UNIVERSITY OF GAZİANTEP
GRADUATE SCHOOL OF
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SEISMIC EVALUATION OF CONVENTIONAL AND VISCOELASTICALLY DAMPED BUILDINGS THROUGH FRAGILITY ANALYSIS

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Seismic Evaluation of Conventional and Viscoelastically Damped Buildings Through Fragility Analysis

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This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science.

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all materials and results that are not original in this work.

Nazlı Deniz ŞAHİN

ABSTRACT

SEISMIC EVALUATION OF CONVENTIONAL AND VISCOELASTICALLY DAMPED BUILDINGS THROUGH FRAGILITY ANALYSIS

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In this study, the seismic fragility based assessment of conventional and viscoelastically damped buildings was investigated. The effectiveness of viscoelastic dampers was evaluated for 5 and 12 story buildings. For the case study buildings, similar type of viscoelastic dampers was used and designed to provide the structure with two different effective damping ratios of 10% and 20%. In order to exhibit the performance of conventional and viscoelastically damped framed buildings, the fragility based seismic vulnerability analyses were carried out. The buildings were designed as: a) Case 1: Conventional moment resisting frame, b) Case 2: Frame with viscoelastic dampers providing supplemental effective damping ratio of 10%, and c) Case 3: Frame with viscoelastic dampers providing supplemental effective damping ratio of 20%. Thus, a total of six different buildings were examined as a case study. Nonlinear time history analyses were performed to develop structural fragility curves and nonstructural fragility curves for drift sensitive and acceleration sensitive components. For the seismic fragility assessment, a database including 15 natural earthquake ground motion records with markedly different characteristics was used. The analysis of the results shows that depending upon the effective damping ratio, frames designed with viscoelastic dampers have noticeably lower probability of exceedance of performance limit states in comparison to conventionally designed moment resisting frame system.

Keywords*:* Building, Fragility curves, Performance levels, Seismic reliability, Viscoelastic dampers

ÖZET

KIRILGANLIK ANALİZİ İLE GELENEKSEL VE VİSKOELASTİK SÖNÜMLEYİCİLİ BİNALARIN SİSMİK DEĞERLENDİRİLMESİ

ŞAHİN, Nazlı Deniz
Yüksek Lisans Tezi, İnşaat Mühendisliği Bölümü
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Bu çalışmada deprem etkisine maruz geleneksel ve viskoelastik sönümleyicili binaların hasar görebilirliği karşılaştırmalı olarak incelenmiştir. Viskoelastik sönümleyicinin etkinliği 5 ve 12 katlı örnek binalar üzerinde değerlendirilmiştir. Bu amaçla, taşıyıcı sistemi geleneksel moment aktaran çerçeve ve viskoelastik sönümleyicili çerçeve sistemli olarak tasarlanmış 5 ve 12 katlı çelik binaların tasarımında üç farklı durum dikkate alınmıştır. Bunlar a) Durum 1: Geleneksel moment aktaran çerçeve, b) Durum 2: Viskoelastik sönümleyicilere sahip çerçeve, etkin sönüm oranı %10 ve c) Durum 3: Viskoelastik sönümleyicilere sahip çerçeve, etkin sönüm oranı %20 koşullarını içermektedir. Böylece, araştırmada toplam 6 faklı bina üzerinde inceleme yapılmıştır. Yapısal ve yapısal olmayan (ötelemeye duyarlı ve ivmeye duyarlı birleşenler) sismik kırılganlık eğrilerinin oluşturulmasında doğrusal olmayan zaman tanım alanında dinamik analizler yapılmıştır. Sismik güvenilirlikle ilgili güçlü tahminler elde etmek için belirgin farklı özelliklere sahip 15 doğal deprem yer hareketi kaydı analizlerde kullanılmıştır. Elde edilen sonuçlara göre, etkili sönüm oranına bağlı olarak, viskoelastik sönümleyiciler ile tasarlanmış çerçeve sistemleri geleneksel moment aktaran çerçeve sistemlerine kıyasla seçilen performans seviyeleri için oldukça düşük aşılma olasılıkları göstermiştir.

Anahtar kelimeler: Bina, Kırılganlık eğrileri, Performans seviyeleri, Sismik güvenilirlik, Viskoelastik sönümleyiciler

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LIST OF SYMBOLS/ABBREVIATIONS

E	Young modulus in (MPa)
f_y	Yield stress of steel in (MPa)
ξ	Damping ratio
IM	Intensity measure of input ground seismic hazard
LS	Limit state
PGA	Peak ground acceleration
PGD	Permanent ground displacement
PGV	Peak ground velocity
S_a	Spectral acceleration
S_v	Velocity
S_d	Spectral displacement
T_1	First period of original frame
X	Lognormal distributed ground motion intensity parameter
β	Standard deviation of the natural logarithm of ground motion index of damage level
Φ	Standard normal cumulative distribution function
μ	Median value of ground motion index
$\bar{S}_{d,ds}$	Median value of spectral displacement of damage state
ds	Damage state
ε_{ds}	Lognormal random variable with unit median value
C	Damping constant
V	Velocity
K_d	Stiffness
$G'(w)$	Shear Storage
$G''(w)$	Shear loss moduli
A	Total shear area
t	Thickness of material
w	Forcing frequency
C_d	Damping coefficient
M	Earthquake magnitude

CHAPTER 1

INTRODUCTION

In the last few decades, there have been studies on innovative approaches additional to the conventional design approaches, in order to receive less earthquake input force and energy and to dissipate the energy with lower damage and deformation in the structural components. These innovative approaches focus on the materials and systems such as seismic base isolation and passive energy dissipation systems (Kelly, 1986; Hausner et al., 1997; Soong and Dargush, 1997; Symans et al, 2008). Different kinds of passive energy dissipation systems, for example viscoelastic dampers (VEDs), frictional dampers, metallic yield dampers, viscous fluid dampers, liquid dampers, and mass dampers are equipped in actual buildings to diminish the structural vibration due to strong wind and earthquake loads (Soong and Dargush, 1997). The idea behind these devices, usually through non load bearing elements, is that by adding them to a structure, its energy dissipation capacity is enhanced against moderate and strong earthquakes. This technology provides an alternative to the conventional earthquake-resistant design and has the potential for significantly reducing seismic risk without compromising the safety, reliability, and economy of the constructed facilities (Shukla and Datta, 1999).

In civil engineering applications, for years viscoelastic dampers have been demonstrated to be efficient, as it is utilized in the Center of World Trade in New York and Center of Columbia in Seattle to lessen the vibration resulting from the wind forces (Lee and Tsai, 1992). However, the use of viscoelastic dampers to reduce earthquake response in buildings is relatively new in comparison to the use of metallic and friction devices (Craig et al., 2002). Moreover, in the literature, more recent studies included experimental investigations by Asano et al. (2000) and Xua et al. (2004) and analytical investigations by Vulcano and Mazza (2000), Soda and Takahashi (2000), Tezcan and Uluca (2003), Singh and Chang (2009) and Karavasilis et al. (2011) were available, and those researches also propose that there is a potential for the utilization of viscoelastic dampers for the earthquake protection of building type structures.

One of the most appropriate approaches for assessing the effectiveness of an earthquake resistant structural system is through a reliability analysis. In the literature, there are some good examples of applying seismic reliability analysis for evaluating the performance of different kinds of steel structures such as special moment resisting framed systems with welded connections (Song and Ellingwood, 1999 a,b), steel frames with different seismic connections (Kinali and Ellingwood, 2007; Park and Kim, 2010), framed systems with metallic and friction dampers (Dimova and Hirata, 2000; Curadeli and Riera, 2004), original steel building retrofitted with hysteretic and linear viscous dampers (Wanitkorkul and Filiatrault, 2008) and steel framed systems with eccentric braces and buckling restrained braces (Lin et al., 2010; Güneyisi, 2011).

However, there are limited studies mainly focusing on the effectiveness of viscoelastic dampers on the seismic reliability of framed building. For instance, Guo et al. (2002) investigated the seismic vulnerability analysis of the hysteretic structures with added viscoelastic damper systems. The dynamic response of structures under unsystematic earthquake excitation was examined in the state space employing the stochastic response analysis and equal linearization procedure. Then, they proposed a framework for conducting the reliability analysis of structures with and without parameter randomnesses. This proposed reliability analysis procedure is applied to a ten-story hysteretic shear beam type structure with and without viscoelastic dampers. The structures have a target structural damping ratio of the structure as 15%. It is observed that the existence of uncertainties decreases the building reliability. However, the application of viscoelastic dampers of appropriate parameters considerably improves the building reliability.

1.1 Objectives of the Thesis

The main objective of the research presented here is to investigate the seismic fragility analysis of viscoelastically damped frame systems in comparison with that of a conventional moment resisting frame system. For this, steel framed buildings of

5 and 12 stories in height were designed as conventional moment resisting frame and viscoelastically damped frame systems having different supplemental effective damping ratios of up to 20%. Then, a series of nonlinear time history analyses were conducted for developing seismic fragility curves of case study steel frames by using fifteen natural ground motion records with different characteristics. The fragility analysis were carried out both considering the response of structural and non-structural components of the frame systems. Nonstructural components are classified as drift sensitive and acceleration sensitive. Thus, the fragility curves were constructed for both structural and nonstructural components, considering different performance limit states. Moreover, the seismic reliability analyses of these frames were performed, which lead to a more general conclusion about the effectiveness of viscoelastic damper systems under seismic effects.

1.2 Outline of the Thesis

Chapter 1-Introduction: Aims of the thesis are introduced.

Chapter 2-Literature review: The previous work in the area is reviewed and collected. For this, first of all, the research and development on fragility curves as well as their categorization in the literature are described. Then, the basic ideas and concepts of using passive energy dissipation systems, particularly for viscoelastic dampers are summarized.

Chapter 3-Case study: This chapter provides a description of the case study structures. The methodology used in the analysis and design of the conventional and viscoelastically damped structures is given. Moreover, the properties of the ground motion records utilized in the nonlinear time history analysis are described. Furthermore, constructing the fragility curves in terms of structural and nonstructural components for the conventional and viscoelastically damped buildings as well as the details of the seismic hazards and seismic risk evaluation used in this study is presented in this chapter.

Chapter 4-Discussion of the results: Results obtained from the fragility based assesment for the original buildings and the buildings designed with the viscoelastic dampers are given. The effect of primary parameters, namely the type and height of buildings, supplemental effective damping ratios, performance limit states, structural and nonstructural components is discussed. Morover, the discussion based on the results of the seismic hazards and seismic risk evaluation is presented in this chapter.

Chapter 5-Conclusion: Conclusions obtained from the results of this study are presented.

CHAPTER 2

LITERATURE REVIEW

2.1 Fragility Analysis

Fragility is described as the conditional likelihood of damage of a structural element or system for a specified set of demand variables. To forecast the fragility of structural components considering aleatory and epistemic randomness, the models of probabilistic capacity are used in a formulation (Gardoni et al., 2002). The models of fragility are utilized to predict the seismic risk acting on the parts of lifeline systems (i.e., oil, water and gas channels, power circulation schemes, networkof transportation, and buildings. They are acquired not only from the statistical investigation of observed damages during past seismic activities but also from the modeling of the earthquake performance of components and systems (Straub and Kiureghian, 2008).

Earthquakes may result in extensive direct and indirect losses in which structural damage lays an important role. One of the most imperative elements in determining the seismic failure to the structures is called as a fragility curve. The fragility curve for certain type of building structures are employed to stand for the likelihoods which the structural failures, under different intensity of seismic activity, go beyond particular damage levels. In other words, points on the seismic fragility curve present the possibility which the spectral displacement under certain level of shaking is higher than the displacement related to the certain damage level. Generally, the damage states are categorized into four different categories, including slight damage (SD), moderate damage (MD), extensive damage (ED), and complete damage (CD) (Cherng, 2001).

In the study of Mander (1999), the development of the fragility curves using the inherent uncertainty and randomness of the structural capacity against the ground acceleration demand for a sample bridge system are illustrated. Figure 2.1 demonstrates the probabilistic explanation of uncertainty and randomness in constructing fragility curves for the analysis of seismic vulnerability. In the figure, the capacity-demand acceleration-displacement spectra indicating randomness and/or uncertainty in structural behavior and ground acceleration response are given as well as the normalized fragility curve which gives an explanation for uncertainty and randomness in both demand and capacity is explained.

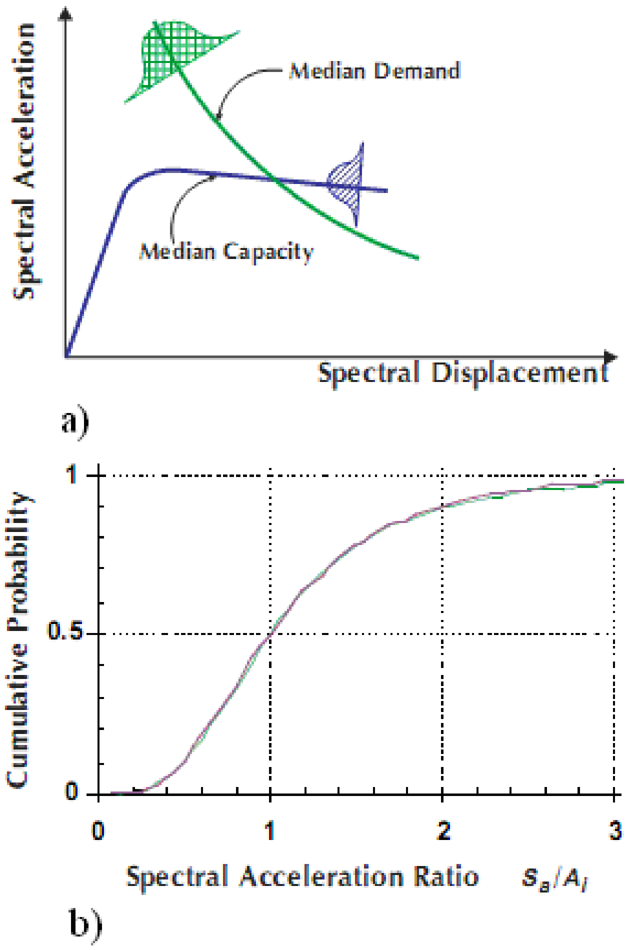

Figure 2.1 Generation of fragility curves for seismic vulnerability analysis (Mander, 1999)

Building failure formulations are in the type of the lognormal fragility curves that associate with the possibility of being in or above, a building failure level to for a certain potential earth science hazard (PESH) demand factor. An illustration of fragility curves for the different damage levels utilized in this method are presented Figure 2.2. The fragility model is described by means of a median value of the demand factor (i.e., peak ground acceleration, PGA or peak ground displacement, PGD) that relate to the threshold of the damage level and the variability in the damage levels. For instance, the spectral displacement, S_d, that describes the threshold of a particular damage level (ds) is considered to be distributed as follows (HAZUS, 1999).

$$S_d = \bar{S}_{d,ds} \times \varepsilon_{ds} \tag{2.1}$$

where $\bar{S}_{d,ds}$ is the median value of spectral displacement of failure level, ds, and

ε_{ds} is a lognormal random variable with unit median value and logarithmic standard deviation, β_{ds}.

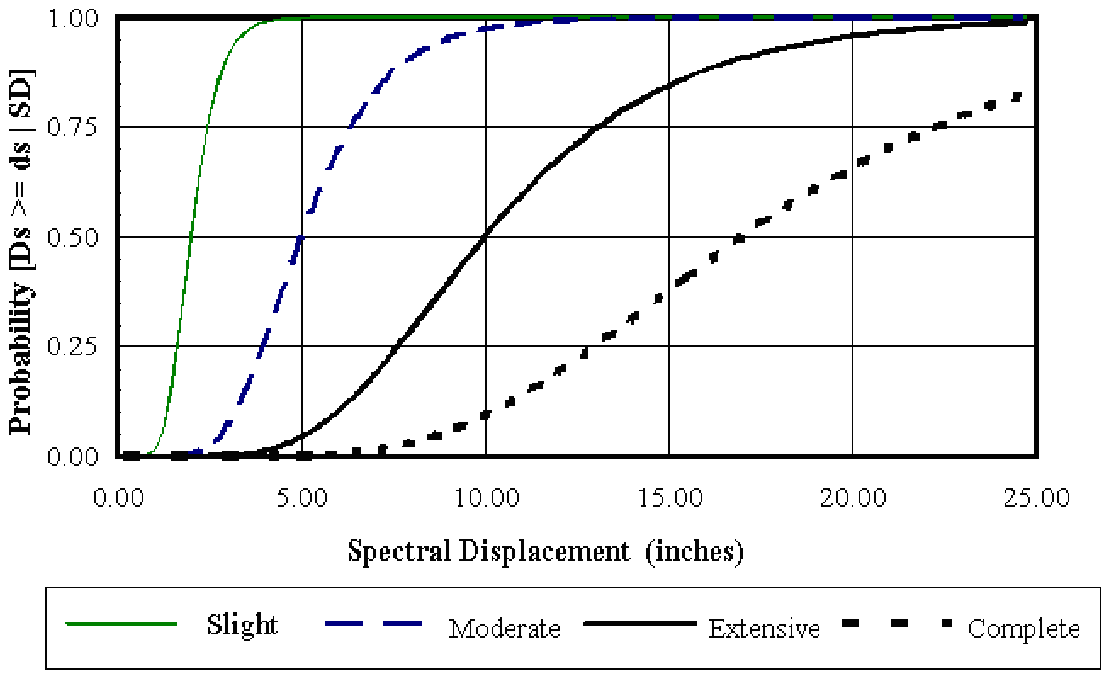

Figure 2.2 Illustration of the fragility curves for different damage levels (HAZUS, 1999)

As a common function of fragility curves, the lognormal standard deviation, β, are stated based on the randomness and/or uncertainty components of variability, β_R and β_U (Kennedy et al., 1980). The combined random variable expression, β, is utilized to generate a composite "best-estimate" fragility curve as it is not taken into account practical to separate uncertainty from randomness. The conditional probability of a particular damage state, ds, specified the spectral displacement, S_d, (or other PESH factor) is described by the formulation as mentioned in the guidelines of HAZUS (1999).

$$P\left[d_s \middle| S_d\right] = \Phi\left[\frac{1}{\beta_{ds}} \mathrm{In}\left(\frac{S_d}{\bar{S}_{d,ds}}\right)\right] \tag{2.2}$$

where: $\bar{S}_{d,ds}$ is the median value of spectral displacement at which the building reaches the threshold of damage state, ds,

β_{ds} is the standard deviation of the natural logarithm of spectral displacement for damage level, ds, and

Φ is the standard normal cumulative distribution function.

