DECISION TREE BASED SEISMIC RETROFIT SELECTION FOR NON-CODE CONFORMING REINFORCED CONCRETE BUILDINGS

by

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ABSTRACT

Pacific Earthquake Engineering Research (PEER) Center has developed a comprehensive framework for quantitative assessment of performance level of structures. The framework relies on integrated work of four consecutive stages to provide probabilistic description of system level performance in terms of repair cost, downtime, casualties, deaths or any other parameter of interest to engineers and stakeholders. This is for the purpose of communicating behaviour of facility under earthquake in term of identified damage states and expected economic losses, thus treats possible disconnection between engineers and stakeholders on the desired performance target for the facility.

Key objective of this dissertation is to present simplified version of the PEER framework to conduct earthquake-related financial loss studies for structures in a computationally efficient manner. The presented framework is utilized in this investigation to examine and compare efficiency of alternative seismic strengthening technique to control earthquake-induced monetary losses of a non-ductile hotel building, representative of 1960s construction. The framework integrates knowledge obtained by analyzing seismic environment at building site, investigation of structural demand, and quantifying levels of structural damage and consequential financial losses. Damage measures are computed, by generating fragility models, to link structural response directly to monetary losses. Seismic-induced economic losses are predicted by converting fragility information (i.e. damage probabilities) into financial losses utilizing inventory and monetary losses data of HAZUS-MH. The economic losses computed in this investigation included direct costs, such as construction cost of retrofit, and repair and replacement cost of the facility. In addition, indirect costs, such as losses due damage of building content and business interruption, as well as consequential losses, such as job and housing losses were also considered. Finally, decision tree model was implemented, as a final component of the framework, to establish a decision-assisting platform that enables transparent comparison and selection of the best retrofit option to reduce owner’s susceptibility for financial losses.

PREFACE

Chapter 3 is part of research work conducted in UBC Okanagan and was published as conference article as outlined:

Decision making tool for seismic retrofit of non-ductile reinforced concrete building, Al-Chatti, Q., Siraj, T., Rajeev. P., and Tesfamariam, S., *Canadian Society of Civil Engineering, General Conference*, Ottawa, Ontario, Canada, June 14-17, 2011.

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Risk management framework for assessment of seismic retrofit, Al-Chatti, Q., and Tesfamariam, S., Canadian Society of Civil Engineering, *3rd International Structural Specialty Conference*, Edmonton, Alberta, Canada, June 06-09, 2012.

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t_j: Thickness of FRP jacket

f'_{cc}: Confined concrete strength

ε_{ju}: Ultimate strain capacity of the jacket in the hoop direction,

φ_f: Flexural strength reduction factor

ε_{cu}: Ultimate concrete strain

ρ_j: Volumetric expansion ratio of the jacket reinforcement

L_p: Length of plastic hinge region

g: Gap between CFRP jacket and supporting member

f_{sy}: Yield strength of steel reinforcement

d_b: Longitudinal bar diameter of column reinforcement.

$P_t[LS]$: Probability of reaching a specified limit state over a given period of time (0,t)

D: Spectrum of uncertain hazards

d: Occurrence of predefined earthquake level

Φ: Standard normal cumulative distribution function

λ_{CL}: Lognormal of median drift capacity

$\lambda_{D|Sa}$: Lognormal of median drift demand

$\beta_{(D|Sa)}$: Uncertainty associated with the fitted power law equation

β_{CL}: Uncertainty related to drift capacity criteria

β_M: Uncertainty related to analytical modelling

$CNSD_{ds,i}$: Cost of drift sensitive non-structural damage for damage state ds and occupancy class i

$CNSD_i$: Cost of drift sensitive non-structural damage for occupancy class i

BRC_j: Building replacement cost of occupancy i

$PONSD_{ds,i}$: Probability of being in non-structural drift sensitive damage state ds for occupancy class i

$RCD_{ds,i}$: Repair and replacement ratio for non-structural drift sensitive damage in state ds and occupancy i.

LOF_{ds}: Loss of function due damage state ds

BCT_{ds}: Construction and clean up time for damage state ds.

MOD_{ds}: Construction time modifiers for damage state ds.

$YLOS_i$: Income loss for occupancy i.

FA_i: Floor area of occupancy class i.

INC_i: Income per day for occupancy i.

$POSTR_{ds,i}$: Probability of being in damage state ds for occupancy i.

RF_i: Recapture factor for occupancy i.

RY_i: Rental income losses for occupancy i.

$\%OO_i$: Percent owner occupied for occupancy i.

FA_i: Floor area of occupancy i.

$RENT_i$: Rental cost ($\$/ft^2/day$) for occupancy i.

$POSTR_{ds,i}$: Probability of being damage state ds for occupancy class i

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CHAPTER 1 : INTRODUCTION

1.1 Background

1.1.1 General

Past earthquakes, such as Saguenay in Canada (1988); Loma Prieta in U.S (1989); Northridge in U.S (1994); Kobe in Japan (1995); Golcuk-Izmit in Turkey (1999); Ji-Ji in Taiwan (1999); Gujarat in India (2001), and Seattle in U.S (2001), have recurrently highlighted vulnerability of non-ductile reinforced concrete (RC) structures. Particularly, pre 1970 design standards utilized strength based philosophy, which lacks ductility measures to achieve adequate overall deformation or energy dissipation. As well, common seismic deficiencies in non-code confirming structures are wide spacing of transverse reinforcement, discontinuity of positive reinforcement in beam and slab, and short lap-splice that may lead to poor seismic performance (Ghobarah, 2000). After the introduction of capacity based design, the current seismic design provisions were introduced in mid to late 1970s.

Despite the fact that these modifications allowed better performance of structures during earthquake, there are other risks that have been traditionally ignored in earthquake-resistant design. Namely, the aim of current design standards is to protect life safety, thereby no attempt was made to control potential economic losses or specify acceptable level of probability by which a structure remains functional after earthquake. This underlying problem is attributed to that current design practices are performed based on prescriptive criteria and simplified analytical methods. Thus, design codes lack to explicitly quantify seismic response of structures, causing inconsistence level of performance (Haselton and Deierlein, 2005). Recent researches (e.g. Krawinkler and Miranda, 2004; Aslani and Miranda, 2005; Mitrani-Reiser and Beck, 2007) proposed that using financial losses as metric to gauge the response of structural system is an adequate measure to quantify earthquake performance.

The need for better quantifiable metrics and constraints to control economic losses in seismic event was further highlighted by the noted monetary losses during past earthquakes. For example, during the 1989 Loma Prieta and 1994 Northridge earthquakes, substantial monetary losses were incurred despite the low loss in life (Insurance Information Institute, 2008). The 1989 Lome Prieta caused 63 deaths, more than 3000 injuries, and an estimate of

$6 to $13 billion in property damage (Benuska, 1990). Similarly, the 1994 Northridge earthquake caused 72 deaths, more than 9000 injured, and resulted in more than $25 billion in economic losses (Hall, 1995).

Further, the non-structural damages sustained by Olive View Hospital building during the 1994 Northridge event represents an example where designing a structure by prescriptive codes falls short to meet owner's and user's needs. Although the earthquake produced relatively moderate ground motion intensity, significant non-structural damages were incurred during the event. Particularly, sprinkler systems, ceilings, and water systems were damaged by earthquake- induced deformation, causing the hospital to evacuate and temporary shutdown. As so, the hospital was not functional to treat patients injured in the event; in addition, 377 patients had to evacuate (Hall, 1995). Similar downtime cases of essential facilities in earthquake events emphasize the conclusion that current design standards may not be enough to achieve satisfactory seismic performance.

The concern to control damage, economic losses, and loss of functionality in earthquakes promoted engineers to formulate documents (Vision, 2000; FEMA, 273; and FEMA, 356) that contain guidelines by which different performance levels can be attained. Thus, stakeholders and design professionals can make more informed decisions to meet a project's needs using performance based criteria. However, the design standards of these documents are qualitative, and often opened to subjectivity.

Pacific Earthquake Engineering Research (PEER) conducted a significant amount of research to address the need for better quantitative measures and improved methodologies to evaluate seismic performance of structures beyond the traditional goal of life safety. Along this trend, PEER proposed framework that quantify seismic performance in terms of parameters that are more relevant to stakeholders, such as deaths, economic losses, and downtime. The framework relies on integrated models and knowledge obtained by earthquake hazard characterization (i.e., suite of ground motions), determination of structural demand, identification of performance levels, and quantifications of degree of structural damages, economic losses and casualties. In other words, PEER framework represents a useful tool for policy- and decision- makers to evaluate earthquake behavior using explicit, quantifiable, and probabilistic matrices. Therefore, it is a suitable measure to justify improvement to building codes, assess performance of older-designed structures, and address the most difficult question of which is the effective mitigation pattern.

1.1.2 Retrofit of non-ductile structures

Major number of Canadian facilities was built before 1970s in response to the increase in population and the rise in immigration level (Gemme, 2009). In general, buildings constructed before 1970s are considered to be seismically vulnerable as they were designed during a stage of inadequate understanding of seismic performance and absence of proper seismic design provisions. As so, these infrastructures worldwide, particularly Canada, pose susceptibility against earthquake disasters, and are in a desperate need for repair and strengthening to protect it against seismic excitations (Gemme, 2009).

The common types of deficient structures in Canada are unreinforced masonry buildings, RC buildings, and steel buildings. These structures usually suffer deficiencies in their structural configuration, such as inadequate shear resistance, poor connection in precast concrete buildings, poor detailing in steel and unreinforced masonry buildings, and soft story mechanism in all types of buildings. As for concrete buildings, most of the aging buildings were only designed to carry gravity loads (Gemme, 2009). Column elements are usually considered as the weakest structural components in concrete buildings because these lack adequate confinement and shear resistance capacity. Further, these buildings commonly suffer detailing deficiencies such as inadequate number of reinforcing bars, short lap-splice, lack of shear reinforcement in the joints, inadequate anchorage of beam longitudinal reinforcement in the joints, and limited shear capacity of the beam (Mitchell, 2007).

Moreover, according to study performed by the Munich Reinsurance Company in Canada in 1992, a moderate size earthquake with magnitude of 6.5 may result in economic losses of $15 to $30 billion dollars. Further, considering the possible rise in population with the growth of economy in future, and given an earthquake of magnitude 8, a much higher financial losses may be expected (Mitchell, 2007). This damage assessment does not include losses attributed to the occurrence of earthquake consequential disasters such as liquefaction, land slide, fire, and services interruption.

As a result, even that seismic risk in Canada is moderate comparing to it in Japan, New Zealand, USA and Pakistan, the occurrence of destructive natural phenomenon with uncertain nature (i.e. earthquake) may result in substantial financial losses and threat to public safety and security in Canada. As a result, high budget needs to be allocated to reduce potential earthquake risk through an effective seismic rehabilitation approach; rather than, having the society being subjected to higher financial losses during earthquake event. This raises the concern for risk management tools to facilitate the seismic performance assessment process of older-designed structures, and helps engineers with the optimal retrofit technique selection.

1.2 Objective and Methodology

The central objective of this research is to propose decision tree tool that utilizes the concept of using earthquake-related financial losses as a quantitative metric of structural performance to prioritize the selection of alternative mitigation measures to control owner's susceptibility for financial losses. The formulation of the methodology is based on a simplified implementation of PEER's framework to minimize the computational effort required to carry cost-benefit assessment of alternative retrofit patterns. The resulting model provide practicing engineers with a tool to select the optimal retrofit using a measure (i.e. dollar loss) that well gauges the suitability of a mitigation to reduce potential damage and subsequent economic losses, thus reflects contribution of retrofitting on the system level performance of structure in term of reduced financial losses.

The framework (Figure 1.1) integrates seismic hazard, structural analysis and seismic performance assessment, fragility analysis of as-built and retrofitted cases, related retrofit and physical damages cost, and decision tree analysis to facilitate the decision making process on the desired retrofit option.

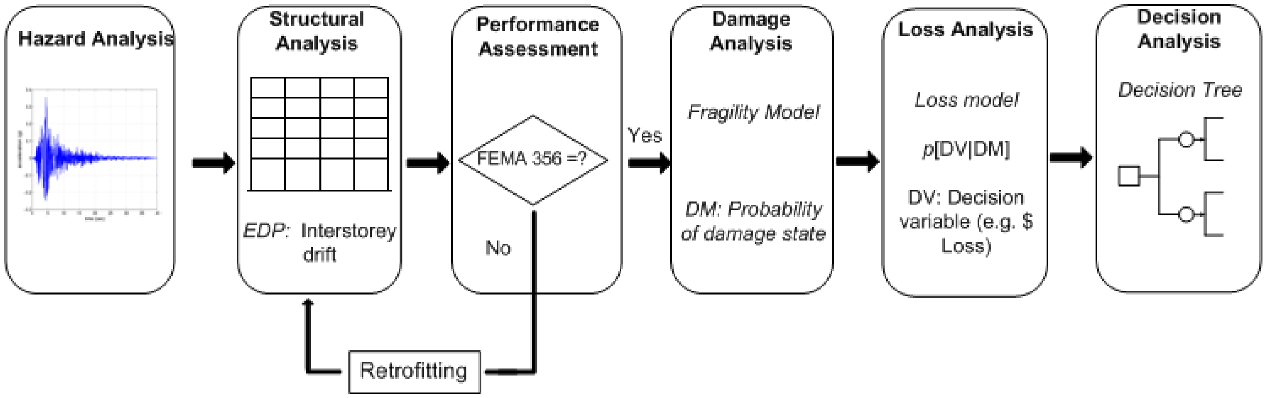


Figure 1.1. Overview of the study methodology (modified from PEER framework (Cornell *et al.* 2005))

Applicability of the methodology is examined in this dissertation to assess and compare the cost-effectiveness of alternative seismic retrofits for typical non-ductile RC building representative of 1960s construction. The case study building is a seven-story RC hotel building located in western U.S. and designed according to 1964 Los Angeles City Building Code. Three retrofitting techniques are considered, including the addition of steel bracing, shear walls, and base isolation. The performance of unretrofitted and retrofitted cases is investigated using nonlinear static and nonlinear time history analyses considering range of seismic hazards. The ground motion records represent 50% (very low) 10% (low), 5% (moderate), and 2% (high) probability of occurrence in 50 years period. Seismic assessment of the facility was conducted according to Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356) criteria. In this engineering approach, design objectives are expressed in terms of desired performance targets for various earthquake return periods. The performance targets are classified based on drift limits to control the damage sustained by structures during seismic events. The target performance levels of FEMA 356 include Immediate Occupancy (IO), Life-Safety (LS), and Collapse Prevention (CP). In FEMA 356, Basic Safety Objective (BSO) requires that LS and CP performance levels are achieved for hazard levels of 10% and 2%, respectively.

Fragility relations were derived for unretrofitted and retrofitted cases using the response data of nonlinear time history analyses. Fragility assessment is intended to describe reduced earthquake-related damages, and thus the enhanced seismic resiliency of the case study building due to rehabilitation. Based on fragility information, damage state probabilities for unretrofitted and retrofitted cases were then converted into monetary losses using inventory losses and economic data provided in HAZUS-MH (2003) manual. Finally, decision tree tool was employed to select the cost-effective seismic risk mitigation strategy for the case study building.

1.3 Scope and Organization

This dissertation deals with the assessment of seismic rehabilitation using economic losses as performance metrics. Structural response predictions are used to estimate earthquake-related economic losses of a seismically deficient RC hotel building representative of 1960s construction. To quantify implication of mitigation, unretrofitted and retrofitted cases are compared to illustrate reduction of owner's susceptibility to financial losses due to mitigation. Decision tree tool is used to provide insight on the cost and benefits associated with implementing each retrofit, where benefits are the reduction in seismic-induced damages and financial losses.

Chapter 2 identifies key characteristics of RC frame structure constructed before 1975. Vulnerability of these structures was mainly controlled by engineering detailing, structural geometry, and occupancy load. It is presumed that all structures were designed according to the governing building codes, but there are differences related to size, function, and design aspects. Collapse or partial collapse of RC frames during past earthquakes was also reported for the purpose of motivating the examination of seismic performance of older RC structures. Chapter 2 also provides overview of the seismic performance assessment, including nonlinear modelling and structural analysis procedures. Further, available retrofit programs to treat seismic deficiencies of RC structures were discussed.

The seismic performance assessment of the non-ductile case study building is carried in **Chapter 3**. Examination of lateral performance characteristics for unretrofitted and retrofitted cases is reported based on nonlinear static analysis and nonlinear time history analysis. The assessment process also included generating fragility relations to measure efficiency of rehabilitation measures to reduce seismic-induced damages. The outcomes of Chapter 3 are family of performance metrics that quantify effectiveness of rehabilitation strategies to manage seismic risk.

Chapter 4 presents a brief literature review of previous studies related to regional and building specific loss estimation methodologies. The chapter also extends assessment of seismic strengthening to predict economic losses of the non-ductile RC frame structures in future earthquake. Simplified version of PEER's framework is proposed and detailed to conduct economic loss studies. It proposes the use of HAZUS (FEMA, 2003) framework in an attempt to provide less computationally expensive and limit resources required to predict reduced seismic-induced monetary losses due to mitigation. Estimation of earthquake-related losses is an explicit measure of seismic rehabilitation to reduce financial losses posed by non-ductile RC structures.

Chapter 5 summarizes the contribution and findings of this investigation. These findings include the use of economic losses as performance metrics to evaluate effectiveness of seismic retrofit to upgrade performance level of non-ductile RC frame structure. Other key outcomes are represented by the use of decision tree model to present and compare cost and benefits associated with each mitigation strategy. Finally, areas of future research are outlined to lay the ground for future research work.

CHAPTER 2 : LITERATURE REVIEW

2.1 Introduction

Reinforced concrete (RC) structures constructed before 1975 experienced various degree of damages during previous earthquakes. This improved understanding of inelastic performance of RC structures, and thus led to evolution in seismic design provisions. This chapter discusses non-ductile features of pre-1975 RC frame structures, and illustrates the difference with modern code-conforming structures. Observed damage to RC frames during past earthquakes was also presented. The concern to highlight the major problems of RC constructions before significant advancement in design codes in 1970s was instituted. The chapter also provides overview of performance-based paradigm, structural analysis, seismic vulnerability assessment, and seismic rehabilitation programs for RC buildings. Review of experiment and analytical works concerning the above areas was also reported.

2.2 Vulnerability of Non-Ductile Reinforced Concrete Structures

Reinforced concrete (RC) structures rely on beam and column elements to resist lateral and gravity forces. Between 1920s and 1930s, RC structures were constructed with masonry infills between frame elements. These infills caused substantial increase in stiffness and strength of structures. The rigid pattern of structures associated with un-engineered masonry infills was changed in newer construction. During 1950s and 1960s, the considered design philosophy relied more on the action of framing members to resist gravity and seismic loads. Thus, considerable attention was devoted for calculations related to member forces, material properties, and allowable deflection. By late 1960s, advancement in design and construction practices allowed the construction of 20 stories frames without the need for shear wall infills (Degenkolb, 1994). These structures have been widely used for industrial, commercial, and residential facilities.

Based on damage observation from past earthquakes and growing understanding of inelastic performance of structures, RC structures constructed before 1975 are considered as seismically deficient. This section discusses the characteristics of pre-1975 non-ductile RC facilities. Also, damage observations and lessons learned from previous earthquake events are presented to reveal major problems of non-ductile structures before significant improvement in seismic design standards in 1970s was instituted. These considerations generate the motivation to extend and simplify performance based earthquake engineering (PBEE) approach to assess and upgrade seismically weak infrastructures using a cost-effective retrofit strategy.

2.2.1 Deficiencies of Non-Ductile Reinforced Concrete Structures

The 1971 San Fernando earthquake served as watershed in the seismic design practice of RC structures. Observations of performance of reinforced concrete facilities during the event and subsequent studies led to improved design provisions that appeared in late 1970s building codes. These design provision have further improved ever since.

An important distinction between the design of older and newer member is associated with design parameters, i.e., the amount of transverse reinforcement in beam, column and joint elements, suggesting susceptibility of non-ductile frame to have brittle shear failure mechanism (Tesfamariam and Saatcioglu, 2008, 2010). This is attributed to the poor concrete confinement level associated with insufficient amount of transverse reinforcement causing the frame to exhibit a limited deformation capacity. The figure also illustrates differences related to detailing of longitudinal reinforcement. It can be noted that in non-ductile frame there is short overlap length of reinforcing bars, indicating susceptibility of non-ductile frame to experience lap-splice failure or pull-out of discontinuous bottom beam bars.

Another type of deficiencies encountered in older-designed structures is represented by design deficiencies. These include (1) inadequate lateral load resisting system (e.g., shear walls or special moment resisting frame, (2) lack of redundancy (i.e., alternate load path), such that collapse is triggered by failure of a few structural members, (3) irregularity in plan or elevation, such as vertical setbacks, and L- or T-shaped plan, (4) storey mechanism due to presence of soft storey (Tesfamariam and Saatcioglu, 2008, 2010). In addition, older design provisions lacked requirement on relative strength of structural components, thus there was no particular hierarchy for the failure mode of structural components. Consequently, some older design structures suffered weak-column strong-beam failure pattern. This is in contrast to modern buildings which are designed so that yielding initiates at beam elements to avoid catastrophic collapse of structures during an earthquake.

A third type of deficiencies suffered by older-design structures are classified as construction deficiencies (Tesfamariam and Saatcioglu, 2008, 2010). These are represented by low-quality workmanship, deviations from structural drawings during construction phase, and the use of poor construction materials.

2.2.2 Observed Mode of Failure of RC members with Ductile Detailing Deficiencies

Reinforced concrete columns designed according to pre-1970s standards commonly lack proper ductile detailing. This deficiency may serve as a factor for three common types of failure modes to occur under seismic activities. The damages and their sequences are (1) development of inclined cracks as the tensile strength of concrete is exceeded, (2) opening of inclined cracks and onset of cover concrete spalling, (3) rupture of transverse reinforcement, (4) buckling of longitudinal reinforcement, (5) disintegration of concrete core (Seible *et al.*, 1997).

The second mode of failure is confinement failure of the flexural plastic hinge region of the column. The process of plastic hinge deterioration consists of (1) flexural cracking, (2) concrete cover spalling, (3) buckling of longitudinal reinforcement, and (4) compression failure of the concrete core (Seible *et al.*, 1997). This failure mode is desirable over the shear mode of failure as it occurs with some displacement ductility, and it is restricted to specified regions within the column length. Such desired failure mode can be achieved by increasing the confinement level of plastic hinge regions through the use of transverse reinforcement, or the adoption of confining device for the case of substandard columns. In case of jacketing retrofit, the added confining pressure intends to prevent concrete cover spalling, provide lateral support for the longitudinal reinforcement, and enhance the strength and ductility of the concrete. These characteristics can be effectively addressed with the use of circular confining device to maintain uniform confinement action along the entire column parameter. As for rectangular columns, oval jacket can be used to provide proper confining pressure along the column parameter, whereas rectangular jacket provided inwards forces only at the corners. This in addition to that the jacket needs to be designed with adequate thickness between the corners of the rectangular cross-section to prevent lateral dilation and buckling of the column longitudinal reinforcement. However, large scale tests indicated that properly designed rectangular carbon jackets addresses the desired characteristics to attain higher displacement capacity levels (Seible *et al.*, 1997).

The third failure mode is related to bond-slip of reinforcement at column ends. This failure occurs as vertical cracks initiates in the concrete cover, and the debonding of lap-spliced reinforcement progresses with the increased lateral expansion and spalling of concrete cover. In case short lap-splice exists, the strength degradation rate can progress rapidly with a low level of flexural ductility demand. This can be mitigated through confining the plastic hinge region with confining device to provide continuous, lateral pressure to prevent the debonding of lap-spliced reinforcement.

2.2.3 Observed Damage to Older Designed RC Structures

This section reports damages experienced by non-ductile RC structures, designed between 1950 and 1975, during past earthquake events. Damage to non-ductile structures during 1971 San Fernando event represented a dividing line, in the seismic design practice, after which ductile detailing requirements became compulsory in building codes.

Olive View Hospital Medical Treatment Building consisted of columns with spiral reinforcement and others with widely spaced ties. Despite the fact that both column types sustained significant damages due to the earthquake, the spiral wrapped reinforcement offered more confinement to the concrete core, and consequently the columns retained their gravity load carrying capacity. Another example where insufficient confinement level, and limited ductility contributed to column failure is in the case of the Olive view Psychiatric, Santa Rosa Social Services Building, Imperial Country Service Building and many others. Likewise, inadequate transverse reinforcement detailing in bean-column joints caused the collapse of the Kaiser Permanente structure during 1994 Northridge earthquake. Further, the collapse of Bullock's Department Store, in the Northridge event, was an evidence of the necessity for continuous longitudinal bottom reinforcement in the slab-column connection.

Lessons learned from previous earthquakes also demonstrated flaws in the design concept of non-ductile RC frames. Columns were frequently subjected to high shear forces, which exceeded the designed capacity, due to the presence of non-structural elements causing short column effect, torsional effect, or over-strength in floor system. As an illustration, in the Barrington building, the columns experienced increased shear forces due presence of deep spandrel beams, causing X-cracking in the columns.

Further, earthquake incurred damages highlighted the effects of irregularity in strength and stiffness, either in plan or elevation, on performance of non-ductile RC frames. For example, the presence of soft-storey acted as the weakest element, and consequently fused failure of the structure. A soft-story is formed either by discontinuity of structural and non-structural shear walls at bottom stories because of architectural reasons, or that the existing beam elements in structure are stronger than columns. Soft-storey mechanism caused failure in several facilities during past earthquakes, such as Olive View Medical Building, Imperial County Services buildings, and May Company Garage collapse.

Asymmetric distribution of structural and non-structural walls is another cause of irregularity, where a facility with this deficiency is a subject for torsional forces during seismic event. This behaviour contributed to failure of Olive View Hospital Psychiatric Building, and the May Company Garage.

These damage observation revealed inadequacies of reinforcement detailing, and are evidence of the seismic vulnerability and potential hazards constituted by non-ductile RC facilities.

2.3 Performance Based Evaluation

Damage observations and understanding of inelastic behaviour of structures under seismic excitations led to advancement in earthquake engineering. This inspired the evolution of seismic design standards and methodologies. In this trend, performance based approach emerged, where the design objectives are stated as performance level target based on potential seismic risk, function of the utility, and the needs of owners and society (Liel, 2008).

Performance based earthquake engineering provides probabilistic description of structural level performance, unlike conventional design methods that rely on evaluating limit state capacity of individual components. In performance based methodology, the operational status (or design) criteria are linked to drift limits for given earthquake levels. This is because past earthquakes highlighted the relation between underwent drift and induced amount of damage, and so drift measure is considered a reliable indicator to evaluate damage state of structures.

As a result, framework of performance based methodology enables engineers to establish communications with client based on identified damage state of a facility for a given earthquake intensity. This is particularly useful as most owners are interested with identification of building performance on deterministic basis, such as whether the building is safe or not, rather than in discussions of building state that involve probabilistic measures or recurrence intervals. Therefore, performance based engineering allows decision makers to quantify seismic performance of facilities in more rigours manner than ever was possible.

As the concept of performance based design becomes more accepted in practice, the procedure of seismic retrofit and rehabilitation has been affected. Consequently, the procedure of attaining retrofit objective is carried out based on the importance and the desired performance level of the facility during seismic event of specified return period. Toward that role, the seismic rehabilitation framework of ASCE-31 (2000) and FEMA 356 (2000) is attractive solutions to practitioners. In this dissertation, FEMA 356 criteria are utilized to assess effectiveness of adopted retrofit patterns to upgrade the performance of exiting seismically deficient RC building, typical of 1960s construction, to match modern standards. The following subsections outline design objective and performance evaluation criteria for FEMA 356 (2000).

2.4 Rehabilitation Objective

The design objectives of this engineering approach are expressed in terms of standardized performance targets for various earthquake return periods. The performance targets are classified based on drift limits to control the damage sustained by structures during seismic events.

Multiple performance objectives are considered by this approach to account for the various needs of owners. The performance targets ranges from state of preventing damage to state of operation. Since performance levels are linked to drift limits, damage state of facility can be identified by computing lateral drift values of the structure. As so, rehabilitation objective determine to great extent the reduced earthquake-related damages due to adopting a retrofit measure, as well as clearly address the reduced risk attained on occupant safety.

Thus, standard of FEMA 356 represents a useful tool to investigate life safety threat posed by older designed structures, and assess suitability of alternative retrofit option to accommodate seismic vulnerability of deficient structures in term of reduced earthquake-induced damages. The target performance levels of FEMA 356 (2000) are included in Table 2.1.

