wood notched into each stud.

Figure 2.10 below shows common diagonal wood frames.

Figure 2.10 - Common diagonal braces in wood frames

The flow of forces in a timber and a steel braced frame of the same layout are essentially the same under earthquake loads in the elastic range. However, the mode of behavior and seismic design philosophy in timber and steel braced frame can be quite different in the inelastic range. Because wood does not have well defined yield points like steel has. The splitting, crushing, and breaking of a piece of wood in the range defined as inelastic does not conform to a classic definition of ductility (Symans, et al., 2002).

• **Concentric Steel Braced Frames**

Concentrically braced frames (CBFs) are widely used as lateral-load resisting systems in buildings throughout the US. The standard bridge or roof truss is loaded vertically by gravity and spans horizontally, while the braced frame is loaded primarily horizontally by seismic inertia loads and acts as a vertical cantilever. The source of truss' stability is based on its basic unit of triangle which is a structural unit that resists structural loads via development of axial forces in its members. Pure truss action results when the forces in the members are aligned with the centerlines of pinned joints. Concentrically braced frames (CBFs) join beams, columns, and braces at common work points such as shown in Figure 2.11. The braced frame is a direct, economical, and effective seismic solution.

Figure 2.11 - Typical Configurations of Concentric Braced Frames

When the strain in a braced frame member exceeds its elastic limit, it may cause the material to have a permanent deformation and the system can't release to the strain safely unless special seismic inelastic design features are incorporated. Buckling of beams and columns cannot represent acceptable means of dissipating seismic energy as such response would endanger the gravity load carrying capacity of the structure. Thus, inelastic action under earthquakes must only take place in the diagonal bracing members and adequate detailing must be provided to ensure that the braces can go through the expected inelastic demand (Tremblay, 2001). A basic seismic design principle is that the structure should gradually deform as the seismic load increases into the inelastic range, allowing it to dissipate energy safely rather than suddenly breaking. In addition, seismic codes encourage redundancy. At a given story, on each line of bracing, diagonal braces should share the lateral load exerted in a given direction by having some resist in tension while others take compression. The failure modes caused by the non-ductile braced frame behaviors must be prevented and include the following (Bruneau, et al., 1997):

- (1) The diagonal delivers too much force to the connection at the beam-column joint and the connection breaks;
- (2) The seismic brace or buckles in compression;
- (3) A tension-only (e.g. tie-rod) diagonal stretches inelastically, but on the next repetition of a cycle when it is again loaded in tension, there is slack in the system and the frame must resist a "slamming" effect, and the hysteresis loop is pinched;
- (4) If diagonals frame into columns (as in a K brace) or beams (as in a V or chevron brace), the force delivered by the brace damages the column or beam.

Concentric steel braced frames are often used in low-rise residential and industrial buildings. The high stiffness of the bracing tends to put these structures at the low-period end of the response spectrum which in turn usually means higher response (e.g., greater spectral acceleration) than in the long-period range. These greater accelerations affect the structure as well as the equipment and contents. The positive aspect to a stiff, low-period structure, however, is that it tends to protect built-in non-structural components such as partitions from drift-induced damage.

• **Eccentrically Braced Frames**

As with any braced frame, the function of the diagonal is to provide stiffness and transmit lateral forces from the upper to the lower level. Eccentrically braced frames (EBFs) is a framing system in which the eccentricity is intentionally planned by an element called “link” beam and the location of inelastic behavior is strategically and very explicitly designed to occur in beams in such a way as to not threaten the gravity-load-resisting system. The diagonal brace, at least at one end, is connected to the end of the link rather than the beam-column joint. All inelastic activity is intended to be confined to the properly detailed links which act as structural fuses and can dissipate applied seismic energy without degradation of strength and stiffness. Figure 2.12 shows eccentric brace frame configuration.

Figure 2.12 - Typical Configurations of Eccentric Braced Frames

Under earthquake loading, eccentrically braced frames (EBFs) dissipate energy as stiffened beam segments and rotate inelastically. These links are typically formed from eccentricities between two brace connections, or between a brace connection and column. Links in EBFs are designed to act as structural fuses, localizing frame damage within link regions during overloading. When links are properly designed, the columns, braces, and beam regions

outside the links will remain essentially elastic (Roeder & Popov, 1978). Shorter links that rotate due to web shear yielding are more common than longer links which develop flexural hinges at each end. **Figure 2.13 - Typical link inelastic behavior** Figure 2.13 shows typical link behavior. These links preferably placed centrally to promote the shear yielding and avoid possible problems related to the connections between the beam and the column. Design procedures are based on capacity design principles and aim to produce frames with stable inelastic response of links and elastic behavior of all other frame members.

Figure 2.13 - Typical link inelastic behavior

Frames with moment-resisting joints respond to earthquakes in a flexible manner and have the potential for high ductility, while concentric braced frames have greater stiffness and reduced drift-induced nonstructural damage. The eccentrically braced frame (EBF) combine the ductility that is characteristic of moment frames and the stiffness associated with braced frames (Chao & Goel, 2005). The excellent ductility and energy dissipation capacity is obtained by 2 factors. First, inelastic activity under severe earthquake loading is restricted mainly to the links which are detailed to sustain large inelastic deformations. Second, braces are designed not to buckle regardless of lateral loading. Due to the ultimate strength of the link can be accurately estimated, the designer can be assured that the brace will not buckle by designing the brace stronger than the link (Popov & Engerlhardt, 1988).

EBFs have also an advantage over concentrically braced frames (CBFs), in terms of architectural freedom permitted by providing larger spaces for doors, windows, and hallways, allowing access through the frame while braces in typical concentric configurations get in the way of such features.

- **Buckling-Restrained Braced Frames (BRBFs)**

Buckling-restrained braced frames (BRBFs) are a relatively new type of brace comprised of a special type of concentrically braced frame (CBF's) which performs equally well in tension and compression. The BRBF's superiority against traditional CBF's is obtained through de-

coupling axial strength from buckling resistance by providing transverse restraint to the brace thus allowing the brace to yield equally in tension and compression.

The symmetric hysteretic behavior of the brace is achieved through its composition. One of the more common methods for precluding brace buckling is to install a steel core element confined in concrete mortar filled steel outer tube. The steel core is designed to axially resist the lateral forces and the concrete confinement prevents buckling of the core while the outer steel tube encases concrete mortar and restraints buckling.

Figure 2.14 shows buckling restraint brace schematic. An important characteristic for BRBs is the prevention of friction between the yielding core element and the confining material thus allowing axial independence. Axial load transfer between the core element and the confining material can be omitted from the system by using a de-bonding layer between the steel and concrete mortar (Sabelli, et al., 2003).

Figure 2.14 - Buckling restraint brace schematic

Under severe earthquake loading, BRBFs dissipate energy through axial yielding of the buckling-restrained brace core. This symmetric hysteretic behavior provides improved ductility over traditional CBF's which are limited by poor post-buckling resistance to compressive loads (Inoue, et al., 2001). Figure 2.15 illustrates the BRB's full, balanced hysteresis loops with compression-yielding similar to tension-yielding behavior. By confining the inelastic behavior to axial yielding of the steel core, great ductility can be achieved by the brace itself. The ductility of the steel material is realized over the majority of the brace length. Thus the hysteretic performance of these braces is similar to that of the steel core material (López & Sabelli, 2004).

Figure 2.15 - Balanced Hysteresis of BRB's

As practical use of the BRB has grown in the United States since the early 2000s, building code provisions have followed. After the AISC Seismic Provisions (American Institute of Steel Construction, 2005b) and the ASCE Minimum Design Loads for Buildings and Other Structures (American Society of Civil Engineers, 2005) which govern the design of these structural steel seismic systems adopted BRB systems, it has been implemented to the International Building Code (International Code Council, 2009).

2.1.3 Moment Frame Seismic Resisting Systems

- **Reinforced Concrete Moment-Resisting Frames**

In many countries all over the world, reinforced concrete moment resisting frames (RC-MRF's) are a widely accepted economical structural system that allows energy dissipation without considerable structural damage. Beams, columns, and beam-column joints in moment frames are detailed to resist flexural, axial, and shearing actions that result as building side-sways through multiple displacement cycles during earthquake ground shaking.

Earthquakes such as in 1957 in Mexico City, 1964 in Alaska, and 1967 in Caracas pointed out the potential for collapse of inadequately ductile concrete frames in severe earthquakes. The damages in the 1971 San Fernando Earthquake clearly illustrated the value of spiral or other close-confinement transverse reinforcing in columns as well as toughness throughout the beam-column joints. Confinement (hoops or spirals in the column, stirrups in the beam near joints), continuity (rebar lap splice details and the presence of bars running through joints or anchored securely in them with 135 degree bar bends rather than just right angles), and other ductile detailing have been developed throughout construction practices (Mahin et al., 1976). RC-MRF concepts were introduced in the U.S. starting around 1960 (Blume, et al., 1961). And significant research by the concrete industry, university researchers, and others was necessary to establish a ductile design philosophy for reinforced concrete moment-resistant frames in the years following 1971. By the time of the 1994 Northridge

Earthquake, reinforced concrete moment-resistant frames, if designed under recent codes and standards, performed well (Yashinsky, 1998).

In order to well behave under an earthquake, a building should possess adequate strength, redundancy and ductility. The response of a RC-MRFs building subjected to severe seismic forces depends primarily on those properties of its members and on proportioning and detailing of its individual component and of connections between the components. Moment frames are generally selected as the seismic force resisting system when architectural space planning flexibility is desired. When concrete moment frames are selected for buildings assigned to Seismic Design Categories D, E, or F, they are required to be detailed as special reinforced concrete moment frames. Proportioning and detailing requirements for a special moment frame will enable the frame to safely undergo extensive inelastic deformations that are anticipated in these seismic design categories.

The proportioning and detailing requirements for special moment frames are intended to provide that inelastic response is ductile. Three main goals are: (1) to achieve a strong-column/weak-beam design that spreads inelastic response over several stories; (2) to avoid shear failure; and (3) to provide details that enable ductile flexural response in yielding regions. Ductile response requires that members yield in flexure, and that shear failure be avoided. Shear failure, especially in columns, is relatively brittle and can lead to rapid loss of lateral strength and axial load-carrying capacity and is the most frequently cited cause of concrete building failure and collapse in earthquakes (Moehle et al, 2008). It is important to avoid a yielding mechanism dominated by yielding of the columns in a story, as this can result in very large local demands in the columns. Instead, it is desirable in a RC-MRF that yielding be predominantly in the beams. This is a fundamental objective in the seismic resistant design.

- **Steel Moment-Resisting Frames**

It was believed that steel moment-frame buildings were mostly invulnerable to earthquake-induced structural damage (SAC Joint Venture, 2000). However, during the Northridge earthquake of January 17 1994, there were unanticipated brittle fractures in welded steel beam to column connections (Mahin, Hamburger & Malley, 1998). Similar damage occurred in the 1995 Kobe earthquake in Japan. The resulting changes to the building code require a more complicated approach to steel frame design than prior to Northridge, especially for the connections. Following these discoveries, a consortium of professional associations and

researchers known as the SAC Joint Venture engaged in a federally funded, multi-year program of research and development to determine the causes of this unanticipated behavior and to develop recommendations for more robust moment-resisting frame construction. The SAC research, resulted in the basis for the current design provisions for moment-resisting frames contained in AISC 341, AISC 358, and AWS D1.8.

The principal advantage of moment frame structures is to provide architectural freedom in design in permitting open bays and clear views by not having structural walls or vertically oriented diagonal braces. However, the moment frames can be more costly to construct than braced frame or shear wall structures because of using heavier sections and requiring more labor intensive connections than is common in braced structures. However, due to being more ductile system and having larger periods, moment frames typically impose smaller forces on foundations than other structural systems, and resulting in more economical foundation systems.

Designers are allowed choose from several types, including special moment frames, intermediate moment frames, ordinary moment frames, and moment frames not specifically designed for seismic resistance. Intermediate moment frames are permitted without restriction in Seismic Design Categories A, B, and C, and are permitted in structures 35 ft or less in height in Seismic Design Category D. Steel special moment frames are permitted without restriction in all Seismic Design Categories, and are required in Seismic Design Categories D, E, and F for most structures exceeding 160 ft in height. Special proportioning and detailing are required in resisting strong earthquake shaking which improve the inelastic response characteristics for Special Moment Frames. The proportioning and detailing requirements of AISC 341 are intended to provide ductile inelastic response. The primary goals are as follows: (1) achieve a strong-column/weak-beam condition; (2) avoid P-delta instability under gravity loads and anticipated lateral seismic drifts; and (3) incorporate details that allow ductile flexural response in yielding regions.

Controlling drift, avoiding P-delta instabilities and proportioning structures to comply with the strong-column/weak-beam criteria of AISC 341 are the primary factors affecting steel special moment frame member size. For design purposes, the P-delta effect is regulated by means of elastic and static concepts in the codes, even though in reality the response of the structure in a severe earthquake is inelastic and dynamic. It is usually recommended to limit the bay width of moment-resisting frames because long-span frames cause section sizes to be

larger required to control drift. However, short bay widths also must be avoided; they can result inelastic behavior caused by shear yielding as opposition to flexural yielding of beams.

In order to avoid development of P-delta instability in multistory structures, it is desirable to achieve a relatively uniform distribution of lateral drift over the structure's height. To achieve this, it is important to avoid early formation of single-story mechanisms in which inelastic response is dominated by formation of plastic hinges at the tops and bottoms of columns within a single story. In order to avoid this, building codes require designs intended to promote formation of multistory sidesway mechanisms dominated by hinging of beams, as opposed to column hinging, like the idealized sidesway mechanism of Figure 2.16 (NEHRP, 2009)

Figure 2.16 - Formation of single story frame mechanism and idealized sidesway mechanism

2.1.4 Diaphragms

• Wood Diaphragms

Diaphragms (roof or floor) are horizontal systems acting to transmit lateral forces to vertical-resisting elements by acting like a horizontal beam in addition to resisting gravity loads. They are required both to distribute seismic forces to the main elements of horizontal resistance, such as frames and shear walls, and also to tie the structure together so that it acts as a single entity during an earthquake.

Like wood shear walls are the most common of the vertically-oriented seismic resistant elements, wood diaphragms are the most common type of diaphragm in the United States. With proper and usually inexpensive seismic detailing, wood diaphragms have performed very reliably in earthquakes in all-wood construction. Usually the seismic design of wood diaphragms doesn't require extra attention in the layout of the building the way often required for shear walls.

Wood diaphragms are often used with large reinforced masonry or tilt-up buildings. The most common cause of the damage with these large diaphragms in mixed wood-concrete/masonry construction has been the connection at the wall. Cross-building ties and collectors are often needed in larger buildings, which pose other connection issues. The problem of inadequate concrete or masonry wall connections to wood diaphragms was experienced in the 1964 Alaska Earthquake. Failure modes included the nails pulling out of or ripping through the plywood, and the ledger splitting in cross-grain tension, both with the same effect. The wall could tilt outward and fall, simultaneously dropping a portion of the roof (Steinbrugge et al., 1967).

Usually, roofs or floors those sheathed with plywood, wood decking, or metal decks without structural concrete topping slabs are modeled as flexible. Flexible diaphragm is not considered to be capable of distributing torsional and rotational forces. For flexible diaphragms, the loads should be distributed to the vertical resisting elements according to the tributary area and simple beam analysis according. Obviously, the performance of most diaphragms is between perfectly rigid and flexible. However, at the current time, there are no design tools available to provide for analyzing diaphragms in the intermediate region. Therefore, building codes simply differentiate between the two extreme conditions. During an earthquake, the diaphragm is subjected to horizontal forces that are based on its own mass and the tributary mass of the walls attached along its edges. The exact amount of the force depends on the design ground acceleration and the type of resisting system being used. Wood-frame shear wall systems will generally produce smaller diaphragm design loads than buildings with heavier concrete or masonry walls.

• **Reinforced Concrete Diaphragms**

Horizontal diaphragms allow the walls or frames to work as a group in resisting lateral forces. The vertical structural elements that resist lateral forces would be a disorganized crowd trying to function alone if they weren't linked together. Due to out-of-plane resistance capacities are very small for walls, moment-resisting frames, and braced frames, it is only through the diaphragms that the vertical elements can transmit their stabilizing effect to other elements. Diaphragms are thus an essential part of the seismic force-resisting system.

Floor and roof diaphragms in reinforced concrete buildings are typically modeled as rigid so that the effect of in-plane diaphragm flexibility on the structure is not considered. For the

rigid diaphragm model, the diaphragm has equal in-plane displacements along its entire length under lateral load such that horizontal forces are transferred to the vertical lateral force resisting system proportional to the relative stiffness of each frame (Barron & Hueste, 2004). The purpose of determining whether a diaphragm is flexible or rigid is to determine whether a diaphragm should have the loads proportioned according to the tributary area or the relative stiffness of the supports. For flexible diaphragms, the loads should be distributed according to the tributary area, whereas rigid diaphragm distributes the horizontal forces to the vertical resisting elements in direct proportion to the relative rigidities (Chen & Lui, 2006).

The seismic analysis of buildings is carried out on the assumption that deflections in the diaphragms are so small compared with those in the main lateral load-resisting structure that the diaphragms can be treated as rigid. In most cases, this is quite satisfactory, because usually diaphragm flexibility affects neither overall structural stiffness nor the distribution of forces within a structure. Rigid diaphragms capable of transferring torsional and shear deflections and forces are also based on the assumption that the diaphragm and shear walls or frames undergo rigid body rotation and this produces additional shear forces in the shear wall. Rigid diaphragms consist of reinforced concrete diaphragms, precast concrete diaphragms, and composite steel deck (Agarwal & Shrikhandle, 2006).