In the guidelines of HAZUS (1999), it is also reported that the median spectral displacement (or acceleration) parameters and the total variability are found for each building type and damage state by combining of performance value, earthquake experience data, expert opinion and judgment. Generally, the total variability of each damage level, β_{ds}, is determined by the following three contributors to damage variability. Each of these three contributors to damage state variability is considered to be lognormally distributed random variables.

- Uncertainty in the damage level threshold,
- Changeability in the response features of the model structures of interest, and
- Uncertainty in response owing to spatial variability of ground motion demand

The fragility curves are derived by a potential earth science hazard parameter. For ground failure, the potential earth science hazard factor utilized to make fragility curves is permanent ground displacement (PGD). For the ground shaking, the potential earth science hazard factor used to generate the fragility curves of building is peak spectral response (based on the displacement or the acceleration). Peak ground acceleration (PGA), rather than peak spectral displacement, is utilized to determine ground shaking caused structural failure to buildings that are parts of the lifelines. Peak spectral response alters considerably for structures that have dissimilar response characteristics. For example, tall, flexible buildings have a tendency to displace more than short, stiff buildings. Therefore, the evaluation of peak spectral displacement necessitates the information of the response features of the buildings (HAZUS, 1999).

Seismic vulnerability analysis through fragility curves ask for a probabilistic method with the purpose of thinking about the dissimilar uncertainties that influence the structures and earthquakes. Based on this intend, many statistical and stochastic approaches are available to evaluate the seismic reliability and the seismic risk of the structures. Comprehensively, the different models can be adopted so as to achieve fragility curves. These can be categorized in empirical, judgmental, analytical, and hybrid (Marano et al., 2011).

2.1.1 Empirical Fragility Curves

The seismic appraisal of the buildings at great geographical scales has been first conducted in the early 70's, through the utilization of empirical methods originally developed and calibrated as a formulation of macroseismic intensities. This is as a result of the fact that, at the time, the hazard maps were, in their enormous majority, described based on these discrete failure scales (previous attempts to relate intensity to physical quantities, such as PGA, brought about inappropriately big scatter).Thus, these empirical procedures comprised the only reasonable and feasible methods that could be initially utilized in the seismic risk analyses at a large scale (Calvi et al., 2006).

Empirical curves utilized the building damage circulations emphasized in post-earthquake surveys as their statistical foundation. The observational source is the most reasonable in the case of all practical details of the exposed stock are taken into consideration such as the effects of soil-structure interaction, topography, location, path, and source properties. On the other hand, similar features that render observational data the most sensible are responsible for the rigorous constraint in their application potential. The relationships of empirical ground motion damage created for European countries are classically founded on very few damage surveys conducted for particular sites or seismic events. No difference between buildings of dissimilar materials, heights or seismic design provisions is generally performed. As a result, the curves are extremely specific to a particular seismo-tectonic, geotechnical and built-environment. A wider range of application is probable if the performance of dissimilar structural systems is thought and if a big amount of trustworthy empirical data, including a broad collection of ground motions is utilized for the construction of curve. In actual fact, this is only attainable through the arrangement of data from disparate earthquakes and sites. Nevertheless, because of the infrequency of big magnitude seismic events close to densely populated regions, the observational data utilized for the curve derivation has a tendency to be scarce and greatly clustered in the low-damage, low ground-motion severity range. This result in big randomness being related with their use in large magnitude occasions. To permit a correct calculation of the ground motion and for the reported damage allocation to be representative of the distinction in earthquake resistance of the buildings, empirical vulnerability curves can utilize post-earthquake surveys conducted for big populations of buildings of similar construction, over regions of uniform soil states in close proximity to ground motion recording stations (Rossetto and Elnashai, 2003).

The development of the empirical fragility curves was inspired by the big amounts of inspection data got from, particularly, two recent urban seismic activities, that is to say the 1994 Northridge and 1995 Hyogoken Nanbu occasions (Mackie and Stojadinovic, 2004) Basöz and Kiremidjian (1997) and Basöz et al. (1999) generated the fragility curves for different observed damage levels from the Northridge and Loma Prieta data. Yamazaki et al. (1999) constructed similar curves using the Kobe event. Furthermore, illustrations of using the empirical approach in developing the fragility curves could be given in the studies of Shinozuka et al. (2000), O'Rourke and So (2000), Der Kiureghian (2002), and Osaki and Takada (2003).

For example, in the work of Shinozuka et al. (2000), a probabilistic analysis of structural fragility curves was carried out. The fragility curves based on empirical data were constructed employing bridge damage data attained from the 1995 Hyogo-ken Nanbu (Kobe) earthquake (for Hanshin Expressway Public Corporation's: HEPC's bridges). Combining the damage level data with that of the PGA, and doing utilization of the maximum probability approach concerning, three fragility curves were constructed together with values of the median and log-standard deviation. Figure 2.3 shows the fragility curves in terms of bridge column damage data from Kobe. The curve with a ‘‘minor’’ description represented, at each PGA value, the likelihood that ‘‘at least a minor’’ state of damage would be maintained by a bridge (randomly selected from the example of bridges) when it was exposed to PGA value. The equal sense applied to other curves with their respective damage level descriptions.

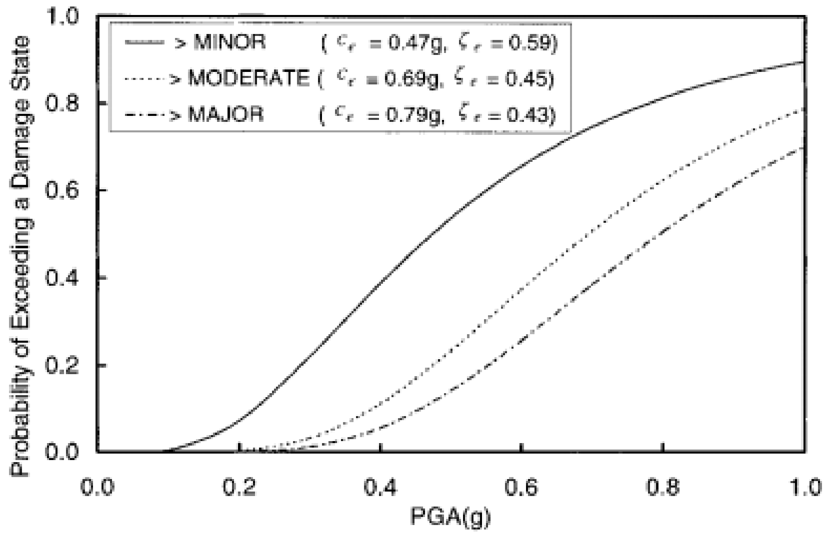

Figure 2.3 Empirical fragility curves for HEPC’s bridges (Shinozuka et al., 2000)

O’Rourke and So (2000) also developed the empirical fragility curves for on-grade steel tanks. The study reported the characterization of the earthquake behavior of cylindrical on grade, steel fluid storage tanks expose to the ground shaking. The performance was evaluated by fragility curves that obtained from an analysis of over 400 tanks in nine separate seismic occasions. The damage levels utilized to differentiate damage (such as slight, moderate, etc.) were proposed to mirror damage level explanations in the report of the HAZUS. The amount of seismic intensity was examined by the peak ground acceleration (PGA) at the location. The effect of height to diameter ratio (which is the property of the tanks studied) and the relative amount of stored contents were examined and were observed to be considerable factors on the seismic performance of the tank. as A final point, the fragility curves constructed in their study were evaluated to the corresponding relations presently available in the literature.

2.1.2 Judgmental Fragility Curves

Judgmental fragility curve, utilizing specialist attitude for the development of fragility formulations, is utilized commonly in the case where there is insufficient availiable data. In this method, the ideas of many experts could be employed as data to generate a likelihood circulation on response, conditioned on the vulnerability intensity. The ranging levels of information of the experts are acquired by being each one self-rate his or her knowledge. The benefit of this procedure is that it is versatile and does not need costly or unavailable failure data. The disadvantage to this method is that specialist view needs a technical foundation that does the data debatable (or prejudiced). One thing that might be difficult in getting reliable data is in the case of new states in that the specialists have no adequate practice (Erberik and Elnashai, 2003).

A judgmental method attempts to capture or explain the functionality states that could be allocated to the structure by inspectors for varied states of experiential damage. Even though this is a more subjective method, post-failure structure functionality is commonly given in this way (Nielson and DesRoches, 2007). As a result, specialist view is the major source employed by the majority existing retrofitting codes in the USA for the development of failure likelihood matrices and reliability curves (e.g. ATC-13 (1985) and ATC-40 (1996)). The vulnerability of judgmental curves is doubtful, but, owing to their dependence on the individual knowledge of the specialists. It is basically impracticable to appraise the level of conservatism related with the foundation of judgment, and intrinsic in the specialist vulnerability forecast is a thought of local structural kinds, representative arrangements, features and materials. Therefore, when the country of the fragility curve development is characterized by building practices which diverge considerably from those utilized in Europe, their uses might be disqualified (Rossetto and Elnashai, 2003).

2.1.3 Analytical Fragility Curves

In the lack of sufficient empirical information and in efforts to concentrate on a selection of various kinds and areas of the country at the same time as preserving homogeneity of the data, the analytical techniques have frequently been utilized to construct the fragility curves. In these analytical works, the demands and/or capacities of the structures utilized to appraise damage likelihood are predicted by such approaches as elastic spectral, non-linear static, and non-linear time history (Padgett and DesRoches, 2008).

Analytical reliability curves assumed failure distributions generated from the analyses of models under raising the seismic loads as their statistical foundation. Analyses of the structural models causes a diminished bias and enlarged reliability of the vulnerability predict for various structures in comparison to the attitude of the expert. Even though this, a small number of analytical vulnerability curves for reinforced concrete (RC) structures have been developed in the past because of the considerable computational attempt involved and constraints in the capabilities of the modeling. Architectural details could not be explained for modeling of infill partitions and intricate soil models could not be accommodated beside complicated structural models and rocking and elevating of structures poses serious mathematical modelling troubles. In addition, numerous available analyses have difficulties converging if the structures are exposed to big demands, and numerical crumple might lead the structural failure. But, the latest progresses have been preceded to the insertion of a number of response characteristics into analytical methods (i.e., shear-flexure-axial interaction, soil-structure interaction, interactive confinement on concrete elements and reinforcement buckling) Moreover, recent analysis methods and solution ways have been developed that facilitate the evaluation of damage data for big and intricate structures at very high rates. Thus, the analytical methods to fragility curve derivation are being very attractive based on the ease and efficiency by that information can be constructed, however, have not yet been completely utilized to the bounds of their potential (Rossetto and Elnashai, 2003).

An approach for the derivation of seismic intensity-damage relations, known as fragility curves and damage likelihood matrices was suggested in the study of

Dumova-Jovanoska (2000). The recommended technique was utilized on RC frame - wall structures. Two sets of fragility curves and damage likelihood matrices were attempted to develop by considering RC frame-wall structures less than or greater than 10 stories. For the former condition, a six-story frame structure was considered while for the later one, a 16-story frame-wall structure was employed as a case study building. During the design of the example structures, Macedonian design code was used. For the needs of this investigation, the program of IDARC-2D was utilized. That selection was taken into account due to fact that the program was primarily aimed for the analysis of failure owing to dynamic forces. There was a opportunity to describe numerous dissimilar factors of damage using this program. Figure 2.4 shows the methodology for defnition of fragility curves utilized in the current work. The key point of describing the seismic intensity- damage relationship is to predict the response of the structure to excitation under a specified intensity. However, two main explanations were considered, namely choice of an indicator of seismic intensity and choice of an indicator of response. As a result of the analytical work, the magnitudes of the global damage index associated to each damage level were computed. Based on the nonlinear dynamic analysis of the example structures, the fragility curves and damage probability matrices were defined. The fragility curves for RC frame-wall structures less than or greater than 10 stories are illustrated in Figures 2.5 and 2.6, respectively.

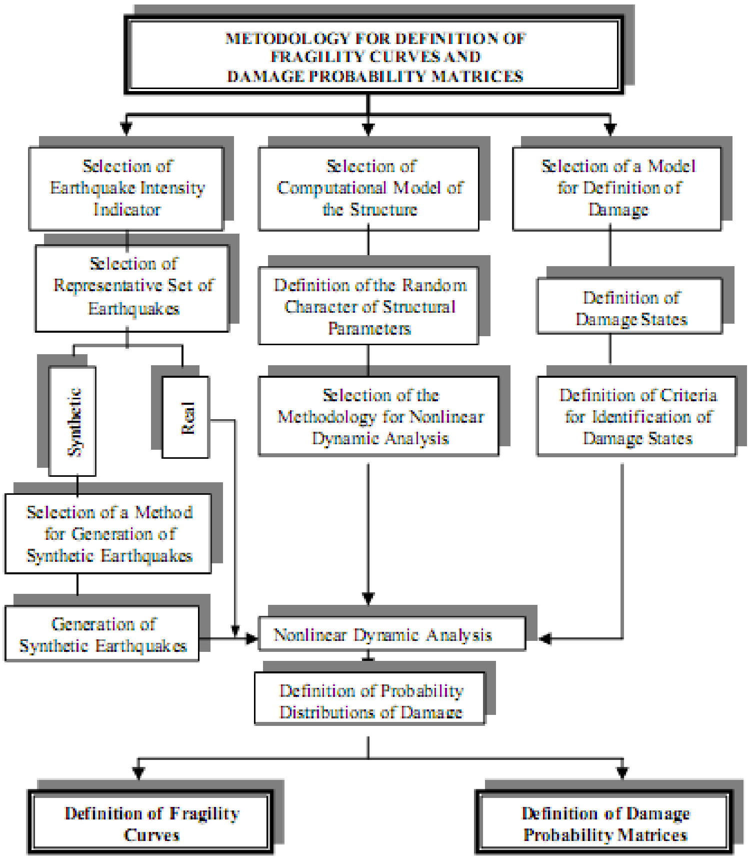

Figure 2.4 Flowchart of analytical vulnerability curves (Dumova-Jovanoska, 2000)

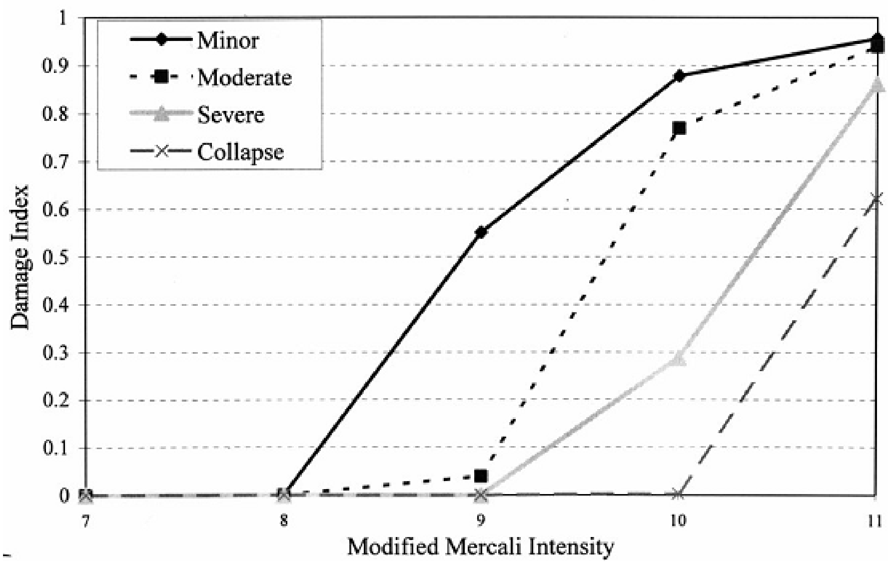

Figure 2.5 Fragility curves for RC structures lower than 10 stories ((Dumova-Jovanoska, 2000)

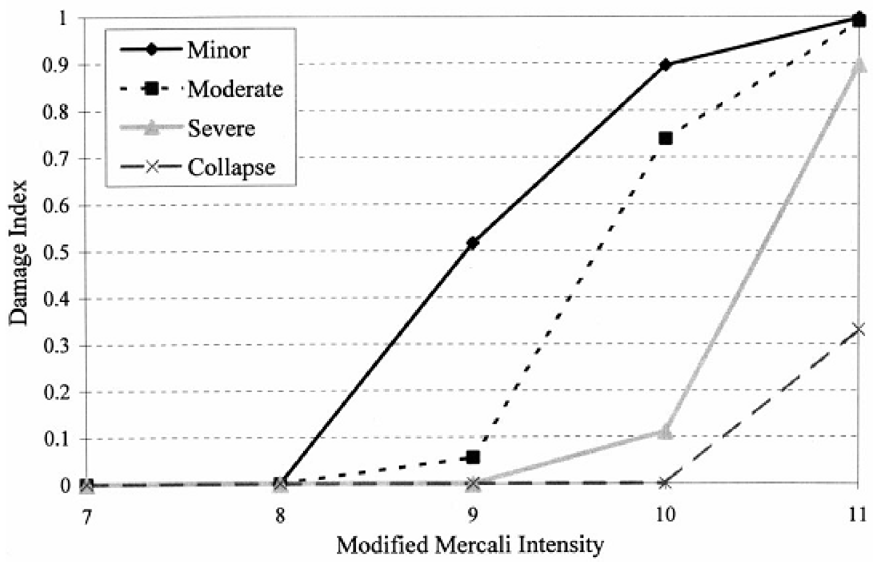

Figure 2.6 Fragility curves for RC frame-wal structures higher than 10 stories (Dumova-Jovanoska, 2000)

A recent study carried out by Ji et al. (2007) is in the field of an analytical framework for earthquake fragility analysis of reinforced concrete high-rise buildings. The main property of the presented work was the method for the derivation of a simple lumped-factor model representative of the intricate high-rise building classification. Aforementioned model was generated in the ZEUS–NL platform to make possible computationally effective dynamic analyses of high-rise structures which were formerly not achievable and could exactly give an explanation for the intricate behaviour and interactions forecasted by more comprehensive analytical models. Utilizing genetic algorithms, the factors for the model were decided. The derivation of a simple lumped-factor model was represented for an available high-rise structure with dual core walls and a RC frame. The precision of the individual parts of the suggested model was determined with the estimation of more comprehensive analytical models and example fragility curves were given. The recommended framework was commonly relevant for generating fragility formulations for high-rise RC building with frames and cores or walls. In Figure 2.7, the flowchart of the suggested framework for constructing analytically based fragility curves for RC buildings is given, that exemplifies the important characteristics. For instance, the choice of sample building structures, proper analytical modeling, uncertainties, and limit level descriptions. Morover, the complete frame model and typical cross-sections are presented in Figure 2.8.