Table 2.1. FEMA 356 rehabilitation objectives (reproduced from FEMA 356 (2000))

Drift limits		Operational	Immediate Occupancy	Life Safety	Collapse Prevention	Probability of Exceedance
		(0.6%)	**(1.0%)**	**(2.0%)**	**(4.0%)**	
Earthquake Return Period	72	a	b	c	d	50%/50 years
	225	e	f	g	h	20%/50 years
	474	i	j	k	l	10%/ 50 years
	2475	m	n	o	p	2%/ 50 years

Notes:
1. Each cell in the above matrix represents a discrete Rehabilitation Objective.
2. The 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:
k + p = Basic Safety Objective (BSO)
k + p + any of a, e, i, b, f, j, or n = Enhanced Objectives
o alone or n alone or m alone = Enhanced Objective
k alone or p alone = Limited Objectives
c, g, d, h, l = Limited Objectives

Immediate Occupancy (IO): this performance level requires that the structural components maintain most of its pre-earthquake capacity. Also, the structure should withstand limited amount of damage, and that it be safe for re-occupancy. This suggests that the risk to public safety constituted by damage state of the facility is very low, and minor repair should be appropriate.

Life-Safety (LS): structure meeting this performance level may experience significant damages but retained margin against partial or total collapse. The repair of the structure may deem to be uneconomical. Nevertheless, life threat posed by building meeting this performance level is low.

Collapse Prevention (CP): the structural components of facility meeting this performance level undergo significant damage and continue to support gravity loads, but retain no margin against collapse. This indicates that the damage state of the structure is on the verge of partial or total collapse, and that the lateral load resisting systems suffered significant degradation in stiffness and strength. In addition, large amount of permanent deformation is undergone by the structural system. And so, the utility may not be practical for repair or safe to reoccupy. Also, injuries due falling from structural debris may exist. In other words, utilities meeting this level may pose risk hazard to life safety and will be great deal of economic loss. However, since the structure is designed not to collapse, then gross loss of life will be saved.

2.4.1 Global level approach

Global level evaluation provides assessment on the overall seismic performance of the facility. The assessment is carried out by comparing the inter-storey drift response parameter with the drift limits as stated by FEMA 356 for each performance level. Table 2.2 summarizes the global level drift limits for concrete frame and concrete wall as specified by FEMA 356 for three performance levels.

Three hazard levels are related to maximum inter-story drift, as a damage indicator parameter, of 1%, 2%, and 4%, respectively. In FEMA 356, Basic Safety Objective (BSO) requires that LS and CP performance levels are achieved for hazard levels of 10% and 2%, respectively.

Table 2.2. Performance levels and damage (reproduced from FEMA 356 (2000))

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	**Life Safety S-3**	**Immediate Occupancy S-1**
Concrete Frames	Primary	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns. Severe damage in short columns.	Extensive damage to beams. Spalling of cover and shear cracking (<1/8" width) for ductile columns. Minor spalling in non-ductile columns. Joint cracks <1/8" wide	Minor hairline cracking. Limited yielding possible at a few locations. No crushing (strains below 0.003).
	Secondary	Extensive spalling in columns (limited shortening) and beams. Severe joint damage. Some reinforcing buckled.	Extensive cracking and hinge formation in ductile elements. Limited cracking and/or splice failure in some non-ductile columns Severe damage in short columns.	Minor spalling in a few places in ductile columns and beams. Flexural cracking in beams and columns. Shear cracking in joints <1/16" width.
	Drift	4% transient or permanent	2% transient; 1% permanent	1% transient; negligible permanent
Concrete Walls	Primary	Major flexural and shear cracks and voids. Sliding at joints. Extensive crushing and buckling of reinforcement. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Some boundary element stress, including limited buckling of reinforcement. Some sliding at joints. Damage around openings. Some crushing and flexural cracking. Coupling beams: extensive shear and flexural cracks; some crushing, but concrete generally remains in place.	Minor hairline cracking of walls, <1/16" wide. Coupling beams experience cracking <1/8" width.
	Secondary	Panels shattered and virtually disintegrated.	Major flexural and shear cracks. Sliding at joints. Extensive crushing. Failure around openings. Severe boundary element damage. Coupling beams shattered and virtually disintegrated.	Minor hairline cracking of walls. Some evidence of sliding at construction joints. Coupling beams experience cracks <1/8" width. Minor spalling.
	Drift	2% transient or permanent	1% transient; 0.5% permanent	0.5% transient; negligible permanent

2.4.2 Member level approach

Member level evaluation identifies the members that are vulnerable. This shall assist with selecting proper retrofit option to upgrade the performance of deficient components. In other words, local level evaluation provides detailed information on the structural behaviour. This evaluation is carried based on comparing plastic hinge rotation of individual structural components with the maximum permissible plastic rotation corresponding to each performance level as stated by FEMA 356.

Plastic rotation is the amount of rotation experienced by structural component beyond the yield rotation of that component, and it assesses the inelastic response of the member. When plastic rotation limit of structural member is exceeded at given performance level, then the element has exhibited inelastic response and plastic hinge mechanism is formed. It is also noteworthy to mention that if plastic hinge is formed at the ends of column members that are supporting a storey, then storey mechanism may be exhibited by the structure.

2.5 Seismic Retrofit Technique for RC Structures

Potential seismic risk mitigation of existing infrastructures is undertaken using two distinct strategies. The concept of the first strategy is based on enhancing the response parameters of individual members that are found to be vulnerable. This requires treatment of design and detailing deficiencies of components so they exhibit the pre-defined ductility level without reaching their ultimate states. The second approach is based on strengthening the overall capacity of the facility, or reducing demand on the existing structural components (Moehle, 2000). This involves modifications of the global level system of structures.

Various rehabilitation techniques are available in literatures that are used to reduce earthquake vulnerability. The selection process of the most suitable and cost-effective intervention method to mitigate a seismic hazard is of a challenge to earthquake engineering community, and involves technical, sociological, and financial measures (Tesfamariam *et al.*, 2010). Such factors that govern the selection process are as follows (Thermou and Elnashai, 2002):

- Function and importance of the facility;
- Quality of the available workmanship;
- Duration of downtime; desired performance level as stated by the owners;
- Aesthetical compatibility of the rehabilitation measure with existing building configuration;
- Type of irregularity in the building structural characteristics (i.e., strength, stiffness, or ductility);
- Capacity of the foundation system, and the technology required for the adoption process of the retrofit scheme.

2.5.1 Local intervention method

These methods tend to upgrade the structural characteristics of vulnerable members so that they exhibit adequate deformation capacity without reaching their limit states while responding to the pre-defined performance level. It is a useful approach to treat seismic deficiencies of individual members, and upgrade the overall performance of the structure to the desired performance level. The commonly used methods are discussed below.

2.5.1.1 Steel plate adhesion

This technique is mainly used to enhance shear and flexural strength of beams. In case of thick steel plates are required, it is recommended to use several thin layers in order to minimize the susceptibility of bonding shear failure mechanism. The application of this method requires understanding of the short and long term behavior of the adhesive material used. Further, quality workmanship is required during the installation stage to ensure composite action between the adherents. In this method, focuses are on preventing de-bonding failure mode of the externally bonded plates.

Beres *et al.,* (1992) investigated the effectiveness of confining deficient beam-column joint using steel plates. The objective was to maintain concrete integrity and prevent spalling of the concrete. Further, steel channels were attached to the bottom face of the beam in order to treat potential bond-slip failure of inadequate anchored steel reinforcement. The results show applicability of the scheme to prevent bars' slippage, increasing shear resistance capacity of the joint, and reduce strength deterioration.

Ghobarah *et al.* (1996) conducted experimental test to confirm the applicability of treating design deficiencies of beam-column joint using corrugated steel jackets. The gap between the jacketing scheme and the concrete was filled by grout. The results showed considerable increased shear resistance of the retrofitted joint, and improved ductility of the joint causing the plastic hinge formation mechanism to take place in the beam. Ghobarah and Youssef (1999) studied the benefit of upgrading shear strength and bond-slip resistance of beam-column joints in order to eliminate brittle mode failure, and ensure ductile response of the overall frame performance. Estrada (1999) examined the rehabilitation of interior beam-column joint by attaching steel plate to the bottom face of the beam at each side of the joint. The concept was to treat the inadequate anchored steel bars with the added steel plates. The results suggested ineffective improvement of the joint behaviour due to slippage of the steel plates.

2.5.1.2 Steel Jacketing

This is one of the coating techniques where thin steel plates are used to enchase deficient RC members. The space between the placed plates and the existing member is filled by non-shrinkage grout (Priestley *et al*., 1994; Aboutaha *et al.,* 1999). Steel caging scheme is an application form of steel jacketing retrofit. The steel caging scheme consists of attaching steel angles to the corner of existing cross-section and either steel plates or lateral steel straps are welded on the angles. The ensuing gaps between the steel casing and the existing concrete are filled with non-shrinkage grout. Further, shotcrete cover or grout concrete may apply in cases where corrosion or fire protection is required.

This retrofit scheme can be applied to increase the flexural capacity of RC columns and as an additional source of confinement to treat brittle failure mechanism of beam-column joints.

2.5.1.3 Fiber reinforced polymer:

Fiber reinforced polymer (FRP) composites are gaining popularity for use in practice as seismic hazard mitigation strategy. Their characteristics as that of high strength to weight ratio, high durability versatility and flexibility of use, ease of installation, low installation time, ease of transportation, low maintenance, and high corrosion resistance make them ideal for use as seismic retrofit schemes. This is particularly true in structural applications where restrictions related to weight of structure, time, or space apply. They are also marked with their ability to develop higher strength capacity than that of steel material. However, the FRPs lack the ability to transfer shear and axial loads, and are sensitive to lateral strain actions (El-Amoury and Ghobarah, 2005). Further, they behave in elastic fashion till failure. This indicates incapability of the material to develop proper amount of yielding and dissipate energy during extreme events. This concludes that the failure mode of FRPs is characterized to be of force-controlled mechanism. The composites are also considered as anisotropic material. This is reflected in being that the coefficient of thermal expansions in longitudinal and transverse direction are different. This anisotropic property in addition to the fact that the strength resistance of the material in longitudinal direction is significantly larger than it in the transverse direction causes de-bonding failure mechanism, splitting of concrete problems, and low fatigue resistance under thermal loadings.

This retrofit scheme is suitable to treat inadequate lap-splice, lack of proper transverse reinforcement and poor lateral confinement level of RC elements (e.g., beam-column joints). The technique is also considered effective to enhance flexural capacity of beam elements that suffer inadequate reinforcement anchorage, and upgrade the strength and ductility supply of deficient walls. Further, it is useful strategy to enhance punching shear resistance of flat slab systems. The application of FRPs scheme showed effectiveness of the method in enhancing deformation capacity of non-ductile structures, and eliminating their brittle failure mechanism developed under seismic excitation (El-Amoury and Ghobarah, 2005).

Fiber reinforced polymer composites are available in a form of constituents such as carbon (CFRP), glass (GFRP), and aramid (AFRP). The FRPs can be applied either as sheets or strips. The application of FRP sheets involves wrapping them around the selected structural element, while the FRP strips are glued to the member.

FRP schemes are considered advantageous over other conventional strategies because its application does not impact the mass or stiffness of the structure (Cheung *et al.*, 2000). This is important as increasing system stiffness is associated with an increase in the seismic demand placed on the structure. Further, the fact that implementation of this scheme involves minimal mass being added to the structure, makes it attractive solution to upgrade ductility and flexural strength of deficient masonry walls (Willis *et al.,* 2009).

In term of design aspects, FRP jacket are adopted to enhance confinement level of flexural plastic hinge region, and achieve stated ductility level by concluding the required thickness of the composite jacket. For circular columns, the equation to determine thickness of FRP sheets to achieve a desired ductility level is expressed as follows (Priestley *et al.*, 1996):

$$t_j = 0.09\frac{D(\varepsilon_{cu} - 0.004)f'_{cc}}{\varphi_f . f_{ju} . \varepsilon_{ju}} \qquad [2.1]$$

where f'_{cc} is the confined concrete strength that depends on the nominal concrete strength and the lateral confining pressure, and it can be conservatively taken as $1.5f'_c$ (Priestley *et al.*, 1996); f_{ju} and ε_{ju} are the strength and ultimate strain capacity of the jacket in the hoop direction, respectively; φ_f is the flexural strength reduction factor and typically considered as 0.9; and, ε_{cu} is the ultimate concrete strain that depends on the confining pressure induced by the composite jacket. The ultimate concrete strain can be computed as (Priestley *et al.*, 1996):

$$\varepsilon_{cu} = 0.004 + \frac{2.8\rho_j f_{ju} \varepsilon_{ju}}{f'_{cc}} \quad [2.2]$$

where ρ_j describes the volumetric expansion ratio of the jacket reinforcement. And, the length of plastic hinge region to which the sheets are applied is computed using the following formula (Priestley *et al.*, 1996):

$$L_p = g + 0.044 f_{sy} . d_b \quad [2.3]$$

where g represents the gap between CFRP jacket and supporting member; and, f_{sy} and d_b are the yield strength and longitudinal bar diameter of column reinforcement.

The confinement action of the jacket constitutes of radial pressure supplied by jacket curvature, and tensile hoop stress generated by lateral dilation of the concrete. In case of rectangular column, the induced radial pressure forces vary with changing radii of curvature in different loading directions. This is accounted for by designing the thickness of the jacket to be twice as that the theoretical thickness derived for equivalent circular diameter (Seible *et al.*, 1995). This is recommended for columns with size ratio of less than 1.5. Further, experimental test on rectangular jackets designed with the assumption of doubling the equivalent circular column jacket indicated suitability of the jacket to perform well up to the designed ductility level.

2.5.1.4 Carbon fiber reinforced polymer sheets and strips

Carbon fiber reinforced polymer (CFRP) jacketing is effective solution to treat detailing deficiencies of older designed reinforced concrete buildings, such as poor confinement level, missing of adequate transverse reinforcement, and short lap-splice (Cheung *et al.*, 2000). Although CFRPs are commonly used for masonry and concrete structures, they are also used as alternative to steel jacketing schemes in upgrading performance level of steel frames (Hollaway, 2003). Their ease of application makes them attractive solution in cases where limited space is available to construct adequate bolted or welded connections (Gemme, 2009).

2.5.1.5 Fiber Reinforced Cement

Fiber reinforced cement is an effective scheme to manage serviceability problems of unreinforced masonry structures. This type of problems are conventionally treated either through replacing the deficient load bearing walls with a lighter frame, or by applying structural jacketing methods. However, such strategies alter the overall stiffness of the structure leading to an increase in the seismic demand sustained by the system. Hence, an attractive solution is to apply FRP overlay on the deficient walls. The scheme system consists of FRP layer reinforced by high strength fiber-glass mesh. Application of this scheme enhances the strength and ductility of masonry walls without modifying the overall stiffness of the building (Cheung *et al.,* 2000).

Ghobarah and Said (2001) performed experimental test to examine the effectiveness of encasing deficient beam-column joint by a U-shaped glass fiber reinforced polymer (GFRP). The objective of their work was to evaluate the improved ductile response obtained by upgrading shear resistance capacity of the joint, and allowing the plastic hinge to form in the beam. The results showed elimination of brittle shear failure in the joint, and ductile mode failure of the beam has occurred.

Prota *et al.* (2001) carried experimental work to investigate the benefit of rehabilitation beam-column joint using FRP rods and laminates. The objective of their work was to limit failure mechanism to the beam by strengthening the flexural capacity of the column using FRP rods, and enhancing the confinement level of the deficient joint. The results showed that the strength and failure mechanism can be altered by varying the combined use of the FRP sheets and rods.

El-Amoury and Ghobarah (2002) investigated the effectiveness of upgrading shear strength and bond-slip resistance of beam-column joints constructed according to the pre-1970s building codes. The case study joints were wrapped with GFRP sheets as additional source of confinement and to maintain concrete integrity. The retrofitted joints exhibited ductile failure mode response, while the unretrofitted joint showed brittle shear response combined with bond-slip failure modes. Their study also examined the suitability of treating bond slip failure of bottom reinforcement of the beam with inadequate anchorage. Their results suggested improved bonding condition of bottom reinforcement and delayed slippage of the reinforcement.

Mukherjee and Joshi (2004) studied the application of FRP sheets and strips with different configurations on the performance of deficient RC beam-column joint. Carbon and glass FRP material were examined as retrofitting schemes. Both of the schemes contributed to the stiffness of the joint, and allowed the reinforcing bars of the joint to yield at higher stresses. The results also demonstrated suitability of the intervention methods to increase deformation capacity of the joint, suggesting a delayed collapse mechanism of the joint. Further, FRP specimens exhibited higher energy dissipation capabilities attributed to the debonding and delamination of the FRP layers, in comparison to the controlled specimen.

2.5.2 Global Intervention Method

This retrofitting strategy is considered when the retrofit objective is not to upgrade ductility level, or when no interruption in lateral load path exists. The most popular global retrofitting schemes are discussed below.

2.5.2.1 RC jacketing

RC jacketing is a commonly applied scheme to treat seismic deficiencies of older designed RC structures. This method can be used to either upgrade selected structural components that are found to be vulnerable, or enhance the performance of the overall system (Thermou and Elnashai, 2006). When the longitudinal reinforcement of the jacket passes through holes drilled in the slabs, then the retrofit scheme is considered as global intervention strategy. However, when the jackets are applied to encase particular members, then the strategy is applied as local intervention method.

Incorporation of this scheme allows uniform distribution of lateral load strength capacity over the building height. Hence, it prevents failure mechanisms attributed to concentration of lateral load resistance, which usually occur in cases where few shear walls are available. The disadvantages of this method lie in the difficulties associated with the construction of the added members. For example, the presence of existing column members does not allow placement of cross-ties for the longitudinal reinforcements that are not at the corner of the jacket member. Another disadvantage is in the uncertainty regarding the bond between the existing concrete surface and the jacket member. This shall influence the shear stress transfer between the original and added member.

To date, the application of this retrofit scheme lack proper design and detailing guidelines to upgrade performance level of the selected group of members, or structure to a stated performance target.

2.5.2.2 Addition of walls

This intervention method is commonly used to strengthen lateral load resistance capacity, and mitigate story mechanism failure modes (i.e. soft story) of vulnerable structures. It is also advantageous in limiting lateral drift of the structure. This shall reduce the amount of damage sustained by the structure under seismic excitation. During the design process, attention should be given to establish proper distribution of shear walls in plan and elevation to avoid the formation of irregular system configuration. Further, application of this scheme usually involves modification to the existing load path of the structure. This is to ensure that inertia forces are transferred to the walls. In addition, proper attention should be paid to design adequate shear connections in order to guarantee composite action between the added walls and the existing frame structure.

The design and details of the added walls are carried by considering them as part of the new system configuration. The walls are designed for shear throughout their heights, and over designed for flexure above the plastic hinge region. This shall ensure that the inelastic response is restricted to the base of the wall, and that the remaining height of the wall exhibits elastic performance.

This retrofit scheme can also be adopted through infilling the openings of frame bays. The walls, in such cases, incorporate the surrounding beams and columns where they serve the wall as boundary elements. This is useful in relieving the stress concentration at the corners of the opening. The main issue is to ensure integrated action between the existing frame and the added walls.

Implementation of the shear wall retrofit scheme may require rehabilitation of the foundation system in order to account for the increased overturning moment effect associated with the increase in building mass and stiffness. This rehabilitation process usually involves heavily and costly operations. Hence, the application of this scheme is considered impractical for buildings without adequate foundation system.

Extensive experimental programs to investigate the performance of reinforced concrete infilled frames were conducted (e.g. Higashi and Kokusho, 1975; Klingner and Bertero, 1976; Klingner and Bertero, 1978; Kahn and Hanson, 1979; Axley and Bertero, 1979; Axley, 1980; Sugano, 1980; Higashi *et al.,* 1982; Higashi *et al.,* 1984; Aoyama *et al.,* 1984; Liauw and Kwan, 1985; Jirsa and Kreger, 1989). In most of these studies, one bay, one-storey or one bay, two-storey infilled frame was examined under monotonic loading. The test results demonstrated the enhanced lateral load resistance capacity, and the reduced drift demand of the frames due to the implementation of the infill walls. The outcomes of the studies indicated that system level performance of infilled frames is controlled by material type of the infill wall (e.g. masonry or reinforced concrete) and reinforcement arrangements, reinforcement detailing in frame, such as amount of flexural reinforcement in column and ratio of transverse reinforcement in beam and column, and type and effectiveness of the connections between the infilled wall and enclosing frame. The results also highlighted that the contribution of infilled walls to strengthen nonductile RC frame can be limited due local failure at the lap splice region.

Miller and Reaveley (1996) and Gregorian (1996) performed feasibility studies on alternate retrofit scheme to improve seismic behaviour of buildings with substandard performance level. These studies showed that providing adequate number of infill walls constitutes an effective role to enhance the lateral stiffness and relive the existing structural components of the infilled system from the applied lateral loads. Turk (1998) and Canbay (2003) conducted studies to research the effectiveness of utilizing RC infill walls to retrofit damaged nonductile RC frames. The results of the studies illustrated that the damage state of the frames did not significantly affect the behaviour of RC infilled frames. The performance of the RC infilled frames mainly depends on the connections between the introduced walls and existing frame. The results also showed that the level of concrete confinement, especially at the lap-splice region, control the contribution of the introduced RC walls.

Altin *et al*. (2007) carried experimental study to investigate the effectiveness of introducing infill walls to upgrade seismic performance of non-ductile RC frame. Six specimens in which each consist of one bay, two-storey and of one-third scale were constructed and subjected to cyclic loadings. The frames were manufactured to reflect common types of seismic deficiencies. The results highlighted the ability of infill walls to enhance stiffness, strength, and energy dissipation capacity of seismically weak frame. However, the results indicated that short lap-splice of column longitudinal reinforcement adversely affected the integrated response of infill walls with the enclosing frame. Therefore, in the study, three different strengthening techniques were applied to address this deficiency. The application of the local strengthening methods prevented premature failure of column splices and increased the stiffness and strength of the infilled frame substantially.

Negro and Verzeletti (1996) conducted series of pseudo-dynamic tests on full scale four-storey reinforced concrete frame to investigate the influence of introducing different pattern of light non-structural masonry infills on global behaviour of the frame. The study was carried in three phases. In the first phase, the frame was tested as a bare frame using artificial acceleration record derived from real earthquake with nominal acceleration 50% larger than what specified for design. The performance of the structure was as that of strong column weak beam mechanism. And, the fundamental period of the structure, after the test, was half to that of the initial value, though the damage was limited and uniformly distributed. The base shear capacity of the frame was 140 kN and the maximum drift was approximately 2%. In the second phase, hollow brick masonry infills were uniformly distributed along the frame height to address their effectiveness to improve the lateral strength and stiffness of the overall frame. The results indicated that irregular distribution of infills result in severe damage to the frame, suggesting that non-structural infill panels should not be ignored during the design.

Govindan *et al.* (1986) performed study to investigate the influence of installing brick infill walls on the lateral load resistance and deformation capacity of seven-storey RC frame. The results demonstrated the strength of infilled frame was double to that of bar frame. The findings also suggested an increase in the deformation capacity of infilled frame, as the ultimate capacity of the frame was reached at 3.7% with respect to that of 1% for the bare frame.

Zarnic (1995) proposed mathematical model to predict the hysteretic response of infilled frames. The model was developed based on test results of 34 one-bay, one-story frame structure plus infill models. The model simulates the effect of infill wall as pairs of compressive struts, and the model was incorporated into the DRAIN-2D software. Michailidis *et al*. (1995) developed analytical model to study the seismic performance of masonry infilled RC frame. The model is capable of capturing strength and stiffness degradation, hysteretic pinching and slippage. Karayiannis (1995) proposed simple method to conduct nonlinear dynamic analysis to evaluate the performance of RC frames with infill walls.

Zarnic and Gostic (1997) investigated the effectiveness of adopting masonry infills to improve the seismic response of building frame. The study indicated that incorrect implementation of the infills can cause significant damage to the frame. Also, equivalent strut model was developed based on experimental results in order to study the effect of controlling parameters for the design.

Pincheira and Jirsa (1995), Lombard *et al.* (2000), and Inukai and Kaminosono (2000) carried research work to emphasize the strengthening achieved with the infilling process of structures. The outcomes of the studies indicated that the overturning effect and base shear forces are concentrated at the location between the infills and foundation of the structure. This suggests the need of strengthening the foundation at these locations.

Jirsa and Kreger (1989) constructed four specimens of one-bay, one-storey, RC frame to investigate the usefulness of utilizing infill walls to modify the behaviour of the frames. The frames were designed to reflect commonly observed detailing deficiencies. This includes wide spacing of transverse reinforcement in the column, and inadequate lap-splice length that is required to develop the designed tensile yield strength. At first, tests were conducted on three specimens to study the contribution of infill walls on RC frames that exhibit the previously mentioned detailing deficiencies. The results indicated that the application of the infills increased the lateral load capacity of the frames, although the column splice-length deficiency caused the frames to experience brittle mode of failure. As for the fourth specimen, longitudinal reinforcement was added adjacent to the existing columns to overcome the lap-splice deficiency. The test results of the fourth specimen demonstrated an increase in both strength and deformation capacity of the frame. Al-Chatti *et al.* (2011) and Al-Chatti and Tesfamariam (2012) addressed the significance of adopting shear wall retrofit scheme to enhance lateral strength of seismically deficient building, and reduce earthquake-induced damages.

2.5.2.3 Steel bracing

This retrofit scheme is effective solution to strengthen the overall lateral load supply of structures. It is advantageous in utilizing frame openings, improving the overall performance with minimal weight being added to the system, and the installation process of the braces require minimal downtime of the building function, especially when the external frame system is of steel structure. Concentric, eccentric, and post-tensioned braces schemes are available to upgrade performance level of vulnerable structures. The incorporation of the braces enhances the strength and stiffness, controls drift demand, and improve ductility supply of deficient structures (Thermou and Elnashai, 2006). The braces can be installed in frame openings, and so intervention with the foundation may not be required. However, the increased stiffness of the structure suggests increased seismic demand sustained by the structure. This addresses the need to evaluate the suitability of the foundation system for the increased loading effects. Further, adequate connections between the existing frame and the added braces are required for effective implementation of the method.

Several experimental studies were carried to investigate the application of using steel braces to upgrade lateral load resistance capacity of RC frames (Jones, 1985; Sugano, 1989; Goel and Lee, 1990; Yamamoto and Umemura, 1992; Maheri and Sahebi, 1995). The results of the studies demonstrated that the strength and stiffness of the tested RC frames are significantly improved with the use of steel bracings. The experimental studies also indicated that frame systems rehabilitated with the use of X-bracings exhibited higher strength capacity than those retrofitted with other bracing configurations. However, the findings of the studies suggested that careful attention is needed during the design of the connections to ensure proper transfer of the forces between the bracing system and the frame.

Researches to address the contribution of connection design on the composite action between the bracing system and the strengthened frame were conducted by Sugano (1989), Canales and Briseno de la Vega (1992), and Maheri and Sahebi (1995). Their studies concluded that the implementation of steel bracings with proper connection design can be an alternative to the use of shear walls as a retrofit scheme to upgrade deficient structure located in seismic areas.

As most of the experimental studies were performed on small scale models that were examined using static loadings, there is a need for analytical work to demonstrate the application of steel bracing as retrofit scheme on realistic building configuration when subjected to earthquake loading. Such analytical studies were performed by Miranda (1991), Bouadi *et al.* (1994), Pincheira and Jirsa (1995), Al-Chatti *et al.* (2011), and Al-Chatti and Tesfamariam (2012). The outcomes of these studies highlighted the improvement attained to the lateral load capacity of retrofitted structures due the use of steel bracings, especially for low-rise structures.

El-Amoury and Ghobarah (2005) conducted analytical study to examine the effectiveness of adopting X-steel bracings as retrofit scheme to improve the dynamic response of 9- and 18-storey non-ductile RC frames using PC-ANSR software. The contribution of the retrofit option on the overall performance of the structure was evaluated in term of inter-story drift response parameter and the sequence of failure mechanism. The results of the study demonstrated the capability of steel bracings to effectively reduce the inter-storey drift and increase the lateral stiffness of the structures. However, the results suggested brittle failure mechanism, such as joint shear failure, may occur due to the application of steel bracings.

Sarno and Elnashai (2008) performed analytical work to compare the contribution of retrofitting 9-storey steel moment resisting frame structure retrofitted using several bracing configurations. Three structural configurations were assessed: special concentrically braces (SCBFs), buckling restrained braces (BRBFs), and mega-braces (MBFs). Nonlinear time history analysis was employed to investigate the dynamic response of the rehabilitated structures. The overall seismic performance of the structures was assessed using inter-storey drift response parameter. The findings of the study illustrated that MBFs are the most effective bracing configuration to reduce earthquake induced drift demand.