Diaphragms are not intended to be the main source of inelastic deformation in a structure, if it occurs at all, should be restricted to the vertical elements. Thus, one of the principles of earthquake-resistant design is to maintain a relatively stiff and damage-free diaphragm that is capable of tying together the vertical elements of the seismic force-resisting system. To achieve this goal, seismic design of a diaphragm should clearly identify the load paths to the vertical elements, and should aim to provide diaphragm strength along that load path at least equal to the maximum force that can be developed by the vertical elements (Sabelli et al, 2011).

2.1.5 Seismic Isolation and Energy Dissipation Systems

In seismic design of structures, the design forces are generally calculated using an elastic design spectrum. A response modification factor (R-ASCE 7) is used to reduce the calculated forces to account for inelastic energy dissipation. Due to ductility of members and redundancy in load path, the structures can sustain damage without collapse during severe earthquakes. However, the inelastic behavior often results in significant damage to the structural non-structural members. Earthquakes and the hazard they have caused in human life by collapse of structures, become the most important aspect for civil engineers for highly

seismic countries. Therefore, innovative techniques have been developed such as seismic base isolation and energy dissipation systems for seismic hazard reduction.

Energy dissipation systems can absorb a portion of earthquake-induced energy and minimize the energy dissipation demand on the primary structural members. The earthquake's higher frequencies and their destructive energy aren't transmitted to the rest of the structure. Lower accelerations and smaller shear forces lead to lower demands in the structural components. Furthermore, these devices can substantially reduce the inter-story drifts and consequently non-structural damage (Sadek et al., 1996). In general energy dissipation systems are characterized by their capability to dissipate energy either by conversion of kinetic energy to heat (passive dampers) or by transfer of energy by among different modes of vibration (tuned vibration absorbers).

The seismic isolation system doesn't absorb the earthquake energy but deflects it through the dynamics of the system. The transmission of seismic energy to the building in seismic isolators is dampened by lowering the vibrational frequency, allowing the building to move or displace, and lowering the shock acceleration of the seismic event thus reducing the tendency for the upper floors to move faster than the lower floors. In general, buildings that have been isolated in this way are subjected to 1/3 to 1/5 of the horizontal acceleration of conventional structures during a seismic event (Psomas, 2002).

Currently, the most practical and reliable methods of reducing seismic structural response are (1) seismic isolation systems and, (2) passive damping devices. Figure 2.17 shows the response of a building with conventional seismic design and response control systems.

Figure 2.17 - a) Conventional design of seismic resistant building

b) Seismic isolation systems

c) Energy dissipation systems-passive damping devices

d) Tuned vibration absorber

• **Seismic Isolation Systems**

Main goals of the seismic isolation is to shift the fundamental frequency of a structure away from the dominant frequencies of earthquake ground motion and fundamental frequency of the fixed base superstructure and to provide an additional means of energy dissipation, by reducing the transmitted acceleration into the superstructure. To achieve this, the design approach aims the isolation of the structure from the supporting ground, generally in the horizontal direction.

Despite wide variation in detail, the concept of seismic base isolation follows two basic approaches. In the first approach the isolation systems reduce the effect of the horizontal components of the ground acceleration by introducing some form of flexible support. This gives the structure a fundamental frequency that is much lower than fixed-base structure. The first dynamic mode of the isolated structure involves deformation only in the isolation system, the structure above being, for all intents and purposes, rigid. The higher modes producing deformation in the structure are orthogonal to the first mode and, consequently, to the ground motion. These higher modes do not participate in the motion, so that if there is high energy in the ground motion at these higher frequencies, this energy cannot be transmitted into the structure. In the second approach, the isolation system increases flexibility in a structure by providing a sliding or friction surface between the foundation and the base of the structure. This works by limiting the transfer of shear force to the superstructure across the isolation interface. The shear force is directly proportional to the coefficient of friction and the weight of the superstructure which means the lower the coefficient friction, the lesser the shear transmitted. However, it must be sufficiently high to provide a friction force that can sustain strong winds and small earthquakes without sliding (Kelly, 1997).

Elastomeric Bearings:

These bearings are developed mainly for bridge superstructures, which often undergo substantial dimensional and shape changes due to changes in temperature. More recently their use has been extended to the seismic isolation of buildings and other structures.

In the United States the most commonly used isolation system is the *lead-plug rubber bearings*. These bearings are multilayered, laminated elastomeric bearings with lead plugs inserted into one or more circular holes. The lead plugs are used to incorporate damping into the isolation system. Although some isolation systems are composed of only lead-plug rubber bearings, in general they are used in combination with multilayered elastomeric bearings (which do not have lead plugs). Most recent examples of isolated buildings use multilayered laminated rubber bearings with steel reinforcing layers as the load-carrying component of the system. Because of the reinforcing steel plates, these bearings are very stiff in the vertical direction but are soft in the horizontal direction, thereby producing the isolation effect. These bearings are easy to manufacture, have no moving parts, are unaffected by time, and resist environmental degradation (Bozorgnia, & Vitelmo, 2004). Figure 2.18 below shows an example of lead rubber bearing.

Figure 2.18 - Example of lead rubber bearings

Low-damping natural rubber bearings and synthetic rubber bearings are used in conjunction with supplementary damping devices, such as viscous dampers, steel bars, frictional devices, and so on. The elastomer used in bearings may be natural rubber or neoprene. The isolators have two thick steel endplates and many thin steel shims. The rubber bearing consists of layer of rubber 5-20 mm thick, vulcanized between steel shims. The rubber layers give the bearing it's relatively low shear stiffness in the horizontal plane while the steel plates control the vertical stiffness and also determine the maximum vertical load, which can be applied safely. The steel plates also prevent the bulging of the rubber. Low-damping elastomeric laminated bearings are simple to manufacture (the compounding and bonding processes to steel is well understood), easy to model, and their mechanical response is unaffected by rate, temperature, history, or aging. However in the structures they must be used with

supplementary damping systems. These supplementary systems require elaborate connections and, in the case of metallic dampers, are prone to low-cycle fatigue. The advantages of using low-damping elastomeric laminated bearings are: they are easy to manufacture; the compounding and bonding to steel are well understood; they are easy to model; and their mechanical response is unaffected by loading rate, temperature, history or aging. The single disadvantage is that a supplementary damping system is generally needed. These supplementary systems require elaborate connections and, in the case of metallic dampers, are prone to low-cycle fatigue (Naeim & Kelly, 1999).

The energy dissipation in *high-damping rubber bearings* is achieved by special compounding of the elastomer. Damping ratios will generally range between 8% and 20% of critical. The material is nonlinear at shear strains less than 20% and characterized by higher stiffness and damping, which minimizes the response under wind load and low-level seismic load. Over the range of 20-120% shear strain, the modulus is low and constant. At large shear strains, the modulus and energy dissipation increase. This increase in stiffness and damping at large strains can be exploited to produce a system that is stiff for small input, is fairly linear and flexible at design level input, and can limit displacements under unanticipated input levels that exceed design levels. The advantages to using high-damping laminated bearings are: these bearings combine the needed flexibility and energy dissipation in a single element; they are easy to design and manufacture; and they are compact, greatly simplifying the installation process. They have been used for 20 years and are installed in a variety of buildings in the United States, Japan, and Italy. The disadvantages are: the material characteristics of these high-damping elastomers are more sensitive to temperature and frequency than low-damping rubber bearings and the most important characteristic of the high-damping rubber bearings is a dependence on load history (Naeim & Kelly, 1999).

Sliding Systems:

The second most common type of isolation system uses sliding elements. These sliding systems performed very well under a variety of severe earthquake loading and are very effective in reducing the large levels of the superstructure's acceleration. This approach assumes that a low level of friction will limit the transfer of shear across the isolation interface, the lower the coefficient friction, the lesser the shear transmitted. However, to provide adequate resistance to wind load and avoid unnecessary movement under small earthquakes or other disturbances, a fairly high value of frictional coefficient is needed (Bozorgnia, & Vitelmo, 2004). Reducing the coefficient of friction or increasing the sliding

period, reduces base shear and increases displacement. Furthermore, any sudden change in the stiffness of the overall structure when slipping or sticking occurs has the effect of generating high-frequency vibrations in the structure. They offer very predictable performance and can accommodate much larger levels of displacements than rubber bearings. Also, these systems offer more space efficiency than rubber bearings with the same displacement capacity. On the other hand, these devices are more expensive than rubber bearings due to construction costs.

The *pure-friction type base isolator* is essentially based on the mechanism of sliding friction. The use of layer of sand or roller in the foundation of the building is the example of pure-friction base isolator and by this way; the superstructure and the substructure are separated by a sliding joint. Under normal conditions of small magnitude earthquakes, the system acts like a fixed base system due to the static frictional force. For large earthquake, the static value of frictional force is overcome and sliding occurs.

The *resilient-friction base isolation (R-FBI)* system consists of concentric layers of teflon coated plates that are in friction contact with each other and contains a central core of rubber. It combines the beneficial effect of friction damping with that of resiliency of rubber. The rubber core distributes the sliding displacement and velocity along the height of the R-FBI bearing. They do not carry any vertical loads and are vulcanized to the sliding ring. The system provides isolation through the parallel action of friction, damping and restoring force (Naeim & Kelly, 1999).

“Electric de France” (EDF) is standardized for nuclear power plants in region of high seismicity which developed by Gueraud et. al. (1985). The base raft of the power plant is supported by the isolators that are in turn supported by a foundation raft built directly on the ground. The main isolator of the EDF consists of laminated (steel reinforced) neoprene pad topped by lead-bronze plate that is in friction contact with steel plate anchored to the base raft of the structure. The EDF base isolator essentially uses elastomeric bearing and friction plate in series. For lower amplitude ground excitation the lateral flexibility of neoprene pad provides base isolation and at high level of excitation sliding will occur which provides additional protection. This dual isolation technique was intended for small earthquakes where the deformations are concentrated only in the bearings. However, for larger earthquakes the bronze and steel plates are used to slide and dissipate seismic energy (Naeim & Kelly, 1999).

Friction pendulum system (FPS) seismic isolation system is the concept of sliding bearings combined with the concept of a pendulum type response (Zayas et al., 1990). In FPS, the isolation is achieved by means of an articulated slider on spherical, concave chrome surface. The system acts like a fuse that is activated only when the earthquake forces overcome the static value of friction. Once set in motion, the bearing develops a lateral force equal to the combination of the mobilized frictional force and the restoring force that develops as a result of the induced rising of the structure along the spherical surface. If the friction is neglected, the equation of motion of the system is similar to the equation of motion of a pendulum, with equal mass and length equal to the radius of curvature of the spherical surface. The seismic isolation is achieved by shifting the natural period of the structure. The natural period is controlled by selection of the radius of curvature of the concave surface. The enclosing cylinder of the isolator provides a lateral displacement restraint and protects the interior components from environmental contamination. The displacement restraint provided by the cylinder provides a safety measure in case of lateral forces exceeding the design values (Kunde & Jangid, 2003). Figure 2.19 below shows an example of friction pendulum sliding isolation system.

Figure 2.19 - Example of friction pendulum sliding isolation system

- **Passive Damping Devices**

The principal function of a passive energy dissipation system is to reduce the inelastic energy dissipation demand on the framing system of a structure. The result is reduced damage to the framing system (Constantinou & Symans, 1993). A number of passive energy dissipation devices have been developed. Devices that have most commonly been used for seismic protection are viscous fluid dampers, viscoelastic solid dampers, friction dampers, and metallic dampers.

These devices can significantly reduce the displacement and acceleration responses and decrease the shear forces along the building height. By using passive damping devices it is aimed to achieve; (1) to provide the building with additional stiffness and damping to reduce the response, (2) to restrict energy dissipation provided mainly by damping devices, (3) to

limit the possible damage to the supplemental dampers which are easier to replace than structural components (Symans et. al., 2008).

Viscoelastic Devices

Viscoelastic devices consist of dampers whose force output is dependent on the rate of change of displacement across the damper. The behavior of such dampers is commonly described using various models of linear viscoelasticity. Examples of such dampers include viscoelastic fluid dampers and viscoelastic solid dampers (Symans et. al., 2008).

Viscoelastic fluid dampers, consist of a hollow cylinder filled with fluid typically being silicone based. Different viscous materials have been considered to enhance stiffness and damping properties of the main structure. As the damper piston rod and piston head are stroked, fluid is forced to flow through orifices either around or through the piston head. The input energy is dissipated by viscous heating due to the friction between fluid particles and device components. The resulting differential in pressure across the piston head can produce very large forces that resist the relative motion of the damper (Lee and Taylor, 2001). The friction forces may cause temperature increase due to friction and this can be significant, when the damper is subjected to long-duration or large-amplitude motions. The possible fluid leakage over long time and leaking out due to wearing in seals after small motions of structure can be shown as other disadvantages.

Viscous fluid dampers are commonly used passive energy dissipation devices for seismic protection of structures. These dampers generally have minimal influence on the fundamental natural frequency. And they are the most useful where engineers desire to reduce displacement without increasing the structure's frequency. Their ability to be activated at low displacements and minimal restoring force can be shown as other advantages. Figure 2.20 below shows properties of fluid viscoelastic device.

Figure 2.20 - Fluid viscoelastic device

Viscoelastic Solid Devices are intentionally designed to provide stiffness in addition to damping. These dampers provide damping forces via fluid orificing and restoring forces via compression of an elastomer. Typical viscoelastic dampers consist of polymeric material layers bonded between steel plates. These devices are designed to dissipate vibration energy in the form of heat when subjected to cyclic shear deformations. Viscoelastic solid dampers, unlike viscoelastic fluid dampers, exhibit stiffness that the dampers will influence the natural frequencies of the structure.

The steel plates are attached to the structure within chevron or diagonal bracing. As one end of the damper displaces with respect to the other, the viscoelastic material is sheared resulting in the development of heat which is dissipated to the environment. Due to they are displacement and velocity dependent by their nature, viscoelastic solids exhibit both elasticity and viscosity. The damper has the ability to store energy in addition to dissipating energy. The major advantages of solid viscoelastic dampers are, to be activated at low displacements and to provide restoring forces. However, limited deformation capacity, to have properties temperature and frequency dependent and possible tearing of viscoelastic material can be shown as disadvantages (Symans et. al., 2008). Figure 2.21Figure 2.20 below shows properties of solid viscoelastic device.

Figure 2.21 - Solid viscoelastic device

Hysteretic Devices

Hysteretic dampers consist of force output which is not dependent on the rate of change of displacement across the damper but rather upon the magnitude of the displacement and possibly the sign of the velocity i.e., the direction of motion. The behavior of such dampers is commonly described using various nonlinear hysteretic models. Examples of such dampers include metallic and friction dampers (Symans et. al., 2008).

Metallic Dampers utilize the hysteric behavior of the metals in the inelastic range. Therefore, the resisting force of dampers depends on the non-linear stress-strain characteristics of the material. These devices are capable of providing buildings with increased stiffness, strength and energy dissipation capacity. The advantages of this type of dampers are their stable hysteric behavior, long term reliability and good resistance to environmental and thermal conditions. Possible requirement for non-linear analysis and the need for replacement after earthquake can be shown as disadvantages. Different devices that utilize flexure, shear or extensional deformation in plastic range have been developed.

The ADAS (Added Damping and Stiffness) devices consist of a series of steel plates wherein the bottom of the plates are attached to the top of a chevron bracing arrangement and the top of the plates are attached to the floor level above the bracing. As the floor level above deforms laterally with respect to the chevron bracing, the steel plates are subjected to a shear force. The shear forces induce bending moments over the height of the plates, with bending occurring about the weak axis of the plate cross section. Thus, inelastic action occurs uniformly over the full height of the plates due to the geometrical configuration of the plates. To ensure that the relative deformation of the ADAS device is approximately equal to that of the story in which it is installed, the chevron bracing must be very stiff. The dissipated energy in an ADAS damper is the result of inelastic material behavior and thus the ADAS damper will be damaged after an earthquake and may need to be replaced (Symans et. al., 2008). Extensive experimental studies have been carried out to investigate the behavior of ADAS damper (Bergman & Goel, 1987).

Lead Extrusion Device (LED) is another type of damper which utilizes the hysteric energy dissipation of metals. The process of extrusion consists of forcing a material through a hole or an orifice so altering its shape. They advantages of this type of damper are; their load deformation relationship is stable and unaffected by number of cycles, they are insensitive to environmental conditions and aging effects, they have long life and they don't need to be replaced after an earthquake (Sadek et. al.). Figure 2.22Figure 2.20 below shows properties of metallic damper.

Figure 2.22 - Metallic damper

Friction Dampers rely on the friction developed between two solid bodies sliding relative to each other. During severe seismic excitations, the device slips at a predetermined load, providing the desired energy dissipation by friction while at the same time shifting the structural fundamental mode away from the earthquake resonant frequency. The development of friction devices for use in civil structures to control seismic response was pioneered in the late seventies (Pall & Marsh, 1982).

Several design variations of these friction dampers have been proposed and developed for energy dissipation in structures. Slotted-bolted dampers (Grigorian et al. 1993) wherein a series of steel plates are bolted together with a specified clamping force, Pall cross-bracing friction damper which consists of cross-bracing that connects in the center to a rectangular damper (Pall & Marsh, 1982) or cylindrical friction damper in which the damper dissipates energy via sliding friction between copper friction pads and a steel cylinder (Soong & Dargush 1997) can be shown as examples of friction dampers. Most of these devices develop rectangular hysteresis loops. The advantages of these dampers are; simplicity to construct, their large energy dissipation capacity per cycle and their good resistance to thermal conditions. Also, their behavior is relatively less affected by load frequency and number of cycles. The devices differ in their mechanical complexity and in the materials used for sliding surfaces. Possible requirement for non-linear analysis due to non-linear behavior, permanent displacements in the case of not having restoring force and the difficulty in maintaining their properties over prolonged time are the main disadvantages. Figure 2.23 below shows properties of friction damper device.