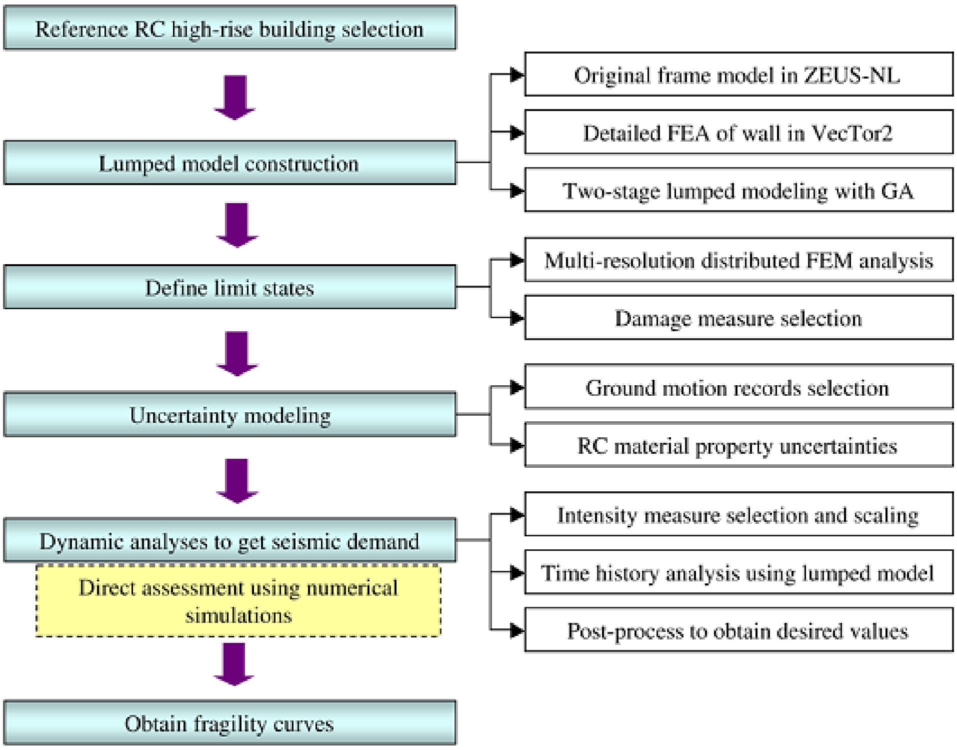

Figure 2.7 Proposed fragility assessment framework (Ji et al., 2007)

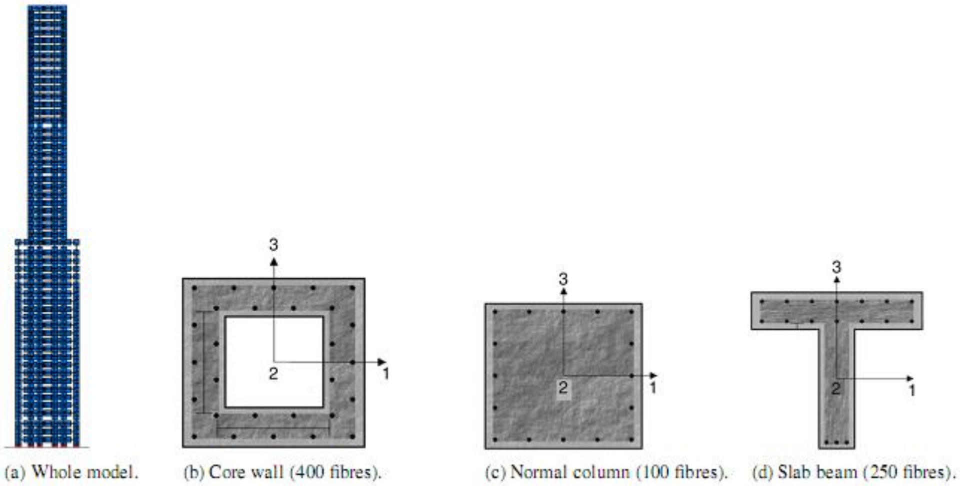

Figure 2.8 Structural model and typical cross-sections (Ji et al., 2007)

Ramanathan et al. (2002) developped an analytical fragility curves for multispan continuous steel girder bridges in moderate seismic zones. In their work, the comparison of fragility curves for seismically and nonseismically designed bridges that are common in the middle and south-eastern United States were done. It was reported that the primary differences between seismically and nonseismically designed bridges were the column details and bridge bearings. Detailed three dimensional (3-D) nonlinear analytical models in which the nonlinear behavior of the column, girders, and abutments, were developed with the use of the OpenSees platform. The fragility curves were obtained considering a set of ground accelerations, indicating the seismic hazard in the area. Unlike most previous studies, the fragility curves were developed with geometric variations such as column height, deck width, and length considered researching the effect of these variations on the fragility curves within the same class of bridges. The results provide insight into the uncertainty state introduced in the analysis of fragility for portfolios of bridges with the use of analytical models and nonlinear time-history analyses. Component and system fragility curves were obtained and were compared for the case of nonseismically and seismically designed bridges. The fragility curves for the seismically and nonseismically designed steel bridges are shown in Figure 2.9.

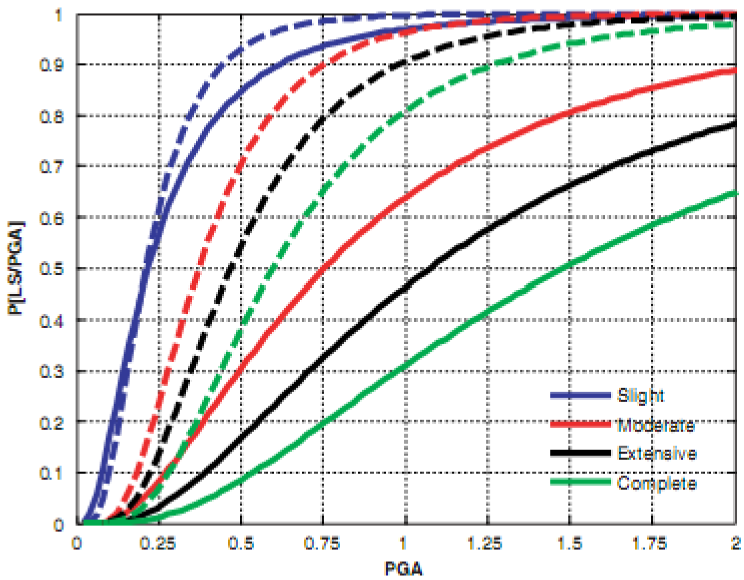

Figure 2.9 System fragility curves for seismically (solid line) sand nonseismically (dashed lines) designed bridges (Ramanathan et al., 2002)

Karim and Yamazaki (2003) studied on a simplifed method for constructing fragility curves. In their work, to construct the fragility curves for highway bridges founded on numerical simulation, an analytical method was employed. Two reinforced concrete (RC) bridges and four piers of typical RC bridges were evaluated.

The bridge structures designed as a non-isolated system and an isolated system in accordance with the earthquake design code in Japan. By using a number of 250 strong acceleration records based on the seismic activities in Japan, United States, and Taiwan, non-linear analysis in time domain was carried out to attain the failure indices for the bridge type structures. Using the index and ground acceleration factors, the fragility curves for the piers of the bridges and the bridge structural system were generated considering a lognormal distribution. Their results indicated that there was a significant efect on the fragility curves because of the difference in structural factors. Moreover, the correlation between the parameters of the fragility curve and the over-strength ratio of the structural systems was archived through a statistical analysis such as linear regression. It was pointed out that the fragility curve factors demonstrated a quite well correlation with the over-strength ratio of the structures. In accordance with the obtained relationship between the fragility curve factors and the over-strength ratio, a simplified approach was suggested to derive the fragility curves for highway bridges using thirty non-isolated bridge structure models. It was reported that the simplifed method might be an extremely functional means to develop the fragility curves for highway bridges (non-isolated) in Japan that considered in identical group of the structures having similar properties.

Marano et al. (2011) suggested an analytical approach for generating the fragility curves of a specified type of existing structures considering a stochastic method. For this, the HAZUS database is utilized. A single degree of freedom (SDOF) system is used in the non-linear modeling of the structure. In their studies, the constitutive rule is defined through a hysteretic model in which the factors are attained by a classification method beginning from the HAZUS data. A special type of structures with essential facilities is analyzed. The HAZUS database was utilized to obtain the factors that express the behavior of the structures, initiating from the curves of capacity that are provided for each building and for each seismic design (from low to high). The reply of the system to seismic activity is modeled through the adopted Clough and Penzien filtered stochastic procedure. That is evaluated by ytilizing the technique of the stochastic linearization and the analysis of covariance. With the intention of developping the fragility curves, a damage index in terms of displacement is employed. Then, the fragility curves are derived by the likelihood of exceeding a given damage state, by utilizing a rough theory of stochastic procedures.

The chief novelty of the recommended method is that it could be straightforwardly enlarged to other types of the structures. They are not comprised in the database of HAZUS. As a summary, the suggested procedure includes the following stages:

- Description of a stochastic based model for the earthquake activity,
- Explanation of a hysteretic formulation in which the actual structure is defined. The constraints of that model would be generated by beginning from the curves of the capacity explained in the HAZUS. Each curve is computed through three controls parameters. They are design capacity, yield (or limit elastic) capacity, and ultimate capacity. By means of these factors, the shape of the nonlinear hysteretic cycle would be attained,
- Appraisal of earthquake response considering stochastic approaches,
- Characterization and determination of the failure levels is needed. For this, the failure levels of the structures associated with the drift ratio chosen by the database of the HAZUS, and
- Development of the curves of fragility.

In the study of Bhargava et al. (2002), the seismic fragility for a typical elevated water-retaining structures were evaluated. The structure was analyzed considering two different cases: (i) empty tank; and (ii) tank filled with water. The different factors that influence the earthquake response of the structure include material strength of concrete and reinforcing steel, effective prestress available in the tank, ductility ratio and structural damping available within the structure, normalized ground motion response spectral shape, foundation and surrounding soil parameters and the total height of water available in the tank. Utilizing the outcomes of this study, the curves of the seismic fragility of the structure was constructed. At two critical locations, the results were given as the relations of conditional likelihood curves generated against peak ground acceleration (PGA). The procedure adopted included the various uncertainties correlated with the factors under consideration (Bhargava et al., 2002).

In the work of Smyth et al. (2004), a cost-benefit analysis method was established for the comparative examination of numerous earthquake rehabilitating remedies adopted to a example apartment type building constructed in Istanbul. In their research, the building was really analyzed in four dissimilar stages: a) the original building without strengthening, b) the building with steel braces, c) the building with partial shear walls, and d) the building with full shear walls. The probabilistic based analysis was conducted for the derivation of analytical fragility curves of the building in its different rehabilitated configurations. Through integrating the probabilistic based earthquake hazard for the areas, anticipated direct losses might be predicted for arbitrary time perspectives. With generating the realistic cost predicts of the strengthening states and costs of direct losses, one might subsequently forecast the net present worth of the different strengthening techniques. The analysis indicated that all of the strengthening ways studied were desirable for all, however the very lowest time horizons. That finding was appropriate for a broad series of predicts concerning the mitigation costs, the discount rates, the amount of fatalities, and cost of human life. The common method derived for a single structure could be lengthened to a whole area by providing supplementary structural types, soil classes, strengthening ways, more precise space- and time-dependent earthquake hazard forecasts, etc. It was reported that this study might act as a benchmark for more reasonable and regular cost-benefit analyses for seismic damage alleviation.

Erberik and Elnashai (2004) carried out a study on determining fragility curves for mid-rise flat-slab buildings having masonry infill walls. A group of seismic ground motion records well-suited to the design spectrum chosen to present the changeability in ground acceleration were utilized. With the purpose of analyzing the arbitrary illustration of structures exposed to the set of records scaled based on the displacement spectral ordinates, at the same time as examining four performance limit levels, inelastic analysis was employed. The analytical fragility curves generated from their investigation were evaluated with those constructed for RC framed structures. According to the findings obtained from their study, it was observed that the seismic losses for flat-slab frames were in the same variety as for RC frames. However, there was a difference. It was also found that the differences were justifiable based on the response characteristics of the two structural types.

In the report of Ramamoorthy et al. (2006), the fragility curves were obtained to evaluate the seismic reliability of reinforced concrete frame building having two stories. The building was designed only for gravity forces. Analytical fragility curves were also constructed for the building moderately rehabilitated through retrofitting of column. A Bayesian method was utilized to make probabilistic demand models to estimate the highest inter-story drifts, specified the spectral acceleration at the natural period of the structures. Two-dimensional inelastic time history analyses were conducted for obtaining the data for the models. For this, a set of synthetic ground motions developed for the Memphis area was employed. Utilizing equality and lower bound data, the model were developed and the model obtained correctly gives an explanation for both randomness and uncertainty. In the lack of statistical capacity models for the structures designed with gravity load, the capacity limit levels were taken into account according to the guidelines of FEMA 356 and deterministic nonlinear pushover analyses. The results quantified the reliability of reinforced concrete buildings (low-rise) and showed the efficiency of earthquake strengthening in mitigating the likelihood of damage.

Kircil and Polat (2007) studied the fragility analysis of middle rise RC buildings. For this, three, five, and seven story frame buildings were designed in accordance with the local seismic design code. With the purpose of examining the yielding and collapse capacity of the structures, the example buildings were evaluated by using incremental dynamic analyses using twelve artificial earthquake records. Then, fragility curves were constracted based on the elastic pseudo spectral acceleration, peak ground acceleration (PGA) and elastic spectral displacement with lognormal distribution presumption. To study the influence of storey height of the building on fragility curves, a statistical regression analysis was conducted between fragility factors and the number of stories. It was obtained that fragility factors varied considerably, depending on the number of stories of the building. Lastly, utilizing generated curves of fragility and statistical approaches, the highest permissible inter-story drift ratio and spectral displacement values which assured the immediate occupancy and collapse prevention performance state conditions were predicted.

In the study of Güneyisi and Altay (2008), the seismic fragility analysis was performed to assess the effectiveness of using viscous dampers in retrofitting the reinforced concrete buildings under scenario earthquakes. The analytical fragility curves were developed for comparative seismic evaluation of various strengthening measures by inclusion of fluid viscous (VS) dampers. Those were adopted to a sample high-rise reinforced concrete (RC) office building located in Istanbul. In the study, the same type of VS dampers was utilized and designed to supply the building with three dissimilar effective damping ratios of 10, 15, and 20%. In the seismic fragility assessment, a suite of 240 synthetically generated earthquakes compatible with the design spectrum chosen to stand for the changeability in ground acceleration was used to examine nonlinear dynamic responses of the buildings before and after strengthening. Different damage levels including slight, moderate, major, and collapse were described to state the failure condition. The fragility curves in their work are presented by lognormal distribution formulations with two factors and developed in terms of peak ground acceleration (PGA), spectral acceleration (S_a), spectral displacement (S_d). Evaluation of the fragility models demonstrated that the viscous dampers were very effectual in mitigating the response of the structures under various earthquakes. Moreover, it was observed that a two-fold decrease in the likelihood of exceeding failure levels could be attained by the incorporation of passive viscous dampers.

Kazantzi et al. (2008) studied the influence of joint ductility on the fragility of a regular moment resisting steel frame designed to EC8 provisions. The change in structural demand was predicted at rising ground shaking intensity states, and analytical fragility curves, provisional on a specified ground acceleration record, were developed for two dissimilar performance states. The inherent uncertainty in the seismic activity was considered by utilizing an ensemble of obtained accelerograms. It was pointed out that the joint rotation capacity was very effective, which is clearly seen in average fragility curve plots.

In the study of Parka and Kim (2010), the fragility examination of steel moment frames with different seismic connections exposed to sudden loss of a column were performed. For this, the progressive collapse potential of steel structures having welded unreinforced flange-bolted web (WUFB), reduced beam section (RBS), and welded cover-plated flange (WCPF) connections was studied based on the uncertainty in properties of material (i.e., yield stress, live load, and modulus of elastic). The beam-end rotation was utilized as a member-level limit level for progressive collapse. Analyses of pushdown of the model structure with various connection types were conducted after getting rid of one of the first-story columns of the structure. Analytical fragility curves were determined in terms of the likelihood of exceeding a given limit level for vertical displacement utilizing the first-order second moment (FOSM) method. The results indicated that the reduced beam section connections supplied the maximum force resisting capacity versus collapse because of their extremely ductile performance and that the failure of an exterior column produced to be more vulnerable for progressive collapse in comparison to than of an interior column.

Celik and Ellingwood (2010) investigated the seismic fragilities for non-ductile RC frames. Effects of aleatoric and epistemic randomnesses were discussed. Fragilities for performance states identified in recent earthquake guidelines were developed for RC frames designed for gravity load in low-seismic areas utilizing probabilistic non-linear finite element analysis. An assessment of the role of uncertainties in material and structural factors depicted that structural damping, strength of concrete used, and cracking strain in beam–column connections had the maximum impact on the fragilities of such structures. On the other hand, fragilities that include these sources of uncertainty were somewhat apart from those based mainly on the uncertainty in seismic demand from seismic ground acceleration, proving that fragilities that were generated under the assumption that all structural factors were deterministic and equivalent to their average values were adequate for aims of seismic failure and loss prediction in areas of moderate seismic activity. Moreover, confidence intervals on the fragility curves were presented as a measure of their precision for the decision of seismic risk, for prioritizing risk alleviation attempts in areas of low to moderate seismic activity.

In the study of Buratti et al. (2010), the fragility based seismic evaluation for RC frame buildings were performed considering the randomnesses in both structural factors and earthquake events. The models of response surface (RS) with random block influences were utilized to get a solution on the difficulty in an approximate means with fine computational effectiveness. The response surface models were adjusted by means of numerical data achieved by non-linear incremental dynamic analyses conducted utilizing various suites of ground acceleration motion records, strength distributions in frame members, and magnitudes of the arbitrary changeable adopted to explain the randomnesses in the behavior of the structures. The study was generally concentrated on the difficulty of getting a realistic compromise between the result soundness and computational attempt. In connection with a frame structure having three stories, a series of numerical tests was represented. Various simulation arrangements described the following the theory of design of experiments (DOE), and simplified polynomial response surface models were utilized. The analytical fragility curves derived by dissimilar methods were evaluated with each other utilizing the results from full Monte Carlo simulation as the control solution.