Goel and Masri (1996) carried experimental test to investigate the effectiveness of adopting steel bracing to upgrade seismically weak slab-column RC structure. Two-storey, two-bay RC slab-column frame at one-third of full scale was selected as a testing model. The dynamic response of strengthened RC frame was investigated in two phases. In the first phase, the braces were located in the exterior bays to strengthen the RC frame; whereas, in the second phase, the braces were used to strengthen the interior bay of the RC frame. Their results demonstrated dramatic increase in the stiffness, strength, and energy dissipation capacity over the original case of the RC frame. Further, the results indicated that the application of the braces allowed the frame to behave in ductile manner through fifteen cycles with no failure.

2.5.2.4 Base isolation

This retrofit scheme is selected for the rehabilitation of facilities with historical value, valuable content, or when limitations in the conditions to modify the superstructure exist (Tena-Colunga *et al.,* 1997). It is also advantageous for cases where the desired performance level is well above the performance of the vulnerable structure. The objective of this scheme is to isolate the structure from the input seismic energy, and reduce the impact on the structure and non-structural components. The isolation devices are inserted between the superstructure and substructure. The disadvantage of this method lies in the exhaustive procedure required to install the isolators.

It is an effective solution for the rehabilitation of masonry structures, which are characterized by their brittle response during seismic events. This is particularly true in cases where the addition of walls or steel braces is impossible.

Extensive effort has been devoted to several base isolator and base-isolated structures. Experimental programs to develop and test isolator systems were performed in early 1970s (Skinner *et al.*, 1993). For example, laminated-rubber bearings has been examined and used for bridge isolation since 1970, although they have also been utilized in building structures. Lead rubber bearings were developed by Robinson in New Zealand in 1975 and have been used ever since (Skinner *et al.*, 1993). Comprehensive testing of Teflon bearings and friction-pendulum isolators has been performed by Mokha *et al.* (1990) and Zayas *et al.* (1993), respectively. Shaking table tests of base-isolated models for different isolator systems and structures have been conducted by Griffith *et al.* (1990); Yaghoubian (1991); Zayas *et al.* (1993), and Foutch *et al.* (1993). Dynamic test on full-scale building model isolated with the use of laminated-rubber bearings was conducted in Italy by Giuliani (1993).

Analytical studies to propose constitutive models for different isolator systems have been carried by Koh and Kelly (1990); Skinner *et al.* (1993); Buckle and Liu (1993), and Ali and Abdel-Ghaffar (1995). Several methods were proposed to analyze the dynamic response of base isolators. These methods range from the use of non-classical damped modes for isolating superstructural system (Skinner *et al.,* 1993) to the use of 3-D analyses where the nonlinear performance of the isolating system can be evaluated, while elastic response of the superstructure is considered using its most representative fixed-base mode shape (Nagarajaiah *et al.*, 1991). Further, standard 2-D and 3-D finite element software have been used to study the nonlinear response of isolated structural systems.

Several numerical simulations and parametric studies are reported in the literature on the seismic performance of hypothetical shaking table models and full-size structural system to evaluate the associated usefulness of adopting base isolators. Lee and Medland (1979); Kartoum *et al.* (1992), and Chen and Ahmadi (1992) conducted parametric studies to highlight the contribution of base isolators to reduce the seismic motion placed on the superstructure systems when subjected to real or artificial ground motion records. Su *et al.* (1990) and Fan and Ahmadi (1990) carried comparative studies to assess the contribution of different base-isolation systems under real earthquake motions. Juhn *et al.* (1992) and Nagarajaiah *et al.* (1992) performed analytical studies to evaluate the suitability of mathematical formulations to predict the hysteretic response of base isolators with respect to table test results. Nagarajaiah *et al.* (1993) and Jangid and Datta (1994) performed studies to investigate the performance of base isolators considering torsional coupling effect induced by earthquake loadings.

Tena-Colunga *et al.* (1997) executed numerical study to examine the application of different base isolation system to be used in typical building structures. In the study, hypothetical buildings were designed both as base-isolated and conventionally fixed based structures, and assumed to be located on hard soil conditions. 3-D Time history analyses were performed to investigate the suitability of lead-rubber bearing (LRB) and steel-hysteretic damper (SHD) to reduce the seismic demand placed on base-isolated structure. The results of the study confirms the effectiveness of implementing base isolators to considerably reduce the displacement, acceleration, and shear forces induced during earthquake on the stories and overall superstructure of base-isolated structures with respect to their counterpart rigid-base design. The results also demonstrated the effectiveness of the isolator systems to ensure elastic response of base-isolated structures. Conversely, in fixed based options, the structures experienced large inter-storey drifts suggesting strong inelastic response of the structures and potential to severe structural damages. Furthermore, the findings of the study illustrated that adoption of base isolators offers important saving on the volume of concrete and steel material needed to build base-isolated project comparing to their counterpart rigid-based structures. However, the application of base isolators can be significantly diminished when the superstructure or the isolator system is subjected to large torsional action. This is because response of structure under torsion shows that some of the installed isolators are subjected to more strength and deformation demand than others. Therefore, some isolators yield and displace substantially while others remain in their elastic range. Moreover, the findings of the study confirm with what published in the literature regarding the contribution of base-isolators and the effect of torsional response.

CHAPTER 3 : CASE STUDY BUILDING AND VULNERABILITY ASSESSMENT

3.1 Building Description

This section summarizes the structural and architectural details of hotel building located at Roscoe Boulevard freeway in Van Nuys city of Los Angeles county, California. The hotel building was designed during 1965 according to the 1964 Los Angeles City Building Code, and constructed in 1966. The Holiday Inn hotel is seven-story with floor area of 66,000 ft^2 (6,200 m^2). The building is located at latitude of 34.221^o N and 118.471^o W in the San Fernando Valley of Los Angeles metropolitan area. The building has been studied by many researchers such as Jennings (1971), Scholl *et al.* (1982), Islam (1996), Islam *et al.* (1998), Li and Jirsa (1998), Trifunac *et al.* (1999), Krawinkler (2005), Al-Chatti *et al.* (2011), and Al-Chatti and Tesfamariam (2012).

It has been instrumented with self-contained tri-axial accelerographs since 1967. The sensors recorded many earthquake such as 1971 San Fernando Van Nuys, 1987 Whittier-Narrows, 1992 Big Bear, and 1994 Northridge earthquakes.

The building sustained light damages during the 1971 San Fernando earthquake (M6.6), where the epicentre was located 20 km northeast of the building. The building also underwent the 1994 Northridge earthquake, and sustained severe damages. The epicentre of the event was located 1.5 km southwest of the building. After the 1994 Northridge event, the building was rehabilitated with shear walls to upgrade its seismic performance. However, this study is concerned with the building configuration as it existed before the 1994 Northridge earthquake.

The building plan dimensions are 19 m (62 ft, 8 inch) by 46 m (151 ft, 2 inch) in the north-south and east-west directions, respectively. The frame in east-west direction consists of 8 bays spaced at 18 ft, 9 in centers; and, the frame in the south-north direction consists of 3 bays at approximately 20 ft centers. The building is 19.8 m (65 ft) tall with uniform mass and stiffness distribution, and the structural system suffer no irregularities. The height of the first story is 4.11 m (13 ft, 6 inch); the height of the second story through seventh is 2.60 m (8 ft, 6 inch), and the height of the roof 2.61 m (8 ft, 6.5 inch). The building foundation system consists of pile cap supported by two to four groups of concrete friction piles. The columns are founded on the centerline of the pile cap. The caps are connected together using tie beams and grade beams. Each pile is of 600 mm (24 in) diameter and 13 m (40 ft) depth. Each pile was designed to provide vertical capacity of over 445 kN (100 kips) and lateral capacity of 89 kN (20 kips). The site geology consists of fine sand silts and silty fine sand.

Four bays of the perimeter frame, in the north side of the structure, are infilled with brick walls. The brick occupies the openings between the ground and the second floor from the east side of the structure. The infill walls are separated from the surrounding beams and columns by 1 inch thick expansion joint. Although the infill walls are not designed as part of the lateral load resisting system, they appeared to contribute to the lateral resistance of the system based on the damage observations from 1971 San Fernando earthquake and 1994 Northridge earthquake (Islam, 1996).

The internal partitions are of gypsum wallboard on metal studs at 0.4 m (16 inch) centers. Cement plaster is used as for the exterior surfaces of the building and on the stair and elevator bays on the long side of the building. The cement plaster is supported using double 16 gauge metal studs.

3.2 Structural Configuration

The system of the building is a reinforced concrete moment resisting frame with non-ductile detailing. The lateral load is primarily resisted by the flexure and shear yielding of the perimeter column spandrel beam frame members, although the interior flat-slab system and columns contribute to the lateral stiffness. Further, the light frame members supporting the stairways and elevator openings participate in resisting the induced lateral loading (Islam, 1996). Further, the presence of brick infill walls in bays of the longitudinal frame may cause short column effect during seismic motion (Islam, 1996). The gravity loads are resisted by two way action of flat slab floors supported by square columns along the interior frames, and square columns at the exterior frames. The slab is 254 mm (10 in) thick at 2nd floor 215 mm (8.5 in) at the third to seventh floor, and 200 mm (8 in) thick at the roof. The roof is covered by a lightweight concrete topping that vary in thickness between 100 mm (3.75 in) to 200 mm (8 in). Further, penthouse with mechanical equipments covers 10% of roof floor area.

The building is regular in plan and elevation. The columns are oriented to bend around their weak axis while responding to lateral loadings. The spandrel beam dimensions are 400 mm (16 in) wide by 700 mm (30 in) deep at the 2nd floor level, 400 mm (16 in) wide by 575 mm (22.5 in) deep at the 3rd through the 7th floors, and 400 mm (16 in) wide by 560 mm (22 in) deep at the roof. Figure 3.1 demonstrates elevation view of the frame, concrete dimensions, and typical reinforcement detailing of beams and columns. Properties of construction material are presented in Table 3.1. Reinforcement detailing of beam and column elements can be found in Cornell *et al.* (2005).

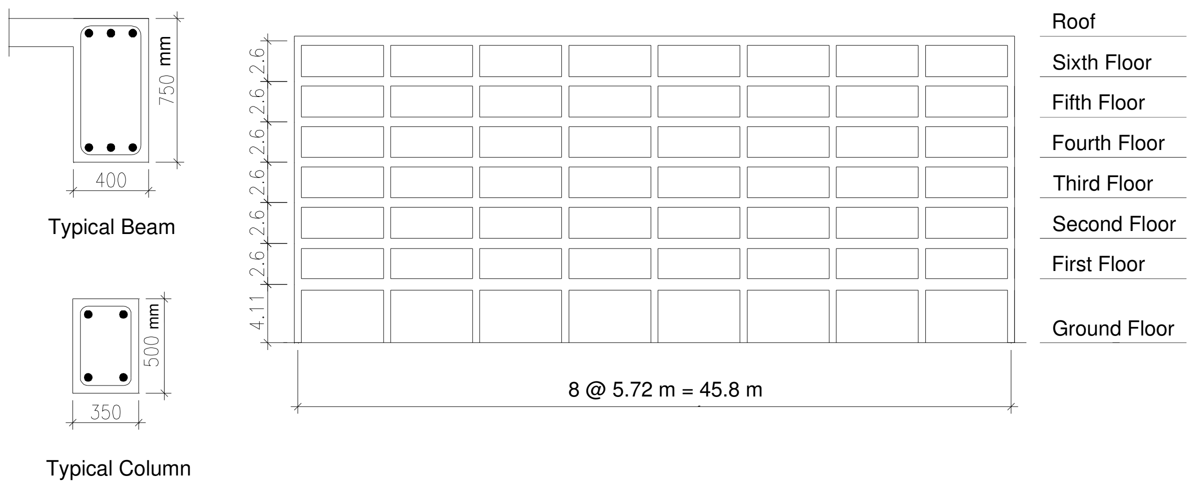


Figure 3.1. Elevation view and typical section detailing of the case study frame

Table 3.1. Properties of construction materials

Material	Element	Floor level	Specified
Concrete	Column	First floor	35 MPa (5 ksi)
	Column	Second floor	28 MPa (4 ksi)
	Column	Third to seventh floor	20 MPa (3 ksi)
	Beam and slab	Second floor	28 MPa (4 ksi)
	Beam and slab	Third to roof	20 MPa (3 ksi)
Steel Reinforcement	Column	All column elements	Grade 60
	Beam and slab	All elements	Grade 40

3.3 Retrofit Strategies

Three retrofit options are considered to upgrade the structural capacity, such as

- Retrofit 1: Addition of Steel Bracings
- Retrofit 2: Addition of Shear walls
- Retrofit 3: Addition of Base Isolators

Each of these retrofit techniques is discussed below.

3.3.1 Retrofit 1: Addition of Steel Bracings

X-steel bracings were implemented to stiffen the structural system and control dynamic response of the structures. The advantages of this retrofit scheme lie in its ability to accommodate frame openings, ease of application, and that steel material inherits high strength-to-weight ratio. Since hollow sections are featured with their effective slenderness ratio and high compressive capacity, square tube steel considered as bracing members. The braces were applied in the middle bay and distributed along the height of the structure. The steel bracing members were designed using AISC-LRFD [2005] code. It was concluded that steel bracings of 203.2 mm × 15.9 mm is required at the ground floor. And, bracing members of 177.8 mm × 15.9 mm is required at the above stories. Figure 3.2 shows elevation view of the frame structure retrofitted with steel bracings.

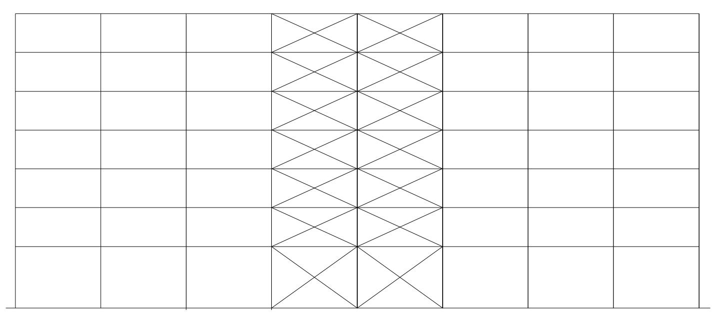
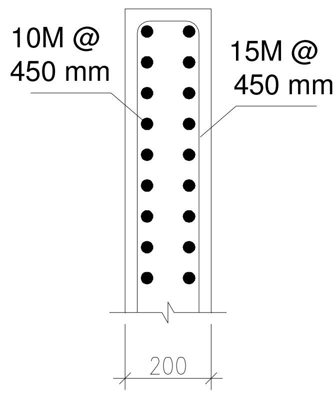


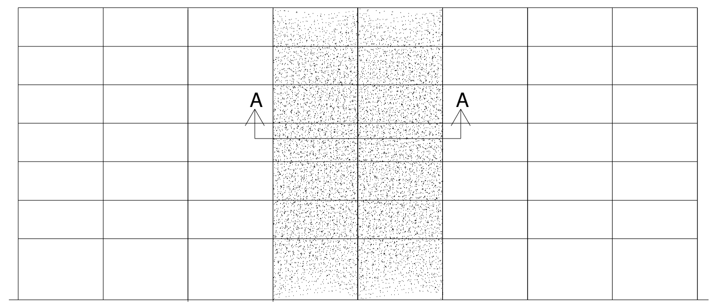
Figure 3.2. Steel bracing retrofitting at the middle bays

3.3.2 Retrofit 2: Addition of Shear walls

The addition of infill wall is a common retrofit method to enhance lateral strength and stiffness of structures. Two shear walls were added to the middle bays of the structure as mitigation measures (Figure 3.3). Design load calculations were determined using The *International Building Code 2003* (IBC 2003). The shear walls were designed according to the standards of ACI-318 (2005). The shear walls are 203 mm (8 in) thick. Two layers of #4 longitudinal reinforcement, and #3 transverse reinforcement placed at 457 mm (18 in) spacing. Figure 3.3a shows detailing of shear wall members, and elevation view of the retrofitted case frame with infill walls.



(a)



(b)

Figure 3.3. Shear wall retrofitting
(a) reinforcement detailing of shear wall section, (b) shear wall added to middle bays

3.3.3 Retrofit 3: Addition of Base Isolators

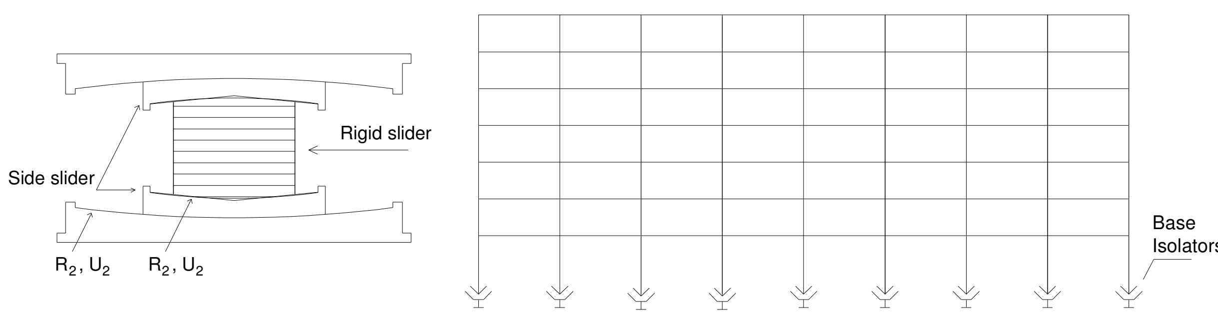
The design of base isolators is iterative procedure, where the response of the facility controls the properties of the isolator devices, which in turn influence the overall performance of the facility. Primary step in designing base isolating device is to determine the maximum displacement to be experienced by the isolator during earthquake. The design displacement of the isolator is essential factor that control the size and consequently the cost of the device. When the design displacement of the isolators is computed the effective damping and stiffness properties of the isolators can be determined. There are two factors involved in determining the displacement demand placed on the isolators: the site specific seismic hazards, and the fundamental period of the isolation bearings. It is noteworthy to mention that performance of base isolator devices is influenced by the sustained axial loading, and the induced earthquake level.

In this study, triple-friction-pendulum devices were selected as the base isolating system. This isolator device has been extensively studied and is proved to be effective for seismic protection (Fenz and Constantinou, 2008). The base-isolated structure was designed according to the performance based design paradigm of ASCE 7-05 code (ASCE, 2005). The target natural period of the isolated building was selected as three times larger than the fundamental period of the fixed base structure, as recommended by ASCE 7-05 (ASCE, 2005).

The isolator devices were selected with theoretical period of 3 sec and displacement limit of 345 mm on the basis of performance and cost, following the work of Zekioglu *et al.* (2009). The properties of the designed isolators are presented in Table 3.2. The main parameters are the effective stiffness, friction coefficient, rate parameter, and radius of sliding surface.

Table 3.2. Properties of base isolator device

Properties	**Value**
Stiffness, U_1 (kN/mm)	0.5
Stiffness, U_2 (kN/mm)	1.45
Friction coefficient, Slow	0.068
Friction coefficient, Fast	0.075
Rate parameter (sec/mm)	0.256
Radius of sliding surface (mm)	750



(a) (b)

Figure 3.4. Base isolation (a) isolator details and material property, (b) isolators added to the base of the frame

3.4 Selection and Scaling of Ground Motion Records

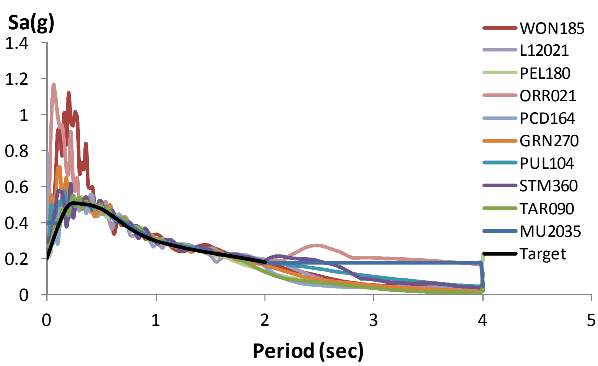
The case study building is located on soil site of class D in the San Fernando Valley and surrounded by variety of active faults such as the San Andreas fault that is located 50 km northeast of the structure. Although the building is located near active faults, none of the faults that dominate the seismic hazard at the site is oriented in a way to generate seismic excitations with near-fault features (Cornell *et al.*, 2005). As so, the use of ground motions that display such characteristics, and use of separate uniform hazard spectrum to capture near-fault rupture directivity effects are not required.

Cornell *et al*. (2005) conducted study to define the parameters by which proper ground motion records can be selected to accompany hazard spectra at the building site, such as expected magnitude, fault distance, and soil condition at the building site. These parameters were used in this study for the selection of ground motion records. This ensures that the suites of ground motion records are compatible with the design response spectrum of the region. Moreover, detailed information on the de-aggregation process of the hazard spectrums and the identification of proper ground motion records to accompany intensity measure is provided by Cornell *et al.* (2005). The de-aggregation process indicated that 1971 San Fernando, 1986 North Palm Springs, 1994 Northridge, and 1997 Whittier Narrows earthquake are suitable for the seismic assessment of the facility. Table 3.3 provides properties of the selected ground motions to carry this study.

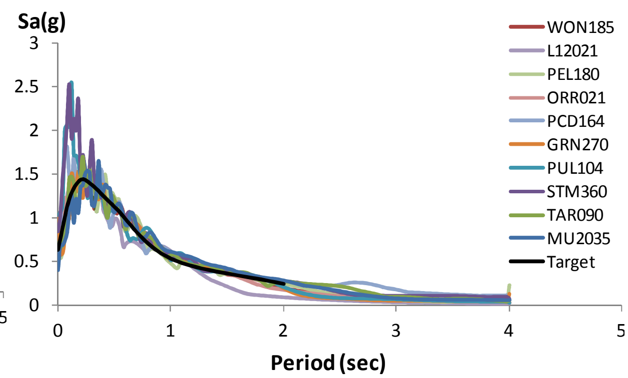
Table 3.3. Properties of ground motion records

Earthquake	**Station**	**Distance (kM)**	**Magnitude (M)**	**Ground Motion**
Northridge 1994/01/17	90014 Beverly Hills - 12520 Mulhol	20.8	6.7	MU2035
	90017 LA - Wonderland Ave	22.7		WON185
	24436 Tarzana, Cedar Hill	17.5		TAR090
	24538 Santa Monica City Hall	27.6		STM360
	24207 Pacoima Dam	8.0		PUL104
	90019 San Gabriel - E. Grand Ave.	41.7		GRN270
San Fernando 1971/02/09	279 Pacoima Dam	2.8	6.6	PCD164
	24278 Castaic - Old Ridge Route	24.9		ORR021
	135 LA - Hollywood Stor Lot	21.2		PEL090
	128 Lake Hughes #12	20.3		L12021

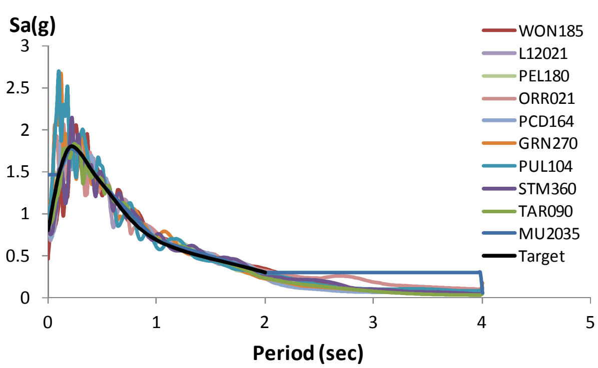
USGS software (2006) was used to generate uniform hazard spectrums at the building site for 50%, 10%, 5%, and 2% probability of exceedance in 50 years return period. Suite of ten accelerograms was used to represent each hazard spectrum. This allows the structure to simulate range of response spectrums, and thus establish statistical sampling of the system level performance. It shall also demonstrate and compare the effectiveness of retrofit measures, to upgrade the structure, under the influence of different hazard levels. SeismoMatch (2010) was used to scale mean value of the representative suite of ground motion records to match the target spectrum of each seismic hazard. The scaling process of the ground motion records is intended to eliminate variability in the records, and the scaled records reflect the inputted seismic energy at the assumed hazard level (Liel, 2008). Spectral acceleration (S_a) at 5% damping level was used as intensity measure. Figure 3.5 presents ground motion records after scaling at each hazard levels.



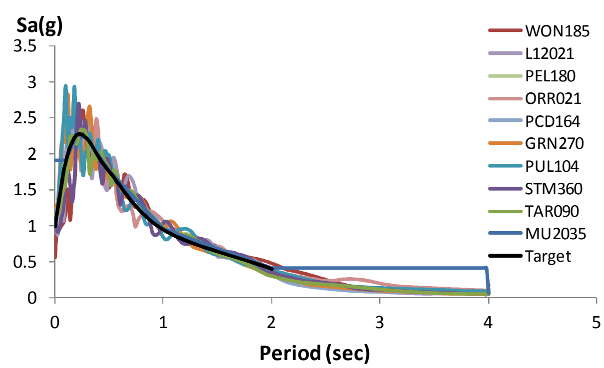
(a) 50% hazard level



(b) 10 % hazard level



(c) 5% hazard level



(d) 2 % hazard level

Figure 3.5. Selected ground motions to represent hazard curves

3.5 Description of Analytical Models for the Case Study Building

Simulation of the structural system to assess its seismic performance requires the development of mathematical models that captures the nonlinear force-deformation properties of the case study building. In this study, mathematical model of the structure was developed utilizing the modelling capabilities of SAP2000 package (2002). SAP2000 is general purpose structural analysis software to perform static and dynamic analyses of structures. The platform can be used to compute mode shapes and periods for any stressed state of the structure. Nonlinear characteristics are represented by assigned plastic hinge at elements end where flexural yielding is expected to occur.

Two dimensional model of the exterior moment resisting frame was created to represent the dynamic characteristics of the structure considering that components of the exterior frames are the primary components in resisting lateral loadings. Beam and column elements were modelled as linear elastic elements. Bracing elements were modelled as linear truss members by releasing elements ends to exhibit no resistance against rotation and bending moment forces. Shear wall elements were modelled as layer shell members. Base isolators were modelled as link elements utilizing "Friction Isolator" model, as provided by the library of SAP2000.

To simulate nonlinear characteristics of the structure, inelastic properties of structural components were lumped to member ends as hinges, following the principle of lumped plasticity approach. Nonlinear force-deformation characteristics of hinges were defined according to FEMA 356 (2000) criteria. The following describes the hinge type used to represent nonlinear properties of the structural elements of the frame.

The flexural characteristics of column member are defined using three-dimensional interaction surface that consists of five equally spaced axial force-bending moment and moment-rotation relationship. As so, there is need to account for the interaction of axial load and bending moment at hinge regions of column. P-M2-M3 hinge type yields based on the interaction of axial force and moment, and thus it was used to represent the inelastic properties of column elements. Conversely, axial load on beam element is assumed to be zero. Thus, moment-rotation relationships of beam element were represented using M3 hinge type, assigned at plastic hinge regions. Further, as behaviour of bracing members is controlled by the application of axial loading, P hinges were used to simulate inelastic properties of bracing elements.

Length of plastic hinge region was taken as half the section depth of the member in the direction of loadings, following the recommendation of ATC-40 (1996). Typical force-displacement relationship of plastic hinges incorporated in SAP2000 is presented in Figure 3.6.

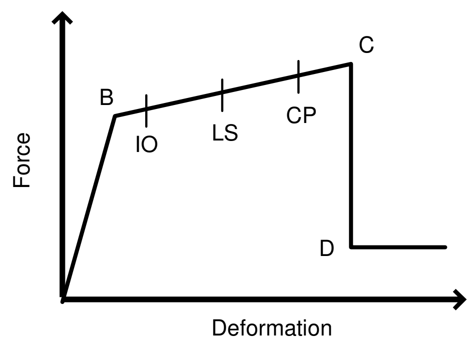


Figure 3.6. Force-deformation response of non-degrading plastic hinge properties (reproduced from FEMA 356 (2000))

Li *et al.* (1998) conducted research work to examine the capability of nonlinear dynamic analysis to predict earthquake incurred damages to RC structures. The case study structure was selected as the Van Nuys hotel building. The modelling of the structure aimed at investigating the influence of varying different structural characteristics of components on the system level performance. The study compared the results for using effective stiffness (cracked section) and non-reduced stiffness (uncracked section) of structural components. The findings suggested that considering the effective or non-reduced stiffness may alter the overall estimated displacement, but pose no effect on the lateral capacity of the structure. The two values also did not influence the plastic hinge formation mechanism. This concludes that stiffness of the structure is not important when the lateral capacity of the system is to be evaluated using push-over analysis. However, it was indicated that stiffness of the structure should be treated (or selected) carefully when the deformation capacity (or level) of the system is of concern. The study also explained that input parameters such as damping ratio, improved material strength since construction, effective stiffness, and residual lateral capacity needed to be carefully considered to obtain response pattern correlated with the observed damage. This is particularly true when nonlinear analysis is considered as evaluation tool. The researchers also illustrated that failure mechanism of columns was governed by the insufficient shear capacity of the columns to develop their flexural capacity while responding to the applied seismic loadings. Most of the column shear failure took place (or occurred) at the fourth floor. This was mainly attributed to the change in column shear reinforcement, which is associated with change in the shear capacity, at the fourth floor level.