2.2. Earthquake Design Philosophy

Intensity of an earthquake at a location during an earthquake can be minor, moderate and strong. Minor shaking occurs frequently; moderate shaking occasionally and strong shaking

rarely. Due to uncertainties involved in estimating magnitude and return period of an earthquake ground shaking during the time of a structure, the performance level specified by the code regulations includes an implied risk. The engineering intention behind earthquake resistant design is not to make earthquake-proof buildings that will not get damaged even during the rare but strong earthquake; such buildings will be too robust and also too expensive. Instead, the engineers make buildings to resist the effects of ground shaking, although they may get damaged severely but would not collapse during the strong earthquake. Thus, safety of human life and contents inside of the building are assured in earthquake resistant buildings. This is a major objective of seismic design codes throughout the world. The earthquake design philosophy may be summarized as follows;

- i) Under minor but frequent shaking, the main members of the building resist earthquake impact without being damaged (staying at elastic range); however building parts that do not carry load may sustain repairable damage.
- ii) Under moderate but occasional shaking, the main members may sustain some repairable damage, while the other parts of the building may be damaged even may need replacement.
- iii) Under strong but rare shaking, the main members may sustain severe (even irreparable) damage, but the building should not collapse.

The important buildings, like hospitals and fire stations, play a critical role in post-earthquake activities and must remain functional immediately after the earthquake. These structures must sustain very little damage and should be designed for a higher level of earthquake protection. Likewise, dams, nuclear power plant, etc. should be designed for higher level of earthquake motion not to cause another disaster after a strong ground motion. Figure 2.24 shows schematic system behavior for seismic demands.

Figure 2.24 - Schematic system behavior for low, moderate and high seismic demands (Hines & Fahnestock, 2010)

Design of buildings to resist earthquakes involves controlling the damage to acceptable levels at a reasonable cost depending on the occupancy category of the structure and intensity of the ground shaking (Ambrose and Vergun, 1999).

Therefore, the engineers are concerned about ensuring that the damages in buildings during earthquakes are of the acceptable levels, and also that they occur at the right places and in right amounts. Due to seismic loading is cyclical; earthquake forces can cause destructive damage. Under a major earthquake, the frame members are expected to resist several cycles of reversed inelastic deformation and yield beyond elastic limit of the structure without any significant loss in stiffness or strength. In other word, lateral load resisting system should have a good energy absorption capacity by making sure that gravity load carrying system is not damaged. This is expressed as ductility and it is one of the most important factors affecting the building performance. Earthquake resistant design philosophy requires the main elements of the structures need to be built with ductility in them with the ability to sway back-and-forth during an earthquake, and to withstand earthquake effects with some damage, but without collapse. Thus, earthquake-resistant design predetermines the locations where damage takes place and then to provide good detailing at these locations to ensure ductile behavior of the building (Murty, 2002).

The seismic design philosophy in the codes relies on the ductility of the system to the design lateral loads. ASCE 7 account ductility by reduction from elastic forces by response modification factors. Different structural systems have different energy absorption capacity of ductility which codes specify different values for the response modification factors. The level of the design force for an inelastic behavior is lower than elastic material and it is a function of the structure's ability to dissipate energy during earthquake. However, if the structure was expected to respond elastically during a major event, no reduction would be allowed in the level of lateral forces. To ensure ductile behavior during a major earthquake, current design provisions require special detailing of frame members and connections such as strong column-weak beam concept which intends that the plastic hinges may occur occur in beams by providing columns with enough ductility. Although it is possible to design a structure to respond elastically during a seismic event, it would be extremely conservative and highly uneconomical.

2.3. Building Codes & Standards

All structures must be designed and constructed in accordance with the design requirements of the applicable codes in order to get permission from local jurisdictions. For instance, areas with high seismicity would require stricter code guidelines compared to areas of low seismicity. Although the differences of seismic regulations among jurisdictions, seismic codes are written with the goals given below: i) provide collapse prevention and a minimum life safety ii) increase the expected structural performance depending on the occupancy or use iii) maintain the continuity of service to function after earthquake for essential facilities.

By the mid-1900s, three organizations were publishing model building codes for adoption by U.S. communities and each represented a major geographic region:

- The Building Officials and Code Administrators International (BOCAI) published the National Building Code that served as the basis for most building regulation in the northeastern and central states.
- The Southern Building Code Congress International (SBCCI) published the Standard Building Code that was commonly adopted throughout the southeastern part of the country.
- The International Conference of Building Officials (ICBO) published the Uniform Building Code that was commonly adopted in the western United States.

In the late 1990s, the three original code development organizations (BOCAI, ICBO, and SBCCI) agreed to merge into a single organization called the International Code Council (ICC) and published a single series of model building codes (called the International or I-Codes) every three years (i.e., 2000, 2003, 2006, 2009, 2012) that intended to be nationally and internationally applicable and include:

- The International Building Code (IBC) that addresses almost all types of buildings including residential, commercial, institutional, government, and industrial structures;

As the model building codes were evolving, professional associations from various industries developed their technical criteria for the design and construction of structures. The various industry standards typically are revised and updated every five years. Among the more important consensus standards presently referenced by the building codes are the following:

- Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7, published by the Structural Engineering Institute of the American Society of Civil Engineers;
- Building Code Requirements for Reinforced Concrete, ACI 318, published by the American Concrete Institute;
- National Design Specification, NDS, published by the American Forest and Paper Association;
- Specification for Steel Buildings, AISC 360, published by the American Institute of Steel Construction;
- North American Specification for the Design of Cold Formed Steel Structural Members, AISI S100, published by the American Iron and Steel Institute; and
- Building Code Requirements and Specification for Masonry Structures, TMS 402/ACI 530/ASCE 5 and TMS 602/ACI 530.1/ASCE 6, jointly published by the Masonry Society, the American Concrete Institute, and the American Society of Civil Engineers.

During the early years of seismic code provision development, the principal basis for code changes was observation of the performance of actual buildings in earthquakes. When an earthquake occurred, engineers and building officials would survey the damage and, when certain types of construction performed poorly, they would develop code changes to address the observed problems. The development of seismic requirements for building codes occurred primarily in the western states, notably California. These earthquake design requirements initially were developed by volunteers from the Structural Engineers Association of California (SEAOC) in cooperation with International Conference of Building Officials (ICBO). These initial requirements appeared as a non-mandatory appendix in the 1927 Uniform Building Code. The 6.6 magnitude earthquake occurred in San Fernando Valley, California in 1971, which resulted in extensive damage and the collapse of some such structures, made it clear that the building code needed significant improvement. First, SEAOC formed a nonprofit entity – the Applied Technology Council (ATC) – to seek the funding needed to assemble the best available talent to research problems with the building code requirements and to develop recommendations for improving those requirements.

At about the same time, the National Earthquake Hazards Reduction Program (NEHRP) was established. Under the NEHRP, four federal agencies – the Federal Emergency Management Agency (FEMA), the National Institute of Standards and Technology (NIST), the National Science Foundation (NSF), and the United States Geological Survey (USGS) – were

authorized to develop effective ways to mitigate earthquake risks to the national economy and the life safety of building occupants. Under the NEHRP, the USGS focuses on identification of the level of earthquake hazard throughout the United States and develop the national seismic hazard maps that serve as the basis for the design maps incorporated into the NEHRP Recommended Seismic Provisions and building codes and standards. NSF fosters technological leadership by sponsoring basic research and the development of new generations of scientists and engineers. NIST conducts research and development work and also supports public/private partnerships that perform such work with the goal of improving the technological competitiveness of the United States. FEMA provides public and individual assistance after an earthquake disaster occurs, speeding community recovery and minimizing the disaster's impact on the nation (FEMA P-749, 2010).

2.4. Seismic Hazard & Accepted Risk

Physically, an earthquake is a result of a sudden move between main tectonic plates, covering the surface of the globe. There are several procedures to represent the seismic actions such as map-based and site-specific procedures. Map-based procedures use maps of peak ground accelerations to define seismic input at different hazard levels and under different site conditions (Fardis et al., 2005). With the aid of maps of peak ground accelerations, the earthquake ground motion at a given site is usually characterized by the response spectrum.

To obtain the objectives of a seismic design or a seismic retrofit of the existing buildings, earthquake ground motions representing seismic hazards have to be defined. The seismic hazard analysis is concerned with getting an estimate of the strong-motion parameters at a site for the purpose of earthquake resistant design or seismic safety assessment. Two basic methodologies used for the purpose are the “deterministic” and the “probabilistic” seismic hazard analysis approaches. In the deterministic approach, the strong-motion parameters are estimated for the maximum expected earthquake, assumed to occur at the closest possible distance from the site of interest, without considering the likelihood of its occurrence during a specified exposure period. On the other hand, the probabilistic approach integrates the effects of all the earthquakes expected to occur at different locations during a specified life period, with the associated uncertainties and randomness taken into account. However, regardless of the approaches, the final aim is to present seismic engineering characteristics of the shaking at the site for a given earthquake.

The term serviceability earthquake is used to refer to an earthquake level that has only 50% chance of being exceeded in a 50-year period. No damage is expected in this level of seismic hazard that has a good chance of occurring during service life of building. In an extreme event called maximum considered earthquake (MCE) which has 2% chance of being exceeded in a 50-year period that serious damage without collapse can be expected. The risk involved depends on the level of the earthquake. The seismic zone and site characteristics will govern the prescribed maximum considered earthquake. But the chance of such an extreme event during the life time of the structure can be very low. It is possible of course to design buildings not have damage even at maximum considered earthquake, however it would be extremely costly and not feasible. Figure 2.25 illustrates an approximate relationship between expected damage and the earthquake level per recommended in design codes.

Figure 2.25 - Relationship between expected damage to building and earthquake levels

The building codes, industry standards and NEHRP Recommended Seismic Provisions are based on the concept of “acceptable risk,” which attempts to balance the cost of seismic-resistant construction against the chance of unacceptable losses in future earthquakes.

3. SEISMIC EVALUATION AND REHABILITATION OF EXISTING STRUCTURES

3.1. Introduction

Unfortunately many existing structures that were built according to past design codes & standards are often found vulnerable to earthquake damage. This fact becomes more obvious every time when there is a major earthquake and the same patterns of damage are observed. Structures like steel moment frames designed before Northridge earthquake with non-ductile connections can experience significant damage or unreinforced masonry without certain characteristics may collapse by a separation of their elements or a wood frame house can be subject to substantial damage or even collapse if it lacks interconnection of the various elements, etc. The 1985 Mexico City and 1994 Northridge earthquakes demonstrated that even large steel frame buildings can be vulnerable to major damage or even collapse. Past earthquakes have also shown that damages are not only concentrated in a few types of buildings, often many poor buildings perform well in major earthquakes. Local ground conditions, focusing effects, construction quality, and building shape and size have all been suggested as possible explanations. These experiences show that no building is automatically immune from damage because of its style of construction. Unlike the traditional structural design for gravity loads, seismic design anticipates that the buildings will be damaged after a major seismic event. According to design codes, buildings are aimed to be strong enough to resist small earthquakes without damage and major earthquakes without collapse. To accomplish this goal, the structural design requires a combination of basic lateral-force-resisting strength with a proper structural detailing and appropriate ductile connections of the structural elements.

The high cost of new construction and historical importance of older buildings has led many residential home owners and major industries including governmental agencies to renovate rather than replace the existing structures. This has caused governmental institutions to implement mandatory seismic strengthening regulations. The awareness of damage potential by owners and the observation that older non-code-complying buildings sometimes perform successfully in major earthquakes, pointed out the need for a specific evaluation and strengthening standard for existing buildings. In this way, seismic rehabilitation techniques for buildings need to include both the evaluation of the existing lateral-force-resisting elements of the building and the addition of new elements where necessary.

The major difference between designing a new building and evaluation of an existing structure is the design perspective according to the applicable codes. The design of new buildings usually employs force-based procedures and the buildings are modeled as elastic systems with stresses proportional to strains. Although actual earthquake forces and deflections may be larger than the calculated forces and deflections per code, a building may survive by dissipating energy in the yielding of its components. The design lateral forces are obtained from a base shear formula that includes a response modification factor (R) that reflects the relationship between required strength and ductility. In this concept, structural systems with greater ductility and consequently larger factor of R have lower required forces for design. On the other hand, existing buildings are usually analyzed with displacement-based procedures which assess the ductility of each element action individually. Due to the ductility of individual elements in the structural system may not be consistent with each other. In evaluation the aim is to determine how a building will respond to design earthquake by finding weak links in the system and to identify how they will affect the response of the structural systems.

3.2. Code Basics

The initial basic provisions of evaluating existing buildings were based on the thought that buildings resisted earthquakes by strength alone. The 1971 San Fernando Earthquake have brought a new understanding that buildings needed both strength and ductility as such provisions of detailing for new buildings. Recent code provisions also evolved performance-based seismic design to focus on better building behavior and performance to reduce and limit economic losses. While the main focus has been on improving seismic provisions for new buildings, seismic provisions for evaluation of existing buildings are limited. Existing buildings are already constructed, the materials are defined and details of construction are in place. Therefore, a different approach was needed for existing buildings and various guidelines have been published over the years (Darrick & Poland, 2004).

The current seismic evaluation and rehabilitation philosophy were based on the following documents:

- ASCE 31-03, Seismic Evaluation of Existing Buildings
- ASCE 41-06, Seismic Rehabilitation of Existing Buildings

These documents are meant to be used together in the rehabilitation process, with the first step being an evaluation using ASCE 31-03 to determine deficiencies, followed by a rehabilitation scheme being developed (if necessary) using ASCE 41-06. These documents

were developed specifically to address the evaluation and retrofit of existing buildings. If a building is determined to have deficiencies per ASCE 31-03, then the upgrade scheme is developed using the more rigorous requirements of ASCE 41-06.

Applying ASCE 31-03 and ASCE 41-06 to develop the upgrade schemes has several advantages. Primarily, ASCE 41-06 addresses existing materials and types of construction that the current building code does not address. This allows one to take advantage of existing structural elements that the current building code would not, which can significantly reduce the scope of the seismic upgrade work. Additionally, ASCE 41-06 is a performance based approach so the seismic hazard (i.e. how large an earthquake is considered) and building performance (i.e. how much damage is acceptable) can be targeted to the owner's needs. This provides additional flexibility to the owner in case they want a building that either exceeds the requirements of the building code or is not fully code compliant.

3.2.1 ASCE 31-03

ASCE 31-03: *Seismic Evaluation of Existing Buildings (2003)*, has been the first nationally applicable seismic evaluation standard has improved the previous seismic evaluation documents including ATC-14: *Evaluating the Seismic Resistance of Existing Building (1987)*, FEMA 178: *NEHRP Handbook for the Seismic Evaluation of Existing Building (1992)* and FEMA 310: *Handbook for the Seismic Evaluation of Buildings (1998)*.

Since the beginning of the 20th century, structural engineers have learned the most about the behavior of structures by visiting the sites damaged by earthquakes. The first seismic code provisions were the result of observations from earthquakes. ASCE 31-03 has its roots both in the lessons learned from past earthquakes and latest developed performance based seismic analysis techniques. Also it has adopted a methodology which allows focusing only on the elements that have been shown to be weak links in earthquakes. The result is either the verification of the adequacy of an existing building or the identification of the extent of seismic strengthening required. ASCE 31 accepts a philosophy of permitting existing buildings to be evaluated and even upgraded to a lower hazard than new buildings. The concept was contained with ATC-14 and carried through to ASCE 31. In Tier 2 procedures of ASCE 31, 75% of new building design forces were carried out by using the m-factors. One of the reasons for this procedure is the increased risk due to the lower hazard is acceptable because of the presumption that an existing building has a shorter remaining life than a new building (Pekelnicky, Poland, 2012).

Evaluation process in ASCE 31-03 consists of three different tiers: Screening Phase (Tier-1), Evaluation Phase (Tier-2) and Detailed Evaluation Phase (Tier-3). Tier-1 Screening Phase intends to identify potential deficiencies. This phase of ASCE 31-03 consists of checklists defining building features that have vulnerabilities to behave poorly during seismic events. Based on the performance level and seismicity, the design professional chooses the appropriate structural, geologic and nonstructural checklists using information from existing documentation, testing results and site visits. If the building doesn't meet the chosen performance level, it may be classified as to have some potential deficiencies and the design professional may choose to continue on to a Tier 2 Evaluation, or stop the evaluation and report the findings.

Tier-2 Evaluation intends to identify any weak links in the building through linear analysis and the deficiencies that are identified in the Tier 1 Phase are analyzed. ASCE 31-03 uses the performance-based methodology of pseudo lateral forces originally developed for FEMA 273: NEHRP Guidelines for the Seismic Rehabilitation of Buildings instead of the traditional equivalent lateral force methodology in current codes. The building is evaluated at the expected displacement of the structure during the demand earthquake. The forces for each component are determined, and then the components are evaluated based on the ductility of the element. This ductility, or m-factor, reduces to the pseudo force level to a point where the component can be evaluated on a realistic force level. In order to evaluate the components, each element must be classified as either deformation or force-controlled. Deformation-controlled elements are those that provide deformation to the entire building through inelastic behavior like a flexural hinge in a beam and they are evaluated using m-factors. Force-controlled elements are those that experience little or no inelastic behavior before loss of strength and they are evaluated for the maximum force that can be delivered by surrounding elements. In a Tier 2 Evaluation to meet the chosen performance level, force-controlled elements must be prevented from experiencing substantial yielding and the deformation-controlled elements must not exceed their ductility capacity.