Another research forwarded by Özel and Güneyisi (2011) examined the influences of eccentric type steel bracing systems on seismic fragility curves of middle rise reinforced concrete (RC) buildings. For this purpose, a six storey mid-rise RC building was selected as a case study. The example building was designed in reference to 1975 version of the Turkish seismic code. The effects of utilizing various types of eccentric steel braces and distribution of the steel bracing over the height of the RC frame in mitigating the seismic response of the case study building were evaluated by means of a fragility analysis. For the retrofitting the original frame structure, D, K, and V type eccentric bracing systems were employed. For fragility assessment, the study used a group of 200 generated ground motion records suitable to the elastic code design spectrum given in the code. Nonlinear analysis in time domain was performed considering this suite of earthquake accelerations derived in terms of peak ground accelerations (PGA). The analytical fragility curves were developed in terms of PGA for performance limit levels, namely slight, moderate, major, and collapse with lognormal distribution assumption. The enhancement in the seismic reliability attained by the utilization of D, K, and V type eccentric braces was evaluated by comparing the median values of the fragility curves of the existing building before and after strengthening. For each type of steel braces, the analytical fragility reduction curves were also plotted by means of a statistical regression analysis as given in Figure 2.10. Analysis of the results indicated that the enhancement in seismic performance of this type of middle rise RC building retrofitted with eccentric steel brace systems was obtained by the formulation of the fragility reduction.

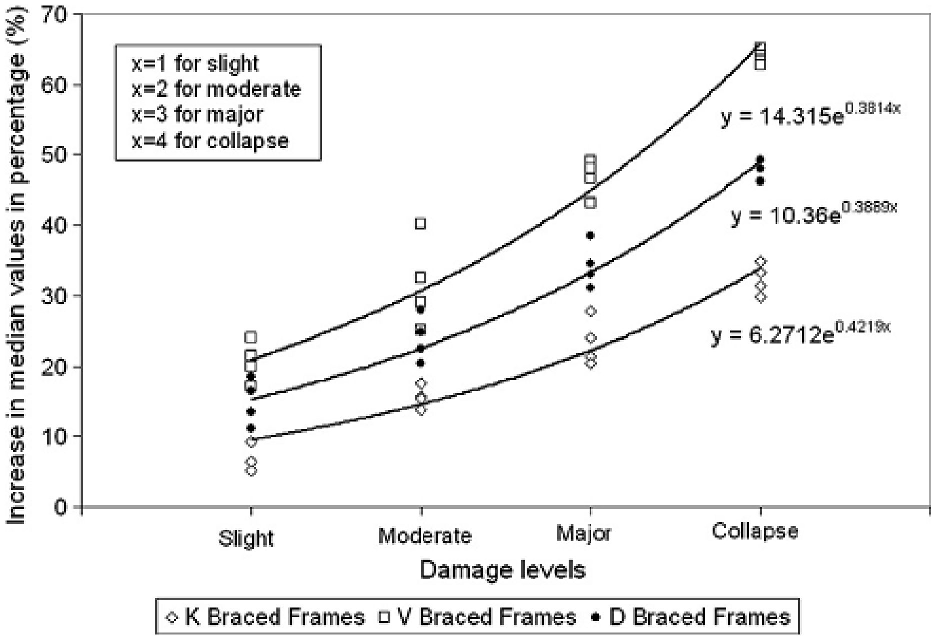

Figure 2.10 Fragility reduction curves (Özel and Güneyisi, 2011)

2.1.4 Hybrid Fragility Curves

Hybrid damage likelihood matrices and seismic vulnerability formulations came together post-seismic failure statistics with simulated, analytical failure statistics from a numerical model of the building typology under thought. The hybrid approaches could be especially beneficial if there is an absence of failure data at certain intensity states for the geographical region under thought and they also permit the calibration of the analytical formulation to be conducted. In addition to this, the utilization of observational data decreases the computational attempt which would be needed to generate an entire suite of analytical vulnerability functions of damage probability matrices (Calvi et al., 2006).

In the report of Schultz et al. (2010), it is stated that the empirical methods have a tendency to be bounded by the accessibility of observational information. On the other hand, the judgmental methods have a tendency to be bounded by subjectivity of specialist evaluations. Alternatively, the analytical procedures have a tendency to be bounded by modeling deficiencies, limiting deductions, or computational burdens. From this point of view, there are a lot of choices for implementing a hybrid approach. One approach is to construct a fragility curve using one approach over one segment of the load and a different approach over a remaining segment of the load. This approach is illustrated by the fragility curves generated by interagency performance appraisal task force (IPET) for formulating flood risk in New Orleans following Hurricane Katrina. The first order second moment (FOSM) method was utilized to get fragility curves for nonovertopping water elevations, and an empirical method was utilized to derive fragility curves for overtopping water elevations. Another possibility is to combine fragility curves developed using judgmental or analytical approaches with observational data through Bayesian updating.

Moroever, in the literature, it can be found more details on the application of hybrid approach (Barbat et al., 1996; Kappos et al., 1998; Singhal and Kiremidjian, 1998; Jeong and Elnashai, 2007).

2.2 Passive Energy Dissipation Systems

Passive energy dissipation systems include a variety of materials and devices for improving damping, stiffness, and strength, and could be employed both for mitigating seismic hazard and for retrofitting aging or deficient structures. Generally, such systems are categorized by their capability to improve the energy dissipation in the structural systems in which they are installed. These apparatus commonly activate on principles such as frictional sliding, yielding of metals, phase transformation in metals, deformation of viscoelastic (VE) solids or fluids and fluid orificing (Soong and Spencer-Jr, 2002).

Symans et al. (2008) also explains the basic principles of energy dissipation systems for applying in seismic regions. It is reported that the main reason to use passive energy dissipation devices in a structural system is to control damaging deformations in structural elements. The degree to which a certain device is able to accomplish this goal depends mainly upon the inherent properties of the basic structure, the properties of the device and its connecting elements, the features of the earthquake acceleration, and the limit state being investigated. Given the large variations in each of these parameters, it is usually necessary to perform an extensive suite of nonlinear response-history analyses to determine which particular passive energy dissipation system is best suited for a given case.

Examples of passive energy dissipation systems can be listed as metallic dampers, friction dampers, viscoelastic dampers (VEDs), viscous dampers, tuned mass dampers, and tuned liquid dampers. In the current study, to mitigate dynamic effects, viscoelastic dampers were utilized as an energy dissipation system in the design of the mid and high rise buildings. For this, more information on this issue are given below.

2.2.1 Viscoelastic dampers

Viscoelastic dampers have been used successfully in high-rise buildings to minimize wind effects. However, the application of viscoelastic dampers to reduce seismic response in buildings is relatively new in comparison to the use of metallic and friction devices. Analytical and experimental investigations on the use of viscoelastic devices under seismic loading have been carried out over only the past decade. The long-term properties and characteristics of these devices are not well defined, and the viscoelastic materials used are also highly dependent on environmental effects, such as the ambient temperature (Craig et al., 2002).

For viscoelastic materials, the mechanical behavior is typically presented in terms of shear stresses and strains rather than forces and displacements. The mechanical properties then become the storage and loss moduli that describe the characteristics of the viscoelastic material rather than properties of the damper. In general, the storage and loss moduli are dependent on frequency of motion, strain amplitude, and temperature. At a given frequency and shear strain amplitude, the storage and loss moduli have similar values that increase with an increase in the frequency of motion. Thus, at low frequencies, viscoelastic dampers exhibit low stiffness and energy dissipation capacity. Conversely, at high frequencies, stiffness and energy dissipation capacity are increased (Symans et al., 2008).

Viscoelastic materials utilized in structural applications are generally copolymers or glassy matters which dissipate energy by means of shear deformation. A typical viscoelastic damper is shown in Figure 2.11. If adopted in a structure, shear deformation and hence energy dissipation occurs whilst the vibration of the structure persuades relative motion between the flanges of the outer steel and the center plates. Considerable progress in research and development of viscoelastic dampers, especially for seismic applications, has been done in recent years through analyses and experimental tests (Soong and Spencer-Jr, 2002). In the study of Symans et al. (2008), the properties of viscoelastic dampers are also described. For instance, the viscoelastic solid dampers generally contains solid elastomeric pads (viscoelastic material) bonded to steel plates. The steel plates are attached to the structure within chevron or diagonal bracing. As one end of the damper moves in accordance with the other, the viscoelastic material is sheared resulting in the development of heat which is dissipated to the environment. By their very nature, the viscoelastic solids exhibit both elasticity and viscosity (i.e., they are displacement and velocity dependent).

Viscoelastic dampers are devices that behave in a manner that both viscous damping and elastic spring characteristics. The elastic component has a linear relationship with deformation. However, the viscous force has a phase difference. The stress-strain relationship is illustrated in Figure 2.12 (Conner, 2003).

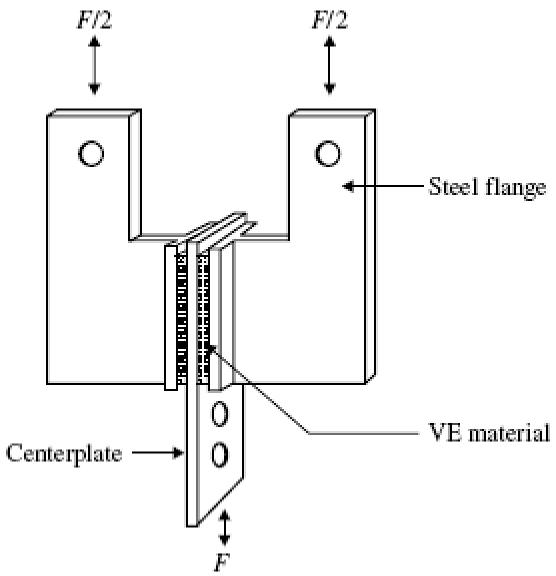

Figure 2.11 Typical view of viscoelastic damper (Soong and Spencer-Jr, 2002)

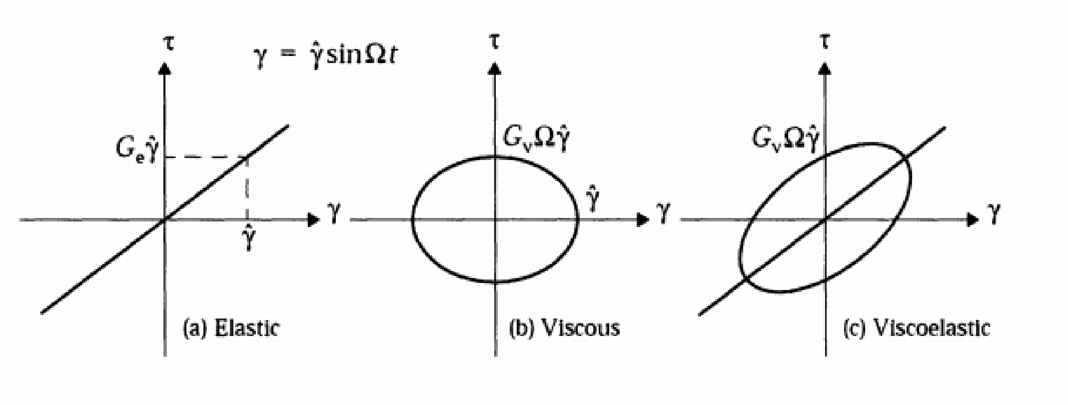

Figure 2.12 Stress-strain relationship for different materials (Conner, 2003)

On the other hand, the behavior of the viscoelastic damper can be explained by very simple viscoelastic linear models comprised by an elastic spring and a dashpot acting in parallel (Kelvin model) or in series (Maxwell model). Figure 2.13 shows the models for the explanation of the viscoelastic damper response (Mazza and Vulcano, 2011). The use of viscoelastic dampers in the frame system is also illustrated in Figure 2.14. As seen from the figure, the modeling of a damper-brace constituent included a viscoelastic damper linked with braces in series (Ou et al., 2007).

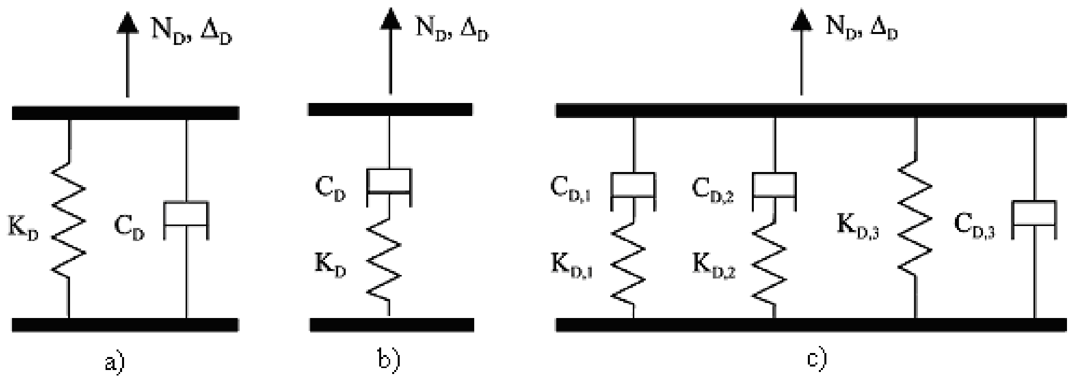

Figure 2.13 Models for the response of viscoelastic damper as (a) Kelvin, (b) Maxwell, and (c) Generalized models (Mazza and Vulcano, 2011)

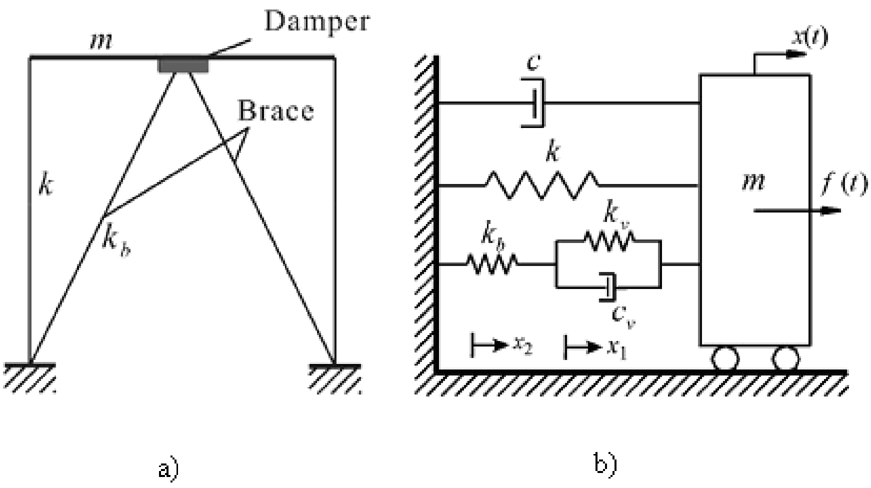

Figure 2.14 Analytical models of the structure with a) a damper and b) a viscoelastic damper (Ou et al., 2007)

In the study of Choi et al. (2003), with the intention of simulating a building with viscoelastic dampers, a non-linear response spectrum was derived utilizing a non-linear oscillator model. The spectra of earthquake response were achieved for a non-linear system, including a model of a comparatively flexible four-story building braced with viscoelastic dampers, as demonstrated in Figure 2.15. The bilinear hysteretic performance of a viscoelastic damper is illustrated in Figure 2.16. The experimental and numerical hysteretic cycles of the viscoelastic damper at different displacements are also presented in Figure 2.17. A force was applied dynamically to the viscoelastic damper using four different displacement levels (0.5, 1.0, 3.0, and 6.0 cm) to examine the nonlinear features of the viscoelastic damper. The analysis of the outcomes demonstrates that the numerical hysteretic cycles explain the experimental results well, irrespective of the applied displacement. In their study, it is observed that the viscoelastic dampers effectively diminish the earthquake response of a model structure in terms of displacement, however, convey more seismic load to the structure that fundamentally raises its earthquake acceleration response.