Oguz (2005) carried out a study to compare the influence of moment-rotation relationships for default and user-defined hinges on pushover results. Their study aimed to test the contribution of the hinges on 2, 5, 8 and 12 storey RC frames subjected to various lateral load patterns. The comparison illustrated that: (1) user-defined hinges exhibited higher plastic rotation capacity yielding higher displacement capacity as respect to the results when default hinges are used, (2) interaction diagrams for default and user-defined hinges are the same for tensile and low level of compressive force, however substantial inconsistency was observed at high level of compressive axial forces, (3) characteristics of default and user-defined hinges did not impact the estimated base shear capacity. Nevertheless, it was suggested in the study that significant difference in pushover curve for the two types of hinges can be observed in case plastic hinges are widely formed in the columns. It was also suggested that more variation in the pushover curves shall be expected in case three-dimensional model is used to represent the structure. In addition, pushover analysis using default and user-defined hinges showed differences in the formation and pattern of plastic hinges.

Inel *et al.* (2006) conducted analytical work to study the possible difference in pushover curves due to the use of default and user-defined hinge properties. The study was carried on RC frames that consisted of four- and seven-storey structures, and by varying key design parameters (e.g., plastic hinge length) to conclude the most influencing factor on the pushover results. The results illustrated that the differences in the properties of default and user-defined hinges did not significantly contribute to the base shear capacity of structures. The results also show that varying the length of plastic hinge region considerably impact the deformation capacity of the structures. It was also concluded that the pattern of plastic hinge formation is different for default and user-defined hinges at ultimate condition of members. It was observed that models with default hinge properties are characterized with ductile mode of failure, and that failure is restricted to beam elements. The results indicated that user-defined hinges are more successful in capturing hinging mechanism, and yield better estimate of nonlinear response of members with respect to that the use of default hinge model. However, the use of built-in hinge model is preferred due to its simplicity, and may be more compatible for building designed by modern standards. Therefore, it was recommended that, in case default hinge model is used for older designed structure, the user should be aware of the underlying assumptions to avoid misuse of the model.

3.6 Results and Discussion

3.6.1 Eigenvalue analysis

Eigenvalue analysis was conducted to compute the fundamental period of the structure. Table 3.4 shows the fundamental period of the case study structure for the unretrofitted case and after applying three retrofit options.

Table 3.4. Fundamental period for unretrofitted and retrofitted cases

Model	**Fundamental period (sec)**
Unretrofitted case	1.09
Retrofitted with steel bracings	0.44
Retrofitted with shear walls	0.25
Retrofitted with base isolation	3.34

3.6.2 Pushover analysis

Nonlinear static analysis was conducted using inverted triangular load pattern (Li *et al.*, 1998). Intensity of the lateral forces is proportional to the product of mass and first mode shape response of each storey. P-Delta effects were considered during the analysis to account for geometric nonlinearities. Pushover analysis was performed from the end of gravity analysis to consider the effects of gravity loadings. The behaviour of the structure in the gravity analysis was considered to be nonlinear. The results of pushover analysis were expressed in the form of capacity curve. Base shear to building drift relationships for unretrofitted and retrofitted case structures are compared in Figure 3.6. The results highlight the impact of each retrofit scheme on the structural characteristic of the building.

Both shear wall and steel bracing schemes enhance lateral stiffness and strength of the structure. An increase of about 575% in base shear is attained due implementing the intervention methods as compared with unretrofitted case. This suggests effectiveness of the retrofit patterns to relief structural components from sustaining earthquake induced forces, and allows slower strength and stiffness degradation. However, it can be observed that with the use of steel bracings, the structure exhibited higher deformation capacity (higher ductility) as compared with that obtained using shear walls.

In case of base isolated structure, application of the isolators caused the pushover curve to flatten. Also, the increase in fundamental period of the base-isolated structure suggests less seismic demand to be placed on the structure.

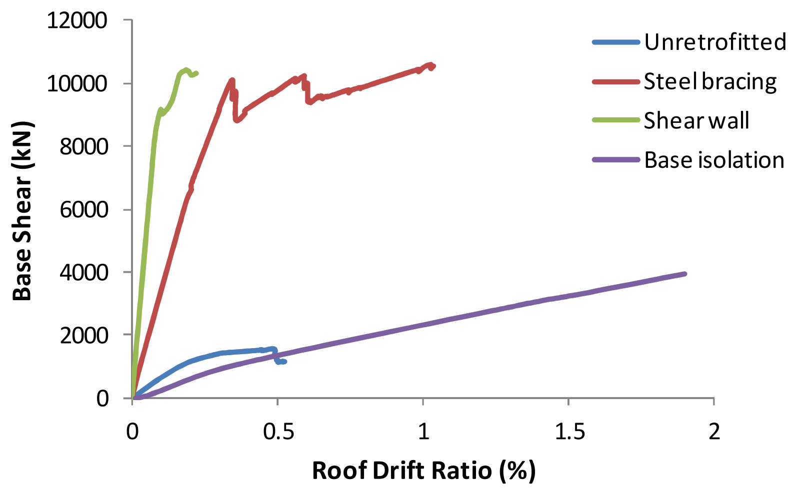


Figure 3.7. Comparison of pushover analysis for unretrofitted and retrofitted cases

3.6.3 Nonlinear time history analysis

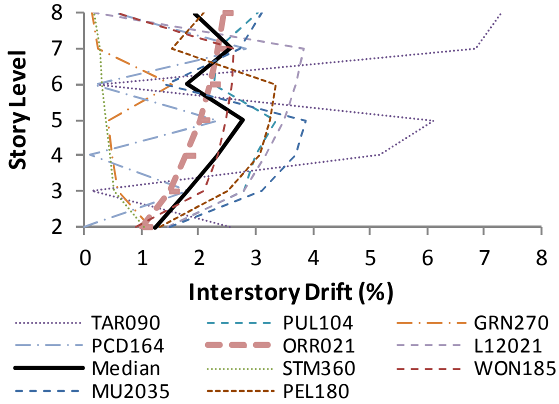
Nonlinear time history analyses were performed to quantify dynamic response parameters of the retrofitted and unretrofitted cases. The modeling of seismic action was performed through incorporating ground motion records to the time domain of structural models to compute nonlinear force-deformation properties, of the structure, at each time increment. SAP2000 platform was employed to carry out the set of nonlinear time history analyses. This analytical procedure accounts for the change in strength and stiffness of structural components during inelastic response. Therefore, it is useful tool to capture the change in dynamic properties of the structure during inelasticity (Saatcioglu and Humar, 2003).

The seismic demand of the retrofitted and unretrofitted structures was quantified using inter-storey drift as performance indicator. This performance indicator accounts for flexural demand, or amount of rotation, placed on the columns (Ghobarah, 2000). It also provides indication on shear distortion and mode response of the floor system. The inter-storey drift is defined as the relative displacement with adjacent storey divided by the storey height. This approach may not be suitable to investigate member level performance. However, it provides assessment on the overall performance of the system under given seismic demand.

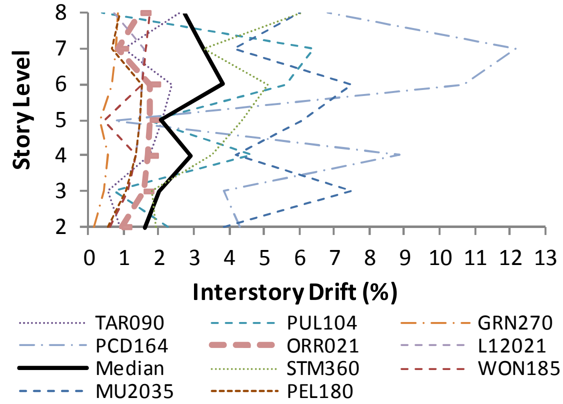
As discussed in section 2, vulnerability assessment of the non-ductile case study building is carried according to the Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356). Evaluation criteria of FEMA 356 are 1%, 2%, and 4% inter-storey drift limits for Immediate Occupancy (IO), Life-Safety (LS), and Collapse Prevention (CP) performance levels, respectively.

3.6.3.1 Unretrofitted case

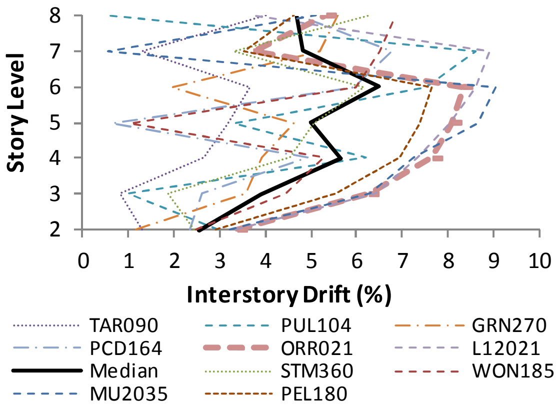
Existing case of the structure was subjected to the representative set of ground motion records at each seismic hazard. The computed maximum median inter-storey drift values were compared with the criteria of FEMA 356 (2000) for the seismic evaluation. Figure 3.8 demonstrates the inter-story drifts corresponding to the 50%, 10%, 5%, and 2% hazard levels.



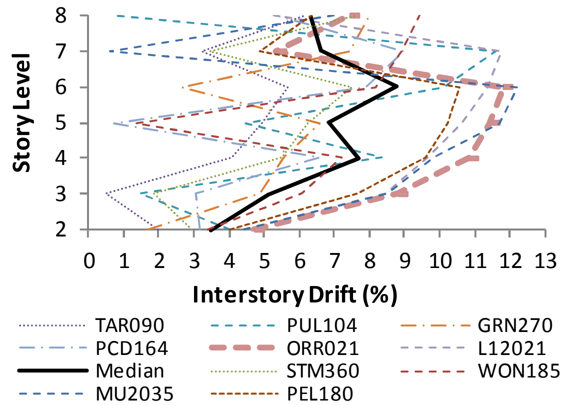
50% hazard level



10% hazard level



5% hazard level



2% hazard level

Figure 3.8. Inter-storey drift for unretrofitted case

The results indicate that the maximum median inter-storey drift values exceed the Basic Safety Objective (BSO) requirements of FEMA 356 (2000) for 10% and 2% in 50 years seismic hazards. Therefore, alternate seismic retrofit method is considered to reduce the seismic vulnerability of the structure. This shall also demonstrate the application of each retrofit option in protecting the structure against potential seismic threat.

3.6.3.2 Steel bracings

The maximum median inter-storey drift corresponding to the use of steel bracings is presented in Figure 3.9. The findings demonstrate that utilizing steel bracings enhanced the lateral stiffness of the structure. As for 2% earthquake hazard, the existing and steel braced frame structure experienced maximum averaged inter-storey drift ratio of 8.7 and 2.5, respectively. This indicates an increase in the lateral stiffness of the structure of about 250%

attributed to the installation of steel bracings. It should be noted that the minimum requirements dominated the design of the steel bracing scheme.

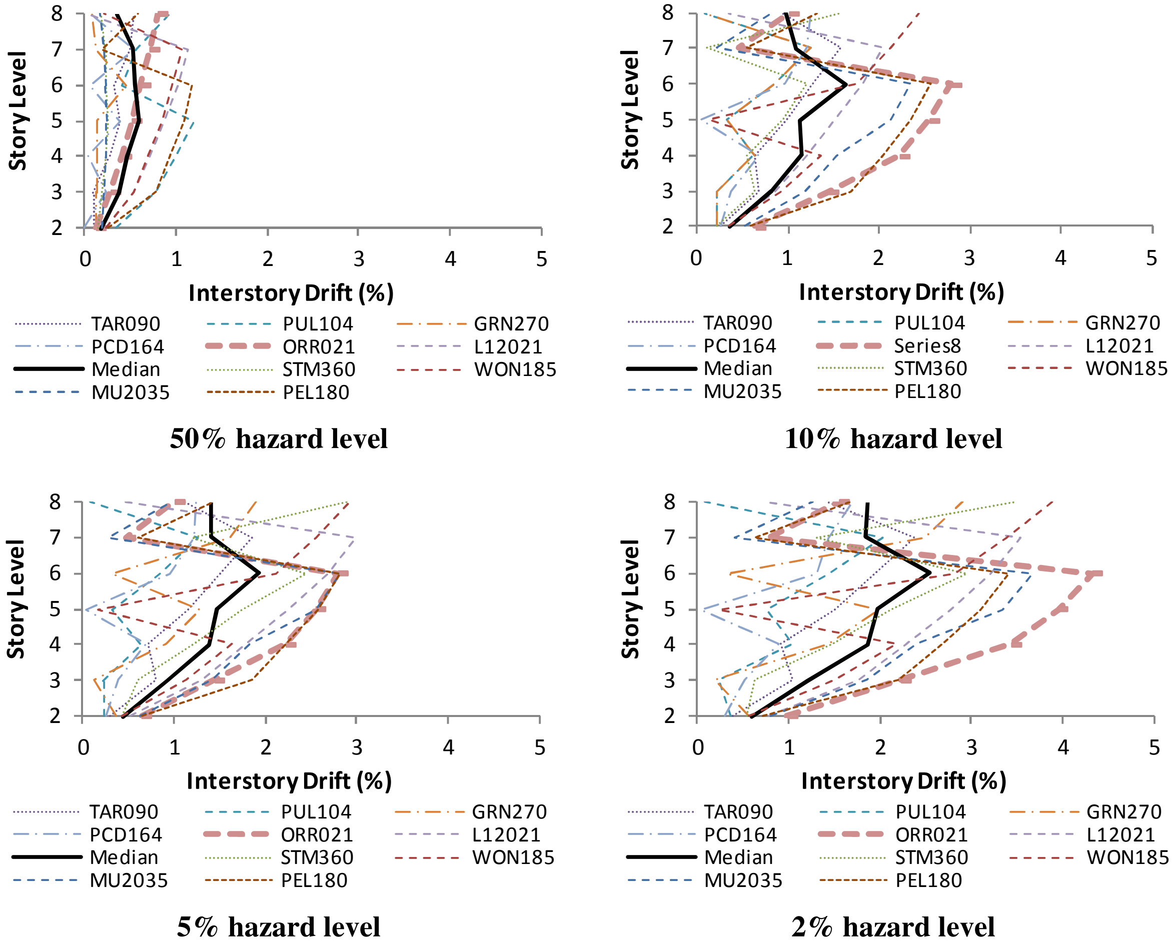


Figure 3.9. Inter-storey drift for retrofitted case with steel bracing

The results also illustrate change in the lateral load deflection pattern due to the implementation of steel bracings. This is well observed using the inter-storey drift performance indicator as it provides measure over the seismic demand and response sustained by each storey. In other words, the reduction in the inter-storey drift for steel braced system reveals suitability of the intervention method to control flexural demand placed on the columns of storey.

3.6.3.3 Shear walls

Infill walls were installed to enhance the overall strength and stiffness of the structural system. Figure 3.10 illustrates effectiveness of this retrofit to control storey drift demand, and upgrade the lateral stiffness of the structure.

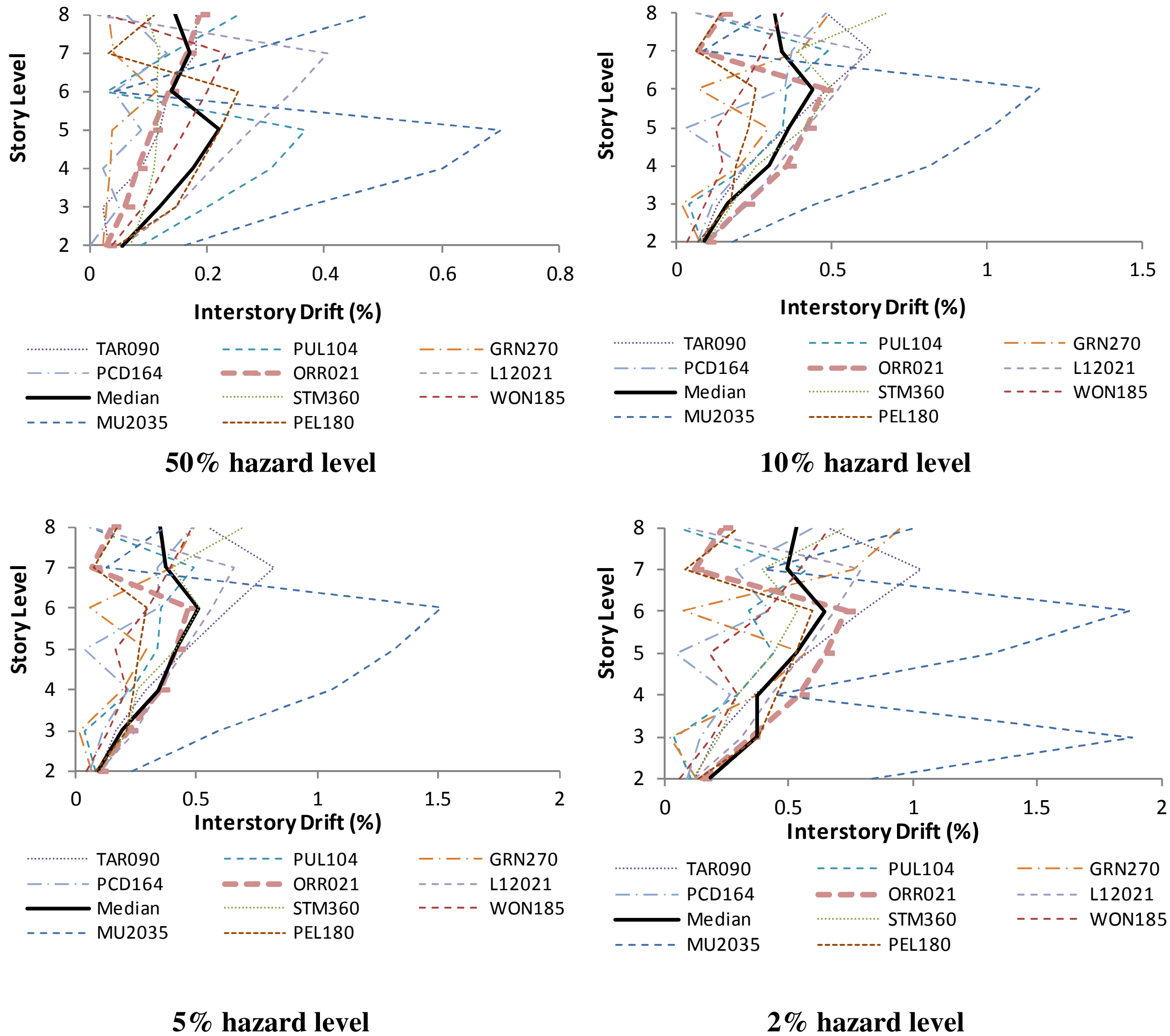


Figure 3.10. Inter-storey drift for retrofitted case with shear wall

The results reveal that, at 2% seismic hazard, an increase of 93% in the lateral stiffness of the structure is attained due the introduction of shear walls with relative to the existing case of the frame structure. It also should be noted that the drift of lower stories were significantly reduced as compared with the unretrofitted case. This is in addition to the improvement in the overall profile of the structure.

3.6.3.4 Base isolation

Figure 3.11 illustrates the decrease in the overall drift attributed to the use of base isolating device as compared with the exiting case of the structure. The results indicate substantial decrease in the overall building response. The maximum building drift reduced by about 66% due the introduction of base isolators. This reduction highlights capability of base isolator devices to decrease seismic design forces, as well as earthquake-induced damages on structural and non-structural components. This suggests smaller member sizes are required for base-isolated structure, indicating cost-saving benefits associated with utilizing base isolation measure. This is in addition to savings achieved due protecting non-structural components from seismic-induced damages. Further, as isolator devices are effective to reduce the inputted seismic energy and subsequent damages, base-isolated buildings shall require less maintenance time, thereby minimize cost due to downtime of the building function.

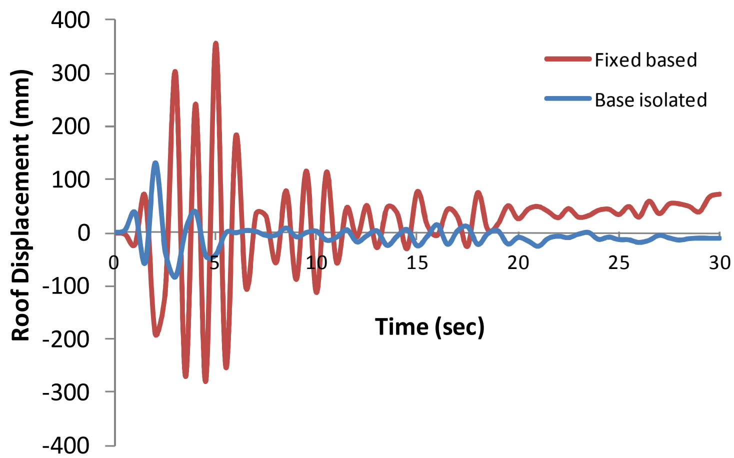


Figure 3.11. Comparison of roof displacement for base-isolated and fixed base model

Results of nonlinear time history analyses demonstrate effectiveness of base isolators to reduce and attain uniform lateral deflection pattern (Figure 3.12). The reduced inter-story drift demand indicate the possibility to conclude simplified envelop detailing of the structure. This reduction in storey response suggests decrease in the shear forces placed on the stories, and consequently less flexural demand and damages to the columns. It also indicates that base-isolated structure is less vulnerable to stability problems related to P-Δ effect. This is in addition to that, in base-isolated models, nonlinearity is restricted to the base isolators, whereas the superstructure remains elastic.

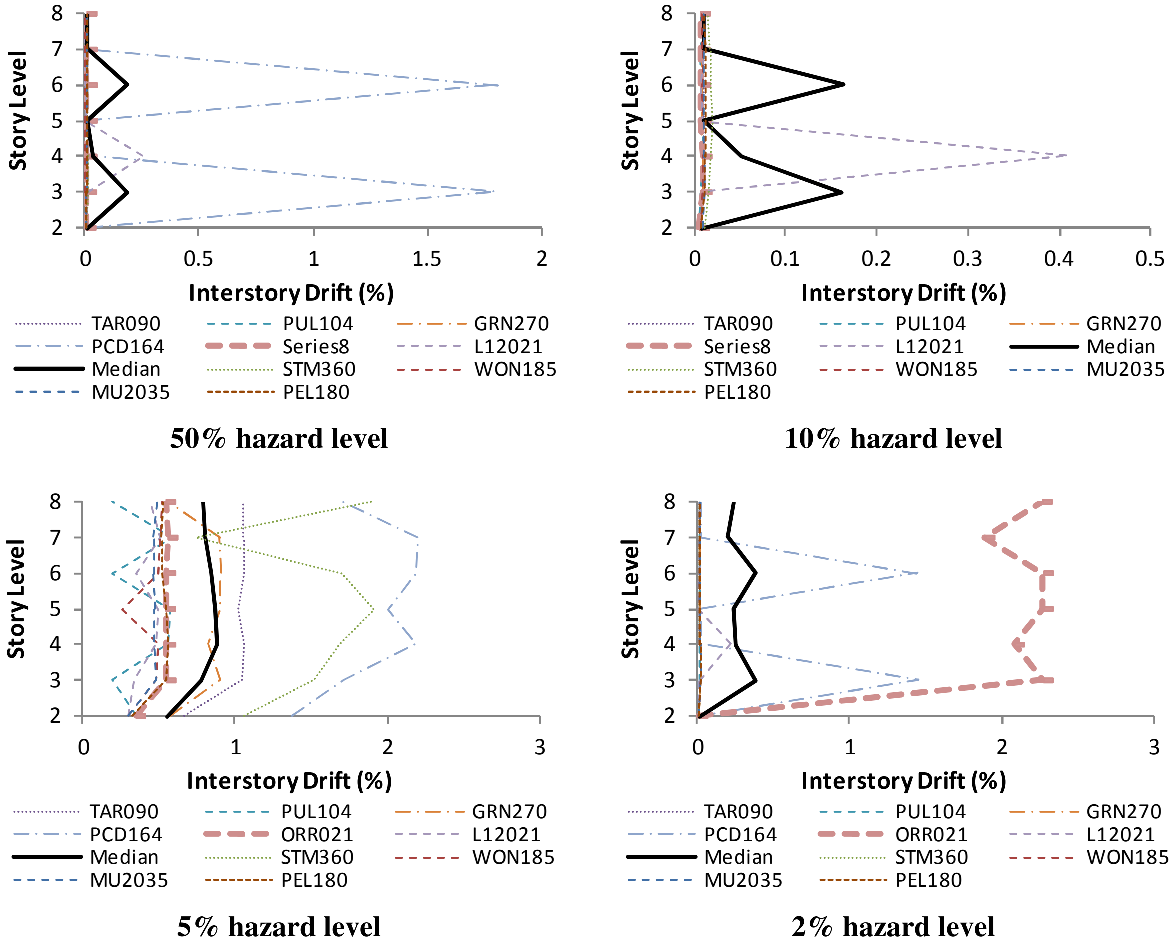


Figure 3.12. Inter-storey drift for retrofitted case with base isolation

3.6.4 Seismic Fragility Assessment

Vulnerability functions are required to assess direct physical damage to facility during earthquake. Seismic fragility functions describe the cumulative probability of being in or exceeding specified damage state for a given shaking intensity. The fragility functions can be developed using analytical methods. The analytical procedures may rely on intensive computations involving series of nonlinear time history analyses to evaluate structural characteristics of the facility.

The seismic vulnerability of facility can be evaluated by establishing the pushover curve of the structure. The curve is then transformed into spectral displacement-spectral acceleration to obtain the so-called capacity spectrum curve. The capacity spectrum curve is compared with seismic demand spectra to determine the performance point. Performance point is defined by the intersection of capacity spectrum with demand spectrum, and represents the expected displacement demand of the structure for a given ground motion. The performance point is used as input in fragility function to determine the probability of exceeding number of limit states of the structure.

Vulnerability function refers to damage state, such as slight, moderate, extensive and complete. Each function is characterized by median and lognormal standard deviation, which accounts for the uncertainties related to the estimation of building response and seismic demand. The capacity spectrum method was adopted by several earthquake loss assessment methodologies, such as HAZUS (FEMA, 2000), EQRM (Robinson *et al.,* 2005), and ELER (Demircioglu *et al.,* 2009).

Limit state probability $P_t[LS]$ is defined as conditional probability of reaching given limit states at a given location or period of time. The conditional probability is calculated as follows (Wen *et al.,* 2003):

$$P_t[LS] = \sum P[LS|D = d]P[D = d] \tag{3.1}$$

where $P_t[LS]$ is probability of reaching a specified limit state over a given period of time (0,*t*), D is spectrum of uncertain hazards, d occurrence of predefined earthquake level, $P[LS|D = d]$ is conditional limit state probability is given D=d for all values of D, and $P[D = d]$ defined the hazard in terms of cumulative distribution function.

Many research studies were conducted regarding seismic vulnerability and development of fragility functions. Cornell *et al.* (2002) proposed probabilistic framework for seismic design and assessment of steel moment resisting structure for the guidelines of FEMA. Demand and capacity were expressed in term of maximum inter-storey drift parameter using nonlinear dynamic relationships. The framework was developed with assuming that the parameters are distributed in closed forms. In addition, probabilistic models were employed to account for uncertainties in structural demand and capacity.

Hassan and Sozen (1997) investigated the seismic vulnerability of low-rise structures with and without masonry infills damaged by the 1992 Erzincan earthquake in Turkey. Gulkan and Sozen (1999) proposed methodology to identify higher seismic vulnerability constructions based on wall and column indices. Dumova-Jovanoska (2000) derived fragility relations for 6- and 16- storey RC buildings located in Skopje, Macedonia using 240 ground motion data. Shama *et al.* (2002) conducted seismic vulnerability assessment of bridges supported by steel piles. The objective of the study is to investigate the effectiveness of retrofitting to reduce the seismic vulnerability of the structure. Bai and Hueste (2007) conducted study to illustrate the effectiveness of reducing seismic vulnerability of five storey RC building using alternate retrofit patterns.