Once the Tier2 Evaluation is complete, if the building doesn't meet the chosen performance level, the design professional may choose to continue on to a Tier 3 Evaluation. The Tier 3 provisions require the modeling of the nonlinear response of the building, with the purpose of determining the failure mechanism for the structure due to a linear analysis was already performed in Tier 2. As a result it uses the nonlinear analysis techniques developed such as those in FEMA 356: *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*.

3.2.2 ASCE 41-06

ASCE 41-06: *Seismic Rehabilitation of Buildings (2006)* serves to provide a standard for nationally applicable provisions in the seismic rehabilitation of existing buildings and supersedes the previous standards; FEMA 273: *NEHRP Guidelines for the Seismic Rehabilitation of Buildings (1997)* and FEMA 356: *Prestandard and Commentary for Seismic Rehabilitation of Buildings (2000)*. While ATC 14 (1987) created the concept of screening buildings for potential deficiencies, FEMA 273 (1997) was the first standard that provided "displacement based" methodologies for nonlinear analysis of all types of structures. Prior to those documents, seismic evaluation and retrofit was primarily depended on to the judgment of the design professional by using the standards for new building design to evaluate and retrofit existing buildings.

ASCE 41-06 defines seismic rehabilitation by improving the seismic performance of structural and/or nonstructural components of a building by correcting deficiencies identified in a seismic evaluation. Unlike ASCE 7, which employs "force-based" procedures by utilizing a global building ductility factor (R-factor), ASCE 41 uses "displacement-based" procedures which assess the ductility of each element action (shear, flexure, etc.) individually. Also, ASCE 41 contains specific guidance on the use of nonlinear analysis procedures which ASCE 7 doesn't contain. ASCE 41 can also be used to rehabilitate the historic structures where performance based rehabilitations are desired.

If seismic upgrading interest is found after defined methodology to identify the deficiencies, several considerations should be studied such as; structural characteristics, site seismic hazards, results from prior seismic evaluations, historic status, economic considerations and societal issues. Economical considerations are proven to be one of the most decisive aspects for whether a retrofit consideration goes from planning to implementation. After the initial phase, if the rehabilitation project is decided to be done, rehabilitation must be done in

accordance with target building performance level, earthquake hazard level and rehabilitation objective classification. ASCE 41-06 defines six structural performance levels expected for post earthquake state shown in Table 3.1 below:

- Immediate Occupancy (S1)
- Damage Control Range (S2)
- Life Safety (S3)
- Limited Life Safety Range (S4)
- Collapse Prevention (S5)
- Not Considered (S6)

Table 3.1- Target building performance and levels from ASCE 41-06 Table C1-8

Nonstructural Performance Levels	Structural Performance Levels and Ranges					
	Immediate Occupancy (S-1)	Damage Control Range (S-2)	Life Safety (S-3)	Limited Safety Range (S-4)	Collapse Prevention (S-5)	Not Considered (S-6)
Operational (N-A)	Operational 1-A	2-A	Not recommended	Not recommended	Not recommended	Not recommended
Immediate Occupancy (N-B)	Immediate Occupancy 1-B	2-B	3-B	Not recommended	Not recommended	Not recommended
Life Safety (N-C)	1-C	2-C	Life Safety 3-C	4-C	5-C	6-C
Hazards Reduced (N-D)	Not recommended	2-D	3-D	4-D	5-D	6-D
Not Considered (N-E)	Not recommended	Not recommended	Not recommended	4-E	Collapse Prevention 5-E	Not rehabilitation

Operational Occupancy Performance Level: The post earthquake damage to the structure is very light. There is no permanent building drift. The structure maintains its original strength and stiffness. There is very little damage. The backup building services maintain function.

Immediate Occupancy Performance Level: The post earthquake damage to the structure is light. There is no permanent building drift. The structure maintains most of its original strength and stiffness. The risk to life threatening injury from structural damage is very low. Some minor repairs may be appropriate, but are not required for re-occupancy.

Life Safety Performance Level: The post-earthquake damage to the structure is significant, but some margin against either partial or total structural collapse remains. Some structural elements and components are severely damaged but this has not resulted in large falling debris hazards, either inside or outside the building. Injuries may occur during the earthquake; however the overall risk of life threatening injury as a result of structural damage is expected to be low. It should be possible to repair the structure; however for economic

reasons this may not be practical. Although the damaged structure is not an imminent collapse risk, it would be prudent to implement structural repairs or install temporary bracing prior to reoccupying the building.

Collapse Prevention Performance Level: The post-earthquake damage is so significant that the building is on the verge of partial or total collapse. Substantial damage to the structure has occurred, potentially including significant degradation in the stiffness and strength of the lateral-force-resisting system, large permanent lateral deformation of the structure, and (to a limited extent) degradation in vertical load carrying capacity. However, all significant components of the gravity load resisting system must continue to carry their gravity loads. The structure may not be technically practical to repair and is not safe to reoccupy.

It should be noted, that Immediate Occupancy and Operational Performance Levels are very costly and typically not practical for structures unless they are needed to maintain their service after an earthquake like hospital, police stations, etc. Expected post-earthquake damage states for target building performance levels are shown in the Figure 3.1 below.

Figure 3.1 - Target Building performance levels from ASCE 41-06 Figure C1-2

The expected post-earthquake damage states are shown in the Figure 3.2 below;

Figure 3.2 - Expected post-earthquake damage states from ASCE 41-06 (Courtesy of R.Hamburger)

Figure 3.3 below shows the performance and structural deformation demand for ductile systems.

Figure 3.3 - Performance and structural deformation demand for ductile systems (FEMA 274 Figure C2-3)

Rehabilitation objectives are divided into three groups:

Buildings meeting the *Basic Safety Objective (BSO)* are expected to experience little damage from relatively frequent, moderate earthquakes but significantly more damage and potential economical loss from the most severe and infrequent earthquakes. It is typical for office or residential buildings. As shown in Table 3.2; for life safety building performance (3-C) BSE-1 earthquake hazard level and for the collapse prevention building performance level (5-E) BSE-2 earthquake hazard level are aimed.

Enhanced Rehabilitation Objectives can be obtained by designing higher target building performance levels (method 1) or by designing using higher earthquake hazard levels

(method 2), or a combination of those. It is typical for critical and essential structures such as hospitals, police stations, fire stations, etc.

Limited Rehabilitation Objective provides building performance less than BSO. It is typical in less critical structures.

Table 3.2- Rehabilitation objectives from ASCE 41-06 Table C1-1

Earthquake Hazard Level		Target Building Performance Levels			
		Operational Performance Level (1-A)	Immediate Occupancy Performance Level (1-B)	Life Safety Performance Level (3-C)	Collapse Prevention Performance Level (5-E)
	50%/50 year	a	b	c	d
	20%/50 year	e	f	g	h
	BSE-1 (~ 10%/50 year)	i	j	k	l
	BSE-2 (~ 2%/50 year)	m	n	o	p

¹Each cell in the above matrix represents a discrete Rehabilitation Objective.

²The Rehabilitation Objectives in the matrix above may be used to represent the three specific Rehabilitation Objectives defined in Sections 1.4.1, 1.4.2, and 1.4.3, as follows:

Basic Safety Objective (BSO)	k and p
Enhanced Objectives	k and m, n, or o p and i or j k and p and a, b, e, or f m, n, or o alone
Limited Objectives	k alone p alone c, d, g, h, or l alone

ASCE 41-06 divides earthquake hazard levels into four possibilities as shown below in Table **3.3** where two basic earthquake hazard levels, Basic Safety Earthquake 1 (BSE-1) and Basic Safety Earthquake 2 (BSE-2) are defined. BSE-1 event is an earthquake with a 10% probability in 50 years of being exceeded. This is an earthquake with a 500 years reoccurrence period or the design event defined in ASCE 7 or IBC. And BSE-2 event is an earthquake with a 2% probability in 50 years of being exceeded. This is an earthquake with 2500 years reoccurrence period or the MCE as defined in ASCE 7 or IBC. These mean return periods shown in

Table **3.3** are typically rounded to 75, 225, 500 and 2,500 years, respectively.

Table 3.3- Earthquake hazard levels from ASCE 41-06

Earthquake Having Probability of Exceedance	Mean Return Period (years)
50%/50 year	72
20%/50 year	225
10%/50 year	474
2%/50 year	2,475

After the rehabilitation objective is defined, data collection (from existing plans or tests), analysis requirements definition, modeling, results evaluation, design and rehabilitation steps are followed.

3.3. Categories of Seismic Deficiencies

Earthquakes cause a serious threat to human life and create serious impacts to the economic and social development. Besides the cost of repairs of buildings and infrastructures, the disruption of the businesses and services can have a continuous negative effect. The requirement for earthquake-resistant design to reduce the hazards of new constructed buildings were first acknowledged in the 1906 San Francisco earthquake but appropriate design considerations were not employed until after 1933 Long Beach earthquake. Today earthquake resistant structure in new constructions is accepted practice in seismic regions of the US. However, a large number of existing buildings in the country can be assumed to have inadequate resistance and creates a serious risk.

Seismic vulnerability can be determined by a careful evaluation of the structural system, material and the detailing used. An analysis should quantify the resistance capacity of the structure. An important part of the analysis is the application of a model with elastic and inelastic properties that takes into consideration all structural components that participate in the resistance of the building. A building's design and construction characteristics with the condition of materials used affect the seismic performance. In order to find an appropriate cost-effective seismic strengthening solution for an existing building; the structural engineer must understand the existing structural system that resist the lateral loads and any constraints on the desired performance of the system due to design or construction characteristics or deteriorated materials. In a building, strength, stiffness, ductility and damping govern the dynamic response of a structure to a ground motion. It is usually possible to compensate for a deficiency by enhancing one or more to the other. There are some design and construction characteristics that have an undesired effect on structural response by preventing the effective

development of structural capacity of structural components such described below (FEMA 547, 2006).

Global Strength: Global strength typically refers to the lateral strength of the vertically oriented lateral force-resisting system and probably it is the most obvious consideration in seismic rehabilitation. A deficiency in global strength is commonly found in older buildings due to a lack of seismic design completed in accordance with early code requirements which are inadequate per current regulations. It can be rehabilitated by strengthening the existing members or adding new members that will increase the overall strength of the structure. A seismically weak structure can be rehabilitated by strengthening existing members or by adding new members that increase the overall strength of the structure. Shear walls and braced frames are effective elements for this purpose.

Global Stiffness: Global stiffness refers to the stiffness of the entire lateral force-resisting system. Although strength and stiffness are often controlled by the same elements or the same retrofit techniques, the two deficiencies are typically considered separately. In the building codes it is intended to provide a minimum structural stiffness to control the potential for seismic damage and drift limitations. It results in higher seismic resistance by reducing the fundamental period of the building. Increasing the global stiffness of the structure is an effective strategy in the case of seismic deficiency. The location of added member and the added stiffness is important due to transfer of loads among the elements of the structure depends on the relative stiffness of those elements. The main philosophy is to provide added members with enough stiffness compared to the existing elements so that added members shall contribute efficiently into the lateral load resisting system as desired. Also, added members should be located in a way to minimize the eccentricities to limit torsional responses and the load path is not modified in a way to create other problems.

Ductility: The ductility can be defined as the ability of dissipating energy inelastically without loss in load carrying capacity and it is a significantly important consideration in seismic rehabilitation. Without of enough ductility, structures would be vulnerable to brittle failures during an earthquake. Ductility capacity depends remarkably upon material type, loading mechanism (flexural, shear, tension/compression), loading history (monotonic or cyclic) and if cyclic loading the number of cycles. Connection detailing is a key to achieve robustness through ductility. Perhaps the most common example of a detailing deficiency is poor confinement in concrete gravity columns. Ductile response of a structural component is

characterized by several cycles of inelastic deformation without significant degradation of strength or stiffness. The most desirable type of ductile response in a structural system is sustained by hysteric force-deformation cycles that dissipate energy. ASCE 7 refers to “Response Modification Factor, R” which accounts for the ductility for the building and adjusts the design lateral loads accordingly. Studies of existing buildings during earthquakes have demonstrated that ductile buildings (like flexible moment frames) perform much better in seismic events than rigid buildings (like masonry shearwall buildings) because of the inherent ability of flexible systems to dissipate the energy of the ground motion. "R" values range from 1-1/2 for unreinforced concrete and masonry shearwalls (very brittle/stiff systems) to 8 for properly detailed shearwalls, braced frames and moment frames (all very ductile/flexible systems).

Continuous Load Path: A proper load path is the most crucial requirement for a structure. A continuous load path or paths with adequate strength and stiffness shall be provided to transfer the earthquake inertia forces from the point of application to the ground through diaphragms, vertical elements and foundation. All parts of the structure between seismic separation joints shall be interconnected to form a continuous path to the seismic force resisting system and the connections shall be capable of transmitting seismic force induced by the parts connected. The ability of structure to maintain its integrity and prevent collapse during a severe earthquake is crucial to the structural safety and collapse prevention. The structural engineer should give a special importance into connection deficiencies in order to have a continuous load path for structural integrity.

Configuration Irregularities: According to the IBC, structures are designated as structurally regular or irregular. A regular structure has no significant discontinuities in plan, vertical configuration, or lateral force resisting systems. An irregular structure, on the other hand, has significant discontinuities such as those in ASCE 7. The buildings with regular geometry and uniformly distributed mass and stiffness in plan as well as in elevation suffer much less damage compared to irregular configurations. But nowadays structural irregularities are commonly found in many buildings both in plan and elevation. It is vital for the structural engineer to understand that severe irregularities can cause uncertainties in the structure's ability to meet the desired performance objectives. Vertical and horizontal irregularities can result in significantly increase in loads and deformations from those assumed by the linear procedures. In most buildings, irregular conditions exist to some degree. However; minor irregularities have little or no significant effect on the structural response. If the significant

irregularities can't be avoided, the structural engineer should act in accordance with special provisions described by the code.

The *vertical irregularities* that may unfavorably affect the seismic performance are;

- *Stiffness Irregularity* is defined to exist one or more stories that are softer than the one above.
- *Weight (Mass) Irregularity* is defined to exist where the effective mass of any story is more than 150% of the adjacent one
- *Vertical Geometric Irregularity* is defined to exist where the horizontal dimension of the story is more than 130% of the adjacent one, caused by setbacks and discontinuities.
- *Vertical Discontinuity in Load Path* is defined to exist where the lateral force-resisting elements is not continuous in a way to create reduction in stiffness.
- *Vertical Discontinuity in Lateral Strength* is defined where the story strength, which is the total strength of all seismic-resisting elements sharing the story shear, in a story is significantly less than the story above.

The *horizontal irregularities* that may unfavorably affect the seismic performance are;

- *Torsional Irregularity* occurs in buildings with rigid diaphragms when the center of the mass in a story is eccentric with respect to the center of rigidity. ASCE 7 also requires additional accidental torsion of 5% of the dimension to the inherent torsion. Torsion should be minimized by making the building symmetrical and regular in geometry and stiffness by providing lateral load resisting elements at the building's perimeter.
- *Reentrant Corner Irregularity* is defined to exist where plan configuration of the structure creates an excessive shear stress at the corners. If it is not avoidable, reentrant corners should be strengthened by using drag struts to distribute the local concentrated forces or by seismic separation.
- *Diaphragm Discontinuity Irregularity* is defined to exist where there are diaphragms with abrupt discontinuities or variations in stiffness such as 50% or more changes in open areas compared to the enclosed diaphragm area or effective stiffness one story compared to the next one. It can be improved by stiffening of the diaphragm segments. Stress concentrations at the corners of a diaphragm with large openings can

be reduced by adding collector members or drag struts to distribute the forces safely into the diaphragm in order to provide an adequate load path.

- *Out-of-Plane Offsets Irregularity* is defined to exist where there are discontinuities in the lateral force-resisting elements such as out-of-plane offsets of the vertical elements.
- *Nonparallel System Irregularity* is defined where vertical lateral force resisting elements are not parallel or symmetric about the major orthogonal axes of the seismic force resisting system. It can be strengthened by providing an adequate load path for resulting forces from diaphragm to vertical lateral load resisting systems.

The existence of an asymmetry in the plan is usually leading to an increase in stresses of certain elements that consequently results in a significant destruction during an earthquake. The most common way is to avoid these irregularities or to reduce the irregularities by modifying the existing structural elements or adding new structural elements such as locating stiff resisting elements that reduce the eccentricity and increasing the lateral force resisting capacity. The analysis of the seismic response of irregular structures is complex due to nonlinear and inelastic response and more difficult than that of regular structures. However, the IBC does not prohibit irregular structures. Instead, it contains specific design requirements for each type of irregularity. In some cases of irregularity, the static lateral force procedure is not permitted. Accordingly, the nonlinear dynamic analysis method is the best choice for solving these problems since they provide more realistic models of structural response to strong ground motion. However, such an approach is not feasible most of the time. The seismic analysis based on nonlinear static procedure (NSP) known as "pushover", can be effectively used as evaluation method to check a particular structural design.

Redundancy: Redundancy means providing multiple continuous load paths in the structural system. The overstress of a component in a redundant structure may be redistributed by creating alternate load paths to resist the seismic loads. In the lack of redundancy in a structure, the seismic capacity is mostly dependent on nonlinear behavior of the lateral load resisting elements. In the concept of rehabilitation of existing structures, a new lateral load resisting elements are added to increase the weakness in the existing seismic resistance by providing redundancy such as addition of new steel braced frame systems.