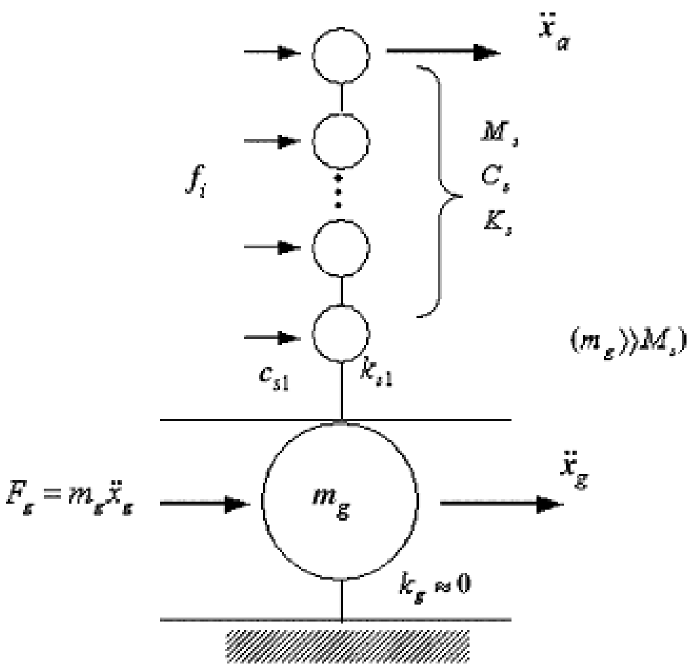

Figure 2.15 Analytical model of a building with viscoelastic damper (lumped mass model) (Choi et al., 2003)

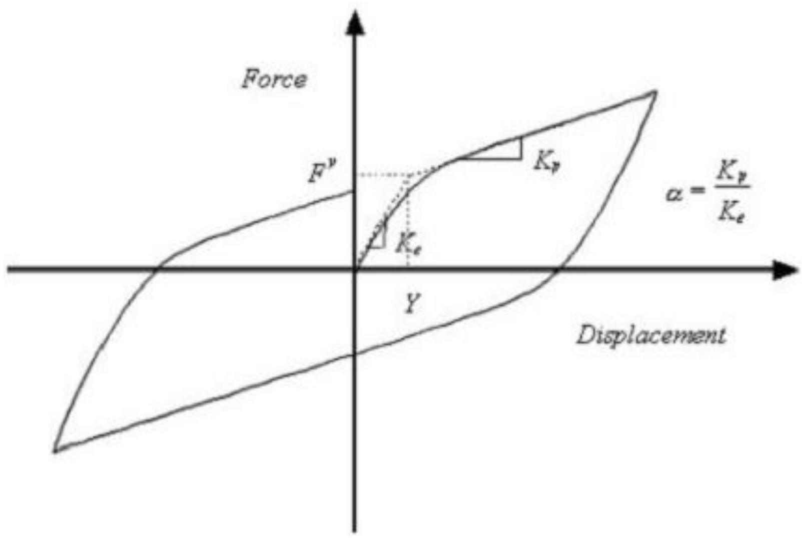

Figure 2.16 A viscoelastic damper under bilinear hysteretic behavior (Choi et al., 2003)

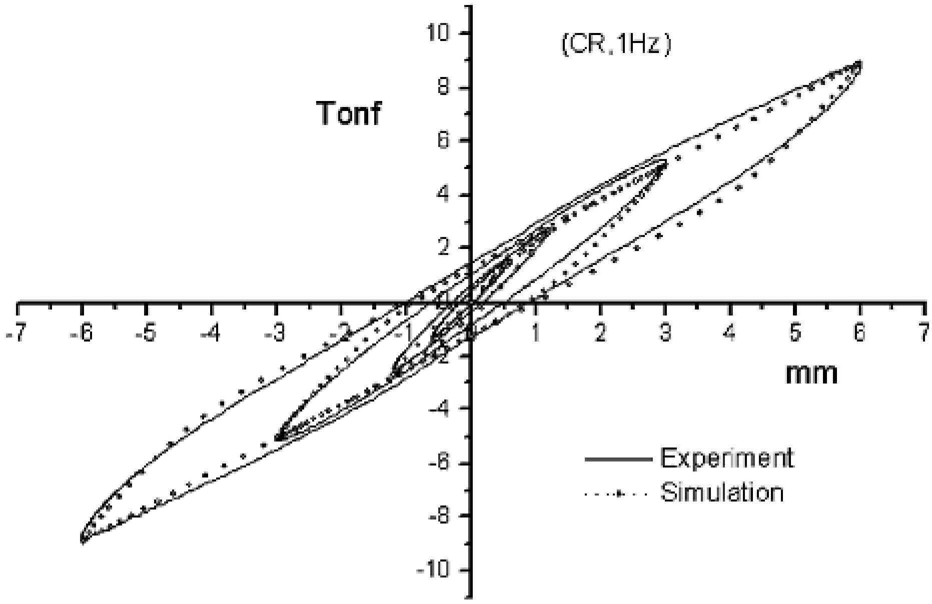

Figure 2.17 Hysteresis cycles of a viscoelastic damper achieved experimentally and numerically (Choi et al., 2003)

CHAPTER 3

CASE STUDY

3.1 General Methodology

In the methodology, the following three main parts were taken into consideration. First of all, the earthquake intensity indicator, namely ($S_a(T_1)$) was selected and 15 natural ground motion records were generated. Secondly, the building configuration and material properties were defined, and then the design and analytical idealization of the conventional and viscoelastically damped structures was done. Thirdly, the damage states were selected as immediate occupancy, life safety, collapse prevention for structural fragility curves and for nonstructural drift sensitive and acceleration sensitive fragility curves, slight, moderate, major, and collapse damage levels were used. After all, the nonlinear time history analyses were conducted and the fragility curves were developed. So as to better examine the seismic reliability of viscoelastically damped frame systems with reference to that of conventional moment resisting frame systems, the seismic risk of the structures were also evaluated.

3.2 Description of Original Buildings

With the purpose of examining the reliability of the frame systems with viscoelastic dampers (VEDs), the seismic reliability analyses were carried out for steel framed buildings, 5 and 12 stories in height, designed by considering : a) Case 1: Conventional moment resisting frame, b) Case 2: Moment resisting frame with viscoelastic dampers providing supplemental effective viscous damping ratio of 10% (ξ_{VED} =10%), and c) Case 3: Moment resisting frame with viscoelastic dampers providing supplemental effective viscous damping ratio of 20% (ξ_{VED} =20%). The framed building were designed in accordance with the method of direct displacement-based design suggested by Lin et al. (2003) by using elastic displacement response spectrum. The elastic displacement response spectrum was obtained in reference to Eurocode 8 (1998) considering the acceleration of peak ground (PGA) of 0.4 g for the level of seismic hazard that had 10% exceedance probability in fifty years. The greatest story drift ratio was supposed to be 0.5% under the seismic excitations. As shown in Figures 3.1 to 3.4, the 5-storey and 12 storey steel buildings had the similar floor plan (4x4 bays) with 8 m bay spacing while the elevation of the stories was 3.8 m. The characteristic loads for floor finishes were taken as 1.0 and 0.8 kN/m^2 at floor levels and roof, respectively, at the same time as for imposed load 2 kN/m^2 was considered. Yield stress of steel with 240 N/mm^2 was employed for columns and beams. Table 3.1 summarizes the structural member sizes determined for the six different case study frames. As seen from the table, for the columns and beams of the frames, the box sections were utilized in the design.

Table 3.1 Dimensions of structural members for the case study frames

Cases		C1 Box Section (mm)	C2 Box Section (mm)	B1 Box Section (mm)	B2 Box Section (mm)	VED K (N/mm)
5 Storey	Case1: Conventional Frame	670x670x25	-	320x160x25	270x135x25	-
	Case2: Frame with VED, ξ_{VED}=10%	510x510x20	-	460x230x20	390x185x20	7610
	Case3: Frame with VED, ξ_{VED}=20%	450x450x20	-	410x205x20	342x171x20	14556
12 Storey	Case1: Conventional Frame	700x700x25	600x600x25	420x210x25	360x180x25	-
	Case2: Frame with VED, ξ_{VED}=10%	510x510x25	440x440x25	480x240x25	420x210x25	9214
	Case3: Frame with VED, ξ_{VED}=20%	460x460x25	390x390x25	440x220x25	380x190x25	16984

The analytical model of the conventional and viscoelastically damped frames were performed by using a finite element program of DRAIN-2DX (Prakash et al., 1993). Considering the implemented modeling procedures, the masses were applied to the joint of beam and column by means of flat translation slaving assigned at the nodules of the identical floor stage and assumes in the direction of displacing only in the horizontal path. The columns were supposed fixed at the bases and the role of the floor slab to the beam strength and stiffness was disregarded. The beams and columns of the frames were modeled as the member of beam and column that permits for the occurrence of plastic hinges at the focused points close to the endings employing lumped plasticity based models with a defined strain-hardening ratio and moment-axial interaction. Beam to column links were selected as stiff joints and the column near base links were modeled as fixed joints. Bilinear elastoplastic behavior through a strain hardening ratio of 0.05 was employed to define plastic hinges. A linearized biaxial plastic domain was used to explain bending axial interaction. The shear behavior of beam and column elements of the frame systems was supposed to stay linearly elastic.

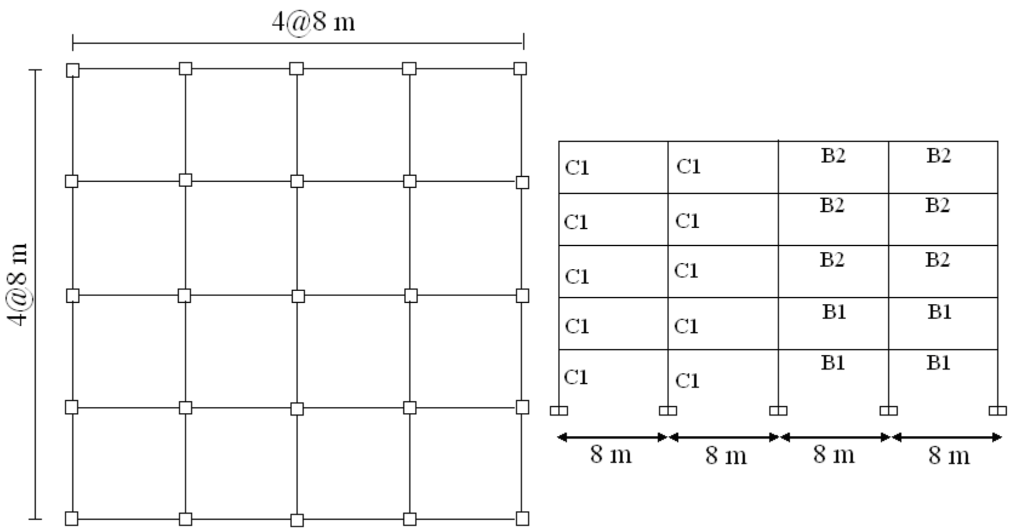

Figure 3.1 Plan and elevation views of 5 storey case study building

A representative viscoelastic damper contains thin stratums of viscoelastic substance connected within the steel plates and dynamic performance of viscoelastic dampers is commonly being a symbol of a spring and a dashpot linked in parallel (Soong and Dargush, 1997; Valles et al., 1997; Kim and Choi, 2006). In the analytical model of the viscoelastically damped frames, inelastic truss finite elements were employed for the linear spring dashpot illustration of the viscoelastic dampers. The rigidity (K_d) and the damping coefficient (C_d) are attained as seen below (Kim and Choi, 2006):

$$K_d = \frac{G'(w)A}{t}$$

$$C_d = \frac{G''(w)A}{wt} \tag{3.1}$$

where $G'(w)$ and $G''(w)$ are shear storage and shear loss moduli, A and t are total shear area and the width of the substance, respectively, and w is the loading frequency. In the current work, firstly, the required stiffness of the VEDs in the frame systems were determined. In all stories of the frames, VEDs with the same stiffness were used as seen in Table 3.1. Additionally, based on these equations, in the calculation of the damping coefficient, the loss factor which is the proportion of the shear loss factor to shear storage moduli was assumed to be 1. For the forcing frequency (w), the natural frequency of the frames were used.

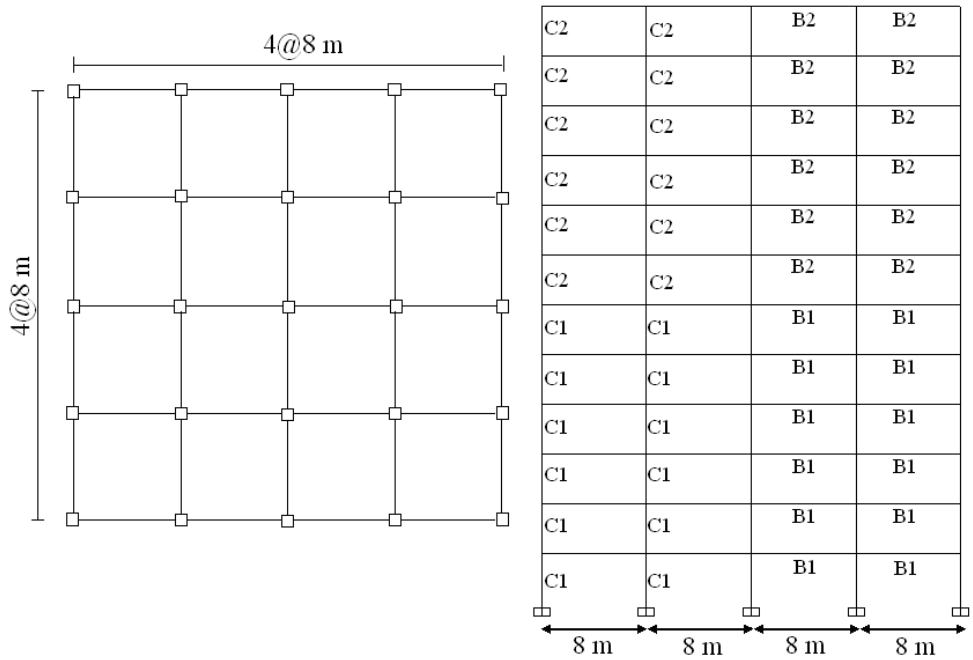

Figure 3.2 Plan and elevation views of 12 storey case study building

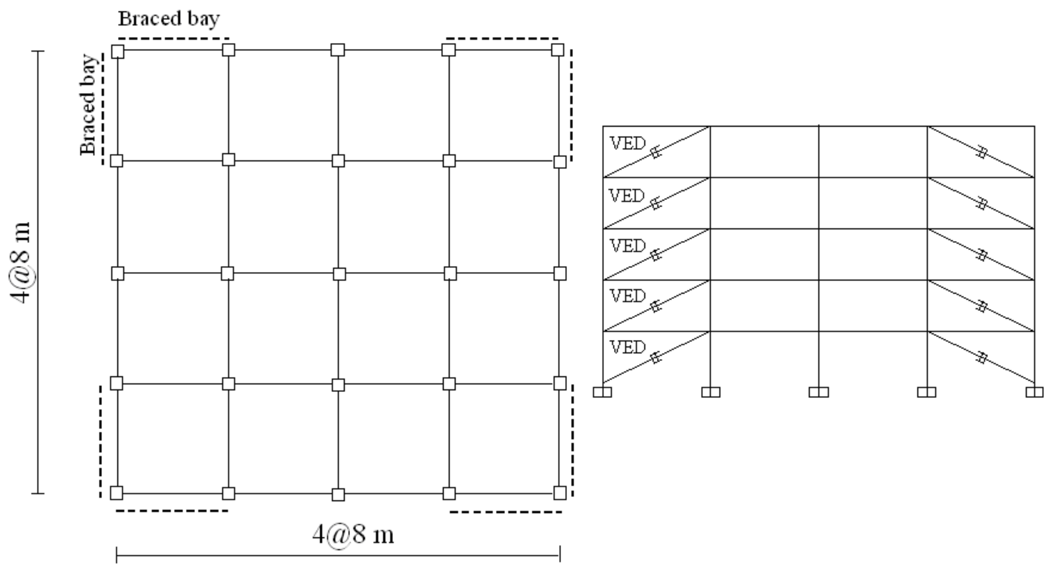

Figure 3.3 Plan and elevation views of 5 storey case study building with viscoelastic dampers (VEDs)

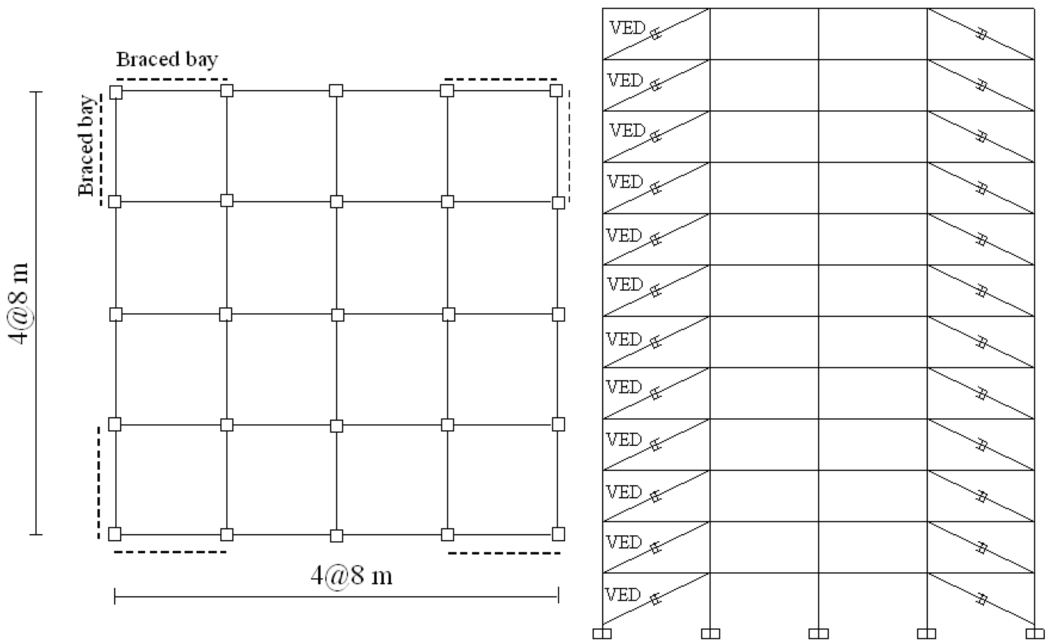

Figure 3.4 Plan and elevation views of 12 storey case study building with viscoelastic dampers (VEDs)

3.3 Seismic reliability analysis

With the intention of giving an explanation for the randomness concerned in the content of frequency, period, and other properties of the earthquake acceleration, the most proper way to evaluate the efficiency of a damper can be done by means of the analysis of the reliability (Curadelli and Riera, 2004). For this, firstly, the fragility analysis of the buildings designed as conventional moment resisting frames and moment resisting frames with viscoelastic dampers suplying supplemental effective viscous damping proportions of 10% (ξ_{VED} =10%) and 20% (ξ_{VED} =20%) was evaluated by considering both structural and nonstructural components. Furthermore, the efficiency of the viscoelastically damped frame systems in comparison to the conventional moment resisting frame systems were evaluated by means of the annual probability of exceedance of performance levels for structural components.

3.4 Earthquake ground motions

The inherent uncertainties in the earthquake acceleration itself such as maximum intensity, time changing amplitude, strong activity duration, and content of frequency, etc., make the damage estimation as probabilistic. Therefore, in the current study, a set of 15 natural ground motion records (Ambraseys et al., 2004 a,b) representing extreme ground motions with dissimilar features were employed.

In the selection of the earthquake ground motions, the limitations for the seismic magnitude ($M \geq 6.5$), peak ground velocity ($PGV \geq 15$ cm/s), and peak ground acceleration ($PGA \geq 0.2$ g) were taken into account. In addition to these limitations, in order to avoid main near field and soft soil causes, all ground motion records recorded at a significant distance from the fault ($D \geq 10$ km) and recorded on firm soil conditions (which correspond to shear wave velocities in 30 m equal or greater than 180 m/s) were selected. The properties of the set of selected earthquake acceleration records utilized in this study are listed in Tables 3.2 and 3.3. Moreover, the 5% damped response spectra of the selected ground motions are illustrated in Figure 3.5.