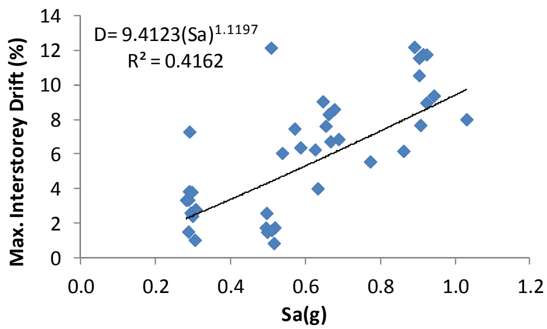
Fragility curves are intended to relate probability of exceeding stated performance level to earthquake intensities. In this study, the models were developed to assess the effectiveness of adopting retrofit option to reduce the probability of exceeding certain damage state. Thereby, it is an effective approach to compare the capability of each mitigation measure to reduce damage. The fragility functions were developed using several parameters including earthquake intensity, structural response characteristics, and demand and capacity uncertainties. The seismic demand was quantified in term of inter-storey drift values, as obtained using series of nonlinear time history analyses. The following equation was used to develop fragility curves, which assumes demand and capacity to be lognormally distributed (Wen *et al.,* 2004)

$$P(LS|S_a) = 1 - \Phi\left[\frac{\lambda_{CL} - \lambda_{D|S_a}}{\sqrt{{\beta_{D|S_a}}^2 + {\beta_{CL}}^2 + {\beta_M}^2}}\right] \qquad [3.2]$$

where $P(LS|S_a)$ is probability of exceeding damage state for a given earthquake return period; Φ is standard normal cumulative distribution function; λ_{CL} is lognormal of median drift capacity for a particular limit state, where drift capacity is expressed in term of percentage of storey height; $\lambda_{D|Sa}$ is lognormal of median drift demand for a given earthquake intensity, where drift demand is computed using fitted power law equation; $\beta_{(D|Sa)}$ is uncertainty associated with the fitted power law equation; β_{CL} uncertainty related to drift capacity criteria, considered as 0.3 based on the work of Wen *et al.* (2004), and β_M uncertainty related to analytical modelling, considered as 0.3 in this study based on Wen *et al.* (2004) work. The equation assumes demand and capacity to be lognormally distributed.

Based on results of time history analyses, sets of fragility curves were developed using FEMA 356 global level performance criteria. The median drift capacity λ_{CL} parameter was computed as lognormal of the drift limit for IO, LS, and CP performance level. The drift demand $\lambda_{D|Sa}$ value is the lognormal of maximum inter-storey drift experienced by the structure when subjected to certain ground motion.

The unretrofitted case study building is considered to demonstrate the construction process of fragility curves, as summarized by Figure 3.13. Figure 3.13 demonstrated the relationship between the spectral acceleration and the corresponding maximum inter-storey drift for 2, 5, 10 and 50% in 50 years seismic hazards. The spectral acceleration values are the values obtained by scaling each representative suite of ground motion records to match the target spectrum of each seismic hazard. The graph also shows the fitted power law equation. The value of S^2 for unretrofitted case is 0.125 which yields $\beta_{D|S_a}$ value of 0.344. The developed fragility curves for the three performance levels including Immediate Occupancy (IO), Life Safety (LS), and Collapse Prevention (CP) for unretrofitted case.



Damage State	FEMA 356 limits
Immediate Occupancy (IO)	1%
Life Safety (LS)	2%
Collapse Prevention (CP)	4%

λ_{CL}

$$P(LS|S_a) = 1 - \Phi\left[\frac{\lambda_{CL} - \lambda_{D|S_a}}{\sqrt{\beta_{D|S_a}{}^2 + \beta_{CL}{}^2 + \beta_M{}^2}}\right]$$

$\lambda_{D|Sa}$

$\beta_{D|S_a}$

$\beta_{CL} = 0.3$

$\beta_M = 0.3$

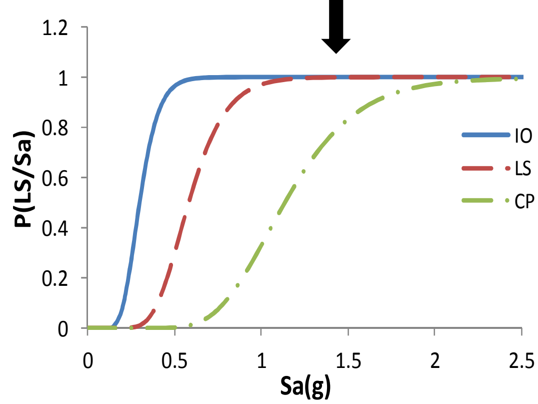
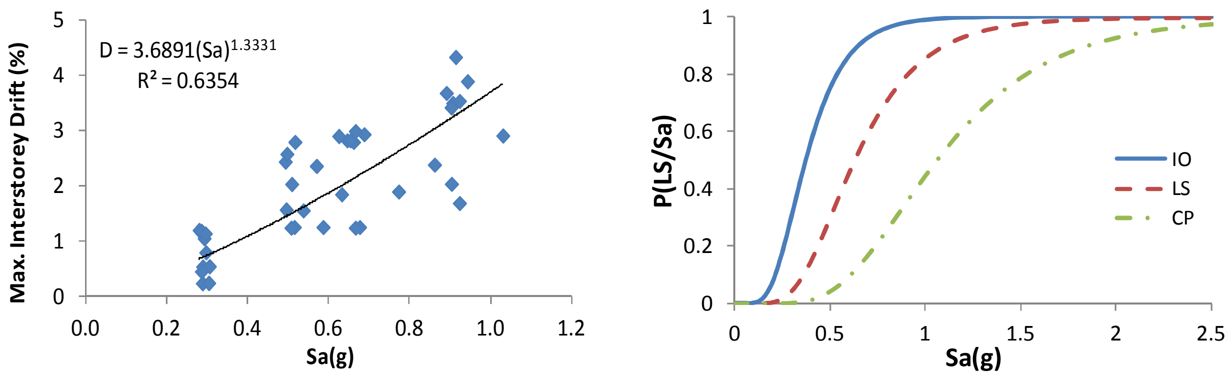


Figure 3.13. Development of fragility model for unretrofitted structure

In similar manner, spectral acceleration value of the scaled ground motions, corresponding to each seismic hazard, was used to derive relationship between demand and structural response (i.e., maximum inter-storey drift). These relations were then used to compute parameters for the development of fragility models. Value of the computed parameters is provided in Table 3.5. Fitted power law equation and derived fragility functions that describe vulnerability of retrofitted cases are shown in Figures 3.14, 3.15, and 3.16.

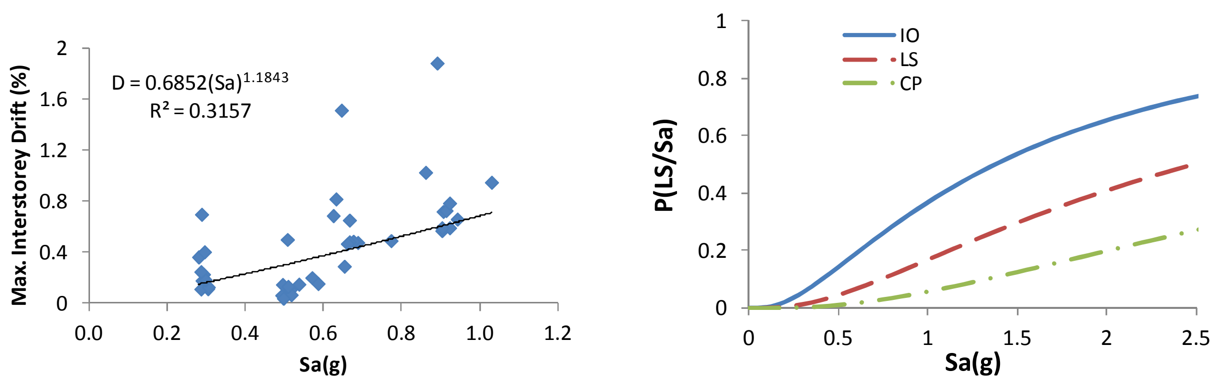
Table 3.5. Parameters used to develop fragility relationships for retrofitted cases

Model	S^2	$\beta_{D\|S_a}$	β_{CL}	β_M
Retrofit 1: Addition of steel bracings	0.514	0.644	0.3	0.3
Retrofit 2: Addition of shear walls	0.934	0.812	0.3	0.3
Retrofit 3: Installation of base isolators	0.536	0.655	0.3	0.3



(a) **(b)**

Figure 3.14. Steel bracing retrofitting a) probabilistic seismic demand model, and b) seismic fragility curve



(a) **(b)**

Figure 3.15. Shear wall retrofitting, a) probabilistic seismic demand model, and b) seismic fragility curve

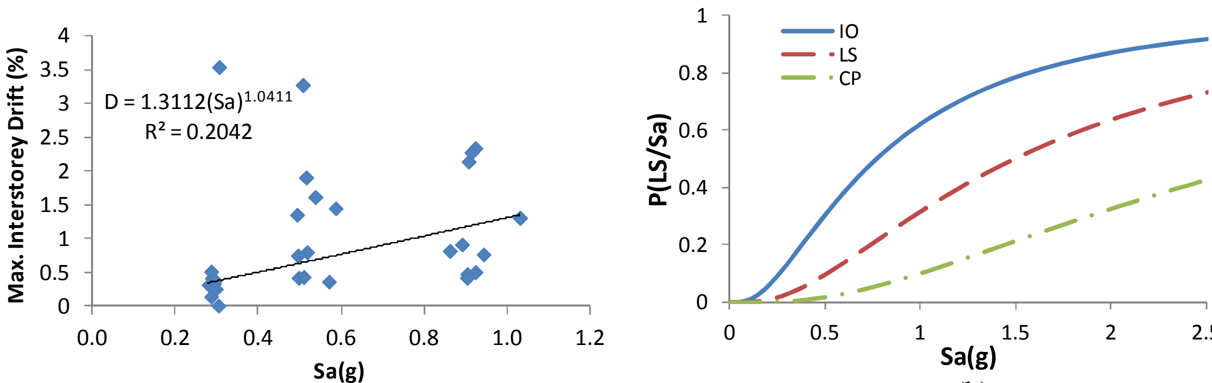


Figure 3.16. Base isolation retrofitting, a) probabilistic seismic demand model, and b) seismic fragility curve

The drift limits for IO, LS, and CP performance levels were defined based on the global level criteria of FEMA 356 as 1%, 2%, and 4%, respectively. Based on fragility models for retrofitted cases, the selected retrofit options were effective to reduce the probability of exceeding each limit state with relative to unretrofitted case. It is noteworthy to mention that spectral acceleration of interest may vary before and after retrofitting the structure, thus direct comparison for specific spectral acceleration may not be suitable.

3.7 Summary

This section described the seismic fragility assessment of a typical 1960's RC building. Results from nonlinear time history analyses and FEMA 356 performance criteria were used to generate the fragility curves. The study indicated that the structure is seismically inadequate as per FEMA 356 standards. Therefore, three intervention methods were considered: i) addition of steel bracings, ii) installation of infill shear walls, and iii) implementation of base isolation. Application of the retrofits was examined using series of nonlinear time history analyses and nonlinear static analysis. Based on analytical results, fragility relations were derived to compare the enhanced reliability of the non-ductile building due to rehabilitation. The results highlighted effectiveness of the selected intervention method to improve the seismic behaviour of the structure, reduce drift demand on existing components, and control earthquake incurred damages. In the next chapter, fragility information will serve as inputs to evaluate efficiency of seismic strengthening in dollar terms. Further, a decision tree based retrofit selection will be discussed.

CHAPTER 4 : DECISION ANALYSIS FOR RETROFIT SELECTION

4.1 Introduction

The previous chapter evaluated effectiveness of retrofit measures to upgrade seismic performance of non-ductile RC structure, typical of 1960s construction. In this chapter, the performance assessment is extended to include additional response metrics in term of economic losses. Understanding susceptibility of facility for financial losses shall help owner with deciding whether to design the new structure beyond code standards, as well as provide justifiable inducement to invest in seismic rehabilitation of existing non-code confirming facilities. At political level, life safety remains the most important incentive for upgrading seismically weak structures, but if rehabilitation can play effective role to reduce financial losses, it would be wise to treat deficient structures.

Prediction of earthquake-related losses relies on calculating engineering demand parameters (e.g., drift, acceleration) to derive damage measures of structural components, non-structural components, and building content (Liel, 2008). Based on the damage state of the building, direct financial losses, such as repair and replacement costs, and indirect financial losses, such as business interruption, can be computed. Seismic-induced motion, such as induced drift or accelerations, is cause of damage during earthquake. Fragility functions link structural response parameters to damage state, and thus the functions are used to quantify damage level of component and/or facility of interest. Once damage measures are estimated, subsequent economic losses can be determined.

In this chapter, analysis process of economic losses utilizing fragility information is detailed. The intent is to provide probabilistic measures of expected economic losses for the case study building when subjected to earthquakes, and quantify significance of the retrofit measures in term of controlling earthquake-related losses. This study utilizes framework of HAZUS (FEMA, 2003) to perform the financial loss analysis. Decision tree tool was implemented to present transparent comparison on the efficiency of each retrofit to limit financial losses due to physical damages.

4.2 Decision Tree

Investors, owners, and engineers are faced with the decision of which is the most effective retrofit strategy to reduce potential seismic threat, and subsequent damages and economic losses to a community. The decision of managing seismic risk through retrofitting is considered to be problematic due to the various factors involved, and that affect the consequences of the decision. These factors include, type of ownership (i.e. public or private), type of the facility, expected earthquake level, desired performance target, economic consideration, and perceived benefits obtained from seismic strengthening. As so, it is essential to implement effective assessment approach to establish informed decision on whether considered intervention method is advantageous or appropriate for a facility.

Decision tree tool is an effective approach to aid with the decision making process. The tool is similar to flow chart that braches out like tree. It consists of parent nodes, branches, and leaves (child node) to which decisions are assigned Bilen and Buyuklu (2006). It is a useful mean to compare between alternate decision options. The advantage of utilizing such tool lies in the graphical visualization ability to present and select among the considered options. The selection criterion can be defined in term of cost or any other parameter that is of interest to stakeholder and professionals not related to the technical field. This allows decision makers to conclude transparent and justifiable decision on which is the most contributing factor.

The decision tree proceeds in chronological order from left to right, such that earlier events/decisions are followed by later events/decisions. The branches of decision tree are composed of two types of forks. The first type is referred to as "Decision fork" from which decision options are generated. The number of branches generated from decision fork depends on the number of decision alternatives. The second fork type is called "Chance fork" and represents the events that can take place as a consequence of selecting a decision option. In other words, the objective of "Chance fork" is to evaluate and compare the suitability of each decision alternative. Typical components of decision tree tool are presented in Figure 4.1.

The branches of decision tree are composed of two types of forks. The first type is referred to as "Decision fork" from which decision options are generated. The number of branches generated from decision fork depends on the number of action required by the decision making process. The second fork type is called "Chance fork" and represents the events that can take place due to selecting decision option.

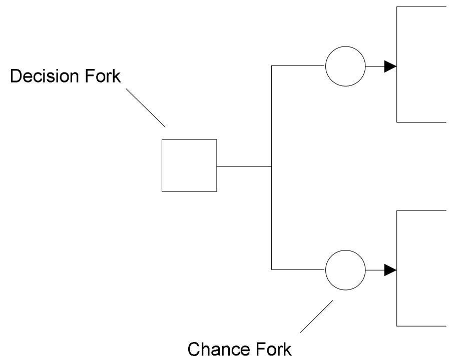


Figure 4.1. Components of decision tree tool

As illustrated by Figure 4.1, each decision fork yields an event fork. At the end of every branch, there will be corresponding consequences and probabilities. In addition, the considered events of each chance fork must be mutually exclusive and the sum of their probabilities needs to be equal to one.

Matin (2006) implemented the concept of decision tree to prioritize the selection of three different retrofit strategies for selected case study structure. The selection criterion was expressed in term of cost, thus the outcomes of the study highlighted the cost saving attained due to implementing mitigation strategy. Sengezer *et al.* (2008) utilized decision tree to determine the most controlling factor to building damage during earthquake. von Winterfeldt *et al.* (2000), Kappos and Dimitrakopoulos (2008), and Al-Chatti *et al.* (2011) implemented decision tree model to facilitate the decision making process among several alternatives to improve seismic safety of structures. Alesch *et al.* (2003), Park (2004), and Al-Chatti and Tesfamariam (2012) proposed decision support platform, involving decision tree analysis, to aid with the comparison of seismic consequences of alternative rehabilitation schemes.

4.3 Methodology for Estimating Financial Losses

Prediction of earthquake-induced economic losses is an alternative measure of building performance, and intended to provide more reliable assessment for rational decision making about the risk management approach (Krawinkler and Miranda 2004). Economic losses communicate incurred damage to building contents, repairs required in structural and non-structural components, and downtime; as well as, reflect seismic vulnerability of design and construction practice of facility in term of parameters that are of interest to stakeholders and decision makers not related to the technical field. Several studies were concerned with developing regional loss estimation methodologies, such as ATC-13 (ATC, 1985) and HAZUS (FEMA, 2003).

Kutsu *et al.* (1982) executed one of the first building-specific loss estimation methodologies. The researchers reported and utilized laboratory test data to assess damage of high-rise building components through implementing a proposed component based methodology. The researchers used the collected data to statistically predict the vulnerability of a component to exceed particular damage states. The tested components include, reinforced concrete members, steel frames, masonry walls, partitions and glazing. The researchers also collected and used published building cost data to statically determine construction cost of the examined components. The estimated costs were used in combination with the developed damage relations to measure damage factor of the components. The damage factors were considered as percentage of the replacement value of the component. The proposed relationships were used by Scholl *et al.* (1982) to construct theoretical motion-damage relation of structural and non-structural components.

Scholl *et al.* (1982) demonstrated the development of component damage relations (i.e. component fragility functions) with utilizing experimental test data in order to measured damage on component by component basis. Application of the proposed methodology was illustrated by examining three buildings that sustained damages during 1971 San Fernando earthquake event. The performance of the facilities was quantified using rudimentary elastic analyses in combination with response spectrum analysis. The analytical results were used to derive theoretical motion-damage relations. The derived relationships computed damage using damage factor. The damage factor was considered as the ratio between repair cost due earthquake damage, and replacement value of the structure. However, the developed relations were limited as the executed analyses do not capture higher mode effects and nonlinear response of the facilities. The researchers also recommended the improvements of both empirical and theoretical loss estimation procedure.

Gunturi and Shah (1993) illustrated the process of computing monetary losses using structural response parameters. The parameters were derived by nonlinear time history analyses. The used ground motions were scaled to peak ground acceleration (PGA) levels of

0.4g, 0.5g, and 0.6g. Damage to building components was categorized as structural and non-structural damages. An energy based damage index proposed by Park and Ang (1985) was used to measure damage of structural components, while inter-storey drift and acceleration parameters were used to assess damage of non-structural components. The damage indices were linked to monetary losses using probabilistic approach. The approach relies on data from expert opinions. The study also investigated variation of damage levels due to the used of different ground motion records.

Singhal and Kiremidjian (1996) proposed systematic approach to derive motion-damage relationships of a structure subjected to suite of artificial ground motions with wide range of parameter variations. The structural analysis stage was carried using DRAIN-2DX. The objective of the work was to address the effect of ground motions variability on the economic loss estimation process of building. Monte Carlo simulation was used to account for the variability in structural parameters. The probability of exceeding damage states was measured using building level fragility functions and Damage Probability Matrices (DPM). The damage states were defined as ratio between repair cost over replacement cost of the building. For fragility functions, ground motion records were characterized using root mean square acceleration and spectral acceleration. The proposed approach was implemented to compute damage measures for low-, mid-, and high-rise reinforced concrete structure. The study was limited to damage measures of structural components.

Porter and Kiremidjian (2001) proposed assembly based framework. The framework accounts for uncertainties related to damage and repair costs. Monte Carlo simulation was used to generate vulnerability functions, which relate expected losses to seismic intensity. The approach was adopted to analysis fragility of office building. The structure was analyzed using linear and nonlinear dynamic analyses. The researchers also considered performing sensitivity analysis to test the influence of different uncertainty sources on the estimated losses. It was found that uncertainties related to ground motion intensities is the most influential factor on the loss results.

Recent studies aimed at utilizing the PEER framework for performance-based earthquake engineering to establish methods and database of building-specific loss estimation, such as Porter (2002), Aslani (2005) and Mitrani-Reiser (2007). Aslani and Miranda (2005), as part of the Pacific Earthquake Engineering Research (PEER) center's effort toward developing performance-based assessment methods, introduced a component based methodology that is capable of capturing the effect of collapse on monetary losses. This was achieved by an explicit consideration of collapse probability at increasing level of ground motion intensities. Two collapse mechanisms were integrated into the framework including, side way collapse and loss of vertical load carrying capacity. The researchers also proposed techniques to disaggregate and determine the most contributing factor on building losses.

Zareian and Krawinkler (2006) proposed simplified version of PEER's framework. The study involved the use semi-graphical approach to assess building losses. The approach calculates losses by grouping components into subsystems, instead of component by component basis. Thereby, components related to single subsystem are represented using single a structural response parameter. The proposed framework is considered easy to work with and simple. However, the application of the framework by the researchers involved many assumptions related to structural response and consequential economic losses due to lack of damage estimation and loss data.

Mitrani-Reiser and Beck (2007) developed computer software called MATLAB Damage and Loss Analysis that implement the PEER loss estimation framework. The methodology was applied on 4-storey reinforced concrete moment resisting frame office building. The researchers also addressed the influence of different structural and modelling parameters on monetary losses. This was achieved by computing mean losses as a function of ground motion intensity level, and expected annual losses were computed for multiple design variants. Losses related to non-collapse were estimated on a component by component basis.

Ramirez and Miranda (2009) proposed storey based loss estimation in an attempt to expand and simplify PEER framework for engineers to perform loss estimation in practice. The proposed approach explicitly account for losses due collapse and demolishing of structure after earthquake. Applicability of the proposed methodology was examined considering four case study buildings. The results indicated that demolishing cost was significant for 4- and 12- storey ductile reinforced concrete moment resisting frames. Thereby, it was indicated that current loss estimation methodologies may underestimate loss results by not accounting for the effect of permanent displacement and consequential damage in structures. It was also highlighted the use of re-centering devices in structure may substantially reduce cost losses.

Al-Chatti and Tesfamariam (2012) presented simplified version of PEER's framework to carry loss estimation studies in a computationally efficient manner. The methodology relies on converting damage measures into monetary losses using inventory losses and economic data provided in HAZUS-MH (2003) manual. Applicability of the methodology was demonstrated by managing susceptibility of non-ductile building to financial losses due to earthquake through seismic rehabilitation. Direct relationships between structural response parameters and seismic-induced economic losses were established by constructing fragility models. The findings highlighted efficiency of the proposed methodology to examine and compare suitability of alternative mitigation strategy to reduce earthquake-induced financial losses of non-code conforming facilities.

4.4 Financial Losses

In this section, conversion of physical damage into monetary losses is discussed. The loss estimation process considers cost to repair structural and non-structural components, and cost due to damage of building contents and business interruption. Downtime of building function impact financial resources of a community in variety of ways based on the occupancy class, such as job and accommodation losses. These consequential losses were as well accounted for in this study.

Economic losses are estimated using building damage measures from physical damage module (i.e. fragility function). The measures are expressed in the form of probabilities of exceeding a damage state. In this study, the probabilities are converted into monetary losses using inventory losses and economic data provided in HAZUS (FEMA, 2003).

HAZUS (FEMA, 2003) estimates economic losses using three methods of different accuracy level. The first method lies on implementing data from national database (i.e., demographic data, and building stock estimates) provided in HAZUS (FEMA, 2003) manual. This method provides rough estimates of the losses. The second method is more accurate and based on professional and expert judgment that involve utilizing detailed information on demographic data, buildings and infrastructure on local level. The third method, which is the most accurate method, involves the use of detailed engineering data into customized methodology developed for specific community.

Table 4.1. Use-related classifications of facility (reproduced from HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Description
	Residential		
1	RES1	Single Family Dwelling	Detached House
2	RES2	Mobile Home	Mobile Home
3-8	RES3a-f	Multi Family Dwelling	Apartment/Condominium
9	RES4	Temporary Lodging	Hotel/Motel
10	RES5	Institutional Dormitory	Group Housing (military, college), Jails
11	RES6	Nursing Home	
	Commercial		
12	COM1	Retail Trade	Store
13	COM2	Wholesale	Trade Warehouse
14	COM3	Personal and Repair Services	Service Station/Shop
15	COM4	Professional/Technical Services	Offices
16	COM5	Banks/Financial Institutions	
17	COM6	Hospital	
18	COM7	Medical Office/Clinic	Offices
19	COM8	Entertainment & Recreation	Restaurants/Bars
20	COM9	Theaters	Theaters
21	COM10	Parking	Garages
	Industrial		
22	IND1	Heavy	Factory
23	IND2	Light	Factory
24	IND3	Food/Drugs/Chemicals	Factory
25	IND4	Metals/Minerals Processing	Factory
26	IND5	High Technology	Factory
27	IND6	Construction	Office
	Agriculture		
28	AGR1	Agriculture	
	Religion/Non-Profit		
29	REL1	Church	
	Government		
30	GOV1	General Services	Office
31	GOV2	Emergency Response	Police/Fire Station
	Education		
32	EDU1	Schools	
33	EDU2	Colleges/Universities	Does not include group housing

Earthquake loss assessment of HAZUS (FEMA, 2003) is conducted by classifying facilities into three use-related categories. This is to determine the nature and value of the non-structural components that describe the facilities. The occupancy classes are residential, commercial/industrial, and industrial facility. Several categories are considered under each occupancy class to establish refined economic loss analysis. The categories of occupancy classes are provided in Table 4.1. In addition, the economic data to carry the loss assessment are repair and replacement costs, values of the content for the use-related class, annual gross sales and income by occupancy. The following subsections describe the direct economic losses that are considered in this study.

4.4.1 Building repair and replacement cost

The losses are estimated for structural and non-structural damages by converting the probabilities of being in a damage state to equivalent dollar losses for a given occupancy class. The building repair and replacement cost is computed as the product of the floor area of each building type within the occupancy class, the probability of the building type to exceed damage state, and the repair cost of the building type per square foot for the identified damage state.

$$CS_{ds,i} = BRC_i \times PMBTSTR_{ds,i} \times RCS_{ds,i} \quad [4.1]$$

$$CS_i = \sum_{ds=2}^{5} CS_{ds,i} \quad [4.2]$$

where $CS_{ds,i}$ is cost of structural damage for damage state ds and occupancy class i; BRC_i is building replacement cost of occupancy *i*; $PMBTSTR_{ds,i}$ is probability of being in structural damage state *ds* for occupancy class *i*, and $RCS_{ds,i}$ repair and replacement ratio for structural damage in state *ds* and occupancy i (Table A.1 in Appendix A). It is noteworthy to indicate that damage state (ds) of 1 refers to none, and so it is not to be considered for the loss assessment process. This explains the reason that the summation of (Equation 4.2) starts from 2 to 5.

Similar calculation is performed to quantify the dollar loss due to non-structural damage. Non-structural components are classified into acceleration sensitive components (e.g., piping ceiling, elevators, and mechanical and electrical equipments.); and, drift sensitive components (e.g., partitions, exterior walls, and glass). The dollar loss is computed as follows:

$$CS_{ds,i} = BRC_j \times PONSA_{ds,i} \times RCA_{ds,i} \quad [4.3]$$

$$CNSA_i = \sum_{ds=2}^{5} CNSA_{ds,i} \quad [4.4]$$

$$CNSD_{ds,i} = BRC_i \times PONSD_{ds,i} \times RCD_{ds,i} \quad [4.5]$$

$$CNSD_i = \sum_{ds=2}^{5} CNSD_{ds,i} \quad [4.6]$$

where $CNSA_{ds,i}$ is cost of acceleration sensitive non-structural damage for damage state ds and occupancy class i; $CNSA_i$ is cost of acceleration sensitive non-structural damage for occupancy class i; $CNSD_{ds,i}$ is cost of drift sensitive non-structural damage for damage state ds and occupancy class i; $CNSD_i$ is cost of drift sensitive non-structural damage for occupancy class i BRC_j is building replacement cost of occupancy i; $PONSA_{ds,i}$ is probability of being in non-structural acceleration sensitive damage state ds for occupancy class i; $PONSD_{ds,i}$ is probability of being in non-structural drift sensitive damage state ds for occupancy class i; $RCA_{ds,i}$ repair and replacement ratio for non-structural acceleration sensitive damage in state ds and occupancy i (Table A.2 in Appendix A), and $RCD_{ds,i}$ repair and replacement ratio for non-structural drift sensitive damage in state ds and occupancy i (Table A.3 in Appendix A).

To determine the total loss due to non-structural damage, the losses for drift and acceleration sensitive components are summed as:

$$CNS_i = CNSA_i + CNSD_i \quad [4.7]$$

Finally, the total loss due to non-structural and structural damages can be determined using the following equation:

$$CBD_i = CS_i + CNS_i \quad [4.8]$$

4.4.2 Building content losses

Building content is defined as equipment, furniture, and facilities not integral with the structure. It does not include ceiling, lightening, mechanical and electrical equipment. It is assumed that damage to content is attributed to sliding, thus acceleration is considered proper damage indicator and contents are classified as acceleration sensitive non-structural components. The damage cost of contents is computed as:

$$CCD_i = CRV_i \sum_{ds=2}^{5} CD_{ds,i} \times PMBTNSA_{ds,i} \qquad [4.9]$$

where CCD_i cost of content damage for occupancy class i, CRV_i replacement value of contents damage for occupancy class i, $CD_{ds,i}$ content damage ratio for occupancy class i and damage state ds (from Table A.4 in Appendix A), and $PMBTNSA_{ds,i}$ probability of occupancy class i to experience non-structural acceleration sensitive damage state ds.