Foundation: Foundation deficiencies can occur within the foundation element itself, or due to inadequate transfer mechanisms between foundation and soil. Element deficiencies include

inadequate bending or shear strength of spread foundations, inadequate axial capacity or detailing of piles and weak and degrading connections between piles and caps. Transfer deficiencies include excessive settlement or bearing failure, excessive rotation, inadequate tension capacity of deep foundations, or loss of bearing capacity due to liquefaction. Analysis and identification of transfer deficiencies is problematic and often expensive and disruptive, it requires careful consideration. Explicit modeling of soil resistance to foundation movement can affect the overall dynamic characteristics of the structure as well as base fixity of rigid elements.

Adjacent Buildings: When the gap between buildings is insufficient to accommodate the seismic deformations of the buildings, both may be vulnerable to structural damage from the "pounding" action that results when the two collide. This condition is particularly more serious when the floor levels of the two buildings do not match and the stiffer floor framing of one building impacts on the more fragile walls or columns of the adjacent building. For buildings separated by expansion joints if the lateral systems on either side of the joint are of considerably different stiffness or strength, an independent analysis of both portions may be inadequate as loads can be transferred from one portion to the other. To minimize the possibility of pounding, a seismic separation between the buildings should be provided at least equal to the sum of the expected drifts of the two buildings. Rehabilitation technique for potential impact from adjacent buildings may be increasing the stiffness of one or both building that results the seismic deformations to desired levels. However, as mentioned above this technique may not be feasible for stiff shear wall buildings. In that case, consideration should be given to alternative load paths for vertical load resisting members in the case of damage after impact.

Deterioration of Structural Materials: Structural materials that are seriously deteriorated may create an undesirable effect on the seismic performance of an existing building during a ground shaking. The the damage or deterioration must be evaluated according to both the existing condition and the proposed seismic strengthening of the building.

Timber: Common problems with timber members that require rehabilitation include termite attack, fungus ("dry rot" or "damp rot"), warping, splitting, checking due to shrinkage, etc.

Unreinforced masonry: The weakest element in older masonry usually is the mortar joint, particularly when the lime used excessively and it was leached out by exposure to the

weather. Thus, cracks in masonry walls caused by differential settlement of the foundations or other causes generally will occur in the joints.

Unreinforced concrete: Unreinforced concrete may be subject to cracking due to excessive drying shrinkage during the curing of the concrete, spalling due to extreme temperatures, and disintegration due to dirty or contaminated aggregates, old or defective cement, or contaminated water.

Reinforced concrete or masonry: Reinforced concrete and masonry are subject to the same types of deterioration as unreinforced concrete and masonry. In addition, poor or cracked concrete or masonry may cause moisture and oxygen to penetrate to the steel reinforcement and initiate corrosion.

Structural steel: Poorly configured structural steel members may allow moisture from rainfall and initiate corrosion process and subsequently can cause loss of section for the steel member. Even well-configured steel members exposed to a moist environment require periodic maintenance such as painting or other corrosion protection to maintain their effective load-bearing capacity.

3.4. Techniques For Seismic Strengthening of Existing Structures

In order to design an efficient seismic retrofit, it is crucial to have a comprehensive understanding of the expected seismic response of the existing building and all of its deficiencies. Due to the significance in providing lateral stability and gravity load resistance, the retrofit plan focus is on vertically oriented components (e.g. column, walls, braces, etc.). However, the walls and columns may be adequate for seismic and gravity loads while the building is inadequately tied together for seismic loads.

In the traditional sense of improving the performance of existing structures, increasing strength, stiffness or deformation capacity and improving connections with a complete load path are the most common. The technical issues to be considered in the scope of retrofitting can be summarized as below; (FEMA 547, 2006)

- To have a complete load path
- To have sufficient strength and stiffness to meet the design standards
- To be compatible with the existing lateral and gravity system

The non-technical issues which have an important impact in the chosen solution system are as given as below; (FEMA 547, 2006)

- *Cost*, construction cost, short-term disruption cost to building users or the value of contents to be seismically protected are always important to define the retrofit methods.
- *Seismic Performance Expectation by Governing Jurisdiction*, objectives that require a limited amount of damage or continued occupancy will significantly affect the retrofit methods.
- *Long-term Functionality of Building*; the additional members in the interior of the building will always change the functional use plan and reduce the flexibility.
- *Aesthetics*; in old buildings performance objectives are controlled by limitations imposed by preservation of historical significance.

3.4.1 Add Elements

The most general technique of retrofitting is adding new elements such as; new shear walls, braced frames, collectors or moment frames are added to an existing building to overcome the deficiencies in strength, stiffness, configuration, or to reduce the displacement, etc. The structural engineer must assure that the new loads to be carried by these elements can be adequately delivered to other existing components by not creating another deficiency in load path that did not exist initially.

3.4.2 Enhance Performance of Existing Elements

Instead of providing additional retrofit measures in the structure, deficiencies can also be eliminated at the component level. This can be done by altering the element in a way that allows additional deformation without compromising vertical load-carrying capacity. Such as; beams yielding before columns, bracing members yielding before connections, flexural yielding before shear failure in columns and walls are preferred yielding sequences. This kind of retrofits can be provided in a variety of ways, such as strengthening connections, enhancing the shear capacity of columns and walls can be to be stronger than the shear that can be delivered by the flexural strength, providing adequate confinements in concrete columns, designing the columns stronger than the beams in the moment frames, etc.

3.4.3 Improve Connections between Components

Improving the connections is very important in the retrofit strategies because the acceptable performance level is often achieved by local strengthening of connections rather than adding new lateral force-resisting system. For example, connections with non-ductile behavior caused a threat for partial or complete collapse in the past earthquakes. A deficiency in the

load path is most often created by a weak connection, rather than by a completely missing link.

3.4.4 Reduce Demand

The examples of the techniques to reduce demand on the seismic system by modification of dynamic response of a structure are mass reduction, seismic isolation and damping devices. Removal of several floors may be economical and practical method of providing acceptable performance. However the noise and disruption might be an issue. Seismic isolation is relatively expensive compared to alternate techniques and is normally employed in existing buildings for historic preservation or for occupancies that cannot be disturbed. The addition of damping devices in a structure is often economically competitive. The added damping may reduce deformations significantly to prevent unacceptable damage in the existing system.

3.4.5 Remove Selected Components

The deformation capacity can be enhanced locally by uncoupling brittle elements from the deforming structure, or by removing them completely. Examples of this procedure include placement of vertical sawcuts in unreinforced masonry walls to change their behavior from shear failure to a more acceptable rocking mode and to create slots between spandrel beams and columns to prevent the column from being a “short column” prone to shear failure.

3.5. Performance Based Seismic Analysis Methods

3.5.1 Introduction

The initial work on Performance-based design (PBD) was conducted by the Applied Technology Council (ATC), the Structural Engineers Association of California (SEAOC), and the Federal Emergency Management Agency (FEMA). The basis of performance-based design can be found in these separate documents, “Vision 2000: Performance-Based Seismic Engineering of Buildings,” published by SEAOC in 1995; “Seismic Evaluation and Retrofit of Concrete Buildings” published by ATC in 1996; and “FEMA 273/274 NEHRP Guidelines and Commentary for the Seismic Rehabilitation of Buildings” published by FEMA in 1997. Later, “FEMA 356, Prestandard and Commentary for the Seismic Rehabilitation of Buildings (2000)”, which brought FEMA 273/274 to the prestandard level. The FEMA 356 document was replaced by the ASCE 41 Standard for Seismic Rehabilitation of Existing Buildings (ASCE, 2006).

ASCE 41-06 provides four analytical procedures for analyzing the performance of a structure:

- Linear Static Procedure (LSP)
- Linear Dynamic Procedure (LDP)
- Nonlinear Static Procedure (NSP)
- Nonlinear Dynamic Procedure (NDP)

The complexity in the procedure and the required computational cost are generally increasing respectively in the list above. The linear static procedure is the most basic and the nonlinear dynamic procedure is the most detailed and requires the most computational effort. The ASCE 7-05 seismic provisions provides a linear static or linear dynamic analysis to obtain the design forces for a structure, however it is not always accurate when compared to actual earthquake loads. It is a common belief among engineers that the limits on building response from the linear analyses will provide conservative results in the nonlinear models. The need for the accuracy in predicting the building's internal forces and deformations, directs the structural engineer into using non-linear methods.

ASCE 7-05 aims in Section 1.4 that, structures should be designed allowing to sustain local damage while the structural system as a whole remains stable. Although there is not a specific limit state defined for the intent of ASCE 7-05 guidelines, the objective described in Section 1.4 correlates well with what ASCE 41-06 defines as a Life Safety Performance Level, in which a structure may have “damaged components but retains a margin against partial or total collapse”. The seismic design methodologies used at ASCE 7's are force-based. Design accelerations are taken from an elastic design spectrum and then multiplied by the structural weight for an elastic design force. Due to damage is expected in the structure in the event of the design ground shaking, significant inelasticity is anticipated in the structure. Expected inelasticity is obtained by dividing the elastic design force by an R-factor, which accounts for the ductility and overstrength for the chosen lateral force resisting system. ASCE 41-06 provides analysis procedures that have more accurate inelastic demands than the ones computed using ASCE 7 design methodologies.

Performance-based design has gained popularity by utilizing performance objectives to determine acceptable levels of damage for a given earthquake hazard. It allows design of new buildings or upgrade of existing buildings with a realistic understanding of the risk of

casualties, occupancy interruption, and economic loss that may occur as a result of future earthquakes. The objectives provided by ASCE 41-06 evaluate individual structural component damage, which then can be related to a performance level for the building as a whole. These performance objectives can vary from limiting story drift to minimizing component damage. It can be used for assessing the performance capability of a building, system or component and to verify the equivalent performance of alternatives, deliver standard performance at a reduced cost, or confirm higher performance needed for critical facilities. Performance-based design begins with the selection of design criteria stated. Each performance objective is a statement of the acceptable risk of damage, and the consequential losses that occur as a result of this damage, at a specified level of seismic hazard. Losses can be associated with structural damage, nonstructural damage, or both. Once the performance objectives are defined, analyses of building response to loading are performed to estimate the probable performance of the building under various design scenario events. In the case of extreme loading, modeling simulations may be performed using nonlinear analysis techniques (FEMA-445 / 2006).

3.5.2 Analysis Methods

- **Linear Static Procedure**

In the linear static procedure (LSP), buildings are modeled with linearly elastic stiffness and equivalent viscous damping values consistent with components responding at near yield level. A lateral force is calculated per seismic properties and weight, then distributed over the height of the building to calculate internal forces and system displacements resulting from the design earthquake. The procedure is keyed to the displacement response of the building because displacements are a better indicator of damage in the nonlinear range of building response than are forces. If the building responds essentially elastically to the design earthquake, the calculated internal forces will be reasonable approximations of those expected during the design earthquake (FEMA 356, 2000). However, at these displacements, the resulting internal forces in the lateral resisting system will typically be larger than the actual ones that would occur if the building responds inelastically to the design earthquake. The internal forces in the lateral resisting system are evaluated by comparing them to an acceptable force level which includes modification factors to account for the anticipated inelastic response of the actual structure.

- **Linear Dynamic Procedure**

In the Linear Dynamic Procedure (LDP), buildings shall be modeled with linearly elastic stiffness and equivalent viscous damping values consistent with components responding at or near yield level. The design seismic forces, their distribution over the height of the building, and the corresponding internal forces and system displacements shall be determined using a linearly elastic, dynamic analysis. As expected with the LSP, the LDP will produce displacements that approximate maximum displacements expected during the design earthquake. Calculated internal forces typically will exceed those that the building can sustain because of anticipated inelastic response of components and elements (FEMA 356, 2000). ASCE 41-06 prohibits neither of the linear procedures from being used if the building has a structural irregularity. The LDP includes two analysis methods, the Response Spectrum Analysis and the Response History Analysis.

Response Spectrum Analysis

The Response Spectrum Analysis is carried out using linearly elastic response spectra and it uses peak modal responses calculated from dynamic analysis of a mathematical model. Only those modes contributing significantly to the response need to be considered. Peak modal responses for sufficient modes shall capture at least 90% of the participating mass of the building in each of two orthogonal principal horizontal directions of the building. Peak member forces, displacements, story forces, story shears, and base reactions for each mode of response shall be combined by either the SRSS (square root sum of squares) rule or the CQC (complete quadratic combination) rule.

Response-History Analysis

The Response-History Method (also called Time History Analysis) involves time-step evaluation of building response, using recorded earthquake records as base motion input. Response parameters shall be calculated for each time history analysis. If three or more time history analyses are performed, the maximum response of the parameter of interest shall be used for design. If seven or more consistent pairs of horizontal ground motion records are used for time history analysis, use of the average of all responses of the parameter of interest shall be permitted for design.

3.5.3 Nonlinear Static Procedure

The nonlinear static procedure (NSP), also known as pushover analysis, was introduced back in 1970's and is becoming a popular tool for seismic for seismic performance evaluation for existing and new structures. Pushover analysis is much more realistic and more

comprehensive than linear methods explained above. On the other hand, compared to nonlinear dynamic analysis, it is relatively simple and much less time consuming (Krawinkler, 1996).

The purpose of the pushover analysis is to evaluate the strength and deformation capacities and compare these capacities with the demands at the performance level of interest, by using a static nonlinear analysis algorithm. It is carried out under constant gravity loads and monotonically increased lateral forces, applied at the location of the masses in the structural model, to simulate the inertia forces until a target displacement is exceeded or a failure mechanism develops. The target displacement is intended to be the maximum displacement likely to be experienced by the building during the design earthquake. If an appropriate lateral load pattern is used, the structural member forces predicted by the model should be a reasonable approximation of the actual earthquake forces (FEMA 356, 2000). The method is able to describe the evaluation of plastic mechanism and structural damage as a function of the lateral forces since they are increased monotonically. The pushover analysis can provide information on many response characteristics that can't be obtained from linear methods such as (Krawinkler, Seneviratna, 1997);

- Evaluate force demands on potentially brittle elements, such as axial force demands on columns, force demands on braced connections, moment demands on beam-to-column connections, etc.
- Estimates the deformation demands for elements that deform inelastically in order to dissipate energy.
- Identify the critical regions in which the deformation demands are expected to be high and requires thorough detailing.
- Estimates of the interstory drifts that account for strength or stiffness discontinuities
- Verify the completeness and adequacy of load path.

The advantage of the outputs given above comes at the cost of additional analysis effort, associated with modeling inelastic properties of all important elements. Three dimensional analytical model of a structure would be the most preferable one, but as expectedly requires more computational effort.

The method is based on the assumption that the response of the structure can be related to the response of an equivalent single degree-of-freedom (SDOF) system. This implies that the

response is controlled by a single mode and that the shape of this mode remains constant throughout the time history response. Clearly, these assumptions are incorrect, but studies have indicated that these assumptions have good predictions of the maximum seismic response of multi degree-of-freedom (MDOF) structures, provided that their response is dominated by a single mode (Krawinkler, Seneviratna, 1997).

The NSP is only permitted to be used on structures with certain characteristics. The strength ratio, R , must be less than the maximum allowable ratio, as defined by ASCE 41-06 Chapter 3. If the strength ratio exceeds the maximum, the structure experiences significant nonlinear degradation, and a nonlinear dynamic analysis is required. Also, the higher mode effects must not be significant as the nonlinear static procedure is typically only valid for first mode-dominated structures, because the procedure fails to account accurately for higher mode dynamic effects (FEMA 356, 2000).

3.5.4 Nonlinear Dynamic Procedure

The nonlinear dynamic procedure (NDP), like the NSP, requires a mathematical model that incorporates the nonlinear load-deformation characteristics of the individual components. However instead of using target displacement as in NSP, the design displacements are determined directly through dynamic analysis using ground motion time histories. The ground motion time-histories should be specific to the building site. The resulting internal forces do not need to be modified since the nonlinear response is explicitly modeled, and the displacements can be directly compared to the acceptance criteria (FEMA 356, 2000).

A minimum of seven ground motion analyses are required by ASCE 41-06. The basis and the modeling approaches of the NDP are similar to those for the NSP. The nonlinear dynamic procedure is capable of providing the best estimates under seismic loading. However, time and engineering cost discourage designers from performing a response history analysis on all buildings.

4. COMPUTER MODELING OF BUILDING DESIGN

The proposed 7-story steel moment frame building, shown in Figure 4.1, is located in Tacoma, WA. The code standards used in the design were ASCE 7-05 and the AISC Steel Construction Manual 13th Edition. The building design resulting from these standards was first evaluated using Linear Static and Linear Dynamic (Response Spectrum) methods in a three-dimensional model. Later, the ground motion was changed to actual Northridge Earthquake (1994) and the model was analyzed with Linear Dynamic (Time History) in a three-dimensional model, as well. Finally a seismic strengthening was proposed and the new model was analyzed with Nonlinear Static method in two-dimensional model. The results from each analysis were compared.

Figure 4.1 - Proposed actual building to be analyzed

4.1. Building Modeling

4.1.1 Description of Structure

The structure is a 7-story special moment frame of structural steel. The building is laid out on a rectangular grid with a maximum of four 32-ft-wide bays in the X and the Y direction. All stories have a height of 12.5 ft except for the first, second and last stories which are 18.75, 15.5 and 14 ft high. The total height of the building above grade is 98.25 ft. Gravity loads are resisted by steel beams and girders that support a normal weight concrete slab on metal deck.