Table 3.2 Lists of earthquake locations and the corresponding recording stations

No	Earthquake Location	Recording Station
1	Friuli /Italy	Tolmezzo Diga Ambiesta
2	Montenegro	Petrovac Hotel Oliva
3	İzmit/Turkey	Yarımca-Petkim
4	Erzincan /Turkey	Erzincan Meteoroloji Müdürlüğü
5	Compano Lucano /Italy	Sturno
6	Alkion/Greece	Korinthos-OTE Building
7	Düzce /Turkey	Bolu Bayındırlık ve İskan M.
8	Montenegro	Ulcinj Hotel Albatros
9	Tabas/Iran	Tabas
10	Manjil/Iran	Abhar
11	Kozani/Greece	Kozani - Prefecture
12	İzmit/Turkey	Yarımca-Petkim
13	Düzce /Turkey	LDEO Station No CO375VO
14	South Iceland	Hella
15	South Iceland (Afterschock)	Solheimar

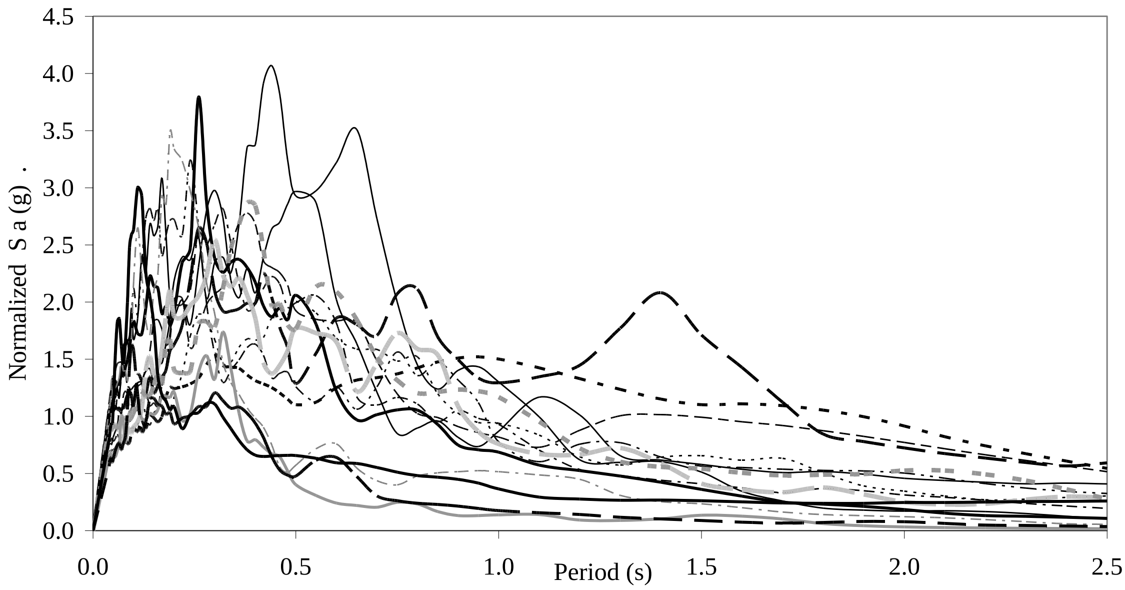

Figure 3.5 Acceleration response spectrum of selected strong earthquake ground motions

Table 3.3 Properties of selected natural ground motions

No	Soil Type	M_w	Date	D (km)	a_{max} (m/s^2)	v_{max} (cm/s)
1	A	6.5	06/5/1976	23	3.35	32.47
2	B	7.0	15/04/1979	25	4.47	39.23
3	C	7.6	17/08/1999	20	3.05	58.50
4	B	6.7	13/03/1992	13	5.31	84.57
5	A	6.9	23/11/1980	32	3.04	62.62
6	C	6.6	24/02/1981	20	3.07	22.61
7	C	7.2	12/11/1999	39	7.88	65.02
8	A	7.0	15/04/1979	21	2.11	26.25
9	B	7.4	16/09/1978	57	10.17	87.64
10	C	7.4	20/06/1990	98	1.94	20.84
11	A	6.6	13/05/1995	17	2.13	8.76
12	C	7.6	17/08/1999	100	3.66	45.17
13	A*	7.2	12/11/1999	39	8.75	38.17
14	B*	6.6	17/06/2000	15	4.58	46.64
15	B*	6.5	21/06/2000	11	7.06	105.28

3.5 Fragility curves

The reliability assessment through fragility curves is derived from a series of nonlinear time history analyses of the conventional and viscoelastically damped frames under the natural earthquake ground motions, performed using DRAIN-2DX structural analysis program (Prakash et al., 1993).

In the literature, several performance limit state criterions in terms of different response measures have been proposed for different type of structures. The determination of a response measure for severely measuring the performance limit state of a building is still an open query. In other wards, post-seismic tragedy analyses have demonstrated a relationship between extreme lateral drifts and structural and/or nonstructural damage (Lin et al., 2010). Furthermore, the earthquake response of outwardly damped structures was evaluated by using the drift technique in the previous studies (Aiken et al., 1988; Filiatrault and Cherry, 1993;

and Pall et al., 1993). They dealt with friction type dampers. In the same way, Chang et al. (1992;1995) utilized the procedure to viscoelastic dampers while Tsai et al. (1993) and Martinez-Romero (1993) considered metallic dampers (Curadelli and Riera, 2004). Consequently, in this work, for the development of structural fragility curves, the inter-storey drift ratio was utilized as the seismic response measure for expression of performance limit state.

For the structural fragilities, the performance intentions in terms of inter-story drifts given in the FEMA 356 guidelines (2000) were considered. The inter-storey drift limits of 0.7%, 2.5%, and 5.0% were selected for describing the performance limit levels of immediate occupancy (IO), life safety (LS), and collapse prevention (CP) for conventional moment resisting frame systems, respectively whereas, the inter-storey drift limits of 0.5%, 1.5%, and 2.0% were utilized for explaning the limit states for frame systems with viscoelastic dampers.

For the development of fragility curves, the nonlinear time history analyses were conducted for a band of seismic ground accelerations scaled to a seismic intensity stage. Existing techniques for derivation of fragility curves utilize frequently peak ground acceleration (PGA), spectral acceleration (S_a), velocity (S_v), or spectral displacement (S_d) to exemplify earthquake intensity. From these, first mode spectral acceleration ($S_a(T_1)$) was chosen as the earthquake intensity factor. For each ground acceleration record, the analyses were done again for rising first mode spectral acceleration values with 0.05g increments. From the outcomes of each nonlinear time history analysis, the peak structural responses in terms of inter-storey drift ratio and floor acceleration were retained. The likelihood of exceedance of a limit level for specified ground motion intensity was then estimated by a lognormal statistical circulation fitted to the data for each intensity level. The likelihood of exceedance of a sure damage state was acquired by determining the area of the lognormal circulation over the flat contour of that limit level. After calculating the likelihood of exceedance of the limit states for each intensity level, the vulnerability curve was constructed by plotting the evaluated data against seismic intensity. Finally, a statistical distribution was fitted to these data points, to achieve the fragility curves which are demonstrations of provisional likelihood indicating the likelihood of getting together or going beyond a state of damage under a presented input earthquake acceleration intensity factor. That provisional likelihood can be stated as (HAZUS, 1997):

$$P\left[LS_i = X\right] = \Phi\left[\frac{1}{\beta}\ln\left(\frac{X}{\mu}\right)\right] \tag{3.2}$$

where Φ is the standard normal cumulative distribution function, X is the lognormal distributed earthquake acceleration intensity factor, and μ is the median value of earthquake acceleration factor at which the structure attains the threshold of limit level LS_i, described utilizing permissible inter-storey drift ratios and β is the standard deviation of the natural logarithm of earthquake acceleration factor of limit level.

Nonstructural components in buildings comprise a big diversity of distinct architectural, mechanical, and electrical constituents. So as to appraise their seismic performance because of a seismic activity, the nonstructural parts are categorized as either drift sensitive or acceleration sensitive parts in accordance with HAZUS (1997). Table 3.4 shows the catalog of representative nonstructural constituents and content of structures. For the nonstructural fragility curves in terms of drift and acceleration sensitive, the level and the severity of failure to the nonstructural constituents are described by four performance limit states: namely, slight, moderate, extensive, and complete.

Therefore, in accordance with HAZUS (1997), for nonstructural drift sensitive components, the limits of 0.4%, 0.8%, 2.5%, and 5.0% based on the inter-storey drift ratio were utilized for defining the performance limit levels of slight, moderate, extensive, and complete, respectively. On the other hand, for acceleration sensitive components, the floor acceleration limits of 0.30, 0.60, 1.20, and 2.40 g were utilized for describing the limit levels of slight, moderate, extensive, and complete, respectively.

Table 3.4 Typical nonstructural parts of buildings according to HAZUS (1997)

Category	Item	Drift-Sensitive	Acceleration-Sensitive
Details of Architecture	Nonbearing Walls/ Panels	xx	x
	Cantilever Elements and Parapets		xx
	Exterior Wall Panels	xx	x
	Layers and Finishes	xx	x
	Penthouses	xx	
	Racks and Cabinets		xx
	Entree Floorings		xx
	Attachments and Decorations		xx
Details of Mechanic and electric	Common Mechanic Details		xx
	Producing and Equipment		xx
	Piping Details	x	xx
	Reservoirs and Spheres		xx
	Systems (i.e. chillers)	x	xx
	Elevators	x	xx
	Trussed Towers		xx
	Common Electrical Part (i.e. channels)	x	xx
	Illumination Contests		xx
Contents	Cabinets, Bookcases, etc.		xx
	Staff Apparatus and Furniture		xx
	PC/Contact Apparatus		xx
	Nonpermanent Producing Apparatus		xx
	Producing /Storeroom Inventory		xx
	Art and Additional Costly Items		xx

xx indicates the main reason of damage, x indicates the minor reason of damage

3.6 Seismic hazards and seismic risk evaluation

In order to better investigate the seismic reliability of viscoelastically damped frame systems with reference to that of conventional moment resisting frame systems, the seismic risk of the structures were also determined.

For this, the structural fragility and equivalent description of the seismic hazard need to be combined, hence leading to a conceptually meaningful estimation of the seismic risk. In this context, the structural seismic risk which is presented in this study based on yearly possibility of exceedance of each damage state is the convolution of fragility curve and earthquake risk curve which is the outcome of probabilistic seismic hazard analysis. Thus, the probabilities of each damage state $P[LS_i]$ was calculated according to the equation given below (Ellingwood, 2001):

$$P\left[LS_i\right] = \sum_{all\ im} P\left[LS_i \middle| IM = im\right] P\left[IM = im\right] \tag{3.3}$$

In this equation, $\mathrm{P}[LS_i \,|\, IM{=}im]$ is the conditional likelihood of exceedance of damage level for a specified seismic intensity, $\mathrm{P}[IM{=}im]$ is the annual probability of exceedance of a given seismic intensity. Since the aim of this work is to compare the seismic reliability of frames, the seismic hazard curve that show annual probability of exceedance in terms of first mode spectral acceleration rates were used with the intention of finding and comparing the annual probability of exceedance of each damage state of case study frames.

These hazard curves given in Figures 3.6 and 3.7 were drawn for 5 and 12 storey frames, respectively in terms of the first mode spectral acceleration values considering a location with soil conditions, which correspond to the shear wave velocities equal or greater than 180 m/s in the upper 30 m (Field et al., 2003). Thus, by using the results obtained from the probabilistic seismic hazard curve and the seismic structural fragility curves of the frame systems, the risk was obtained based on annual likelihood of exceedance of limit states.

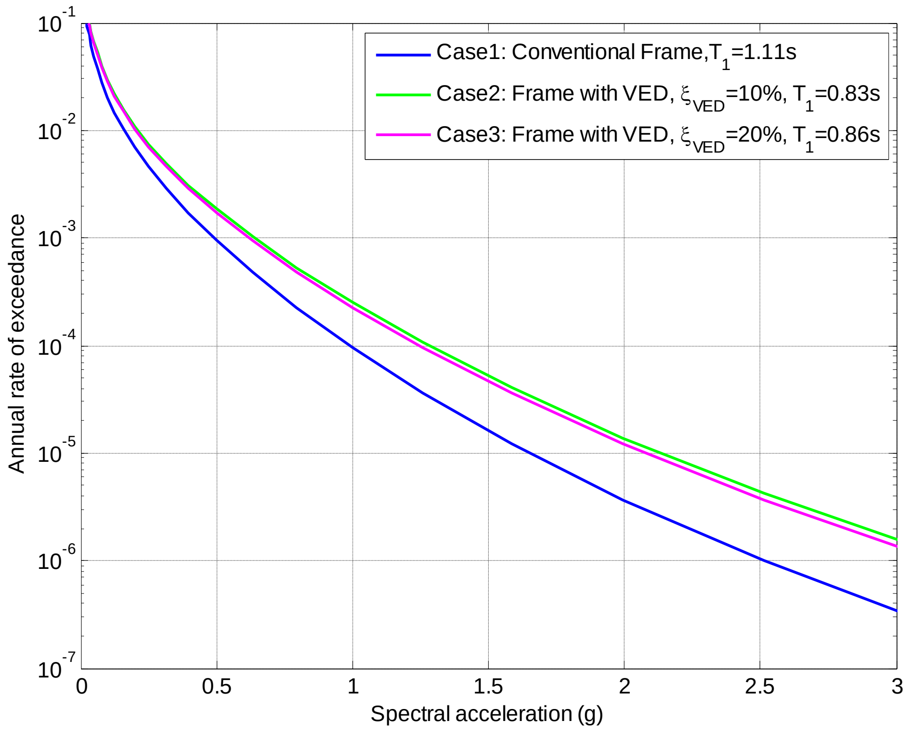

Figure 3.6 Seismic hazard curves for 5 storey frames

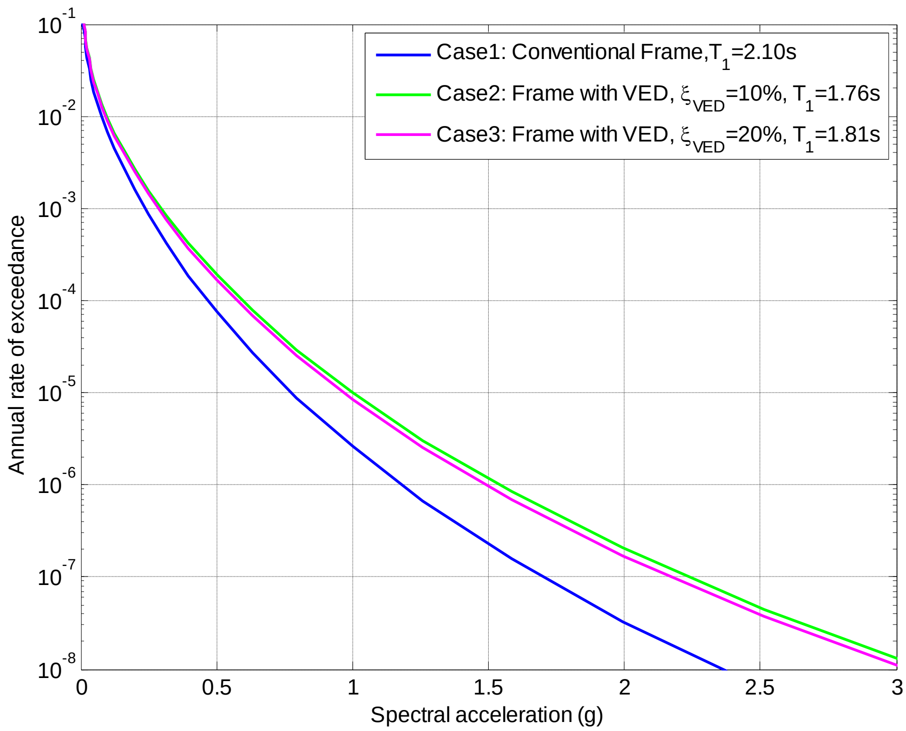

Figure 3.7 Seismic hazard curves for 12 storey frames

CHAPTER 4

DISCUSSION OF THE RESULTS

Computed structural fragility curves for the three performance limit-states are shown in Figures 4.1 and 4.2 for the 5 storey and 12 storey frames, respectively. The analysis of the results are presented for the frames considered in this study such as (a) Case 1: Conventional moment resisting frame, (b) Case 2: Frame with viscoelastic dampers (VEDs) providing supplemental effective viscous damping ratio of 10% (ξ_{VED} =10%), and (c) Case 3: Frame with VEDs providing supplemental effective viscous damping ratio of 20% (ξ_{VED} =20%). In addition to this, the median and standard deviation of these structural fragility curves are given in Table 4.1.

Table 4.1 Median and standard deviation parameters of the structural fragility curves

Frames		Performance Limit Levels					
		Immediate Occupancy		Life Safety		Collapse Prevention	
		μ	β	μ	β	μ	β
5 Storey	Case1: Conventional Frame	0.296	0.248	0.718	0.113	1.616	0.312
	Case2: Frame with VED, ξ_{VED}=10%	0.418	0.116	1.323	0.228	1.770	0.254
	Case3: Frame with VED, ξ_{VED}=20%	0.493	0.207	1.515	0.240	1.940	0.251
12 Storey	Case1: Conventional Frame	0.100	0.313	0.233	0.430	0.500	0.309
	Case2: Frame with VED, ξ_{VED}=10%	0.148	0.166	0.441	0.185	0.590	0.251
	Case3: Frame with VED, ξ_{VED}=20%	0.168	0.162	0.490	0.251	0.639	0.292

Based on the data plotted in Figures 4.1 and 4.2, the frames designed with VEDs providing supplemental effective viscous damping ratios of 10% and 20% are similar in their structural fragility curves. It was found out that the generated fragility curves for Case 3 frame system (Frame with VEDs providing supplemental effective damping ratio of 20%) were less fragile compared to those for Case 2 frame system. An improvement up to about 1.2 times (in accordance with the median first mode spectral acceleration values) was observed. On the contrary, the structural fragility curves of the conventionally designed moment resisting frame were apparently different from the viscoelastically damped frame systems. This difference between the fragility curves of the conventionally designed and viscoelastically designed frames was much more pronounced, especially for the life safety limit state.