4.4.3 Building repair time and loss of function time

Loss of function time is referred to when the facility is incapable of conducting business. In general, downtime of building function is shorter than repair time because business managers may rent alternate space while repairs are being completed. The repairing time of damaged facility can be divided into two categories: construction and clean-up time, and time to manage financial resources and complete design. The length of repair time depends on the level of damage state, and building occupancy (e.g., simple and small buildings require less repair time than heavily serviced or large buildings).

Table A.5 (Appendix A) presents required time for building repair and clean-up including delay time in decision making regarding several tasks: obtaining financing, inspection and recommendation, negotiation with design firms, and start up and occupancy activates after repair completion. It can be noted that the loss time for none and slight damage is considered to be short, thus work can be resumed with slight repairs are done. It is also indicated in Table A.5 that for most commercial and industrial facilities, the business interruption time is assumed to be short for moderate and extensive damages. This is attributed to the assumption that such facilities may found temporary rearrangement to continue their activities while repairs are being completed.

However, for some business, building repair time shown in Table A.5 may be irrelevant. This is because of the possibility that owners may rent alternate space elsewhere. This is accounted for by multiplying the values shown in Table A.5 by factors to arrive at proper estimates of business interruption costs. The factors are shown in Table A.6.

The application resulting from multiplying the factors in Table A.6 by the time shown in Table A.5 represents the median value for the probability of business interruption. The multiplication is done as follows:

$$LOF_{ds} = BCT_{ds} \times MOD_{ds} \quad [4.10]$$

where LOF_{ds} is loss of function due damage state ds; BCT_{ds} construction and clean up time for damage state ds (Table A.5), and MOD_{ds} construction time modifiers for damage state ds (Table A.6).

4.4.4 Loss of income

Business generates several types of income resources. First income resource is related to the ownership of the property. Business activity provides profit, and portion of the profit is paid to individuals and other businesses as dividends; while, the remaining profit is kept for the enterprise. This is in addition to the interest that business pays to banks and bondholders for loans. Further, business generates to owners a category referred to as proprietary income, which portion of it reflects profit and the other portion reflects an imputed salary. Finally, biggest portion of the earned income is paid to the labour. In general, as for businesses in U.S., wages and salary incomes compromise more than 75% of the generated profit.

Income losses occur when building damage interrupts business activities. The losses are computed as the product of floor area, income generated per floor area, and the anticipated days of downtime for each damage state. The following formula describes estimate of income losses:

$$YLOS_i = (1 - RF_i) \times FA_i \times INC_i \times \sum_{ds=1}^{5} POSTR_{ds,i} \times LOF_{ds} \quad [4.11]$$

where $YLOS_i$ is income loss for occupancy i; FA_i floor area of occupancy class i (in square feet); INC_i income per day for occupancy i (Table A.7); $POSTR_{ds,i}$ is probability of being in damage state ds for occupancy i, and RF_i recapture factor for occupancy i (Table A.8).

4.4.5 Rental income losses

This loss applies to residential, commercial, and industrial businesses. It is considered that renter will pay full rent in case of none and slight damages. Thus, rental losses are only computed for moderate, extensive, and complete damage states. Rental income losses are the product of floor area, rental rates realized by floor area, and the expected days for loss of function. The rental income is computed as percentage of floor area as follows:

$$RY_i = (1 - \%OO_i) \times FA_i \times RENT_i \times \sum POSTR_{ds,i} \times RT_{ds} \quad [4.12]$$

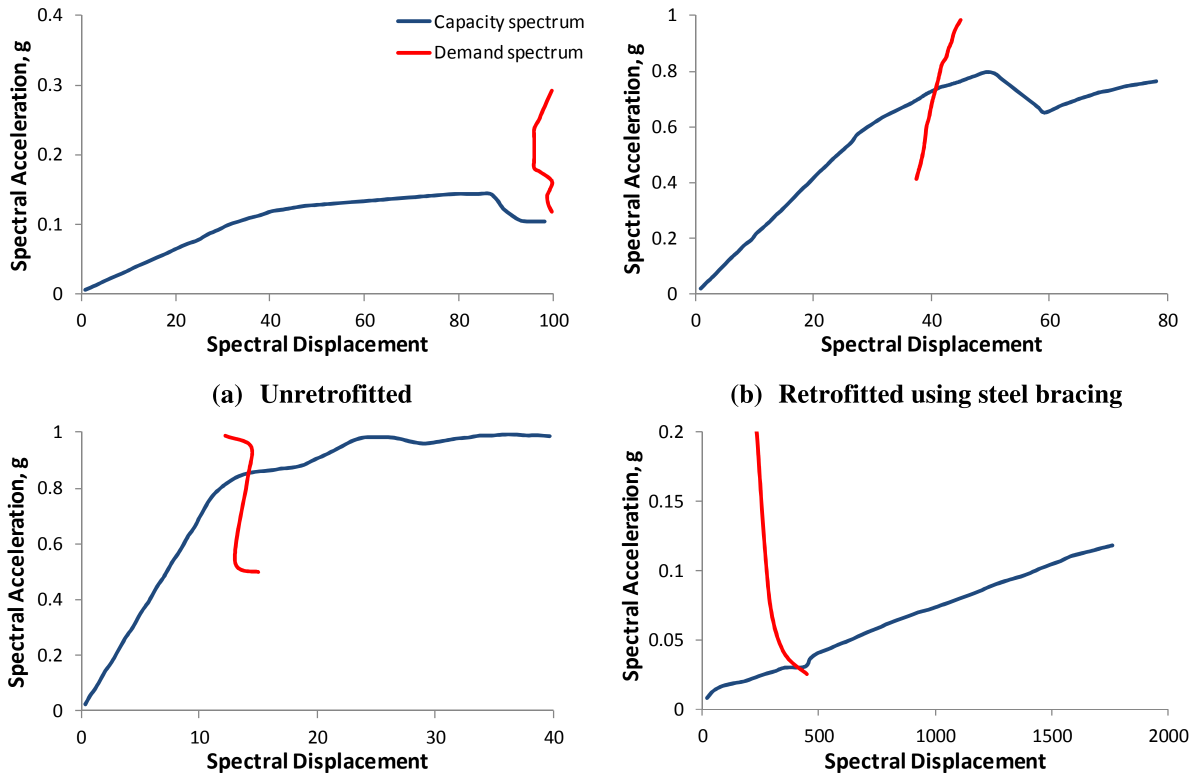
where RY_i is rental income losses for occupancy i; $\%OO_i$ percent owner occupied for occupancy i (Table A.9); FA_i floor area of occupancy i (in square feet); $RENT_i$ rental cost ($/ft^2/day) for occupancy i (Table A.10); $POSTR_{ds,i}$ probability of being damage state ds for occupancy class i, and RT_{ds} recovery time for damage state ds (Table A.5)

It should be noted that rental rates vary based on the desirability of the building and neighbourhood, as well as the local economic conditions (e.g. vacancy rates). The percentage rates given for owner occupancy are based on judgments. Thus, census data may provide more accurate estimates for a given study region.

4.5 Damage Cost Estimation

Prediction of earthquake related losses requires quantification of the expected levels of physical damage. Earthquake induced damages are related to the expected displacement demand of retrofitted and unretrofitted cases. Expected displacement demand is represented by the intersection between capacity spectrum and seismic demand spectrum, and is referred to as performance point. The performance point is used as input in fragility models to determine the corresponding probabilities of exceeding number of damage levels. The estimated damage measures are then converted to compute the equivalent dollar loss as discussed in section 4.3.

Figure 4.2 compares capacity spectrum curve with seismic demand spectra of unretrofitted and retrofitted cases for the determination of performance points. As for unretrofitted case, the method illustrates that capacity spectrum and demand spectra do not intersect. This indicates that the structure fails before reaching the design earthquake, suggesting that the structure needs seismic retrofitting. As for retrofitted cases, performance point for steel bracing shear wall, and base isolation schemes is 0.722g, 0.829g, and 0.042g, respectively. The increased value of spectral acceleration for retrofitted structure using shear walls and steel bracings indicates increase in seismic demand. This attributed to increase in building’s frequency due to its stiffening. In case of base isolated structure, the isolating devices reduce the earthquake induced seismic demand. This causes the capacity curve of base isolated system to flatten. The estimated performance points are inputted into the derived fragility models to determine the corresponding damage state probabilities. The predicted damage measures for exceeding limits states for unretrofitted and retrofitted cases are presented in Table 4.2.



(a) Unretrofitted **(b) Retrofitted using steel bracing**

(c) Retrofitted using shear wall **(d) Retrofitted using base isolation**

Figure 4.2. Performance point for unretrofitted and retrofitted cases

Table 4.2. Probability of exceeding performance levels obtained using fragility curves

Probability of exceeding	**Unretrofitted**	**Retrofitted with steel bracings**	**Retrofitted with shear walls**	**Retrofitted with base isolation**
No Damage	0	0.055	0.695	0.999
IO	0	0.081	0.136	0.00053
LS	0	0.444	0.089	2.40E-05
CP	1.0	0.210	0.040	6.06E-07

The estimated probabilities are converted into equivalent monetary losses using loss estimation methodology of HAZUS-MH (2003). The economic losses consist of cost due repair of structural and non-structural components, damage of building content, downtime of building function, and consequential losses due downtime of building function (e.g., loss of job and/or housing). The computed losses are presented in Table 4.3.

Table 4.3. Computed physical damages cost for unretrofitted and retrofitted cases

Damage state	Building repair and replacement cost	Loss of function	Rental income losses	Total
Slight	$12,268,350	$1,306,689	$137,700	$13,712,738
Moderate	$62,704,900	$1,306,689	$619,650	$64,631,238
Extensive	$310,798,200	$1,306,689	$1,239,300	$313,344,189

In addition, construction cost to install each retrofit scheme is considered to finalize estimate on the total cost for each retrofit. FEMA SRCE (2010) was used to estimate construction cost for steel bracing and shear wall retrofit options. The estimated installation costs for steel bracing scheme are $813,087, $600,000 and $235,532. The selected construction costs for shear wall scheme are $3,132,075, $2,400,000, and $1,178,913. Furthermore, according to Kelly (1998) and Boroschek (2002), construction costs for implementing base isolators are $24,000,000, $27,000,000, and $112,000,000.

4.6 Application of Decision Tree Analysis

Efficiency of retrofit measure to reduce vulnerability and potential damage of the building was quantified in dollar terms. In this section, decision tree analysis is performed to provide insight on the cost-effective mitigation strategy. The cost-effective retrofit is considered as the retrofit option with the minimal expected value. Expected value of an option is the weighted average of all possible values (i.e. cost) multiplied by their probability of occurrence. The expected value parameter provides, on average basis, the anticipated benefits or losses due implementing a mitigation strategy, where benefit represents reduction in earthquake-related monetary losses due mitigation, when compare with unretrofitted case option.

Figure 4.3 presents decision tree model for the comparison of expected values of the alternatives investigated in this study. Result of decision tree analysis indicates that shear wall scheme is the most economical solution, and base isolation scheme is the second option.

Despite the high installation cost of base isolators, solution of the decision model classifies this retrofit as more cost-effective when compared with using steel bracing option. This is mainly attributed to the ability of isolating devices to reduce transmitted seismic energy, and consequently the incurred damages due earthquake. Nevertheless, it is noted that adopting steel bracing offer cost savings of about 70% as compared with as-built case. This is particularly useful in cases where there are constraints to invest in expensive mitigation measure.

Similar decision model was conducted to test the influence of varying construction cost for the retrofits. It was noted that the solution of decision model does not change for steel bracing and shear wall schemes. This indicates that the ability of the retrofits to reduce physical damages during earthquake is a more dominant factor than construction cost factor. However, expensive investment (i.e., $112,000,000) for installing base isolators affects ranking of base isolators as the third desired retrofit pattern, instead of a second option when $27,000,000 was considered as a construction cost.

Furthermore, comparing the benefits related to adopting retrofit measures with the cost to be afforded in case of building collapse emphasize effectiveness of rehabilitation policy to reduce seismic risk; as well as, call for attention to invest in seismic retrofitting in order to save a community economic losses, aftermath an earthquake, that can be avoided.

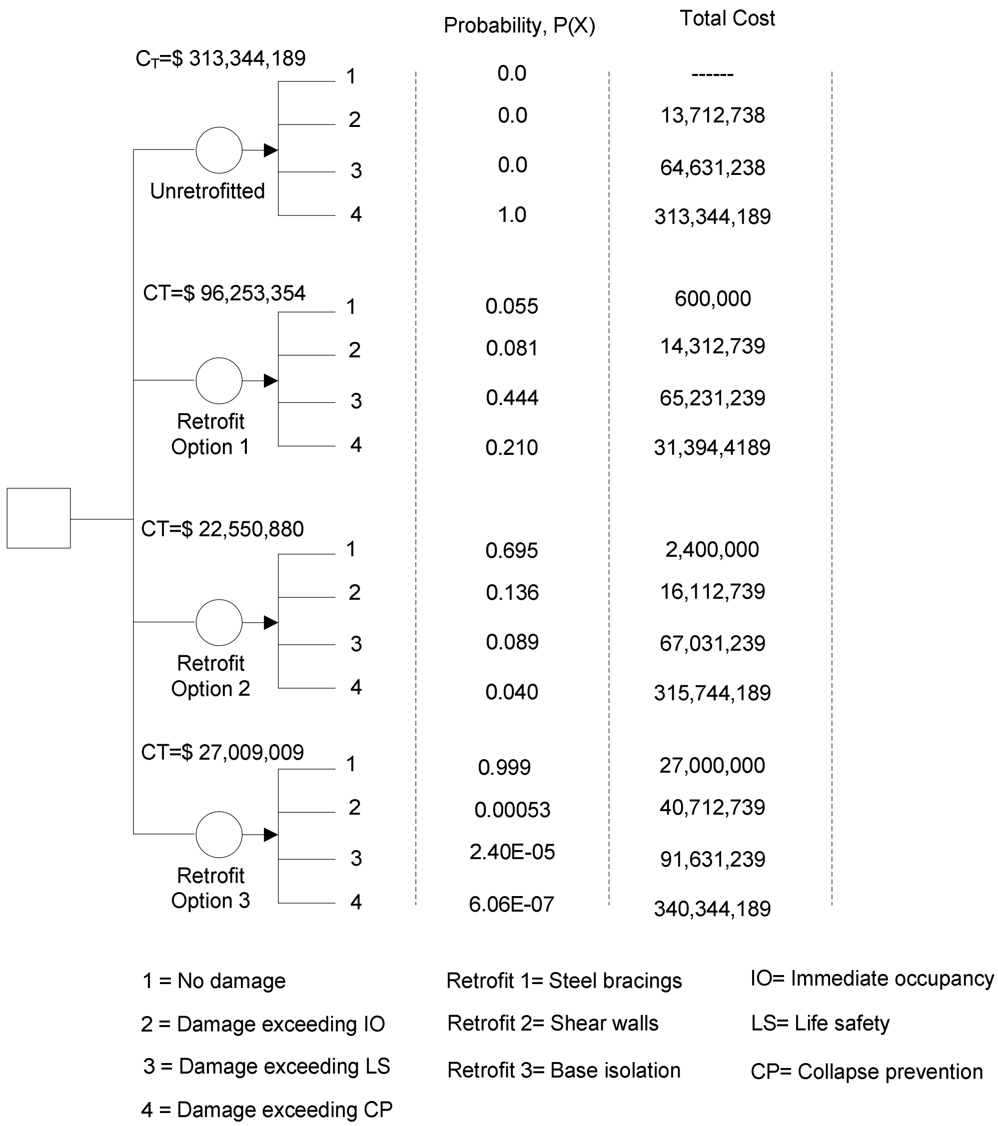


Figure 4.3. Decision tree for retrofit selection

4.7 Discussion

In this chapter, the non-ductile RC frame structure was re-examined to assess reduced earthquake related economic losses due to rehabilitation. Economic losses were predicted following HAZUS (FEMA, 2003) framework by converting fragility information into monetary losses. Indirect losses were considered to establish idea on seismic-induced losses associated with commercial facility. This prediction of losses was intended to evaluate investment in seismic retrofitting. Results indicated that seismic retrofitting was effective to prevent catastrophic collapse of the facility, and consequently avoid building owner to suffer great deal of economic losses as compared to what is needed for investment in seismic rehabilitation. It was reported that retrofit measures offered savings up to %70, %91, and %93 due to implementation of steel bracing, base isolation, and shear wall scheme, respectively. It was also evident that the use of decision tree provided ease of comparison, and facilitated the process of identifying the benefits related to the use of each retrofit option. This is particularly true since the decision criterion was expressed in dollar term. As an illustration, it can be noted the use of steel bracing offer less saving on earthquake-related losses, than the use of base isolation. However, the high installation cost of base isolation scheme is considerably higher than that for incorporating steel bracings. As so, the use of steel bracing option can be more desired in case limited funding is available for investment in seismic retrofit.

Furthermore, determination of performance point, to be used in fragility assessment, depends on the characteristics of the building site. This suggests that desired retrofit option may differ by location. In other words, the framework can be extended for other types of structure, retrofit patterns, and/or locations for screening the cost effective seismic retrofit.

CHAPTER 5 : CONCLUSION AND FUTURE WORK

5.1 Summary

The collaborative research efforts of PEER has resulted in a methodology that quantifies seismic demand in term of parameters of interest to stakeholders (e.g., dollars, fatalities, downtime) that facilitate the decision making process concerning managing seismic risk. Unfortunately, the process of evaluating earthquake-induced economic losses can become complicated due to the significant amount of information required, making it computationally intensive and thus overwhelm structural engineers to conduct loss assessment while delivering structural design. Successful adoption of performance based earthquake engineering approach by practicing engineers may count on providing more computationally efficient version of PEER methodology.

This study proposed simplified implementation of PEER's approach to perform loss estimation assessment in a more efficient way to quantify seismic response parameters. Application of the methodology was demonstrated by quantifying efficiency of seismic retrofit schemes to reduce earthquake-related damage and subsequent economic losses. Three retrofit patterns including steel bracing, shear wall, and base isolation methods were adopted to enhance seismic performance of a typical 1960s non-ductile reinforced concrete building. Nonlinear static analysis and nonlinear time history analysis were used to characterize lateral performance of retrofitted and unretrofitted cases. Range of seismic hazards was considered to derive seismic response parameters of these cases. The ground motion records represent 50% (very low), 10% (low), 5% (moderate) and 2% (high) probability of occurrence in 50 years return period. Based on global evaluation criteria of FEMA 356 (2000), fragility models were constructed for unretrofitted and retrofitted cases. The fragility relations created relationship between structural response and damage measures. The models assessed efficiency of the intervention methods to reduce earthquake-induced damages. The effectiveness of seismic strengthening techniques was further assessed by converting physical damage information, obtained by fragility models, to estimate economic impact of damage. The use of economic loss as a performance metric was intended to measure efficiency of the intervention methods to reduce earthquake-induced losses. The implementation of decision tree model facilitated the process of understanding the benefits related to using each retrofit measure, and thus justifiable decision on the desired retrofit was easy to make, particularly that the decision criterion was expressed in monetary terms.

5.2 Findings

As selection of retrofit option depends on the deficient characteristics and inelastic behaviour of the structure, examination of mitigation strategies is essential to reveal their impact on the system level performance. Numerical results indicated that both steel bracing and infill wall schemes were effective in enhancing lateral stiffness and strength, and offered considerable control over drift demand and deformation pattern of the building. These aspects illustrate that the retrofitted cases experienced less rotation demand on the columns, implying enhanced reliability of the retrofitted structures against stability (or nonlinearity) problems related to P-Δ effect. However, the increase in the frame lateral load capacity was associated with reduction in the deformation capacity of the structure, indicating more potential for brittle mode of failure to occur.

The study also demonstrated the application of base isolators as a mitigation measure. Considerable decrease in the response parameter of the structure was reported due to the capability of the base isolators to absorb the inputted energy. As a result, the structural demand, and acceleration transmitted to non-structural component and equipments are reduced. As so, this retrofit scheme is an effective approach when the goal of design is to protect the building. This shall reflect less need for smaller section sizes during design, and less retrofit actions and/or disruption to building function, implying long term cost benefits for base-isolated structure.

Assessment of seismic strengthening techniques also included measuring their ability to reduce earthquake-related damages by developing probabilistic relationships between the specified limit states, and measures of earthquake demand (e.g., spectral acceleration, ground motion magnitude, etc.). Results of fragility assessment indicated that the selected intervention methods offered varying degree of protection for the system against physical damages. The addition of shear walls provided significant reduction in fragility for LS and CP. However, the use of base isolators was the most effective in reducing the seismic vulnerability of the case study building. It was also noted that each retrofit option modified the fundamental period of the structure, suggesting the need for using different spectral acceleration, if comparison of fragility models for unretrofitted and retrofitted cases is to be conducted.

The presented loss estimation methodology provides alternative measure to assess structural performance that is less computationally expensive than previous studies. The approach is based on creating relationships between structural response and loss measures by estimating damage levels of the structure for a given earthquake intensity. These relations predict losses of the facility when subjected to seismic hazard based on converting damage measures to monetary losses using inventory losses data provided by HAZUS (FEMA, 2003). This allows losses to be estimated without the need to know exact costs that are of interest to be investigated. As a results, engineers using this methodology can focus on the inputs such as, seismic hazard analysis and structural analysis, and on evaluating the outputs (i.e., decision making), rather than on the loss estimation procedure itself. Limiting the time and amount of resources needed on the loss estimation process shall facilitate the use of performance-based earthquake engineering technology by practicing engineers.

5.3 Limitation of the Study and Future work

There are several possible avenues to improve the proposed methodology to characterize seismic performance using economic loss metrics or extend these results. These areas can be organized in several categories including, model improvement and validation, treatment of source of uncertainties, and estimation of inventory losses.

5.3.1 Model improvement

Successful implementation of performance-based earthquake engineering requires development of computational model that accurately represents nonlinear characteristics of RC facilities. Several aspects present in this investigation that may provide opportunities for future studies concerning modelling of non-ductile RC frames:

- Model in this study do not account for the contribution of flat-slab system to the lateral resistance of the structure. These gravity frame elements should be incorporated into the analytical model to improve seismic performance assessment, and consequential seismic-induced economic losses.

- Model of the non-ductile RC frame excludes three-dimensional torsion failure. Further research is required to develop mathematical model that incorporates this failure mechanism.
- Model could also be advanced by accounting for the contribution of non-structural component on the lateral strength and stiffness of the RC frame; as well as, incorporating performance of non-structural components on limit states of fragility models.

The use of performance-based approach to predict economic losses of non-ductile RC structure may require validation with observed long-term performance of building stock in the region of interest. This validation requires documentation of structural response and financial losses to identify discrepancies between the predicted results and reported documentations based on experience. It is also of interest to examine suitability of the presented methodology to prioritize retrofit selection for other type of structures including steel, composite, and masonry structures, as well as bridge structures.

5.3.2 Treatment of uncertainties

This investigation can be expanded in various ways:

- Characterizing and treating the effect of structural modelling uncertainties shall yield a better seismic performance assessment of the frame structure. Also, examination of different types of structural systems would help to generalize the results.
- Investigation of construction and human error in design may influence the estimated financial losses for a given earthquake, and yield different conclusions concerning the best retrofit measure to manage seismic risk.
- Accounting for possible deterioration and aging of material properties, variations in maintenance, damages from past earthquakes since the time of construction; as well as, accounting for site conditions may yield more realistic assessment of seismic performance of the structure.

5.3.3 Economic loss estimation

Possible avenues for future research include the followings:

- Estimation of indirect losses, such downtime losses, illustrated that much higher financial benefits were attained due to mitigation. An interesting extension of loss estimation results is to compare the reduced downtime losses due to mitigation of non-ductile and code-conforming structures.
- Building residual drift induced by earthquake may significantly contribute to owner's susceptibility to financial losses. Consideration of residual drift shall reveal more financial benefits associated with seismic rehabilitation.
- Decreasing uncertainties in damage assessment and improve data for loss assessment shall establish better economic loss assessment for a given earthquake intensity.
- In addition to financial losses, this work can be extended to predict earthquake-related fatalities to illustrate effectiveness of seismic strengthening to reduce life threat posed by non-code conforming structures.

5.3.4 Need for Archetype data for policy development

This study implemented performance-based paradigm to provide informed decision on seismic safety policy. This provides motivation to establish archetype data concerning the following areas:

- Archetype data can be used to characterize influence of different heights, typical key design and detailing features, and type of irregularities commonly found in non-code conforming structures. This shall generalize the conclusion on seismic retrofits to manage seismic risk.
- Variation of building sites and hazard levels may have significant impact on the cost-benefit assessment.
- Structures with irregular infill walls, irregular plan causing torsional demand, and structures susceptibility for shear failures may need to be incorporated in the cost-benefit assessment of seismic rehabilitation.

5.4 Concluding Remarks

Results of this study serve the debate on managing seismic risk through implementation of retrofit schemes by providing loss estimate measures that explicitly illustrate effectiveness of mitigation strategies to reduce owner's susceptibility to earthquake-induced monetary losses. The provided information may be used to establish well-informed decisions related to identification of vulnerable non-ductile RC structures, assessment of polices and performance targets for intervention methods; as well as, provide transparent incentives for stakeholders to invest in seismic safety.

BIBLIOGRAPHY

Abou-Elfath, H., and Ghobarah, A. 2000. Behaviour of reinforced concrete frames rehabilitated with concentric steel bracing. *Canadian Journal of Civil Engineering*, **27**(3): 433–444.

Aboutaha, R.S., Engelhardt, M.D., Jirsa, J.O., and Kreger, M.E. 1999. Rehabilitation of shear critical concrete columns by use of rectangular steel jackets. *ACI Structural Journal*, **96**(1): 68–78.

ACI Committee 318. 1983. *Building Code Requirements for Reinforced Concrete* (ACI 318-83). American Concrete Institute. Detroit (MI).

Ahamd, S.H., Khaloo, A.R., and Irshaid, A. 1991. Behaviour of concrete spirally confined by fibre glass filament. *Magazine of Concrete Research*, **43**(156): 143–148.

Ahmad, S.H., and Shah, S.P. 1982a. Complete triaxial stress-strain curves for concrete. *ASCE Journal of Structural Division*, **108**(4): 728–742.

Ahmad, S.H., and Shah, S.P. 1982b. Stress–strain curves of concrete confined by spiral reinforcement. *ACI Structural Journal*, **79**: 484–490.

Aire, C., Gettu, R., and Casas, J.R. 2001. Study of the compressive behavior of concrete confined by fiber reinforced composites. *Composites in Constructions, Proceeding of the International Conference* A. A. Balkema, Lisse, The Netherlands, 239–243.

Al-Chatti, Q., Siraj, T., Rajeev. P., and Tesfamariam, S. 2011. Decision making tool for seismic retrofit of non-ductile reinforced concrete building. CSCE 2011 General Conference, Ottawa, Ontario, Canada, June 14-17, 2011.

Al-Chatti. Q., and Tesfamariam, S. 2012. Risk management framework for assessment of seismic retrofit, *CSCE 2012 3rd International Structural Specialty Conference*, Edmonton, Alberta, Canada.

Alesch, D.J., Dargush, G.F., Grigoriu, M., Petak, W.J., and von Winterfeldt, D. 2003. *Decision Models: Approaches for Achieving Seismic Resilience. Research Progress and Accomplishments*, 2001-2003, Multidisciplinary Center for Earthquake Engineering Research, University at Buffalo.

Allen, R.M., and Gerald, S. 2007. Earthquake hazard mitigation: new directions and opportunities. *Treatise on Geophysics Earthquake seismology*, H. Kanamori edition, (4): 607-648

Altin, S., Anil, O., Kara, M.E., and Kaya, M. 2007. Strengthening of RC nonductile frames with RC infills: An experimental study, *Cement and Concrete Composite*, 30: 612–621.

Aoyama, H., Kato, D., Katsumata, H., and Hosokawa, Y. 1984. Strength and behavior of postcast shear walls for strengthening of existing R/C buildings, in: *Proceeding of Eight World Conference on Earthquake Engineering*, San Francisco CA, **1**: 485–92.

Applied Technology Council, ATC-40. 1996. *Seismic Evaluation and Retrofit of Concrete Buildings*, vols. 1 and 2. California.

ASCE. 2005. *Minimum Design Loads for Buildings and Other Structures* (ASCE 7-05). American Society of Civil Engineers, Reston, Virginia.