Figure 4.2 below shows the moments frame elevation from actual structural drawings.

SAP2000-V15 finite element structural analysis software was used to create a three-dimensional model of the structure. All beams and columns in the structure were modeled using beam elements (6 DOF per node). Diaphragms were modeled using a combination of rigid and flexible behavior assumptions. The member sizes for the analysis are in Table 4.1;

	Perimeter Beams	Perimeter Columns	Gravity Beams	Gravity Columns	Gravity Joists
Level 1	W36x170	W24x250	W24x76	W14x109/159	W16x57
Level 2	W33x141	W24x250	W24x76	W14x109/159	W16x57
Level 3	W30x132	W24x176	W24x76	W14x90/120	W16x57
Level 4	W30x116	W24x176	W24x76	W14x90/120	W16x57
Level 5	W30x108	W24x176	W24x76	W14x90/120	W16x57
Level 6	W30x108	W24x131	W24x76	W14x61/90	W16x57
Roof	W24x76	W24x131	W27x102	W14x61/90	W16x31

Figure 4.3 - Model layout in SAP2000

4.1.2 Lateral Force-Resisting System

Gravity and lateral frame members were dimensioned in accordance with the actual design of building. As in the case of actual design, a special moment resisting frame was modeled around the perimeter of the frame as lateral force-resisting system. The frame members are as shown in

Figure 4.4 - Frame plan view and member sizes for the first floor

Figure 4.5 - Frame member sizes for outer frame on X-Z plane

In the model to define the moment frame, the gravity frame were separated from the outer moment frame by assigning “releases” and the joints in the moment frame were assigned “panel zone action” by allowing “elastic properties to be transferred from columns” as shown in Figure 4.6 below.

	Release		Frame Partial Fixity Springs	
	Start	End	Start	End
Axial Load	☐	☐		
Shear Force 2 (Major)	☐	☐		
Shear Force 3 (Minor)	☐	☐		
Torsion	☒	☐	0	
Moment 22 (Minor)	☒	☒	0	0
Moment 33 (Major)	☒	☒	0	0

Figure 4.6 - Assigning frame releases and panel zone in SAP 2000

From the structural note page of design drawings following coefficients were obtained for this special moment frame building regarding the lateral force-resisting system:

Occupancy Category:	IV
Importance Factor:	I=1.5
Response Modification Coefficient:	R = 8
System Overstrength Factor:	$\Omega = 3$
Deflection Amplification Factor:	Cd = 5.5

4.1.3 Diaphragm Modeling

Diaphragm of 5.25" thick of 4,000 psi metal deck concrete slab was used in the real design. For modeling in SAP200, a "rigid diaphragm" was assumed for the levels 1 to 6 and the roof level was assumed to be "semi-rigid" due to penthouse and the geometry of the floor system. Thin-shell elements were used to model diaphragm behavior. The rigid diaphragm elements were given the stiffness properties of a 12" thick concrete slab with uncracked section and the flexible diaphragm was given 5.25" thick concrete slab with cracked section. Then diaphragms were meshed according to the geometric layout of the gravity systems as seen Figure 4.4 above.

Earthquake loads which are applied to these stories are transferred to the lateral force-resisting system through the concrete shell elements. It should be noted that using shell elements to model diaphragm behavior results in the addition of many vertical modes of vibration. To cut down on analysis time, these modes were removed from the model.

4.1.4 Foundation

The foundation design was not considered in this report; however the analysis performed considered a pinned support as shown in Figure 4.3 above. Due to this assumption the foundation design would need to accommodate a flexible base connection and does not require special detailing as in fixed base.

4.1.5 Gravity Loads

The gravity loads are taken from the structural note pages of the design drawings and the summed values are as shown below;

- Floor Dead Load: 110psf
- Floor Live Load: 100psf
- Roof Dead Load: 150psf
- Roof Live Load: 40psf

All applied dead and live loads were defined as area loads on the mesh that was created on floor diaphragms. This accurately transferred loads to beams and columns.

4.1.6 Response Spectra per Site Location

The maximum considered earthquake spectral response accelerations for short periods (S_{MS}) and at 1.0 second period (S_{M1}), adjusted for site class effects, are determined by the following equations for "Site Class C" per ASCE7-05:

$$S_{MS} = F_a * S_s = 1.00 * 1.22 = 1.22g$$

$$S_{M1} = F_v * S_1 = 1.38 * 0.42 = 0.58g$$

The 5 percent damped design spectral response accelerations at short periods (S_{DS}) and at 1.0 second period (S_{D1}), adjusted for site class effects are determined by the following equations:

$$S_{DS} = 2/3 * 1.22 = 0.813g$$

$$S_{D1} = 2/3 * 0.58 = 0.386g$$

The design response spectrum and maximum considered earthquake response spectrum (MCE) was plotted according to section 11.4.5 of the ASCE 7-05 and are shown in Figure 4.7 below.

Figure 4.7 - Design and MCE response spectrum

4.1.7 Dynamic Properties

- **Mass**

For three-dimensional modeling, mass was taken from applied loads and elements. The mass moment of inertia about vertical axis and translational masses were computed automatically in SAP2000 by modeling floor diaphragms as shell elements and entering proper mass density of the elements and dead loads. The mass such as windows, walls and cladding on the exterior moment frame was assigned as line mass of 500 plf.

Two-dimensional analysis was modeled without explicit diaphragm elements. Displacement and plane constraints were used to represent the rigid diaphragm behavior and to limit the analysis into analyzed plane. In this case, floor masses, as shown in Table 4.3, were computed by hand and entered at the joints of each level.

- **Period of Vibration and Base Shear**

The approximate fundamental building period can be computed from the building height and the structural system according to equation 12.8-7 of the ASCE 7-05 where T_a is the approximate fundamental period, C_T and x are coefficients defined by Table 12.8-2 of the ASCE 7-05 and h_n is the height in feet above the base to the highest level of the structure.

$$T_a = C_T h_n^x = 0.028 * (98.25 \text{ ft})^{0.8} = 1.10 \text{ seconds}$$

ASCE 7-05 also requires that the calculated period shall not exceed,

$$C_u * T_a = 1.4 * 1.10 = 1.54 \text{ sec.}$$

When a modal-response spectrum is used, it is required that the displacements and member design forces be scaled to a value of 85 percent of the equivalent lateral force base shear computed using the period $C_u * T_a = 1.54 \text{ sec.}$

The calculation of seismic response coefficient (C_s) is performed using equations 12.8-2 of the ASCE 7-05.

$$C_s = \frac{S_{DS}}{\frac{R}{I}} = \frac{0.813g}{\frac{8}{1.5}} = 0.152g$$

The value of C_s need not exceed equation 12.8-3 of the ASCE 7-05 for $T \leq T_L = 16 \text{ seconds}$;

$$C_s = \frac{S_{D1}}{T \left(\frac{R}{I} \right)} = \frac{0.386g}{1.10 * \left(\frac{8}{1.5} \right)} = 0.066g$$

The value of C_s shall not be less than the equation 12.8-5 of the ASCE 7-05;

$$C_s = 0.044 S_{DS} I = 0.044 * 0.813 * 1.5 = 0.054g$$

Therefore, the seismic response coefficient to be considered is 0.066g. The Seismic Base Shear (V) in a given direction is determined in accordance with equation 12.8-1 of the ASCE 7-05;

$$V = C_s W = 0.066 * 13,301 = 875.87 \text{ kips}$$

• **Damping**

When a modal-response-spectrum analysis is performed, the structure's damping is included in the response spectrum. For time-history analysis, SAP2000 allows an explicit damping ratio to be used in each mode. A damping of 5 percent was specified in each mode. This is consistent with the level of damping assumed in the development of the mapped spectral acceleration values.

4.1.8 Vertical Distribution of Seismic Loads per Equivalent Static Method

A node was placed at the center-of-mass of each level and tied to the diaphragm in the corresponding story height. Center of mass is assumed to be center of diaphragm due to relatively regular shape of the building. This ensured that each of these nodes acted as a whole with the diaphragm as lateral loading was applied there. The seismic loads were determined in accordance with ASCE 7-05 Chapters 11 & 12. The following parameters shown in Table 4.2 were used in seismic load calculations.

Table 4.2 - Seismic parameters

S_{DS}	0.813	C_s	0.152438
S_{D1}	0.386	C_{s,max}	0.065851
I	1.5	C_{s,min}	0.053658
R	8	C_{s,used}	0.065851
T	1.10	V_{x-base} shear (k)	875.87
T_L	16.00		

Seismic loads were distributed to each story of the building by exerting 100% of the earthquake forces exerted in the longitudinal direction and 30% of the earthquake forces exerted in the transverse direction in accordance with equations 12.8-11 and 12.8-12 of the ASCE 7-05:

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

$$F_x = C_{ux} V$$

Also, “accidental torsional moment” caused by assumed displacement of the center of mass which is equal to 5% displacement of the dimension in the transverse direction and applied load in longitudinal direction. When using the equivalent lateral force method as the basis for structural design, these two effects must be added to the direct lateral forces. The “orthogonal loading” must be added for the fact that the earthquake can produce inertial forces that act in any direction. Accidental torsion must be added because the structure shall be modeled with additional forces to account for uncertainties in the location of center of mass and center of rigidity and uneven yielding of vertical systems. The seismic forces in kips are shown in Table 4.3 below:

Table 4.3- Seismic forces according to equivalent lateral load procedure

Floor	W _x (k)	Elevation (ft)	W _x * H ^K	C _{vx}	F _x (k)	F _y (k)	M _x (k-ft)
G	0	0	0	0	0	0.00	0.00
2	2,070	18.75	93,519.55	0.0335	29.32	8.80	183.26
3	2,070	34.25	204,672.51	0.0733	64.17	19.25	401.08
4	1,698	46.75	251,585.59	0.0901	78.88	23.66	493.01
5	1,698	59.25	342,345.42	0.1225	107.34	32.20	670.86
6	1,698	71.75	439,074.42	0.1572	137.67	41.30	860.41
7	1,698	84.25	541,016.53	0.1937	169.63	50.89	1060.18
R	2,368	98.25	921,317.75	0.3298	288.87	86.66	1805.41
SUMS	13,301		2,793,531.77	1.00	875.87	262.76	

4.1.9 Load Combinations

The following load combinations were considered to obtain the axial, shear and flexural demands:

Gravity loads,

- $1.2 D + 1.6 L$

Redundancy Factor, $\phi = 1.0$ [ASCE 12.3.4.2]

- $(1.2 + 0.2S_{DS})D + Q_E + L$
- $(0.9 - 0.2S_{DS})D + Q_E$

Over Strength Factor, $\phi = 3$ [ASCE 12.2.1]

- $(1.2 + 0.2S_{DS})D + \phi Q_E + L$
- $(0.9 - 0.2S_{DS})D + \phi Q_E$

4.2. Objective

The main aim of this exercise is; to increase the general knowledge and proficiency in seismic analysis of existing structures in described methods of analyses in Chapter 3.5 by using structural analysis software. The expectation is that the results will provide enough information about structural vulnerabilities under earthquake actions and necessary retrofitting measures can be taken to increase seismic resistance.

4.2.1 Evaluation Of Existing Structure Against Seismic Loads

This example presents the analysis of 7-story steel moment frame building under seismic effects alone. For this reason member stress checks, member design and detailing against gravity loads are not discussed. The analysis of the structure is performed using the methods described above;

- Three-dimensional model was subjected to *Equivalent Lateral Load Procedure* in SAP 2000 based on described procedure on ASCE7-05. Although this method gives more conservative results than the other methods, it provided a practical comparison level for the rest of the analysis.
- Three-dimensional model was analyzed with *Response Spectrum Analysis* in SAP 2000 using mapped linear elastic response values obtained in Chapter 4.1.7 above in accordance with the existing site location of the building. Due to the building has quite a regular shape, this model of the structure gives accurate results that are close to actual response.
- Three-dimensional modal was tested with *Time History Analysis* using recorded ground motion parameters of Northridge, CA Earthquake (1994) in SAP 2000 to observe the building response against a seismic load which is higher than it was constructed in WA. The recorded earthquake ground motions were used to evaluate building response at discrete time-steps. This method approach provided a valuable experience about how to use various recorded motions on a model.

4.2.2 The Need For Seismic Retrofit

Various different types of retrofits can be implemented on structures with regards to particular problems of existing building's. The aim of this study is to analyze the implementation of a retrofit solution in computer software and examine the differences in strength capacity. The analysis of the retrofitted structure is performed as described below;

- Two-dimensional *Nonlinear Static Pushover Analysis* was performed in SAP 2000 to evaluate load deformation characteristics of a single moment frame of the the retrofitted building and examine the differences in strength capacity. The frame was subjected to monotonically increasing lateral load until target displacement described in ASCE 41-06 Table C1-3. This method provided useful information about structural performance by estimating the strength and deformation capacity that can't be obtained through linear methods such as generation of plastic hinge mechanism as a function of lateral displacement.

4.3. Analysis of Existing Structure

4.3.1 Equivalent Static Method

An equivalent static force analysis was used to determine the drift demands placed on the structure. It is typically used for preliminary design and for assessing the three-dimensional response characteristics of the structure and investigating the behavior of drift-controlled structures. The static forces and accidental torsional moments shown in Table 4.3 above were applied at the defined center of mass of each story in the longitudinal, transverse and longitudinal directions respectively and the drifts were determined.

Table 4.4 below shows the drifts and allowable drifts of each story for the structure in the critical direction while Figure 4.8 shows the deformed shape under applied gravity and seismic loads. The drift output from SAP2000 was multiplied by the deflection amplification factor, C_d and divided by importance factor, I . Allowable drift was calculated in accordance with Table 12.12-1 of ASCE 7-05. As shown in

Table 4.4, it can be seen that the story drift requirements weren't met for the structure for the proposed layout of the lateral system.

Figure 4.8 - Deformed shape of structure with vertical and lateral loads

Table 4.4 - The story drifts from equivalent lateral force analysis

Story	Height (ft)	Total Drift-X (in) from SAP2000	Total Drift-Y (in) from SAP2000	Story Drift (in) from SAP2000	Inelastic Story Drift (in)	Allowable Drift (in)
R	98.25	4.89	1.31	0.54	1.98	1.68
7	84.25	4.35	1.26	0.39	1.43	1.5
6	71.75	3.96	1.16	0.43	1.58	1.5
5	59.25	3.53	1.06	0.48	1.76	1.5
4	46.75	3.05	0.94	0.53	1.94	1.5
3	34.25	2.52	0.79	0.84	3.08	1.86
2	18.75	1.68	0.53	1.68	6.16	2.25
1	0	0	0	0	0.00	0

A modal analysis was run to in the generated model and the mode shapes and frequencies are automatically computed by SAP2000. In the model, the mass is taken from applied dead loads and elements and Ritz Vectors were used with 99% target participation ratios. At least 97% mass participation was recorded in each direction after 5 modes, and the periods of the first, second, and third modes of the structure were recorded to be 2.42s, 2.38s, and 2.06s respectively. Table 4.5

below shows the modal analysis results and the mass participation of the structure in each direction for the first 10 modes of vibration for the equivalent static method.

Table 4.5 - Modal analysis results

Mode	Period (Sec)	SumUX	SumUY	SumRZ
1	2.42	0	0.9344	0.3513
2	2.38	0.9281	0.9344	0.6721
3	2.06	0.9292	0.9344	0.9352
4	0.78	0.9792	0.9344	0.9534
5	0.65	0.9792	0.9807	0.9812
6	0.49	0.9794	0.9807	0.9813
7	0.46	0.9799	0.9807	0.9815
8	0.43	0.9923	0.9808	0.9861
9	0.39	0.9927	0.9932	0.9909
10	0.27	0.9927	0.9977	0.9951

Table 4.6 below shows the maximum member forces for an envelope analysis for defined load combinations.

Table 4.6 - Maximum member forces from SAP 2000

	Pu (k)	Vu (k)	Mu(k-ft)
Column	2,191.78	340.98	2,803.43
Beam	296.69	280.21	4,486.27

4.3.2 Modal Response Spectrum Method

A response spectrum analysis was run on the structure to determine the drift demands and member forces placed on the structure. The first step in the modal-response-spectrum analysis is the computation of the structural mode shapes and associated periods of vibration using the same mathematical model generated for equivalent lateral load method. To run this analysis, the site specific ground motion parameters were put into SAP and a response spectrum curve was developed. In the SAP2000 model, the response spectrum load cases for both X and Y directions were defined with the design spectrum curve developed and with a scale of;

$$32.2 * I / R = 32.2 * 1.5 / 8 = 6.038$$

The demands placed on the structure by this curve were applied at 100% in the primary axis and 30% in transverse axis of the buildings. The response parameters were computed and superimposed using complete quadratic combination (CQC). A modal damping ratio of 5 percent of critical was used in the CQC calculations. The computed periods of vibration for the first 10 modes are summarized in

Table 4.7. As it is seen, the requirement in ASCE 7 for minimum 90% mass participation is achieved. The first mode is predominantly X translation, the second mode is primarily Y translation, and the third mode is largely torsion. Note that the longest period, 2.472 seconds, is significantly greater than the calculated upper limit in $C_u * T_a = 1.54$ seconds. Therefore; displacements, drifts, and member forces as computed from the true modal properties may have to be scaled up to a value consistent with 85 percent of the equivalent lateral force method base shear.

