UNIVERSITY OF GAZİANTEP
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ASSESSING EFFECTS OF TYPE AND DISTRIBUTION OF ECCENTRIC STEEL BRACES ON SEISMIC VULNERABILITY OF MID-RISE REINFORCED CONCRETE BUILDINGS

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ADİL EMRE ÖZEL
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Assessing Effects of Type and Distribution of Eccentric Steel Braces on Seismic Vulnerability of Mid-Rise Reinforced Concrete Buildings

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Supervisor
Assist. Prof. Dr. Esra METE GÜNEYİSİ

by

Adil Emre ÖZEL

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Prof. Dr. Ramazan KOÇ
Director

I certify that this thesis satisfies all the requirements as a thesis for the degree of Master of Science.

Assoc. Prof. Dr. Mustafa GÜNAL
Head of Department

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.

Assist. Prof. Dr. Esra METE GÜNEYİSİ
Supervisor

Examining Committee Members	Signature
Assoc. Prof. Dr. Mehmet GESOĞLU	______________________
Assist. Prof. Dr. Esra METE GÜNEYİSİ	______________________
Assist. Prof. Dr. M. Akif KÜTÜK	______________________

ABSTRACT

ASSESSING EFFECTS OF TYPE AND DISTRIBUTION OF ECCENTRIC STEEL BRACES ON SEISMIC VULNERABILITY OF MID-RISE REINFORCED CONCRETE BUILDINGS

ÖZEL, Adil Emre
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In this study, the seismic vulnerability of a mid-rise reinforced concrete (R/C) building retrofitted by eccentric steel braces is investigated through fragility analysis. For this purpose, a representative six story mid-rise R/C building was chosen. The design of sample building was conducted in accordance with 1975 version of the Turkish seismic design code. The effect of different types and distributions of eccentric steel braces on the seismic performance of the retrofitted building was studied. For the strengthening of the original structure, D, K, and V type eccentric bracing systems were utilized and each of these bracing systems was applied with four different distributions in the structure. For fragility analysis, the study employed a set of artificially generated earthquake acceleration records compatible with the elastic code design spectrum. Nonlinear time history analysis was performed in order to analyze the structures subjected to this set of earthquake accelerations generated in terms of peak ground accelerations (PGA). The fragility curves were developed in terms of PGA with lognormal distribution assumption for different limit states, namely slight, moderate, major, and collapse. As a result, the improvement in seismic performance of this type of mid-rise R/C building resulting from retrofits by different types of eccentric steel braces was obtained by formulation of the fragility enhancement.

Key Words: Eccentric steel brace, Reinforced concrete building, Retrofit, Seismic response, Performance assessment, Seismic fragility analysis

ÖZET

DEĞİŞİK TİP VE DAĞILIMDAKİ DIŞ MERKEZLİ ÇELİK ÇAPRAZLARIN ORTA KATLI BETONARME BİNALARIN DEPREM HASAR OLASILIĞI ÜZERİNDEKİ ETKİSİ

ÖZEL, Adil Emre
Yüksek Lisans Tezi, İnşaat Mühendisliği Bölümü
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Bu çalışmada, dış merkezli çelik çapraz sistemler kullanılarak güçlendirilmiş orta katlı betonarme bir binanın depremsel güvenilirliği hasar olasılık analiziyle araştırılmıştır. Bu amaçla, altı katlı betonarme bir bina örnek olarak seçilmiştir. Örnek binanın tasarımı 1975 Türk Deprem Yönetmeliği esas alınarak yapılmıştır. Değişik tiplerdeki ve dağılımlardaki dış merkezli çelik çapraz sistemlerin binanın deprem performansı üzerine etkisi incelenmiştir. D, K ve V tipi dış merkezli çelik çapraz sistemlerin ve bu çapraz tiplerinin bina üzerindeki dört farklı dağılımlarının binanın deprem performansı üzerine etkisi incelenmiştir. Hasar olasılık analizi için, yönetmelikte verilmiş olan elastik spektrumla uyumlu oluşturulmuş yapay deprem yer ivmeleri kullanılmıştır. Yapıların analizinde, nonlineer zaman tanım alanında analiz yöntemi uygulanmıştır. Yapıların, az, orta, ağır ve yıkım hasar durumları için log-normal dağılıma sahip hasar olasılık eğrileri maksimum yer ivmesine bağlı olarak oluşturulmuştur. Sonuç olarak, değişik tip ve dağılımlardaki çelik çapraz sistemlerin, benzer tipteki orta katlı betonarme yapıların deprem performansları üzerinde sağladığı iyileşme hasar olasılık analizi ile formüle edilmiştir.