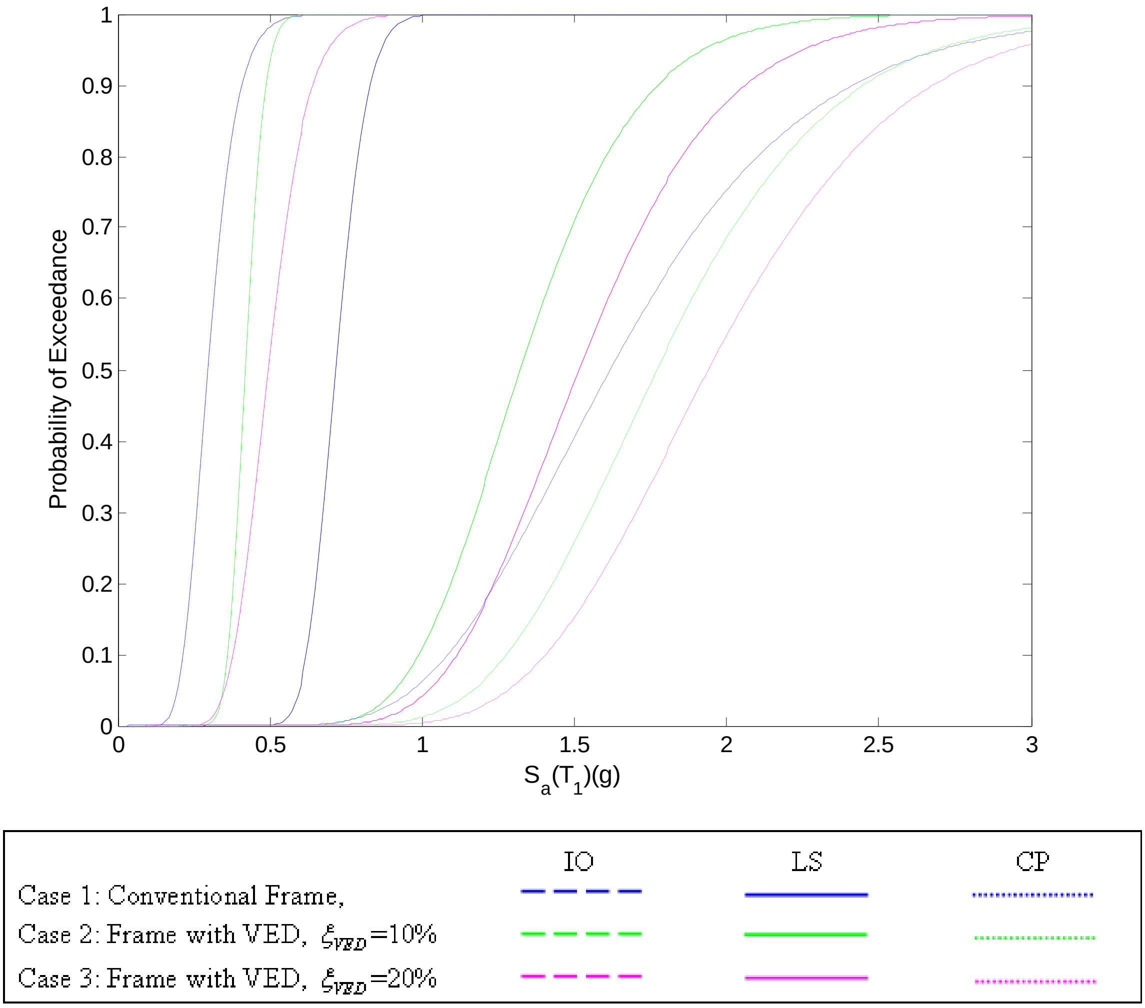

Figure 4.1 Structural fragility curves developed for 5 storey frames

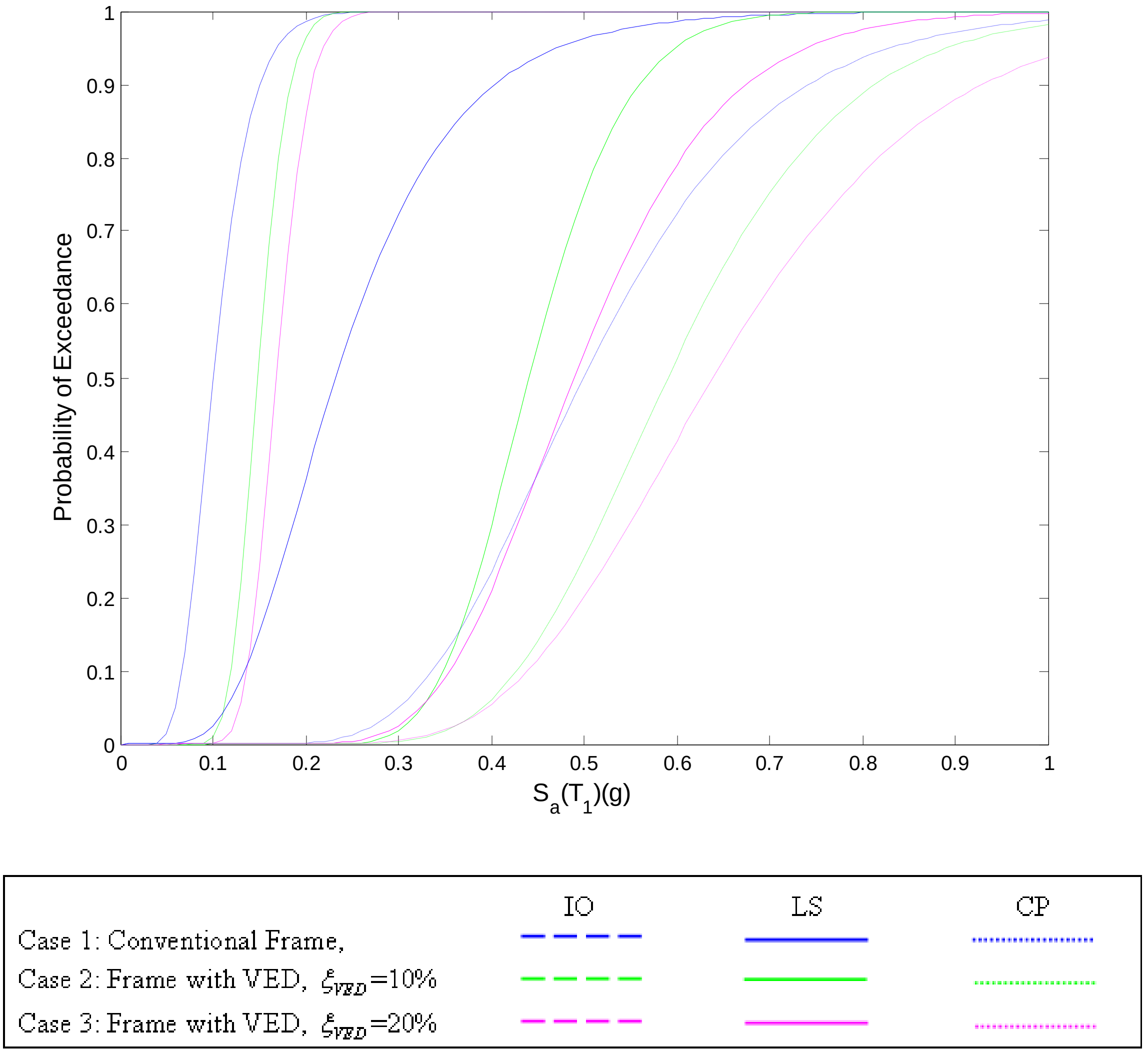

Figure 4.2 Structural fragility curves developed for 12 storey frames

For example, the simulated fragility curves for the Case 3 frame system were less fragile compared to those for conventional frame system by as much as 2.1 times based on median first mode spectral acceleration values. This was explained as the frame systems designed with VEDs could remain in good elasticity with small inter-storey drifts. However, in the moment resisting frame system designed, similar to the observations reported by Lin et al. (2010), the earthquake damage could easily concentrate at the weak story which caused the increase in the inter-storey drift demands on conventionally designed moment resisting frame systems.

The seismic structural fragility curves generated for the case study frames indicated that the fragility curves became flatter when the performance limit state shifted from immediate occupancy to collapse prevention. Thus, the structure became more sensitive to the variations underneath low ground motion intensity values than high ground motion intensity values that pointed out that little alterations in lower first mode spectral acceleration values brought about remarkable discrepancies in the likelihood of exceedance of performance limit states. In the previous studies, similar observations were also stated concerning the fragility curve flatness constructed for different types of buildings (Güneyisi and Altay, 2008; Özel and Güneyisi, 2011; Erberik and Elnashai, 2004).

The changes in the likelihood of exceedance of considered performance limit states for the low and high values of first mode spectral acceleration was connected with the uncertainty levels related with the amplitudes of the inter-storey drift ratio measured. These uncertainties in the inter-storey drift ratio measured produced with the increase in the value of first mode spectral acceleration values since the nonlinear behavior became more considerable as the values of first mode spectral acceleration increased.

The nonstructural drift-sensitive and acceleration sensitive fragility curves generated for the 5-storey and 12-storey studied frames are given in Figures 4.3 through 4.6. Furthermore, the median and standard deviation parameters of these nonstructural fragility curves are given in Table 4.2.

As seen from the figures and table, similar to the structural fragility curves, the drift sensitive and acceleration sensitive nonstructural fragility curves of the frames designed with viscoelastic dampers supplying supplemental effective viscous damping ratios of 10% and 20% were close to each other. Based on median first mode spectral acceleration value, the frame with viscoelastic dampers providing supplemental effective damping ratio of 20% (Case 3) was as much as 1.2 times less fragile compared to frame with viscoelastic dampers supplying supplemental effective damping ratio of 10% (Case 2).

Table 4.2 Median and standart deviation parameters for the drift sensitive and acceleration sensitive nonstructural fragility curves

Frame systems			Damage Levels							
			Slight		Moderate		Extensive		Complete	
			μ*	β**	μ*	β**	μ*	β**	μ*	β**
Drift sensitive	5 Storey	Case1: Conventional Frame	0.168	0.267	0.324	0.250	0.718	0.113	1.616	0.312
		Case2: Frame with VED, ξ_{VED}=10%	0.335	0.116	0.666	0.128	2.184	0.276	3.953	0.339
		Case3: Frame with VED, ξ_{VED}=20%	0.394	0.207	0.795	0.219	2.346	0.271	4.109	0.337
	12 Storey	Case1: Conventional Frame	0.075	0.086	0.114	0.313	0.233	0.430	0.500	0.309
		Case2: Frame with VED, ξ_{VED}=10%	0.118	0.163	0.242	0.159	0.729	0.258	1.266	0.219
		Case3: Frame with VED, ξ_{VED}=20%	0.134	0.155	0.273	0.178	0.776	0.292	1.349	0.288
Acceleration sensitive	5 Storey	Case1: Conventional Frame	0.161	0.383	0.326	0.461	0.395	0.458	1.024	0.524
		Case2: Frame with VED, ξ_{VED}=10%	0.203	0.290	0.407	0.295	0.866	0.317	2.274	0.264
		Case3: Frame with VED, ξ_{VED}=20%	0.239	0.308	0.479	0.305	0.990	0.304	2.309	0.276
	12 Storey	Case1: Conventional Frame	0.049	0.610	0.102	0.716	0.125	0.864	0.317	0.823
		Case2: Frame with VED, ξ_{VED}=10%	0.094	0.650	0.189	0.669	0.387	0.724	0.824	0.763
		Case3: Frame with VED, ξ_{VED}=20%	0.095	0.639	0.192	0.667	0.401	0.738	0.878	0.788

*indicates the median, **indicates the standard deviation

When the 5 storey conventionally designed frames are compared with the viscoelastically damped frames, it was observed that the viscoelastically damped frame were as much as 3.3 times less fragile than conventional frame based on median first mode spectral acceleration values. For 12 storey frames, this ratio reached as much as 3.2. Apart from the behavior observed in structural fragility curves, the difference between the fragility curves of the moment resisting frame system and frame systems with viscoelastic dampers could be apparently observed for all performance limit states, due to the fact that in determination of nonstructural drift sensitive and acceleration sensitive structural fragility curves, different than structural fragility curves, the same inter-storey drift ratio and storey acceleration limit values were utilized in the derivation of nonstructural fragility curves.

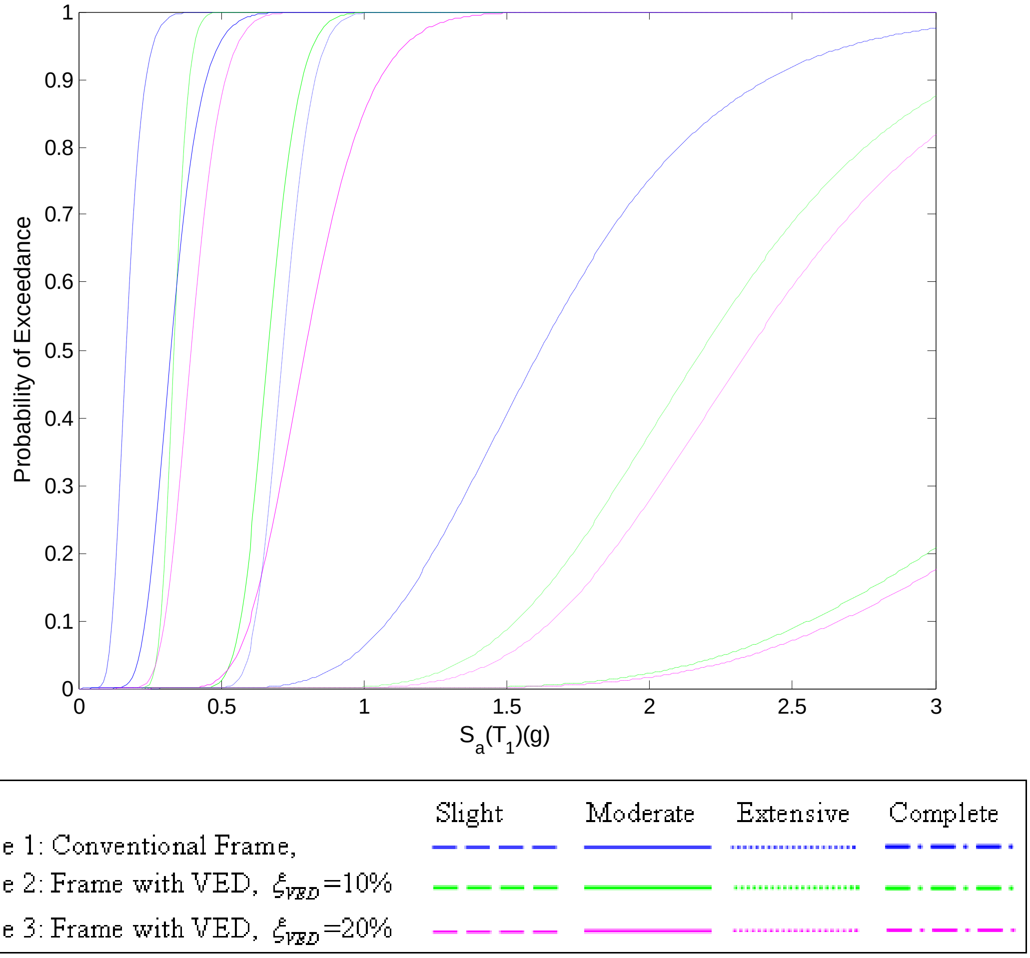

Figure 4.3 Nonstructural drift-sensitive fragility curves developed for 5 storey frames

As a result, for all performance limit states, the physical improvement provided by frame systems with viscoelastic dampers became evident in terms of enhanced nonstructural acceleration-sensitive and drift-sensitive fragility curves, moved those related with the moment resisting frame system to the right if constructed as a function of first mode spectral acceleration.

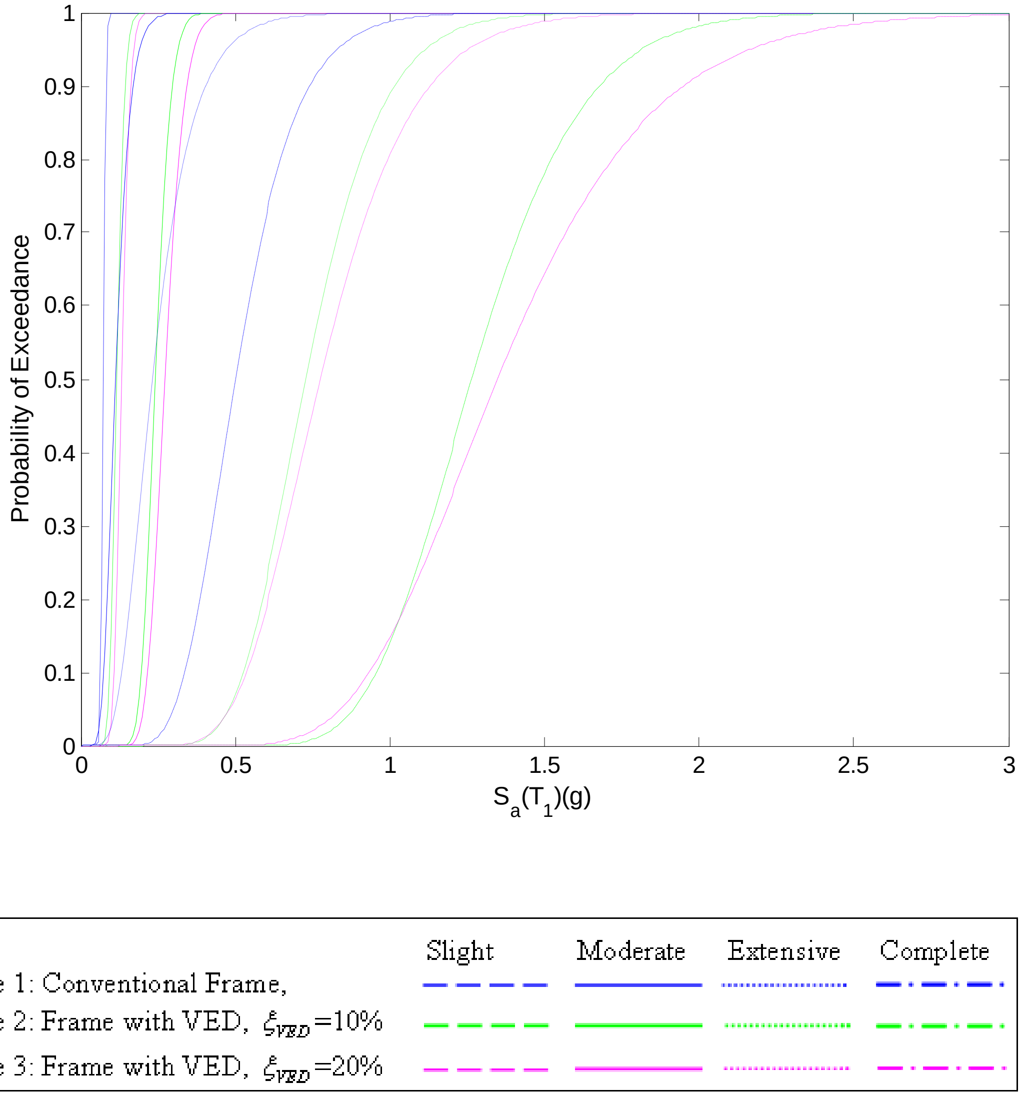

Figure 4.4 Nonstructural drift-sensitive fragility curves developed for 12 storey frames