Table 4.7- Modal analysis results

Mode	Period (Sec)	SumUX	SumUY	SumFZ
1	2.47	0.9441	0	0.3309
2	2.46	0.9441	0.9435	0.6855
3	2.04	0.9448	0.9435	0.9452
4	0.78	0.9842	0.9435	0.9597
5	0.63	0.9842	0.9834	0.9843
6	0.44	0.9855	0.9834	0.9847
7	0.43	0.9931	0.9834	0.9878
8	0.41	0.9943	0.9939	0.992
9	0.40	0.9943	0.994	0.992
10	0.34	0.9943	0.994	0.9943

The base shears obtained from SAP2000 in X and Y directions are as below;

X-direction base shear = 490.62 kips

Y-direction base shear = 490.55 kips

These values are much lower than the equivalent lateral load base shear of 875.87 kips. The modal-response-spectrum shears are less than the equivalent lateral load base shear because the fundamental period of the structure used in the response-spectrum analysis is 2.47 seconds (vs 1.10). Due to X and Y direction base shears are quite similar, only one scale factor will be used. However, member design forces be scaled to a value of 85 percent of the equivalent lateral force base shear which is 713.69 k computed using the upper limit period $C_u \cdot T_a = 1.54$ seconds. Hence, the required scale factors are:

Scale factor for drifts = $0.85(875.87)/490.62 = 1.517$

Scale factor for member forces = $0.85(713.69)/490.62 = 1.236$

Table **4.8** and Table 4.9 below show the displacements in X and Y direction;

Table 4.8- The story drifts in X-direction

Height (ft)	Total Drift-X (in) from SAP2000	Total Drift-X (in) Scaled by 1.517	Scaled Story Drift (in)	Inelastic Story Drift (in)	Allowable Drift (in)
98.25	2.67	4.05	0.30	1.11	1.68
84.25	2.47	3.75	0.26	0.95	1.5
71.75	2.30	3.49	0.29	1.06	1.5
59.25	2.11	3.20	0.33	1.22	1.5
46.75	1.89	2.87	0.56	2.06	1.5
34.25	1.52	2.31	0.71	2.61	1.86
18.75	1.05	1.59	1.59	5.84	2.25
0	0	0.00	0.00	0.00	0

Table 4.9- The story drifts in Y-direction

Height (ft)	Total Drift-Y (in) from SAP2000	Total Drift-Y (in) Scaled by 1.517	Scaled Story Drift (in)	Inelastic Story Drift (in)	Allowable Drift (in)
98.25	2.62	3.97	0.26	0.95	1.68
84.25	2.45	3.72	0.26	0.95	1.5
71.75	2.28	3.46	0.27	1.00	1.5
59.25	2.10	3.19	0.35	1.28	1.5
46.75	1.87	2.84	0.56	2.06	1.5
34.25	1.50	2.28	0.74	2.73	1.86
18.75	1.01	1.53	1.53	5.62	2.25
0	0	0.00	0.00	0.00	0

Table 4.10 below shows the maximum member forces for an envelope analysis for defined load combinations.

Table 4.10 - Maximum member forces from SAP 2000

	Pu (k)	Vu (k)	Mu(k-ft)
Column	2,682.60	199.21	2,502.79
Beam	54.09	210.42	3,019.23

4.3.3 Modal Time History Analysis

A time-history analysis was run on the structure to determine the drift demands and member forces placed on the structure. In modal-time-history analysis, the response in each mode is computed using step-by-step integration of the equations of motion, the modal responses are transformed to the structural coordinate system, linearly superimposed, and then used to compute structural displacements and member forces. The same mathematical model of the structure used for the equivalent lateral load and response-spectrum analysis is used for the time-history analysis.

Modeling Assumptions:

Per ASCE 7-05 Chapter-16, for linear modal time history analysis, the methodology explained below is executed.

- Three ground motion recordings consisting of horizontal pairs of acceleration were selected for Northridge Earthquake-1994 with 6.69 magnitude and downloaded from PEER (Pacific Earthquake Engineering Research Center) database as shown Table 4.11 in below.

Table 4.11- Ground Motion Parameters (Unscaled)

Record Name (PEER)	Orientation	Number of Points and Time Increment	Peak Ground Acceleration (g)	Source Motion
NGA0952	N-S	2398 @ 0.01 sec.	0.594	Northridge
NGA0952	E-W	2398 @ 0.01 sec.	0.467	Northridge
NGA0960	N-S	1999 @ 0.01 sec.	0.466	Northridge
NGA0960	E-W	1999 @ 0.01 sec.	0.337	Northridge
NGA0963	N-S	2000 @ 0.02 sec.	0.488	Northridge
NGA0963	E-W	2000 @ 0.02 sec.	0.534	Northridge

- Time histories and 5-percent-damped response spectra for each of the motions are shown in Figure 4.9 - **Time histories and response spectra for record NGA0952** Figure 4.9 through Figure 4.11.

N-S Direction

E-W Direction

Response Spectra

Figure 4.9 - Time histories and response spectra for record NGA0952

N-S Direction

E-W Direction

Figure 4.10 - Time histories and response spectra for record NGA0960

E-W Direction

Figure 4.11 - Time histories and response spectra for record NGA0963

- For each pair of the horizontal ground motion components, a square root of the sum of the squares (SRSS) spectrum was constructed with 5% percent spectra for the scaled components over the period range $0.2T$ to $1.5T$ per ASCE 7-05 Chapter 16. Target spectra is taken as design spectrum and period is taken as the $T = 2.47$ sec. from the modal response spectrum analysis. Figure 4.12 shows the unscaled response spectra of ground motion and design spectrum computer using PEER online software.

0.2

1.5

Figure 4.12 - Unscaled SRSS spectra of the ground motion pairs with design spectrum

- Figure 4.13 below shows the ratio of average scaled SRSS spectrum to design spectrum over periods of $0.2T = 0.494$ sec. and $1.5T = 3.705$ sec. The scale factor obtained through PEER online software is 1.716.

Figure 4.13 - Ratio of average scaled SRSS spectrum to design spectrum

- The ground acceleration scale factor in SAP 2000 is;

$$1.716 * 32.2 \text{ ft/sec}^2 * (I=1.5) / (R=8) = 10.360$$

When performing linear time history analysis, each of the recorded ground motions is multiplied by this amount above.

- Twelve individual time history analysis were carried out in SAP2000. One for each N-S ground motion in the X direction, one for each N-S ground motion in the Y direction, one for each E-W ground motion in the X direction, one for each E-W ground motion in the Y direction. 5% of critical damping was used in each mode.
- The base shear obtained through the minimum requirement from ASCE 7-05,

$$V = 0.044 * S_{DS} * I_e * W = 0.044 * 0.813 * 1.5 = 0.0537 * 13,301 = 713.7$$

The base shears obtained from SAP2000 in X and Y directions are as below;

$$\text{X-direction base shear} = 907.68 \text{ kips}$$

$$\text{Y-direction base shear} = 872.11 \text{ kips}$$

Due calculated base shears are greater than 85% of the initial shear, there is no need to scale member forces and drifts.

Modal analysis results from the analysis are shown below in Table 4.12;

Table 4.12 - Modal analysis results

Mode	Period (Sec)	SumUX	SumUY	SumRZ
1	2.376	0	0.9359	0.3518
2	2.331	0.9302	0.9359	0.6728
3	2.022	0.9313	0.9359	0.9369
4	0.761	0.9796	0.9359	0.9545
5	0.633	0.9796	0.9809	0.9815
6	0.443	0.9807	0.9809	0.9818
7	0.421	0.9895	0.9809	0.9854
8	0.341	0.9926	0.9931	0.9933
9	0.271	0.9926	0.9979	0.9951
10	0.214	0.9983	0.9979	0.9981

The results from the SAP 2000 modal time history analysis are summarized in Table 4.13 and Table 4.14 below.

Table 4.13 - The story drifts in X-direction

Height (ft)	Total Drift-X(in) from SAP2000	Story Drift (in)	InelasticStory Drift (in)	Allowable Drift (in)
98.25	5.17	0.53	1.94	1.68
84.25	4.64	0.44	1.61	1.5
71.75	4.20	0.45	1.65	1.5
59.25	3.75	0.50	1.83	1.5
46.75	3.25	0.57	2.09	1.5
34.25	2.68	0.87	3.19	1.86
18.75	1.81	1.81	6.64	2.25
0	0	0.00	0.00	0

Table 4.14 - The story drifts in Y-direction

Height (ft)	Total Drift-Y (in) from SAP2000	Story Drift (in)	InelasticStory Drift (in)	Allowable Drift (in)
98.25	4.98	0.45	1.65	1.68
84.25	4.53	0.42	1.54	1.5
71.75	4.11	0.42	1.54	1.5
59.25	3.69	0.54	1.98	1.5
46.75	3.15	0.64	2.35	1.5
34.25	2.51	1.01	3.70	1.86
18.75	1.50	1.50	5.50	2.25
0	0	0.00	0.00	0

4.3.4 Comparison of Results from Various Methods of Analysis

Figure 4.14 below shows the inter-storey drifts of three solution methods explained above. Equivalent static method gives conservative results compared to response spectrum and time history analysis. As explained above in analyses, the expected story drifts are above the allowable limits given in the ASCE-7.

Figure 4.14 - Comparison of inter-storey drifts of analysis methods

The ASCE 41-06 standard describes a performance evaluation process of individual components of the seismic resisting system for the linear procedures by comparing a demand-capacity ratios (DCR) to a unitless m-factor derived from physical testing and the judgment associated with a specific performance level for a member (Adams, 2010). Like the R-factor in ASCE 7-05, ASCE 41-06 employs “mfactors” for deformation-controlled elements and “j-factors” for force-controlled elements to account for material inelasticity anticipated in response to the earthquake. These factors are dependent on the nonlinear deformation capacity of these elements. Primary components of seismic resisting system are considered deformation-controlled if ductile behavior is exhibited. All the elements modeled in this project were considered deformation-controlled elements, since their behavior is

consistent with the criteria discussed in ASCE-46. DCR should be calculated for each controlling action (such as axial force, shear, and moment) of each component.

$$DCR = \frac{Q_{UD}}{Q_{CE}}$$

$$Q_{UD} = Q_G \pm Q_E$$

A performance level is achieved if the DCR for the member is less than the m-factor at that performance level.

$$mKQ_{CE} \geq Q_{UD}$$

Where:

Q_E = Action due to design earthquake loads

Q_G = Action due to design gravity loads

Q_{UD} = Design action due to gravity and earthquake loads

Q_{CE} = Expected strength of the component

K = Knowledge factor (0.75~1.00)

m = Component demand modifier to account for expected ductility at a performance level

The analysis results show the demand-capacity ratios for the columns. The performance levels are based on ASCE 41-06 Table 5.5: Acceptance Criteria for Linear Procedures-Structural Steel Components. The maximum values for the Immediate Occupancy, Life Safety and Collapse Prevention performance levels differ from floor to floor because the acceptance values are based on the slenderness of the columns. The performance objective considered in this study is the BSE-1 Earthquake Hazard Level which is consistent with the design-based earthquake in ASCE 7-05 and is 2/3 of the MCE, or has a 10% probability of exceedance in 50 years. The DCR's in Figure 4.15 below show the column's performance according to different analyses methods in comparison to acceptance criteria of different performance levels.

Figure 4.15 - BSE-1 level earthquake DCR for columns per ASCE 41-06 Acceptance Criteria

The analyses for BSE-1 level hazard showed the building performance for the linear procedures. For an earthquake level such as in Time History Analysis, the building's performance stays in Life Safety region. However, due to the building's seismic importance factor is 1.5; the expected performance for this level of earthquake must be Immediate Occupancy to remain operational after the seismic hazard. Also the expected inter-storey drifts are above the limits ASCE-7 limits as shown in analyses above. Due to all these reasons, a seismic retrofitting measure should be taken to increase the building performance to target levels in terms of global strength and stiffness.

4.4. Seismic Retrofitting

In order to enhance the building's performance to meet the drift demands for a seismic event such as analyzed in Section 4.3.3, the building theoretically retrofitted with concentrically braced frames as shown in Figure 4.16 below.

Figure 4.16 - Retrofitted section

Insertion of braces in moment frame buildings is one of the most common retrofit options. The aim of the strengthening was generally to increase stiffness and reduce the inter-storey drifts. It should be pointed out that the proposed strengthening are usually placed in the outer frame of the structure and continuously run from foundation to roof. Hence, some conditions should be satisfied with the architectural plan of building such as compromising on window view, lesser inner space etc. Also locations of concentrically braced frames need to be determined such that they symmetrical with respect to the both principal axis to minimize the torsional effects.

4.5. Nonlinear Static Method (Pushover Analysis)

Inelastic analytical procedures help to understand the actual behavior of structures by identifying failure modes and the potential for progressive collapse. Inelastic static analysis, or pushover analysis, has been the preferred method for seismic performance evaluation due to its simplicity. It is a static analysis that directly incorporates nonlinear material characteristics. A pushover analysis is performed by applying monotonically increasing lateral loads to a structure which is representing the inertial forces during ground shaking. At each increment, the structure experiences a loss in stiffness and as a result various structural

elements may yield sequentially and the non-linear force displacement relationship, weak links and failure modes of the structure can be determined.

The loading is monotonic with the effects of the cyclic behavior and load reversals being estimated by using a modified monotonic force-deformation criteria and with damping approximations. The ATC-40, FEMA-273 and FEMA-356 documents have developed modeling procedures, force-deformation criteria for hinges, acceptance criteria and analysis procedures for pushover analysis. SAP2000 static pushover analysis allow quick and easy implementation of the pushover procedures prescribed in these documents for both two and three-dimensional buildings.

- **Basis of Procedure**

Analytical models for the pushover analysis of frame structures may be divided into two main types: (1) distributed plasticity (plastic zone) and (2) concentrated plasticity (plastic hinge). The plastic hinge approach is simpler than the plastic zone however it is limited to capture the more complex member behaviors such as combined actions of compression and bending and buckling. Non-linear lateral behavior of two dimensional frames was evaluated by incremental pushover analyses using SAP2000 which utilized lumped plastic hinge formulations based on FEMA 356-2000.

In SAP2000, frame elements were first modeled having linear-elastic properties. Nonlinear force-displacement characteristics of frame elements are modeled as hinges. A generalized force-displacement characteristics of hinge elements in SAP2000 is shown below Figure 4.17.

Figure 4.17 - Force vs. displacement properties of hinges elements

Point A represents the unloaded condition and point B corresponds to yielding of the element. Point C corresponds to nominal strength with the deformation at which significant strength degradation begins and resistance to lateral loads beyond point C is usually unreliable. The residual resistance from D to E allows the frame elements to sustain gravity loads. Point E represents the maximum deformation capacity; beyond this point gravity load resistance can no longer be sustained (FEMA 356/2000). In the model, defined hinge patterns that are shown in Figure 4.18 were assigned the frame members as defined below;

Figure 4.18 - Locations of plastic hinges in the frame elements

• Modeling

To control the drift demands of the building, HSS 10x10x1/2 concentrically steel braces were assigned in the outer frame of the structure. These braces are very efficient and commonly used for rehabilitation of steel structures in resisting seismic loads. These frames behave as vertical cantilever trusses, and the columns, beams, and braces are subjected to axial forces. The braces in these frames are designed to buckle and yield under cyclic loading to dissipate seismic energy and limit damage to gravity components. The 2-D model of the retrofitted moment frame is shown in Figure 4.19.

Figure 4.19 - 2D model of the retrofitted frame

• **Pushover Analysis**

After the 2-D model was defined and assigned with all the properties, the displacement-controlled pushover analysis was performed. The model was pushed monotonically until target displacement was reached or structure loses equilibrium. FEMA 356 defines target lateral roof drifts at identified operational, immediate-occupancy, life safety and collapse prevention performance levels as shown in Table 4.15 below. For this purpose, target displacement is taken 24in which is about 2.5% of the building height. The pushover curve, which is a base shear force versus roof displacement curve, was plotted. The peak of this curve represents maximum lateral load carrying capacity of the structure. **Table 4.15**

Table 4.15- Target lateral roof displacement

Performance Level	Operational	Immediate Occupancy	Life Safety	Collapse Prevention
Lateral Roof Drift Ratio (δ/h)%	0.37	0.70	2.50	5.00

Performance point and location of hinges in various stages can also be obtained from pushover curve as shown in Figure 4.20. The curve represents a ductile behavior where there is an elastic range of from A to B. It is followed by a plastic range where there is a strain hardening from point B to C and strength degradation from point C to D. B to IO is the range of immediate occupancy; IO to LS is the range of life safety and LS to CP is the range

of collapse prevention. When a hinge reaches Point C, that hinge starts to drop load. After Point C, the base shear is reduced until the force reached Point D. After this point, force is again increased and the displacement continues to increase again until Point E where the hinge can't carry any load (FEMA 356/2000).

Figure 4.20 - Component or element acceptance criteria from FEMA 356/2000

• Results

The resulting pushover curve of base shear vs. roof displacement from SAP 2000 is shown below in Figure 4.21. This curve shows overall response of the structure against incremental loading. This load is increased monotonically until the failure occurs in the structure. As the loading is increased, a curve between the base shear and roof displacement is plotted. The curve is initially increasing linear but later begins to change from linearity as the components go through inelastic actions. The maximum base shear of 2700k was reached at a displacement of 12.2in. For the immediate occupancy (IO) range, the maximum displacement is around 7.6in.

Figure 4.21 - Pushover curve from SAP 2000 (k-in)

From the Figure 4.22, it can be seen that the demand curve intersects the capacity curve near an elastic response zone and it means that a safety margin is granted. Therefore, it can be concluded that the safety margin against collapse is high and the structural system has sufficient strength and displacement reserves.