Aslani, H., and Miranda, E. 2005. *Probabilistic Earthquake Loss Estimation and Loss Disaggregation in Buildings*, Report No. 157. Stanford, CA: John A. Blume Earthquake Engineering Center, Stanford University.

ATC-13. 1985. *Earthquake Damage Evaluation Data for California*. Applied Technology Concuil, Redwood City Federal Emergency Management Agency.

Axley, J.W. 1980. Modeling the stiffness contribution of infilled panels to framed structures by a constraint approach, in: *Proceeding of the 7th WCEE*, Istanbul, Turkey, **4**: 249–51.

Axley, J.W., and Bertero, V.V. 1979. *Infill Panels: Their Influence on Seismic Response of Buildings*, Report No. UCB/EERC-79/28, Earthquake Engineering Research Center, University of California, Berkeley.

Badoux, M., and Jirsa, J.O. 1990. Steel bracing of RC frames for seismic retrofitting. *ASCE Journal of Structural Engineering*, **116**(1): 55–74.

Benuska, K.L. 1990. Loma Prieta earthquake reconnaissance report. *Earthquake Spectra*, **6:** 1- 448.

Beres, A., El-Borgi, S., White, R.N., and Gergely, P. 1992. *Experimental results of repaired and retrofitted beam–column joint tests in lightly reinforced concrete frame buildings.* In: Report No, NCEER-92- 25. Buffalo (NY): National Center for Earthquake Engineering Research, State University of New York at Buffalo.

Beres, A., White, R.N., and Gergely, P. 1992. *Seismic performance of interior and exterior beam-to-column reinforced concrete frame buildings.* Detailed experimental results. In: Report No. 92-06. Ithaca, NY: Department of Structural Engineering, Cornell University 1992.

Bilen, O., and Buyuklu, A.H. 2006. Modelling banking sector data using decision tree algorithms and checking these algorithms. *In Proceedings of the International Conference on Modelling and Simulation*, Montreal, Canada, **2**: 843–847

Boroschek, R. 2002. *Base Isolation in Hospitals*. Seminar on Design of Health Facilities to Resist Natural Hazards. Barbados.

Bouadi, A., Engelhardt, M.D., Jirsa, J.O., and Kreger, M.E. 1994. Eccentrically braced frames for strengthening of R/C building with short columns. *Proceedings of the 5th U.S. National Conference on Earthquake Engineering*, Chicago, Ill., **3**: 617–626.

Bouchard, K.M. 2007. A performance-based approach to retrofitting unreinforced masonry structures for seismic loads. Master thesis, Massachusetts Institute of Technology.

Buckle, I.G., and Liu, H. 1993. Stability of elastomeric seismic isolation systems. *Proceedings of ATC 17-1 Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, San Francisco, California, pp. 293-306.

Budnitz, R.J., Apostolakis, G., Boore, D.M., Cluff, L.S., Coppersmith, K.J., Cornell, C.A., and Morris, P.A. 1998. Use of technical expert panels: applications to probabilistic seismic hazard analysis. *Journal of Risk Analysis*, **18**(4): 466.

Bush, T., Jones, E. and Jirsa, J.O. 1991. Behavior of RC frame strengthening using structural steel bracing. *ASCE Journal of Structural Engineering*, **117**(4): 1115–1126.

Canales, M.D., and Briseno, de la., and Vega, R. 1992. Retrofitting techniques used in telephone buildings in Mexico. *Proceedings of the 10th World Conference on Earthquake Engineering*, Madrid, Spain, pp. 5143–5147.

Canbay, E., and Ersoy, U., and O¨ zcebe, G. 2003. Contribution of RC Infills to the seismic behavior of structural system. *ACI Structural Journal*, **100**(5): 637–43.

Chaallal, O., Hassan, M., and Shahawy, M. 2003. Confinement model for axially loaded short rectangular columns strengthened with fiber reinforced polymer wrapping. *ACI Structural Journal*, **100**(2): 215–221.

Chen, Y., and Ahmadi, G. 1993. Stochastic earthquake response of secondary systems in base-isolated structures. *Earthquake Engineering and Structural Dynamics*, **21**: 1039-1057.

Cheung, M., Foo, S. and Granadino, J. 2000. Seismic Retrofit of Existing Buildings: Innovative Alternatuves. Public Works & Government Services Canada, Hull (Quebec) and Vancouver (British Columbia), Canada. Retrieved from http://www.icomos.org.

Chun, S.S., and Park, H.C. 2002. Load carrying capacity and ductility of RC columns confined by carbon fiber reinforced polymer. *Proceedings 3rd International Conference on Composites in Infrastructure (CD-ROM)*, University Of Arizona, San Francisco.

Cornell, A., Zareian, F., Krawinkler, H., and Miranada, E. 2005. *Prediction of Probability of Collapse*. In H. Krawinkler (Ed.), Van Nuys Hotel Building Testbed Report: Exercising Seismic Performance Assessment. 4.5. Vol. 11, pp. 85-93. Pacific Earthquake Engineering Research.

Cornell, C.A., Jalayer, F., Hamburger, R.O., and Foutch, D.A. 2002. Probabilistic basis for 2000 SAC federal emergency management agency steel moment frame guidelines. *Journal of Structural Engineering*, **128** (4): 526-533.

CSI. SAP2000 V-8. 2002. *Integrated Finite Element Analysis and Design of Structures Basic Analysis Reference Manual*. Berkeley (CA, USA): Computers and Structures Inc.

Degenkolb, H. 1994. *Connections* (Interview by Stanley Scott). Oakland, EERI Oral History Series, EERI.

Demers, M., and Neale, K.W. 1994. Strengthening of concrete columns with unidirectional composite sheets. Development in Short and Medium Span Bridge Engineering ’94, *Proceedings of the 4th International Conference on Short and Medium Span Bridges*, A. A. Mufti, B. Bakht, and L. G. Jaeger, eds., Canadian Society for Civil Engineering, Montreal, 895– 905.

Demircioglu, M.B., Erdik, M., Hancilar, U., Sesetyan, K., Tuzun, C., Yenidogan, et al. 2009 Technical manual—Earthquake loss estimation routine ELER-v1.0.Istanbul: Department of Earthquake Engineering, Bogazici University.

Di Sarno, L. and Elnashi, A.S. 2004. Bracing systems for seismic retrofitting of steel frames. *Proceeding of 13th World Conference on Earthquake Engineering*, Vancouver, Canada.

Di Sarno, L., and Elnashai, A.S. 2009. Bracing systems for seismic retrofitting of steel frames. *Journal of Construction Steel Research*, **65**: 452–65.

El-Amoury, T., and Ghobarah, A. 2005. Retrofit of RC frames using FRP jacketing or steel bracing. *Journal of Seismology and Earthquake Engineering*, **7**(2): 83-94.

Erdik, M., et al. 2010. Rapid earthquake loss assessment after damaging earthquakes. *Soil Dynamic Earthquake Engineering*, **31**(2): 247–266.

Fam, A.Z., and Rizkalla, S.H. 2001. Confinement model for axially loaded concrete confined by circular fiber-reinforced polymer tubes. *ACI Structural Journal*, **98**(4): 451–461.

Fan, F.G., and Ahmadi, G. 1990. Floor response spectra for base-isolated multi-storey structures. *Earthquake Engineering and Structural Dynamics*, **19**: 337-388.

Fardis, M.N., and Biskinis, D.E. 2003. Deformation of RC members, as controlled by flexure or shear. In: *Proceedings of the international symposium honouring Shunsuke Otani on performance-based engineering for earthquake resistant reinforced concrete structures*.

Fardis, M.N., and Khalili, H. 1981.Concrete encased in fibre glass reinforced plastic. *ACI Structural Journal*, **78**(6): 440–446.

FEMA. 2003. *HAZUS-MH Technical Manual*. Washington, DC: Federal Emergency Management Agency.

FEMA-273. 1997. *NEHRP Guidelines for Seismic Rehabilitation of Buildings. Washington*, DC: Prepared by the SAC Joint Venture for the Federal Emergency Management Agency.

FEMA-356. 2000. *Prestandard and Commentary for the Seismic Rehabilitation of Buildings*. Washington, DC: Prepared by the SAC Joint Venture for the Federal Emergency Management Agency.

Fenz, D.M., and Constantinou, M.C. 2008. Modeling triple friction pendulum bearings for response history analysis. *Earthquake Spectra*, **24**(4): 1011-1028.

Foutch, D.A., Gambill, J.B., and Garza-Tamez, F. 1993. *Investigation of a Seismic Base Isolation System Based on Pendular Action*. Structural Research Series No. 578, Department of Civil Engineering, University of Illinois at Urbana-Champaign.

Frazier, G.A., Wood, J.H., and Housner, G.W. 1971. <u>Earthquake Damage to Buildings. Engineering Features of the San Fernando Earthquake of February 9, 1971</u>. P.C. Jennings, Ed. Pasadena, California Institute of Technology.

Gemme, M.C. 2007. *Seismic Retrofitting of Deficient Canadian Buildings*. Master thesis, McGill University, Canada.

Ghobarah, A. 2000. Seismic assessment of existing RC structures. *Progress in Structural Engineering and Materials*, **2**(1): 60–71.

Ghobarah, A. 2001. Performance-based design in earthquake engineering: state of development. *Engineering Structure*, **23**(8):878–84.

Ghobarah, A., and Biddah, A. 1999. Dynamic analysis of reinforced concrete frames including joint shear deformation. *Engineering Structure*, **21**: 971–87.

Ghobarah, A., and Said, A. 2001. Seismic rehabilitation of beam–column joints using FRP laminates. *Journal of Earthquake Engineering*, **5**(1):113– 29.

Ghobarah, A., and Youssef, M. 1999. Response of an existing RC building including concrete crushing and bond slip effects. In: *Proceedings of 8th Canadian Conference on Earthquake Engineering, Canadian Association for Earthquake Engineering*, Vancouver, BC, p. 427–32.

Ghobarah, A., Aziz, T.S., and Biddah, A. 1996. Seismic rehabilitation of reinforced concrete beam–column connections. *Earthquake Spectra*, **12**(4):761–80.

Ghobarah, A., Tarek, S.A., and Biddah, A. 1996. Seismic rehabilitation of reinforced concrete beam–column connections. *Earthquake Spectra (EERI)*, **12**(4): 761–780.

Giuliani, G.C. 1993. Structural design, analysis and full-scale tests of seismically isolated buildings. *Engineering Structures*, 15(2): 102-116.

Goel, S.C., and Lee, H. 1990. Seismic strengthening of RC structures by ductile steel bracing system. *Proceedings of the 4th U.S. National Conference on Earthquake Engineering*, Palm Springs, Calif., Vol. 3, pp. 323–331.

Goel, S.C., and Masri, A.C. 1996. Seismic strengthening of an RC slab-column frames with ductile steel bracing. *Proceedings 11th World Conference on Earthquake Engineering*, Acapulco, Mexico (CD- ROM).

Goel, S.C., and Masri, A.C. 1996. Seismic Strengthening of an RC Slab-Column Frames with Ductile Steel Bracing. *Proceedings 11th World Conference on Earthquake Engineering*, Acapulco, Mexico (CD- ROM).

Gomes, A.M., and, Appleton. J. 1998. Repair and strengthening of reinforced concrete elements under cyclic loading. *11th European Conference on Earthquake Engineering, Paris, France*, (CD-ROM).

Goulet, C.A., Haselton, C.B., Mitrani-Reiser, J., Beck, J.L., Deierlein, G.G., Porter, K.A. and Stewart, J.P. 2007. Evaluation of the seismic performance of a code-conforming reinforced concrete frame building - from seismic hazard to collapse safety and economic losses. *Earthquake Engineering and Structural Dynamics*, **36**(13): 1973 - 1997.

Gregorian, Z.B., and Gregorian, G.B. 1996. Seismic upgrading of hospital building No. 6 and 7, VA Medical Center, Bedford, Mass. In: Sabnis GM, Shroff AC, Khan, LF, editors. *Seismic Rehabilitation of Concrete Structures*, SP-160, American Concrete Institute, Farmington Hills, MI, p. 131–47.

Griffith, M.C., Aiken, I.D., and Kelly, J.M. 1990. Displacement control and uplift restraint for base-isolated structures. *ASCE Journal of Structural Engineering*, **116**(4): 1135-1148.

Gunturi, S., and Shah, H. 1993. *Building-Specific Earthquake Damage Estimation*. Ph.D. Thesis, Stanford, CA: John A. Blume Earthquake Engineering Center, Stanford University.

Hall, J.F. 1995. *Northridge Earthquake of January 17, 1994 Reconnaissance Report*. Oakland, CA: Earthquake Engineering Research Institute.

Harries, K.A., and Kharel, G. 2002. Behavior and modeling of concrete subject to variable confining pressure. *ACI Material Journal*, **99**(2): 180–189.

Haselton, C.B., and Deierlein, G.G. 2007. *Assessing Seismic Collapse Safety of Modern Reinforced Concrete Frame Buildings*, Technical Report No. 156. Stanford, CA: John A. Blume Earthquake Engineering Center, Stanford University.

Higashi, Y., and Kokusho, S. 1975. The strengthening method of existing reinforced concrete buildings. *US–Japan Cooperative Research Program in Earthquake Engineering*, Honolulu, Hawaii.

Higashi, Y., Endo, T., Shimuzu, Y. 1982. Effects on behavior of reinforced concrete frames by adding shear walls, in: *Proceedings of the Third Seminar on Repair and Retrofit of Structures*, Ann Arbor, MI.

Higashi, Y., Endo, T., Shimuzu, Y. 1984. Experimental studies on retrofitting of reinforced concrete building frames, in: *Proceeding of the Eight World Conference on Earthquake Engineering*, San Francisco, CA, Vol. 1, pp. 477–84.

Hollayay, L.C. 2003. The evolution and the way forward for advanced polymer composites in the civil infrastructure. Construction and Building Material, Vol. 17, pp. 365-378

Howie, I., and Karbhari, V.M. 1994. Effect of materials architecture on strengthening efficiency of composite wraps for deteriorating columns in the North-east. Infrastructure: New Materials and Methods of Repair, *Proceeding 3rd Materials Engineering Conf., K. D. Basham ed., Material Engineering Division*, ASCE, N.Y., 199–206.

Hueste, M.D., and Bai, J-W. 2007. Seismic retrofit of a reinforced concrete flat slab structure: Part I - seismic performance evaluation. *Engineering Structures*, **29**(6): 1165–77.

Hueste, M.D., and Bai, J-W. 2007. Seismic retrofit of a reinforced concrete flat slab structure: Part II - seismic fragility analysis. *Engineering Structures*, **29**(6): 1165–77.

Inel, M., and Ozmen, H.B. 2006. Effects of plastic hinge properties in nonlinear analysis of reinforced concrete buildings. *Engineering Structures*, **28:** 1494–1502.

Insurance Information Institute 2006. Earthquakes: Facts and Statistics. Retrieved on January 12, 2009, from Insurance Information Institute Web site: http://www.iii.org/media/facts/statsbyissue/earthquakes/.

Islam, S., and Gupta, B. 2004. Seismic retrofit of a building using passive energy-dissipation devices. *Proceedings, ASCE Structures Congress,* Washington, D.C., United States, May 21-23, 2001

Jangid, R.S., and Datta, T.K. 1994. Nonlinear response of torsionally coupled base isolated structure. *ASCE Journal of Structural Engineering*, **120**(1): 1-22.

Jennings, P. Ed. 1971. Engineering Features of the San Fernando Earthquake of February 9, 1971. Pasadena, CA.

Ji, J., Elnashai, A.S., Kuchma, D.A. 2007. *Seismic Fragility Assessment for Reinforced Concrete High-rise Buildings.* Mid-America Earthquake Center, University of Illinois at Urbana-Champaign, CD Release 07-14, September.

Jiang, T., and Teng, J.G. 2007. Analysis-oriented stress–strain models for FRP-confined concrete. *Engineering Structure*, **29**: 2968–86.

Jirsa, J.O., and Kreger, M. 1989. Recent research on repair and strengthening of reinforced concrete structures. *Proceedings, ASCE Structures Congress,* San Francisco, CA, 1, 679-688

Jirsa, J.O., and Kreger, M.E. 1989. Recent research on repair and strengthening of reinforced concrete structures. In: Kircher CA, Chopra AK, editors. *Seismic Engineering*, ASCE, p. 679–88.

Johnson, G.P., Robertson, I.N. 2004. Retrofit of slab-column connections using CFRP. *Procceding Of the 13th World Conference on Earthquake Engineering*, (CD-Rom, paper n. 142), Vancouver, B.C., Canada, August 1-6.

Jones, T. 1985. *Seismic Strengthening of a RC Frame Using Structural Steel Bracing*. Master Thesis, Department of Civil Engineering, University of Texas, Austin, Tex.

Juhn, G., Manolis, D., Constantinou, M.C., and Reinhorn, A.M. 1992. Experimental study of secondary systems in base-isolated structures. *ASCE Journal of Structural Engineering*, **118**(8): 2204-2221.

Julio, E.S., Branco, F. and Silva, V.D. 2003. Structural rehabilitation of columns with reinforced concrete jacketing. *Journal of Progress in Structural Engineering and Materials*, **5**(1): 29–37.

Kahn, L.F., and Hanson, R.D. 1979. Infilled walls for earthquake strengthening. *ASCE Journal of Structural Division*, **105**(ST2): 283–96.

Kalkan, E., and Kunnath, S.K. 2006. Effects of fling-step and forward-rupture directivity on the seismic response of buildings, *Earthquake Spectra*, **22**(2): 367–390.

Kappos, A., and Dimitrakopoulos, E. 2008. Feasibility of pre-earthquake strengthening of buildings based on cost benefit and life-cycle cost analysis, with aid of fragility curves. *Natural Hazards*, **45**: 33–54

Karbhari, V.M., and Gao, Y. 1997. Composite jacketed concrete under uniaxial compression—Verification of simple design equations. *Journal of Material in Civil Engineering*, **9**(4): 185–193.

Kartoum, A., Constantinou, M.C., and Reinhorn, A.M. 1992. Sliding isolation system for bridges: analytical study. *Earthquake Spectra*, **8**(3): 345-372.

Kelly, J.M. 1998. Seismic Isolation as an Innovative Approach for the Protection of Engineered Structures. *Proceedings 11th European Conference on Earthquake Engineering*, Balkema, Rotterdam.

Klingner, R.E., Bertero, V.V. 1976. *Infilled Frames in Earthquake-Resistant Construction*, Report No. UCB/EERC-76/32, Earthquake Engineering Research Center, University of California, Berkeley.

Klingner, R.E., Bertero, V.V. 1978. Earthquake resistance of infilled frames. *ASCE Journal of Structural Division*, **104**(ST6): 973–87.

Koh, C.G., and Kelly, J.M. 1990. Application of fractional derivatives to seismic analysis of base-isolated models. *Earthquake Engineering and Structural Dynamics*, 19: 229-241.

Kramer, S.L. 1996. *Geotechnical Earthquake Engineering*. Prentice Hall, Upper Saddle River, NJ.

Krawinkler, H., and Miranda, E. 2004. *Performance-Based Earthquake Engineering*. In Y. Borzognia and V. Bertero (Ed.), Earthquake Engineering: From Engineering Seismology to Performance-Based Engineering, 1st edition, pp. 1-59. CRC Press.

Kustu, O., Miller, D.D., and Brokken, S.T. 1982. *Development of Damage Functions for High-rise Building Components.* San Francisco, CA: URS / John A. Blume & Associates, Inc.

Lee, D.M., and Medland, I.C. 1979. Base Isolation Systems for Earthquake Protection of Multi-Storey Shear Structures. *Earthquake Engineering and Structural Dynamics*, **7**: 555-568.

Lew, H.S., and Kunnath, S.K. 2002. Assessment of structural systems evaluation methods based on local seismic demands. *Innovations in Design with Emphasis on Seismic, Wind, and Environmental Loading; Quality Control and Innovations in Materials/Hot-Weather Concreting* (SP-209), ACI 5th International Conference, Cancun, Mexico.

Li, Y.R., and Jirsa, J.O. 1998. Nonlinear analyses of an instrumented structure damaged in the 1994 Northridge earthquake. *Earthquake Spectra*, **14**(2): 265–283.

Liauw, T.C., and Kwan, K.H. 1985. Unified plastic analysis for infilled frames. *ASCE Journal of Structural Division*, **111**(7): 1427–49.

Liel, A.B. 2008. *Assessing the Collapse Risk of California's Existing Reinforced Concrete Frame Structures: Metrics for Seismic Safety Decisions*. PhD thesis, Stanford University.

Maheri, M.R., and Sahebi, A. 1995. Use of steel bracing in RC frames. *Engineering Structures*, **19**(12): 1018–1024.

Masri, A.C., Subhash, C., and Goel, C. 1996. Seismic design and testing of an RC slab-column frame strengthened by steel bracing. *Earthquake Spectra*, **12**(4): 645–666.

Miller, J.T., and Reaveley, L.D. 1996. Historic hotel Utah remodel and seismic upgrade. In: Sabnis GM, Shroff AC, Khan, LF, editors. *Seismic Rehabilitation of Concrete Structures*, SP-160, American Concrete Institute, Farmington Hills, MI, p. 115–30.

Miranda, E. 1991. *Seismic Evaluation and Upgrading of Existing Buildings*. Ph.D. thesis, Department of Civil Engineering, University of California, Berkeley, California.

Mirmiran, A. and Shahawy. 1996. A new concrete-filled hollow FRP composite column. *Composites, Part B*, **27**(3–4): 263–268.

Mirza, S., and Haider, M. 2003. *The State of Infrastructure in Canada: Implications for Infrastructure Planning and Policy*. Research and Analysis, Infrastructure Canada.

Mitchell, D. 2007. *Reducing Urban Seismic Risk*. Proposed Canadian Seismic Research Network, Montreal, Canada.

Mitchell, D., DeVall R.H., Saatcioglu, M., Simpson, R., Tinawi, R., and Tremblay, R. 1995. Damage to concrete structures due to the Northridge earthquake. *Canadian Journal of Civil Engineering*, **22**: 361-377.

Mitrani-Reiser, J. 2007. *An Ounce of Prevention: Probabilistic Loss Estimation for Performance Based Earthquake Engineering*, Doctoral Dissertation, California Institute of Technology.

Mitrani-Reiser, J., and Beck, J. 2007. *An Ounce of Prevention: Probabilistic Loss Estimation for Performance-based Earthquake Engineering*. Pasadena, CA: Department of Civil Engineering and Applied Mechanics, California Institute of Technology.

Miyauchi, K., Inoue, S., Kuroda, T., and Kobayashi, A. 1999. Strengthening effects of concrete columns with carbon fiber sheet. *Transportation Japan Concrete Institute*, **21**: 143–150.

Moehle, J.P. 2000. State of research on seismic retrofit of concrete building structures in the US. *US-Japan Symposium and Workshop on Seismic Retrofit of Concrete Structures*, Tokyo, Japan.

Mokha, A., Constantinou, M.C., and Reinhorn, A.M. 1990. Teflon bearings in base isolation. I: testing. *ASCE Journal of Structural Engineering*, **116**(2): 438-454.

Moran, D.A., and Pantelides, C.P. 2002. Stress-strain model for fiber reinforced polymer-confined concrete. *Journal of Composite Construction*, **6**(4), 233–240.

Mukherjee, A., Joshi, M. 2005. FRPC reinforced concrete beam-column joints under cyclic excitation. *Composite Structure*, **70**(2): 185–99.

Nagarajaiah, S., Reinhorn, M.A., Constantinou, M.C. 1991. Nonlinear dynamic analysis of 3-D base-isolated structures. *ASCE Journal of Structural Engineering*, **117**(7): 1666-1682.

Nagarajaiah, S., Reinhorn, M.A., Constantinou, M.C. 1992. Experimental study of sliding isolated structures with uplift restraint. *ASCE Journal of Structural Engineering*, **118**(6): 1666-1682.

Nagarajaiah, S., Reinhorn, M.A., Constantinou, M.C. 1993. Torsional coupling in sliding base-isolated structures. *ASCE Journal of Structural Engineering*, **119**(1): 130-149.

Nanni, A., and Braford, N.M. 1995. FRP jacketed concrete under uniaxial compression. *Construction and Building Material*, **9**(2): 115–124.

Nanni, A., and Norries, M.S. 1995. FRP jacketed concrete under flexure and combined flexure-compression. *Construction and Building Material*, **9**(5): 273–281.

Pantazopoulou, S.J., and Mills, R.H. 1995. Microstructural aspects of the mechanical response of plain concrete. *ACI Materials Journal*, **92**(6): 605–16.

Park, J. 2004. *Development and Application of Probabilistic Decision Support Framework for Seismic Rehabilitation of Structural Systems*. Ph.D. Thesis, Department of Civil and Environmental Engineering, Georgia Institute of Technology, Atlanta, Georgia.

Park, R., and Paulay, T. 1975. *Reinforced Concrete Structures*. New York: John Wiley & Sons, pages 769.

Park, Y.J., and Ang, A.H.S. 1985. Mechanistic seismic damage model for reinforced concrete. *ASCE Journal of Structural Engineering*, **111**(4): 722-739.

Pincheira, J.A., and Jirsa, J.O. 1995. Seismic response of RC frames retrofitted with steel braces or walls. *ASCE Journal of Structural Engineering*, **121**(8): 1225–1235.

Porter, K.A., and Kiremidjian, A.S. 2001. *Assembly-Based Vulnerability of Buildings and Its Uses in Seismic Performance Evaluation and Risk Management Decision-making*, Technical Report No. 309. Stanford, CA: John A. Blume Earthquake Engineering Center, Stanford University.

Porter, K.A., Beck J.L., and Shaikhutdinov R.V. 2002. Sensitivity of building loss estimates to major uncertain variables. *Earthquake Spectra*, **18**(4): 719-743.

Prakash, V., Powell, G.H., and Campbell, S. 1994. *Drain–3DX base Program Description and User Guide, and Element Description Version 1.10*. Structural engineering mechanics and materials report no. UCB/SEMM-94-07 and 94-08. Berkeley: University of California; August.

Prakash, V., Powell, G.H., Campbell, S. 1993. *Drain–2DX Base Program Description and User Guide Version 1.10*. Structural engineering mechanics and materials report no. UCB/SEMM-93-18. Berkeley: University of California; November.

Priestley, M.J.N. Seible, F., and Calvi, G.M.S. 1996. *Seismic Design and Retrofit of Bridges*. New York: John Wiley & Sons.

Priestley, M.J.N., Seible, F., Xiao, Y., and Verma, R. 1994. Steel jacket retrofitting of reinforced concrete bridge columns for enhanced shear strength. Part 2: test results and comparison with theory. *ACI Materials Journal*, **91**(5): 537–551.

Prota, A., Nanni, A., Manfredi, G., and Cosenza, E. 2001. Seismic upgrade of beam–column joints with FRP reinforcement. In: *FRPRCS5 Proceedings of 5th Non-Metallic Reinforcement for Concrete Structures*, Thomas Telford Ltd, UK, p. 339.

RAM International. *Perform-2D*. West Carlsbad, CA 92008 at http://www.ramint.com.

Richart, F.E., Brandtzaeg, A., and Brown, R.L. 1928. *A Study of the Failure of Concrete Under Combined Compressive Stresses*. Engineering Experiment Station, University of Illinois, Urbana, Ill.

Richart, F.E., Brandtzaeg, A., and Brown, R.L. 1929. *The Failure of Plain and Spirally Reinforced Concrete in Compression*. Bulletin No. 190, Engineering Experiment Station, University of Illinois, Urbana, Ill.

Rissman and Rissman Associates. 1965. *Holiday Inn Van Nuys Structural Drawings*, Pacific Palisades, CA.

Robinson, D., Fulford, G., and Dhu, T. 2005. *EQRM: Geoscience Australia's Earthquake Risk Model*. Geoscience Australia Record 2005/01,Geoscience Australia, p.151.

Rodriguez, M., and Park, R. 1991. Repair and strengthening of reinforced concrete buildings for seismic resistance. *Earthquake Spectra*, **7**(3): 439–459.

Scholl, R.E., Kustu, O., Perry, C.L., and Zanetti, J.M. 1982. *Seismic Damage Assessment for High-rise Building*. San Francisco, CA: URS / John A. Blume & Associates, Inc.

Seible, F., Priestly, M.J.N., Hegemier, G.A., and Innamortato, D. 1997. Seismic retrofit of RC columns using continuous carbon fibre jackets. *ASCE Journal of Composite Construction*, 1(2): 52-56.

Sengezer, B., Ansal, A., and Bilen, O. 2008. Evaluation of parameters affecting earthquake damage by decision tree techniques. *Natural Hazards*, **47**: 547–568.

Sheikh, S.A., and Yau, G. 2002. Seismic behaviour of concrete columns confined with steel and fibre-reinforced polymers. *ACI Structural Journal*, **99**: 72-80.

Shitindi, R.V., Kato, Y., Hattori, A., and Miyagawa, T. 1998. Stress strain behaviour of concrete confined by FRP. *Material Science Research Institute*, **4**(3): 163–170.