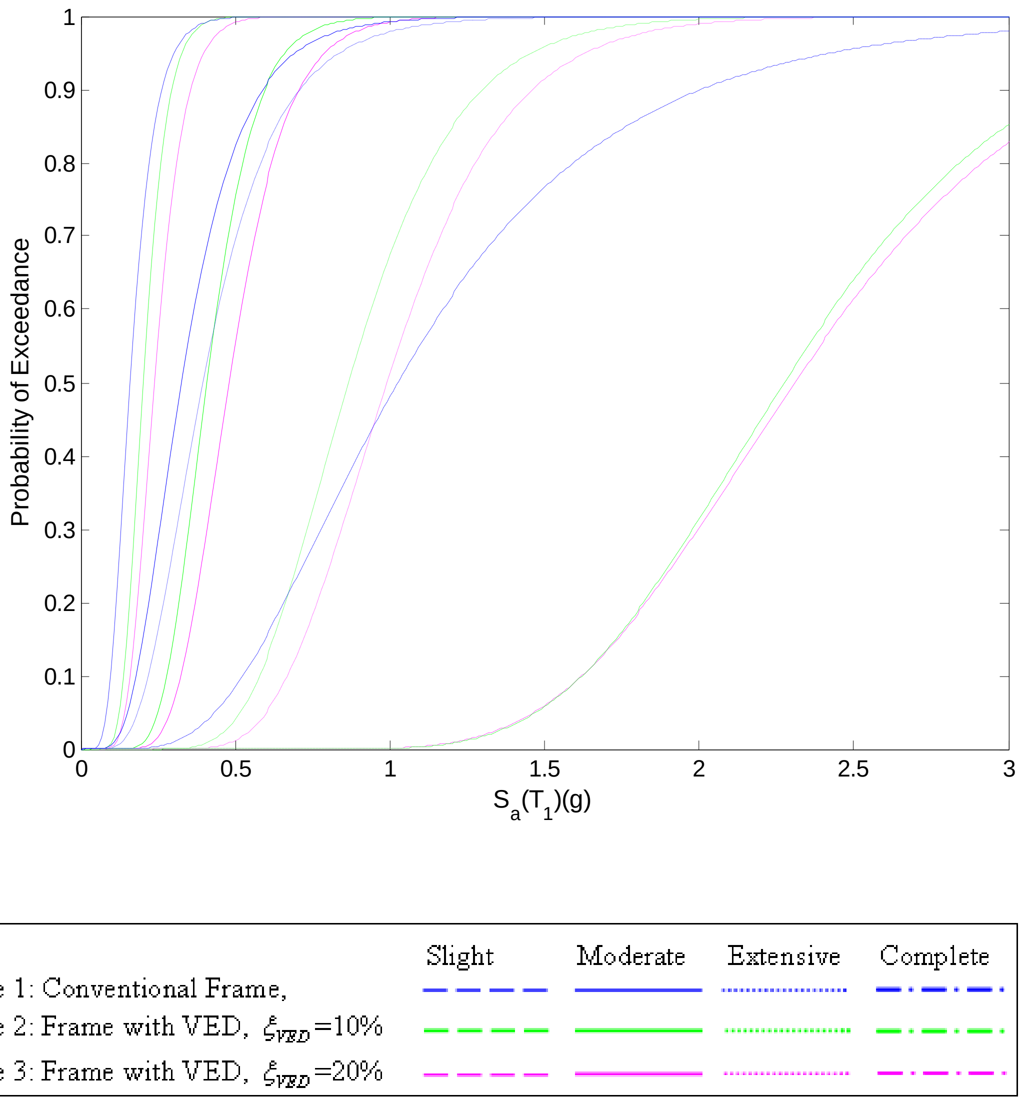

Figure 4.5 Nonstructural acceleration-sensitive fragility curves developed for 5 storey frames

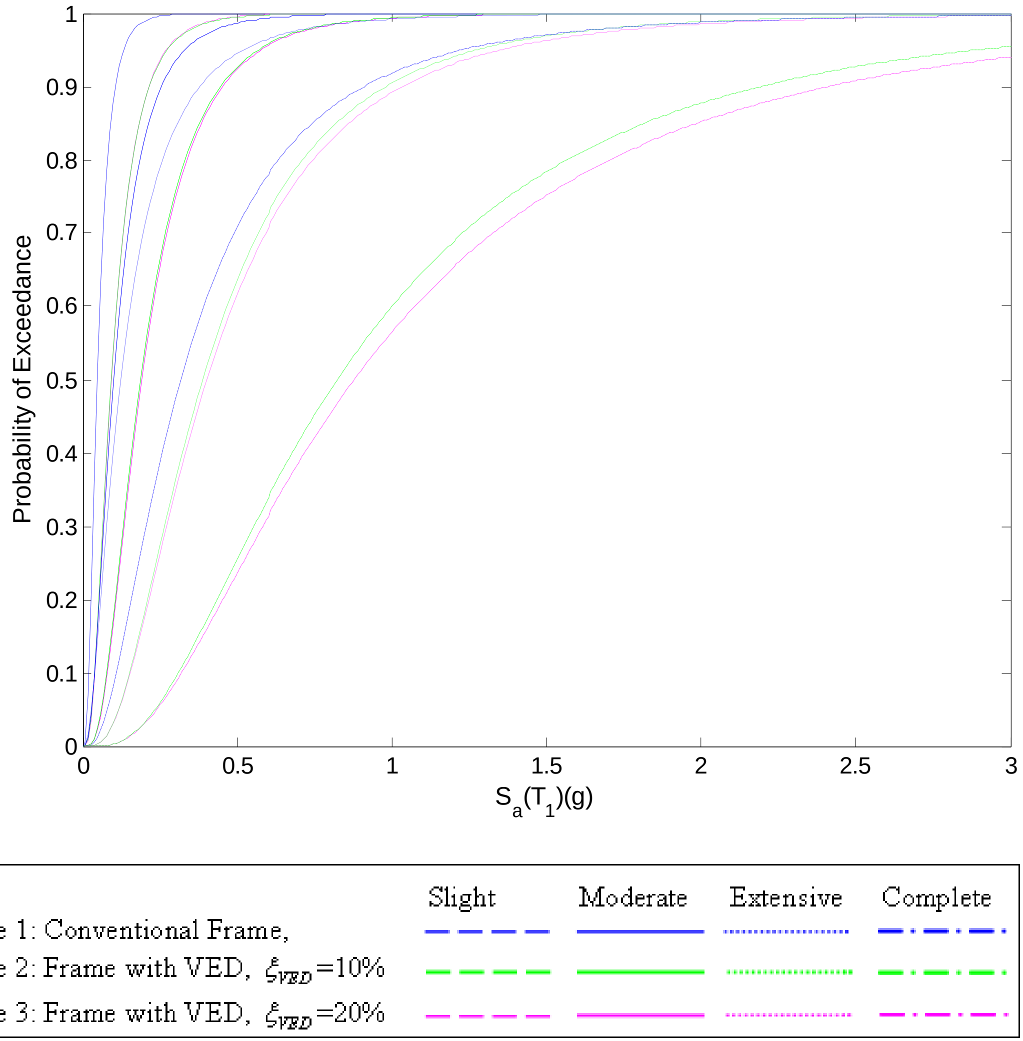

Figure 4.6 Nonstructural acceleration-sensitive fragility curves developed for 12 storey frames

With the aim of better comparing the seismic reliability of frame systems with viscoelastic dampers with that of conventional moment resisting frame systems, the seismic risk which was the convolution of fragility curve and seismic hazard curve was determined for each frame system. The seismic risk of 5 storey and 12 storey case study frames is presented in Figures 4.7 and 4.8 in terms of point estimates of performance limit state probabilities, respectively.

As it is seen from the figures, for both the moment resisting frame system and frame systems with viscoelastic dampers, the annual probability of exceedance was close to each other for small inter-storey drift ratios. However, at larger drifts, the difference became evident. For both 5 and 12 storey frames, the viscoelastic damper in the system reduced the drift demands of the frames which led to a decrease in the annual likelihood of exceedance of inter-storey drift ratio.

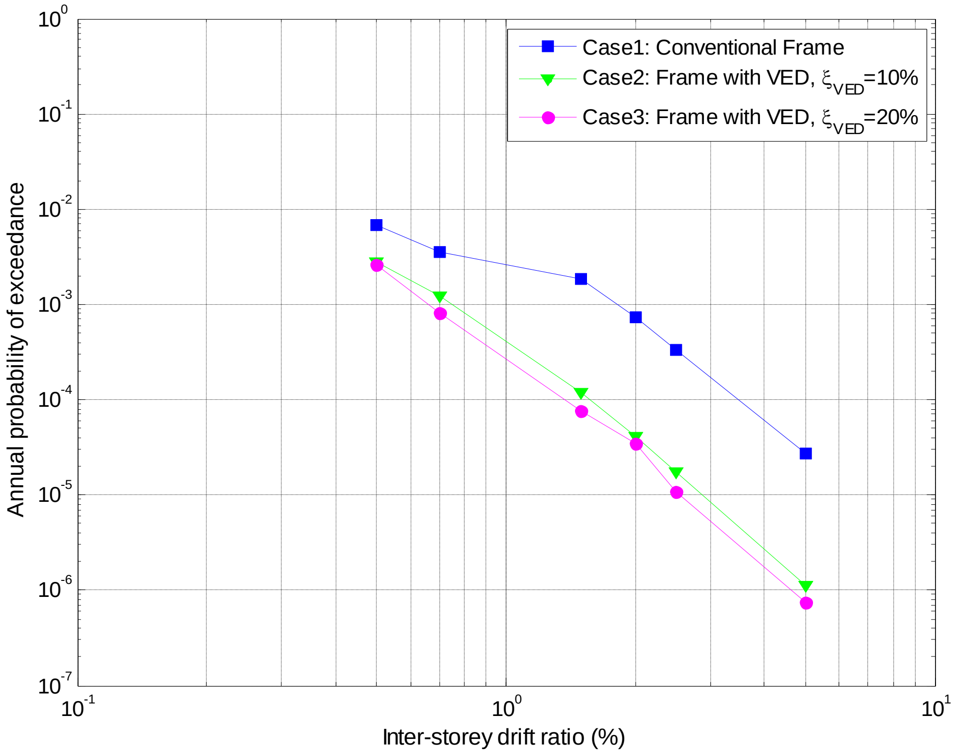

Figure 4.7 Annual probability of exceedance of inter-storey drift ratio for 5 storey frames

The annual probabilities of exceedance of performance limit states of immediate occupancy, life safety, and collapse prevention are summarized in Table 4.3. As seen from the table, all case study frames were close to each other in the annual likelihood of exceedance of immediate occupancy limit level. This might be explained as the drift limits of 0.7% and 0.5% were used for defining the limit state for the moment resisting frame systems and viscoelastically damped frame systems, respectively. Especially, for performance limit state of life safety, the difference between the frame with viscoelastic dampers and the moment resisting frame system became more significant.

For example, when the annual probability of exceedance of life safety performance limit states of 5 storey frames were evaluated, it was pointed out that the annual probability of exceedance obtained for moment resisting frame systems were about 2.8 and 4.3 times the annual probability of exceedance obtained for Case 2 and Case 3 frame systems with viscoelastic dampers, respectively. For 12 storey frames, this ratio became approximately 4.7 and 5.3. It migth be recalled that the drift limits of 2.5% and 1.5% were used for defining the life safety limit state for the moment resisting frame systems and for the viscoelastically damped frame systems, respectively.

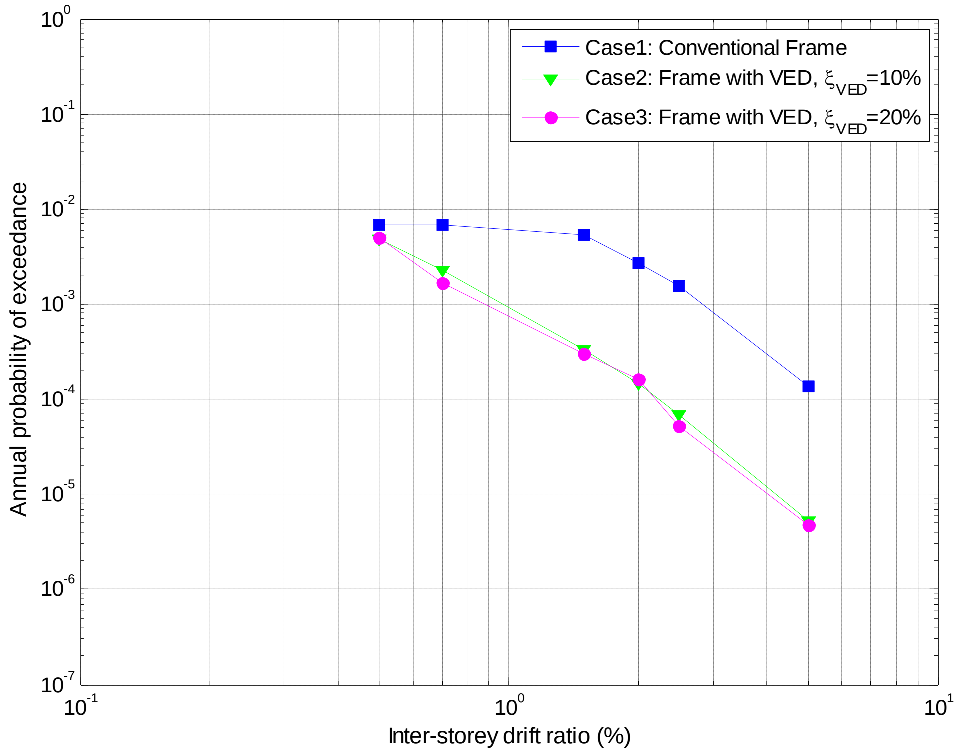

Figure 4.8 Annual likelihood of exceedance of inter-storey drift ratio for 12 storey frames

Furthermore, from Table 4.3, it was found that both 5 storey and 12 storey moment resisting frame systems and the frame systems with viscoelastic dampers had a similarity in the annual probability exceedance of the performance limit level of collapse prevention. In determination of the limit level of collapse prevention, the drift limits of 5.0% and 2.0% were used for the moment resisting frame systems and viscoelastically damped frame systems, respectively. When 5 storey and 12 storey frames were compared with each other, it was pointed out that 12 storey frames had greater annual probability of exceedance of performance limit states than 5 storey frames.

Table 4.3 Annual probability of exceedance of performance limit states

Frames		Performance Limit Levels		
		Immediate Occupancy	Life Safety	Collapse Prevention
5 Storey	Case1: Conventional Frame	$3.55\text{x}10^{-03}$	$3.30\text{x}10^{-04}$	$2.68\text{x}10^{-05}$
	Case2: Frame with VED, ξ_{VED}=10%	$2.77\text{x}10^{-03}$	$1.19\text{x}10^{-04}$	$4.08\text{x}10^{-05}$
	Case3: Frame with VED, ξ_{VED}=20%	$2.64\text{x}10^{-03}$	7.67E-05	$3.49\text{x}10^{-05}$
12 Storey	Case1: Conventional Frame	$6.80\text{x}10^{-03}$	$1.56\text{x}10^{-03}$	$1.35\text{x}10^{-04}$
	Case2: Frame with VED, ξ_{VED}=10%	$4.76\text{x}10^{-03}$	$3.35\text{x}10^{-04}$	$1.45\text{x}10^{-04}$
	Case3: Frame with VED, ξ_{VED}=20%	$4.92\text{x}10^{-03}$	$2.96\text{x}10^{-04}$	$1.62\text{x}10^{-04}$

CHAPTER 5

CONCLUSIONS

In this study, the seismic fragility analysis of 5 storey and 12 storey conventional moment resisting frame and viscoelastically damped frame was conducted. For this, firstly, the analytical structural and nonstructural fragility curves were generated in terms of the first mode spectral acceleration using nonlinear analysis in time domain. A set of 15 natural earthquake records were considered in nonlinear analysis. For structural fragility curves, three limit states namely, immediate occupancy, life safety, and collapse prevention suggested by FEMA 356 were used. Moreover, for nonstructural drift-sensitive and acceleration-sensitive fragility curves slight, moderate, extensive, and complete limit levels were utilized according to HAZUS. Furthermore, by using fragility curves generated in this work, the seismic risk of the structural systems based on annual likelihood of exceedance of performance limit levels was examined. According to the results of this study, the following conclusions are drawn on the efficiency of viscoelastic dampers in mitigating the seismic responses of the case studied buildings:

- The computed analytical fragility curves corresponding to the stated performance limit states and seismic risk analysis appeared to make intuitive sense relative to the performance of the conventionally and viscoelastically damped frames designed.

- It was observed that failure probabilities were highly sensitive to the structural characteristics utilized in the design of the building.

- Comparison of the fragility curves showed that viscoelastic dampers added to the both 5 and 12 storey frames assisted to reduce story drifts, the simulated fragility curves for the frames with the viscoelastic dampers showed significant improvement compared to those of moment resisting frame system

- by rising the median values of the fragility curves (i.e. moving the fragility curves to greater intensity values) and decreasing the likelihoods of exceedance of performance limit states.

- The difference between the fragility curves of the conventionally designed and viscoelastically designed frames is much more obvious, especially at the life safety limit state. For instance, the fragility curves for the frame system designed with a supplemental effective damping ratio of 20% are less fragile compared to those for conventional frame system by as much as 2.1 times based on median first mode spectral acceleration values.

- Furthermore, since the same limits for each performance state were used for all types of frames, the difference in the nonstructural drift-sensitive and acceleration sensitive fragility curves of the moment resisting system and frame systems with viscoelastic dampers became more pronounced than that in structural fragility curves.

- It was pointed out that the results of the seismic risk analysis were parallel with the results of the structural fragility analysis. From the result of the seismic risk analysis, it was observed that 12 storey frames had greater annual probability of exceedance of performance limit states than 5 storey frames.

- When conventionally designed and viscoelastically designed frames were compared with each other, it was found that the frames designed with viscoelastic dampers providing supplemental effective damping ratio of 20% were very effective in decreasing the annual probability of exceedance of performance limit state up to five times when life safety performance limit state considered.

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