Figure 4.22 - Capacity-demand curve for the building (FEMA 440 Equivalent Linearization Method)

Plastic hinges formation for the 2-D frame has been obtained at the target displacement level as shown in Figure 4.23 below. Plastic hinges formation starts with concentric braces as expected and transmits to beams and base columns. But since the yielding in beams and columns occur B, IO levels; the amount of the damage to the building will be limited.

Figure 4.23 - Deformed shape and plastic hinge formation for envelope analysis

The conclusions from the Pushover Analysis about the performance of the retrofitted frame are summarized below,

- The pushover analysis is a relatively simple and direct way to investigate the non-linear behavior of buildings and it can be used after the linear analyses to estimate the seismic resistance. However, the pushover analysis is an approximate method based on static loading; the procedure fails to account accurately for higher mode dynamic effects.
- Most of the hinges occurred in implemented braces; at the final stages of displacement started in the beams and columns but with very limited damage. It shows that proposed retrofit measure works properly to limit drift demands.
- The resulting demand and capacity curves clearly indicate that the building has a sufficient strength and displacement reserve against collapse with the proposed concentrically braced frames.
- The obtained pushover curve (base shear vs. roof displacement) is the load-displacement envelope of the building and represents the global response of the structure. It approximates how the structure behaves after exceeding elastic limits. The maximum value of roof curve represents the lateral load carrying capacity of the structure.

5. CONCLUSIONS

5.1. Final Conclusions

In this study, first the seismic resistance of seven-story steel moment frame building was evaluated. The three-dimensional “Equivalent Static Analysis-ESA”, “Response Spectrum Analysis-RSA” and “Time History Analysis-THA” evaluations were carried out. The 3D ESA and RSA was carried out in order to obtain the fundamental period, dominant mode shapes and the base shear. In ESA, the fundamental period of the structure is 2.42sec. The roof-displacement is 4.89in and the total base shear of the structure is 875.87k which is obtained and distributed as story lateral loads throughout the building as described in ASCE7- 05 Chapter 12. In RSA the fundamental period of the structure is 2.47sec. and the roof-displacement is 4.05in where the first 20 mode shapes are combined with the CQC method. The displacements were scaled by a scale factor of 1.517 due to obtained base shear 490.62k is less than 85% of the equivalent lateral force method base shear. In the 3D THA with recorded ground motion parameters of Northridge, CA Earthquake (1994) at discrete time-steps, the fundamental period of the structure is 2.32sec. and the roof-displacement is 5.19in. As shown in the analysis above, the drift demands haven’t been met with allowable limits in ASCE7.

The steel moment frame structure is retrofitted with concentrically steel braces in the interior spans to decrease the deformations into acceptable limits. The retrofitted section was analyzed by two-dimensional “Pushover Analysis”. The structure was pushed to a target displacement (20in) where serious damage starts to occur. Plastic hinges as expected initially started to occur on diagonal braces of the first three floors as a result of each step of pushing the building. When the target displacement was reached, plastic hinges started to occur in the first floor column which is an indication of significant damage and relatively fewer damages were obtained after the third storey. When evaluating the performance impact of retrofit implementation, it was quite obvious that the addition of braces to the moment frame served to stiffen the building sufficiently and help the structure resist lateral loads.

Generally, the structural retrofit improved the seismic resistance of the building and it can be considered in the retrofit of moment frame structures to prevent the risk of structural collapse under the design load with much more confidence. This study shows how the pushover analysis may be used in order to estimate the seismic resistance of existing or retrofitted

structures as well as how the linear analysis may be followed by a detailed nonlinear analysis of part of the structure. One of the most significant advantages of nonlinear pushover analysis beyond the linear analyses is the opportunity to evaluate damage. The pushover analysis can give valuable information about performance of building in expected future seismic events.

5.2. Recommendations

In this research of seismic retrofits, there were numerous points which were not covered by the scope of this thesis due to limitations in time and resources. However, they might be considered to create a more detailed research for future work. These recommendations for further research work are discussed below.

- For this study, only one computer software was used to run all the analyses. Although, SAP 2000 is largely accepted in academia and engineering offices for structural analysis, it is advisable to use different applications to compare the results rather than to rely on information from one source. In further research, it is highly recommended to use different structural analysis software to validate the results.
- Due to the time constraints in developing this thesis, only the linear analyses were modeled in three-dimensional model. Non-linear pushover analysis was modeled two-dimensional system in retrofitted building. Additionally, the building was simplified in model with basic structural framing which didn't include many elements such as stairs, elevators, mechanical equipment, etc. Furthermore, non-linear time-history analysis was not analyzed which would provide the thesis with more accurate results. In further research, three-dimensional non-linear analyses with both pushover and time-history including the weak zones are highly recommended to capture more realistic behavior of the structure.
- As observed in the Northridge Earthquake (1994), the design of connections and detailing of framing elements are significantly important. However, in the scope of this research many detailing considerations weren't considered. In the scope of future work, it is also recommended to focus on the design of connections in the seismic retrofits.
- Another recommendation to provide better results for a more detailed research is to implement a wider variety of retrofit applications such as seismic isolation and energy dissipation devices or different brace types.
- Finally, the performance analysis of individual components per ASCE-41 may include other components such as beams, slabs or braces in non-linear procedures.

6. REFERENCES

- Adams, Scott Michael (2010); “Performance-Based Analysis of Steel Buildings: Special Concentric Braced Frame”, California Polytechnic State University.
- Agarwal, Pankaj & Shrikhandle, Manish (2006); “Earthquake Resistant Design of Structures”, New Delhi, Prentice Hall.
- AISC 13th Edition (2006); “Steel Construction Manual”, American Institute of Steel Construction
- Ambrose, J. & Vergun, D (1999); “Design for Earthquakes”, John Wiley & Sons, Inc., USA.
- ASCE 31-03; “Seismic Evaluation of Existing Buildings”, American Society of Civil Engineers, VI, USA.
- ASCE 41-06; “Seismic Rehabilitation of Existing Buildings”, American Society of Civil Engineers, VI, USA.
- ASCE 7-10; “Minimum Design Loads for Buildings and Other Structures”, American Society of Civil Engineers, VI, USA.
- Astanek, Abolhassan (2001); “Seismic Behavior and Design of Steel Shear Walls”, Structural Steel Educational Council.
- Barron, Joel M. & Hueste Mary Beth D. (2004); “Diaphragm Effects In Rectangular Reinforced Concrete Buildings”, ACI Structural Journal, Title No:101-S60.
- Benjamin, Jack, & C. Allin Cornell (1970); “Statistics, Probability, and Decision for Engineers” McGraw-Hill, New York.
- Bergman, D. M., and Goel, S.C. (1987). “Evaluation Of Cyclic Testing Of Steel Plate Devices For Added Damping And Stiffness”, Report UMCE 87-10, The University of Michigan, Ann Harbor, MI.
- Blume, John A.; Newmark, N.M. & Corning, L.H. (1961); “Design of Multi-story Reinforced Concrete Buildings for Earthquake Motions”, Portland Cement Association, Skokie, IL.
- Bozorgnia, Yousef & Bertero, Vitelmo V., (2004); “Earthquake Engineering from Engineering Seismology to Performance-Based Engineering”, CRC Press LLC, Florida.
- Breyer, Donald; Fridley, Kenneth & Cobein, Kelly (1999); “Design of Wood Structures: ASD”, McGraw-Hill, New York.
- Bruneau, Michel; Chia-Ming Uang & Whittaker, Andrew (1997); “Ductile Design of Steel Structures”, McGraw-Hill, New York.
- Canadian Wood Council (2001), Engineering Guide for Wood-Frame Construction, Building Performance Series No. 5, Ontario, Canada.
- Chao, S.H. & Goel, S.C. (2005); “Performance-Based Seismic Design of EBF Using Target Drift and Yield Mechanism as Performance Criteria”, Report No. UMCEE 05-05, Department of Civil and Environmental Engineering, University of Michigan, Ann Arbor, MI.
- Chen, W.F. & Lui, E. M. (2006); “Principles of Structural Design”, Taylor & Francis Group, Florida.
- Constantinou, M. C., & Symans, M. D. (1993); “Seismic Response of Structures with Supplemental Damping”, Structural Des. Tall Buildings, Vol. 2, 77 – 92, Buffalo, NY.
- Cowan, Henry (1977); “The Master Builders: A History of Structural and Environmental Design from Ancient Egypt to the Nineteenth Century”, Wiley, New York.

- Darrick B. Hom & Poland Chris D. (2004); “Codes & Standards - Where the Past Meets the Future ASCE 31-03”, Structure Magazine, WI, USA.
- Drysdale, R.G.; Hamid, A.A. & L.R. Baker (1999) “Masonry Structures Behavior and Design”, The Masonry Society, 2nd Edition, Boulder, CO.
- Fardis M. N.; Carvalho E.; Elnashai Amr; Faccioli E.; Pinto P. & Plumier A. “Design of Structures For Earthquake Resistance. General Rules, Seismic Actions, Design Rules for Buildings”, 2005.
- Farzad Naeim & Kelly, James M. (1999); “Design of Seismic Isolated Structures from Theory to Practice”, John Wiley & Sons Inc., USA.
- FEMA 273 (1997); “NEHRP Guidelines for the Seismic Rehabilitation of Buildings”, Federal Emergency Management Agency, Washington, D.C.
- FEMA 350 (2000); “Recommended Seismic Design Criteria for New Steel Moment-Frame Buildings”, Federal Emergency Management Agency, Washington, D.C.
- FEMA 356 (2000); “Prestandard and Commentary for the Seismic Rehabilitation of Buildings”, Federal Emergency Management Agency, Washington, D.C.
- FEMA 445 (2006); “Next-Generation Performance-Based Seismic Design Guidelines”, Federal Emergency Management Agency, Washington, D.C.
- FEMA 547 (2006); “Techniques for the Seismic Rehabilitation of Existing Buildings” , Federal Emergency Management Agency, Washington, D.C.
- FEMA 749 (2010); “Earthquake-Resistant Design Concepts-An Introduction to the NEHRP Recommended Seismic Provisions for New Buildings and Other Structures”, Federal Emergency Management Agency, Washington, D.C.
- Fujitani, H.; Yamanouchi, H.; Okawa, I.; Sawai, N.; Uchida, N. & Matsutani, T. (1996); “Damage and Performance of Tall Buildings in the 1995 Hyogoken Nanbu Earthquake”, Japan.
- Gueraud, R.; N.Leroux, J.P.; Livolant; M. & Michalopoulos, A.P. (1985); “Seismic Isolation Using Sliding-Elastomer Bearing Pads”, Nuclear Engineering and Design, Vol. 84, 363-377.
- Hamburger, Ronald O.; Krawinkler, Helmut; Malley, James O. & Adan Scott M. (2009); “Seismic Design of Steel Special Moment Frames”, “NEHRP Seismic Design Technical Brief No. 2”.
- Hines , E.M. & Fahnestock, L.A. (2010); “Design Philosophy For Steel Structures In Moderate Seismic Regions”.
- <http://peer.berkeley.edu/>; Pacific Earthquake Engineering Research Center
- <http://www.usgs.gov/>; US Geological Survey
- Inoue, K.; Sawaizumi, S., & Higashibata, Y. (2001); "Stiffening Requirements for Unbonded Braces Encased in Concrete Panels." J. Struct. Eng., 127(6), 712-20.
- Kelly, James M. (1997); “Base Isolation: Origins and Development”. National Information Service for Earthquake Engineering.
- Krawinkler, H. (1996); "Pushover analysis: Why, How, When, and When Not to Use It", Convention of the Structural Engineers Association of California, California.
- Krawinkler, H; Seneviratna, G. D. (1997); “Pros and Cons of a Pushover Analysis of Seismic Performance Evaluation”. Engineering Structures, Vol.20.

- Kunde, M.C. & Jangid, R.S. (2003); "Seismic Behavior of Isolated Bridges", Electronic Journal of Structural Engineering Vol:3.
- Lee, D., & Taylor, D. P. (2001); "Viscous Damper Development and Future Trends", Struct. Des. Tall Build., 10-5, 311–320.
- Lopez, W., A. & Sabelli R. (2004); "Seismic Design of Buckling-Restrained Braced Frames". Structural Steel Education Council.
- Mahin, Stephen A./ V. V. Bertero; Chopra, A. K. & Collins, R. G. (1976); "Response of the Olive View Hospital Main Building During the San Fernando Earthquake", Earthquake Engineering Research Center, University of California, UCB/EERC-76/22, Berkeley, CA.
- Mahin, Stephen; Hamburger, Ronald & Malley, James (1998); "National Program to Reduce Earthquake Hazards in Steel Moment-Resisting Frame Structures", Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.
- Malik, A. M. (1995); "Estimating Building Stocks for Earthquake Mitigation and Recovery Planning", Cornell Institute for Social and Economic Research, Ithaca, NY.
- McCormick, Timothy P.; Kiefer, Stephan A., Chylinksi, Richard (2009); "Seismic Retrofit Training For Building Contractors & Inspectors", Participant Handbook.
- Moehle, Jack P.; Ghodsi, Tony; Hooper, John D. & Fields David C. (2012), "Seismic Design of Cast-in-Place Concrete Special Structural Walls and Coupling Beams", NEHRP Seismic Design Technical Brief No. 6, Gaithersburg, MD.
- Moehle, Jack P.; Hooper, John D. & Lubke, Chris D. (2008); "Seismic Design of Reinforced Concrete Special Moment Frames", NEHRP Seismic Design Technical Brief No. 1, Gaithersburg, MD.
- Murty, C.V.R (2002); "Seismic Design Philosophy for Buildings", IITK-BMTPC Earthquake Tips, New Delhi, India.
- Newmark, Nathan & Veletsos, Anestis (1960); "Effect of Inelastic Behavior on the Response of Simple Systems to Earthquake Motions,", Proceedings of the Second World Conference on Earthquake Engineering, Vol. II, Tokyo and Kyoto, Japan.
- Pall, A. S. & Marsh, C., (1982); "Seismic Response of Friction Damped Braced Frames", Journal Struct. Div., 108-6, 1313-1323.
- Paulay, Thomas. & Priestley, M.J.N. (1992); "Seismic Design of Reinforced Concrete and Masonry Buildings," John Wiley & Sons, USA
- Pekelnicky, R. & Poland, C. (2012); "Seismic Evaluation and Retrofit of Existing Buildings". SEAOC 2012 Convention Proceedings, CA, USA.
- Popov, E.P. & Engerlhardt, M. D. (1988); "Seismic Eccentrically Braced Frames", J.Construct. Steel Research.
- Prion, Helmut; Foschi, Ricardo; Lam, Frank & Ventura, Carlos E. (1999); "Proceedings of the Invitational Workshop on Seismic Testing, Analysis and Design of Wood frame Construction", Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.
- Roeder, C.W. & Popov, E.P. (1978); "Eccentrically Braced Steel Frames for Earthquakes." J. Struct. Div. ASCE, 104(ST3), 391-412.
- Sabelli ,Rafael; Sabol Thomas A. & Easterling W. Samuel (2011); "NEHRP Seismic Design of Composite Steel Deck and Concrete-filled Diaphragms".

- Sabelli, R.; Mahin, S. & Chang, C. (2003). "Seismic Demands on Steel Braced Frame Buildings with Buckling-Restrained Braces", Eng. Structures, 25(2003), 655-666.
- Sadek, Fahim, Mohra, Bijan, Taylor, Andrew W. and Chung Riley M., 1996. National Institute of Standards and Technology. "Passive Energy Dissipation Devices for Seismic Application". Maryland.
- Seilie, Ignasius F. & Hooper, John D. (2005); "Steel Plate Shear Walls, Practical Design and Construction", North American Steel Design Conference.
- Soong, T. T. & Dargush, G. F. (1997); "Passive Energy Dissipation Systems in Structural Engineering", Wiley, Chichester, U.K.
- Steinbrugge, Karl V. (1970); "Earthquake Damage and Structural Performance in the United States", Earthquake Engineering, Prentice-Hall, Englewood Cliffs, New Jersey.
- Steinbrugge, Karl V.; Manning, John H. & Degenkolb, Henry J. (1967); "Building Damage in Anchorage Earthquake of 1964 and Aftershocks", Washington, DC.
- Symans, M. D.; Charney F. A.; Whittaker A. S., Constantinou, M. C.; Kircher, C. A.; Johnson, M. W. & McNamara, R. J. (2008); "Energy Dissipation Systems for Seismic Applications: Current Practice and Recent Developments", Journal of Structural Engineering - ASCE / January 2008 / 3
- Symans, Michael, K.; Fridley, W. Cofer, & Y. Du (2002); "Innovative Systems", Wood frame Project Report W-20. Richmond, CA: Consortium of Universities for Research in Earthquake Engineering.
- Thorburn, L. J.; Kulak, G. L. & Montgomery, C. J. (1983); "Analysis of Steel Plate Shear Walls", Structural Engineering Report No. 107. Department of Civil Engineering, University of Alberta. Astaneh.
- Tim Psomas (2002) "Los Angeles City Hall Seismic Rehabilitation Project-Base Isolation Technology" (<http://eng.lacity.org/projects/lacityhall/isohow.html>).
- Tremblay, Robert; (2001); "Seismic Behavior and Design of Concentrically Braced Steel Frames", Engineering Journal, Third Quarter, 1001, pp. 148-166.
- Yashinsky, Mark; (1998); "Performance of Bridge Seismic Retrofits During the Northridge Earthquake," Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.
- Zayas, V.A.; Low, S.S. & Mahin, S.A. (1990); "A Simple Pendulum Technique for Achieving Seismic Isolation", Earthquake Spectra, Vol. 6, 317-334.