Singhal, A., and Kiremidjian, A.S. 1996. *A Method for Earthquake Motion-damage Relationships with Application to Reinforced Concrete Frames*, Technical Report No. 119. Stanford, CA: John A. Blume Earthquake Engineering Center, Stanford University.

Skinner, R.I., Robinson, W.H., Mc Verry, G.H. 1993. *An Introduction to Seismic Isolation*. First Edition, John Wiley and Sons, England.

Somerville, P. and Collins, N. 2002. *Ground Motion Time Histories For The Van Nuys Building*, Prepared for the PEER Methodology Testbeds Project, URS Corporation, Pasadena, CA. http://www.peertestbeds.net/VNY/PrelimVanNuysHazardReport.pdf.

Spoelstra, M.R., and Monti, G. 1999. FRP-confined concrete model. *Journal of Composite Construction*, **3**(3): 143–150.

Steinbrugge, K.V., Schader, E.E., and Moran, D.F. 1975. *Building Damage in San Fernando Valley*. San Fernando, California, Earthquake of 9 February 1971. G. B. Oakeshot, Ed. Sacramento, California Department of Conservation, Division of Mines and Geology.

Stoppenhagen, D.R., Jirsa, J.O., and Wyllie, Jr. L.A. 1995. Seismic repair and strengthening of a severely damaged concrete frame. *ACI Structural Journal*, **92**(2): 177–187.

Su, L, Ahmadi, G., and Tadjbakhsh, I. G. 1990. A comparative study of performance of various base isolation systems, Part II: sensitivity analysis. *Earthquake Engineering and Structural Dynamics*, **19**: 21-33.

Sugano, S. 1980. Aseismic strengthening of existing reinforced concrete buildings, in: *Proceedings of the First Seminar on Repair and Retrofit of Structures*, US/Japan Cooperative Earthquake Engineering Research Program.

Sugano, S. 1989. Study of the seismic behaviour of retrofitted RC buildings. Seismic Engineering, Research and Practice, *Proceedings of the sessions related to seismic engineering at the Structures Congress*, ASCE, pp. 517–526.

Taciroglu, E., and Khalili-Tehrani, P. 2008. *M7.8 Southern San Andreas Fault Earthquake Scenario: Non-ductile Reinforced Concrete Building Stock*. The ShakeOut Scenario Supplemental Study Prepared for United States Geological Survey (USGS) and California Geological Survey (CGS).

Taghdi, M., Bruneau, M., and Saatcioglu, M. 1998. Seismic retrofit of non-ductile concrete and masonry walls by steel-strips bracing, *11th European Conference on Earthquake Engineering Balkema*, Rotterdam. Retrieved from http://www.eng.buffalo.edu.

Teng, J.G., and Lam, L. 2002. Compressive behavior of carbon fiber reinforced polymer-confined concrete in elliptical columns. *Journal of Structural Engineering*, **128**(12), 1535–1543.

Tesfamariam, S. and Saatcioglu, M. 2010. Seismic vulnerability assessment of reinforced concrete buildings using hierarchical fuzzy rule base modeling. *Earthquake Spectra*, **26**(1): 235-256.

Tesfamariam, S., and Saatcioglu, M. 2008. Risk-based seismic evaluation of reinforced concrete buildings. *Earthquake Spectra*, **24**(3): 795-821.

Tesfamariam, S., Sadiq, R. and Najjaran, H. 2010. Decision-making under uncertainty – an example for seismic risk management. *Risk Analysis*, **30**(1): 78-94.

Thermou, G. and Elnashai, A.S. 2002. *Performance Parameters and Criteria for Assessment and Rehabilitation*. Seismic Performance Evaluation and Retrofit of Structures (SPEAR), European Earthquake Engineering Research Network Report, Imperial College, UK.

Thermou, G.E., and Elnashai, A.S. 2006. Seismic retrofit schemes for RC structures and local–global consequences. *Progress in Structural Engineering and Materials*, **8**(1): 1–15.

Thiel, C.C., Willsea, F.J., and Singh, J.P. 1991. Breaking the Pattern: a research and development plan to improve seismic retrofit practices for government buildings, California Seismic Safety Commission.

Tsonos, A.G. 2002. Seismic repair of exterior RC beam-to-column joints using two-sided and three-sided jackets. *Structural Engineering and Mechanics*, **31**(1): 17–34.

Tsonos, A.G. 2008. Effectiveness of CFRP-jackets and RC-jackets in post-earthquake and pre-earthquake retrofitting of beam–column subassemblages. *Engineering Structures*, **30**(3): 777–793.

Turk, A.M. 1998. *Rehabilitation of Reinforced Concrete Infill Walls*. PhD thesis, Bogazici University, Istanbul, Turkey, p. 213.

UNIDO-Building construction Under Seismic Conditions in the Balkan Region. Post-earthquake damage evaluation and strength assessment of buildings under seismic conditions. United Nations Industrial Developing Organization, UNDP/ UNIDO Project REP/79/015, Vol. 4, Vienna, Austria, 1983.

von Winterfeldt, D., Roselund, N., and Kituse, A. 2000. *Framing Earthquake Retrofitting Decisions: The Case of Hillside Homes in Los Angeles*, PEER Report 2000/03, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA.

Wen, Y.K., Ellingwood, B.R., Bracci, J.M. 2004. *Vulnerability Function Framework for Consequence-Based Engineering*. Mid-America earthquake center project DS-4 report. Urbana (IL).

Wen, Y.K., Ellingwood, B.R., Veneziano, D. and Bracci, J.M. 2003. *Uncertainty Modeling in Earthquake Engineering*. Mid-America Earthquake (MAE) Center Project FD-2 Report, 12 Feb. 2003, Urbana, IL.

Willis, C.R., Yang, Q., Seracino, R. Griffith, M.C. 2009. Damaged masonry walls in two-way bending retrofitted with vertical FRP strips. *Construction and Building Material*, **23**: 1591-1604.

Wosser, T.D., Campi, D.E., Fovinci, M.A., and Smith, W.H. 1982. Damage to Engineered Structures in California. The Imperial Valley, California, Earthquake of October 15, 1979. Ed. Washington, D.C., US GPO.

Wu, G., Lu, Z., and Wu, Z. 2003. Stress–strain relationship for FRP confined concrete cylinders. *Proceedings 6th Int. Symp. on Fibre-Reinforced Polymer (FRP) Reinforcement for Concrete Structures (FRPRCS-6)*, K. H. Tan, ed., Vol. 1, World Scientific, Singapore, 551–560.

Xiao, Y., and Wu, H. 2000. Compressive behavior of concrete confined by carbon fiber composite jackets. *Journal of Material in Civil Engineering*, **12**(2): 139–146.

Yaghoubian, J. 1991. Isolation Building Contents from Earthquake Induced Floor Motions. *Earthquake Spectra*, **7**(1): 127-143.

Yamamoto, Y., and Umemura, H. 1992. Analysis of reinforced concrete frames retrofitted with steel braces. *Proceedings of the 10th World Conference on Earthquake Engineering*, Madrid, Spain, Vol. 9, pp. 5187–5192.

Yankelevsky, D.Z., and Reinhardt, H.W. 1989. Uniaxial behavior of concrete in cyclic tension, *ASCE Journal of Structural Engineering*, **115**(1): 166-182.

Youssef, N. 1995. Behavior Characteristics of Reinforced Concrete Structures in Northridge Earthquake, California 1994. *ASCE Structures Congress*.

Zayas, V., Piepenbrock, T., and Al-Hussaini, T. 1993. Summary of Testing of the Friction Pendulum Seismic Isolation System, 1986-1993. *Proceedings of ATC 17-1 Seminar on Seismic Isolation, Passive Energy Dissipation, and Active Control*, San Francisco, California, pp. 337-338.

Zekioglu, A., Darama, H. and Erkus, B. 2009. Performance-based seismic design of a large seismically isolated structure: Istanbul Sabiha Gökçen International Airport Terminal Building. *SEAOC 2009 Convention Proceedings*, p. 409-427.

Appendix A: Inventory and loss estimation data for HAZUS-MH (2003) manual

Table A.1. Repair and replacement ratio for structural damage (Reproduced HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Structural Damage State			
			Slight	Moderate	Extensive	Complete
		Residential				
1	RES1	Single Family Dwelling	0.5	2.3	11.7	23.4
2	RES2	Mobile Home	0.4	2.4	7.3	24.4
3-8	RES3a-f	Multi Family Dwelling	0.3	1.4	6.9	13.8
9	RES4	Temporary Lodging	0.2	1.4	6.8	13.6
10	RES5	Institutional Dormitory	0.4	1.9	9.4	18.8
11	RES6	Nursing Home	0.4	1.8	9.2	18.4
		Commercial				
12	COM1	Retail Trade	0.6	2.9	14.7	29.4
13	COM2	Wholesale Trade	0.6	3.2	16.2	32.4
14	COM3	Personal and Repair Services	0.3	1.6	8.1	16.2
15	COM4	Professional/Technical/ Business Services	0.4	1.9	9.6	19.2
16	COM5	Banks/Financial Institutions	0.3	1.4	6.9	13.8
17	COM6	Hospital	0.2	1.4	7.0	14.0
18	COM7	Medical Office/Clinic	0.3	1.4	7.2	14.4
19	COM8	Entertainment & Recreation	0.2	1.0	5.0	10.0
20	COM9	Theaters	0.3	1.2	6.1	12.2
21	COM10	Parking	1.3	6.1	30.4	60.9
		Industrial				
22	IND1	Heavy	0.4	1.6	7.8	15.7
23	IND2	Light	0.4	1.6	7.8	15.7
24	IND3	Food/Drugs/Chemicals	0.4	1.6	7.8	15.7
25	IND4	Metals/Minerals Processing	0.4	1.6	7.8	15.7
26	IND5	High Technology	0.4	1.6	7.8	15.7
27	IND6	Construction	0.4	1.6	7.8	15.7
		Agriculture				
28	AGR1	Agriculture	0.8	4.6	23.1	46.2
		Religion/Non-Profit				
29	REL1	Church/Membership Organization	0.3	2.0	9.9	19.8
		Government				
30	GOV1	General Services	0.3	1.8	9.0	17.9
31	GOV2	Emergency Response	0.3	1.5	7.7	15.3
		Education				
32	EDU1	Schools/Libraries	0.4	1.9	9.5	18.9
33	EDU2	Colleges/Universities	0.2	1.1	5.5	11.0

Table A.2 Repair and replacement ratio for acceleration sensitive non-structural damage (Reproduced from HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Structural Damage State			
			Slight	Moderate	Extensive	Complete
		Residential				
1	RES1	Single Family Dwelling	0.5	2.7	8.0	26.6
2	RES2	Mobile Home	0.8	3.8	11.3	37.8
3-8	RES3a-f	Multi Family Dwelling	0.8	4.3	13.1	43.7
9	RES4	Temporary Lodging	0.9	4.3	13.0	43.2
10	RES5	Institutional Dormitory	0.8	4.1	12.4	41.2
11	RES6	Nursing Home	0.8	4.1	12.2	40.8
		Commercial				
12	COM1	Retail Trade	0.8	4.4	12.9	43.1
13	COM2	Wholesale Trade	0.8	4.2	12.4	41.1
14	COM3	Personal and Repair Services	1.0	5	15	50
15	COM4	Professional/Technical/ Business Services	0.9	4.8	14.4	47.9
16	COM5	Banks/Financial Institutions	1.0	5.2	15.5	51.7
17	COM6	Hospital	1.0	5.1	15.4	51.3
18	COM7	Medical Office/Clinic	1.0	5.2	15.3	51.2
19	COM8	Entertainment & Recreation	1.1	5.4	16.3	54.4
20	COM9	Theaters	1.0	5.3	15.8	52.7
21	COM10	Parking	0.3	2.2	6.5	21.7
		Industrial				
22	IND1	Heavy	1.4	7.2	21.8	72.5
23	IND2	Light	1.4	7.2	21.8	72.5
24	IND3	Food/Drugs/Chemicals	1.4	7.2	21.8	72.5
25	IND4	Metals/Minerals Processing	1.4	7.2	21.8	72.5
26	IND5	High Technology	1.4	7.2	21.8	72.5
27	IND6	Construction	1.4	7.2	21.8	72.5
		Agriculture				
28	AGR1	Agriculture	0.8	4.6	13.8	46.1
		Religion/Non-Profit				
29	REL1	Church/Membership Organization	0.9	4.7	14.3	47.6
		Government				
30	GOV1	General Services	1.0	4.9	14.8	49.3
31	GOV2	Emergency Response	1.0	5.1	15.1	50.5
		Education				
32	EDU1	Schools/Libraries	0.7	3.2	9.7	32.4
33	EDU2	Colleges/Universities	0.6	2.9	8.7	29.0

Note: damage ratio is expressed in term of percentage of building replacement value.

Table A.3. Repair and replacement ratio for drift sensitive non-structural damage (Reproduced from HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Structural Damage State			
			Slight	Moderate	Extensive	Complete
		Residential				
1	RES1	Single Family Dwelling	0.5	2.7	8.0	26.6
2	RES2	Mobile Home	0.8	3.8	11.3	37.8
3-8	RES3a-f	Multi Family Dwelling	0.8	4.3	13.1	43.7
9	RES4	Temporary Lodging	0.9	4.3	13.0	43.2
10	RES5	Institutional Dormitory	0.8	4.1	12.4	41.2
11	RES6	Nursing Home	0.8	4.1	12.2	40.8
		Commercial				
12	COM1	Retail Trade	0.8	4.4	12.9	43.1
13	COM2	Wholesale Trade	0.8	4.2	12.4	41.1
14	COM3	Personal and Repair Services	1.0	5	15	50
15	COM4	Professional/Technical/ Business Services	0.9	4.8	14.4	47.9
16	COM5	Banks/Financial Institutions	1.0	5.2	15.5	51.7
17	COM6	Hospital	1.0	5.1	15.4	51.3
18	COM7	Medical Office/Clinic	1.0	5.2	15.3	51.2
19	COM8	Entertainment & Recreation	1.1	5.4	16.3	54.4
20	COM9	Theaters	1.0	5.3	15.8	52.7
21	COM10	Parking	0.3	2.2	6.5	21.7
		Industrial				
22	IND1	Heavy	1.4	7.2	21.8	72.5
23	IND2	Light	1.4	7.2	21.8	72.5
24	IND3	Food/Drugs/Chemicals	1.4	7.2	21.8	72.5
25	IND4	Metals/Minerals Processing	1.4	7.2	21.8	72.5
26	IND5	High Technology	1.4	7.2	21.8	72.5
27	IND6	Construction	1.4	7.2	21.8	72.5
		Agriculture				
28	AGR1	Agriculture	0.8	4.6	13.8	46.1
		Religion/Non-Profit				
29	REL1	Church/Membership Organization	0.9	4.7	14.3	47.6
		Government				
30	GOV1	General Services	1.0	4.9	14.8	49.3
31	GOV2	Emergency Response	1.0	5.1	15.1	50.5
		Education				
32	EDU1	Schools/Libraries	0.7	3.2	9.7	32.4
33	EDU2	Colleges/Universities	0.6	2.9	8.7	29.0
Note: damage ratio is expressed in term of percentage of building replacement value.						

Table A.4. Contents damage ratios (Reproduced from HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Structural Damage State			
			Slight	Moderate	Extensive	Complete
		Residential				
1	RES1	Single Family Dwelling	1	5	25	50
2	RES2	Mobile Home	1	5	25	50
3-8	RES3a-f	Multi Family Dwelling	1	5	25	50
9	RES4	Temporary Lodging	1	5	25	50
10	RES5	Institutional Dormitory	1	5	25	50
11	RES6	Nursing Home	1	5	25	50
		Commercial				
12	COM1	Retail Trade	1	5	25	50
13	COM2	Wholesale Trade	1	5	25	50
14	COM3	Personal and Repair Services	1	5	25	50
15	COM4	Professional/Technical/ Business Services	1	5	25	50
16	COM5	Banks/Financial Institutions	1	5	25	50
17	COM6	Hospital	1	5	25	50
18	COM7	Medical Office/Clinic	1	5	25	50
19	COM8	Entertainment & Recreation	1	5	25	50
20	COM9	Theaters	1	5	25	50
21	COM10	Parking	1	5	25	50
		Industrial				
22	IND1	Heavy	1	5	25	50
23	IND2	Light	1	5	25	50
24	IND3	Food/Drugs/Chemicals	1	5	25	50
25	IND4	Metals/Minerals Processing	1	5	25	50
26	IND5	High Technology	1	5	25	50
27	IND6	Construction	1	5	25	50
		Agriculture				
28	AGR1	Agriculture	1	5	25	50
		Religion/Non-Profit				
29	REL1	Church/Membership Organization	1	5	25	50
		Government				
30	GOV1	General Services	1	5	25	50
31	GOV2	Emergency Response	1	5	25	50
		Education				
32	EDU1	Schools/Libraries	1	5	25	50
33	EDU2	Colleges/Universities	1	5	25	50
Note: damage ratio is expressed in term of percentage of building replacement value.						

Table A.5. Building repair and clean-up time (Time in days) (Reproduced from HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Structural Damage State				
			None	Slight	Moderate	Extensive	Complete
		Residential					
1	RES1	Single Family Dwelling	0	5	120	360	720
2	RES2	Mobile Home	0	5	20	120	120
3-8	RES3a -f	Multi Family Dwelling	0	10	120	480	960
9	RES4	Temporary Lodging	0	10	90	360	480
10	RES5	Institutional Dormitory	0	10	90	360	480
11	RES6	Nursing Home	0	10	120	480	960
		Commercial					
12	COM1	Retail Trade	0	10	90	270	360
13	COM2	Wholesale Trade	0	10	90	270	360
14	COM3	Personal and Repair Services	0	10	90	270	360
15	COM4	Professional/Technical/ Business Services	0	20	90	360	480
16	COM5	Banks/Financial Institutions	0	20	90	180	360
17	COM6	Hospital	0	20	135	540	720
18	COM7	Medical Office/Clinic		20	135	270	540
19	COM8	Entertainment & Recreation	0	20	90	180	360
20	COM9	Theaters	0	20	90	180	360
21	COM1 0	Parking	0	5	60	180	360
		Industrial					
22	IND1	Heavy	0	10	90	240	360
23	IND2	Light	0	10	90	240	360
24	IND3	Food/Drugs/Chemicals	0	10	90	240	360
25	IND4	Metals/Minerals Processing	0	10	90	240	360
26	IND5	High Technology	0	20	135	360	540
27	IND6	Construction	0	10	60	160	320
		Agriculture					
28	AGR1	Agriculture	0	2	20	60	120
		Religion/Non-Profit					
29	REL1	Church/Membership Organization	0	5	120	480	960
		Government					
30	GOV1	General Services	0	10	90	360	480
31	GOV2	Emergency Response	0	10	60	270	360
		Education					
32	EDU1	Schools/Libraries	0	10	90	360	480
33	EDU2	Colleges/Universities	0	10	120	480	960

Table A.6. Multipliers for cost estimates of building and service interruption time (Reproduced from HAZUS (FEMA, 2003)

No.	Label	Occupancy Class	Structural Damage State				
			None	Slight	Moderate	Extensive	Complete
		Residential					
1	RES1	Single Family Dwelling	0	0	0.5	1.0	1.0
2	RES2	Mobile Home	0	0	0.5	1.0	1.0
3-8	RES3a-f	Multi Family Dwelling	0	0	0.5	1.0	1.0
9	RES4	Temporary Lodging	0	0	0.5	1.0	1.0
10	RES5	Institutional Dormitory	0	0	0.5	1.0	1.0
11	RES6	Nursing Home	0	0	0.5	1.0	1.0
		Commercial					
12	COM1	Retail Trade	0.5	0.1	0.1	0.3	0.4
13	COM2	Wholesale Trade	0.5	0.1	0.2	0.3	0.4
14	COM3	Personal and Repair Services	0.5	0.1	0.2	0.3	0.4
15	COM4	Professional/Technical/ Business Services	0.5	0.1	0.1	0.2	0.3
16	COM5	Banks/Financial Institutions	0.5	0.1	0.05	0.03	0.03
17	COM6	Hospital	0.5	0.1	0.5	0.5	0.5
18	COM7	Medical Office/Clinic	0.5	0.1	0.5	0.5	0.5
19	COM8	Entertainment & Recreation	0.5	0.1	1.0	1.0	1.0
20	COM9	Theaters	0.5	0.1	1.0	1.0	1.0
21	COM10	Parking	0.1	0.1	1.0	1.0	1.0
		Industrial					
22	IND1	Heavy	0.5	0.5	1.0	1.0	1.0
23	IND2	Light	0.5	0.1	0.2	0.3	0.4
24	IND3	Food/Drugs/Chemicals	0.5	0.2	0.2	0.3	0.4
25	IND4	Metals/Minerals Processing	0.5	0.2	0.2	0.3	0.4
26	IND5	High Technology	0.5	0.2	0.2	0.3	0.4
27	IND6	Construction	0.5	0.1	0.2	0.3	0.4
		Agriculture					
28	AGR1	Agriculture	0	0	0.05	0.1	0.2
		Religion/Non-Profit					
29	REL1	Church/Membership Organization	1	0.2	0.05	0.03	0.03
		Government					
30	GOV1	General Services	0.5	0.1	0.02	0.03	0.03
31	GOV2	Emergency Response	0.5	0.1	0.02	0.03	0.03
		Education					
32	EDU1	Schools/Libraries	0.5	0.1	0.02	0.05	0.05
33	EDU2	Colleges/Universities	0.5	0.1	0.02	0.03	0.03

Table A.7. Proprietor's income ((Reproduced from HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Income		Wages per Square Foot per Day	Employees Per Square Foot	Output per Square Foot per Day
			per Square Foot per Year	per Square Foot per Day			
		Residential					
1	RES1	Single Family Dwelling	0.000	0.0000	0.000	0.000	0.000
2	RES2	Mobile Home	0.000	0.0000	0.000	0.000	0.000
3-8	RES3a-f	Multi Family Dwelling	0.000	0.0000	0.000	0.000	0.000
9	RES4	Temporary Lodging	32.065	0.088	0.206	0.003	0.46
10	RES5	Institutional Dormitory	0.000	0.000	0.000	0.000	0.000
11	RES6	Nursing Home	53.442	0.146	0.345	0.005	0.767
		Commercial					
12	COM1	Retail Trade	19.785	0.054	0.189	0.004	0.401
13	COM2	Wholesale Trade	32.449	0.089	0.233	0.002	0.521
14	COM3	Personal and Repair Services	42.754	0.117	0.276	0.004	0.614
15	COM4	Professional/Technical/ Business Services	336.882	0.923	0.328	0.004	0.897
16	COM5	Banks/Financial Institutions	384.421	1.053	0.534	0.006	2.912
17	COM6	Hospital	53.442	0.146	0.345	0.005	0.767
18	COM7	Medical Office/Clinic	106.884	0.293	0.689	0.01	1.534
19	COM8	Entertainment & Recreation	196.013	0.537	0.427	0.007	0.967
20	COM9	Theaters	64.13	0.176	0.414	0.006	0.921
21	COM10	Parking	0	0	0	0	0
		Industrial					
22	IND1	Heavy	81.098	0.222	0.368	0.003	1.555
23	IND2	Light	81.098	0.222	0.368	0.003	1.555
24	IND3	Food/Drugs/Chemicals	108.131	0.296	0.492	0.004	2.073
25	IND4	Metals/Minerals Processing	245.687	0.673	0.38	0.003	1.645
26	IND5	High Technology	162.196	0.444	0.737	0.006	3.109
27	IND6	Construction	79.065	0.217	0.398	0.005	1.54
		Agriculture					
28	AGR1	Agriculture	75.031	0.206	0.081	0.004	0.767
		Religion/Non-Profit					
29	REL1	Church/Membership Organization	42.754	0.117	0.276	0.004	1.534
		Government					
30	GOV1	General Services	35.112	0.096	2.646	0.025	0.614
31	GOV2	Emergency Response	0	0	4.023	0.038	0.705
		Education					
32	EDU1	Schools/Libraries	53.442	0.146	0.345	0.005	2.973
33	EDU2	Colleges/Universities	106.884	0.293	0.689	0.01	4.518

Table A.8. Recapture factors
(Reproduced from HAZUS (FEMA, 2003)

Occupancy	Wage Recapture (%)	Employment Recapture (%)	Income Recapture (%)	Output Recapture (%)
RES2	0	0	0	0
RES3a-f	0	0	0	0
RES4	0	0	0	0
RES5	0.60	0.60	0.60	0.60
RES6	0.60	0.60	0.60	0.60
COM1	0.87	0.87	0.87	0.87
COM2	0.87	0.87	0.87	0.87
COM3	0.51	0.51	0.51	0.51
COM4	0.90	0.90	0.90	0.90
COM5	0.90	0.90	0.90	0.90
COM6	0.60	0.60	0.60	0.60
COM7	0.60	0.60	0.60	0.60
COM8	0.60	0.60	0.60	0.60
COM9	0.60	0.60	0.60	0.60
COM10	0.60	0.60	0.60	0.60
IND1	0.98	0.98	0.98	0.98
IND2	0.98	0.98	0.98	0.98
IND3	0.98	0.98	0.98	0.98
IND4	0.98	0.98	0.98	0.98
IND5	0.98	0.98	0.98	0.98
IND6	0.95	0.95	0.95	0.95
AGR1	0.75	0.75	0.75	0.75
REL1	0.60	0.60	0.60	0.60
GOV1	0.80	0.80	0.80	0.80
GOV2	0	0	0	0
EDU1	0.60	0.60	0.60	0.60
EDU2	0.60	0.60	0.60	0.60

Table A.9. Owner percentage of income (Reproduced from HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Percentage Owner Occupied
		Residential	
1	RES1	Single Family Dwelling	75
2	RES2	Mobile Home	85
3-8	RES3a-f	Multi Family Dwelling	35
9	RES4	Temporary Lodging	0
10	RES5	Institutional Dormitory	0
11	RES6	Nursing Home	0
		Commercial	
12	COM1	Retail Trade	55
13	COM2	Wholesale Trade	55
14	COM3	Personal and Repair Services	55
15	COM4	Professional/Technical/ Business Services	55
16	COM5	Banks/Financial Institutions	75
17	COM6	Hospital	95
18	COM7	Medical Office/Clinic	65
19	COM8	Entertainment & Recreation	55
20	COM9	Theaters	45
21	COM10	Parking	25
		Industrial	
22	IND1	Heavy	75
23	IND2	Light	75
24	IND3	Food/Drugs/Chemicals	75
25	IND4	Metals/Minerals Processing	75
26	IND5	High Technology	55
27	IND6	Construction	85
		Agriculture	
28	AGR1	Agriculture	95
		Religion/Non-Profit	
29	REL1	Church/Membership Organization	90
		Government	
30	GOV1	General Services	70
31	GOV2	Emergency Response	95
		Education	
32	EDU1	Schools/Libraries	95
33	EDU2	Colleges/Universities	90

Table A.10. Rental and disruption cost (Reproduced from HAZUS (FEMA, 2003))

No.	Label	Occupancy Class	Rental Cost ($\$/ft^2/month$)	Disruption Costs ($\$/ft^2$)
		Residential		
1	RES1	Single Family Dwelling	0.68	0.82
2	RES2	Mobile Home	0.48	0.82
3-8	RES3a-f	Multi Family Dwelling	0.61	0.82
9	RES4	Temporary Lodging	2.04	0.82
10	RES5	Institutional Dormitory	0.41	0.82
11	RES6	Nursing Home	0.75	0.82
		Commercial		
12	COM1	Retail Trade	1.16	1.09
13	COM2	Wholesale Trade	0.48	0.95
14	COM3	Personal and Repair Services	1.36	0.95
15	COM4	Professional/Technical/ Business Services	1.36	0.95
16	COM5	Banks/Financial Institutions	1.70	0.95
17	COM6	Hospital	1.36	1.36
18	COM7	Medical Office/Clinic	1.36	1.36
19	COM8	Entertainment & Recreation	1.70	N/A
20	COM9	Theaters	1.70	N/A
21	COM10	Parking	0.34	N/A
		Industrial		
22	IND1	Heavy	0.20	N/A
23	IND2	Light	0.27	0.95
24	IND3	Food/Drugs/Chemicals	0.27	0.95
25	IND4	Metals/Minerals Processing	0.20	0.95
26	IND5	High Technology	0.34	0.95
27	IND6	Construction	0.14	0.95
		Agriculture		
28	AGR1	Agriculture	0.68	0.68
		Religion/Non-Profit		
29	REL1	Church/Membership Organization	1.02	0.95
		Government		
30	GOV1	General Services	1.36	0.95
31	GOV2	Emergency Response	1.36	0.95
		Education		
32	EDU1	Schools/Libraries	1.02	0.95
33	EDU2	Colleges/Universities	1.36	0.95