Anahtar kelimeler: Dış merkezli çelik çapraz, Betonarme bina, Güçlendirme, Deprem performansı, Performans değerlendirmesi, Hasar olasılık analizi

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LIST OF SYMBOLS/ABBREVIATIONS

C	Capacity
CFRPC	Carbon fiber reinforced polymer composite
D1,D2,D3,D4	D braced frame types from 1 to 4
D	Demand
E	Young modulus in (MPa)
f_y	Yield stress of steel in (MPa)
g	Gravitational acceleration
I	Intensity
IM	Intensity measure of input ground seismic hazard
K1,K2,K3,K4	K braced frame types from 1 to 4
LS	Limit state
M3	Uniaxial moment type hinge
MR	Magnetorheological
P	Probability
PGA	Peak ground acceleration
PMM	Axial force biaxial moment type hinge
R^2	Correlation coefficient
R/C	Reinforced concrete
S1,S2,…,S6	Column types from 1 to 6
S_a	Spectral acceleration
S_d	Spectral displacement
S_v	Spectral velocity
St	Steel
T_1	First period of original frame
W	Weight of the building
x	Intensity level
X	Lognormal distributed ground motion intensity parameter
V1,V2,V3,V4	V braced frame types from 1 to 4
Z1	Local site class according to 2007 Turkish Seismic Code
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
ø	Radius of steel bars in (mm)

CHAPTER 1

INTRODUCTION

Mid-rise reinforced concrete (R/C) frame buildings constitute the major part of the building stock in urban areas, in Turkey and they are used as residential or commercial buildings. Recent devastating earthquakes in Turkey such as, 1999 İzmit, 1999 Bolu-Düzce, 2002 Afyon-Sultandağ, 2003 Tunceli, İzmir and Bingöl, and the last 2010 Elazığ earthquakes demonstrated that the majority of this building stock, which were built before the enforcement of the 1998 Turkish Earthquake Code (ABYYHY, 1998), are highly vulnerable to seismic action. Since, it is inevitable to experience devastating earthquakes in the near future in Turkey, currently, residential buildings and industrial facilities are in urgent need of retrofitting to provide life safety to inhabitants and to protect the developing economies from the results of significant damages. In other words, recent changes and updates of the earthquake codes and lessons learned from destructive earthquakes also implies the need for possible upgrading of existing building stock for safe environment.

However, it is of critical significance that the structures that need seismic retrofitting must be identified correctly, and an optimal retrofitting must be conducted economically. Once the decision for retrofitting is made, seismic retrofitting can be performed through several methods with objectives of increasing the load, deformation, and/or energy dissipation capacity of the structure (FEMA 356, 2000). Conventional retrofitting methods generally consist of addition of new structural elements and/or enlarging the existing members (Newman, 2001). Among the conventional design methods, in general, addition of shear walls or steel bracings is the most preferred strengthening methods because of their effectiveness, relative ease, and lower total cost in comparison to the enlargement of the existing structural members. In this study, from the conventional retrofitting approaches, addition of different types and distributions of eccentric steel bracing systems for improving the seismic performance of the existing structure is investigated.

For rational estimation and reduction of the potential seismic losses associated with existing and also with the retrofitted structures, the seismic performance level should be quantitatively measured through risk assessment of structures. Development of fragility curves is an effective tool for risk assessment of structural systems as it can be used for probabilistic estimation of seismic losses and eventually enables decision-making activities for seismic risk reduction. Therefore, in this thesis, for comparison of the seismic performance of the selected sample structure before and after retrofit by using eccentric steel bracing systems, fragility analysis is utilized.

1.1. Objectives of the Thesis

The aim of this thesis is to develop fragility curves for the original and the retrofitted buildings by addition of different types and distributions of eccentric steel bracing systems. From the developed fragility curves, the improvement in the seismic reliability of the sample structure resulting from such retrofits is also formulated by providing basic information for fragility enhancement.

Firstly, the analytical model of the selected structure is constructed. The structure was designed according to 1975 Turkish Seismic Code (ABYYHY, 1975). Then, the analytical models of the retrofitted buildings with eccentric steel braces were also constructed with the bracing having the same slenderness ratio. Secondly, nonlinear time history analyses were performed to evaluate the seismic response of the original and retrofitted buildings. For nonlinear time history analysis, 200 available synthetic earthquakes consistent with the code design spectrum were used. Therefore, 200 maximum inter story drift were obtained from each nonlinear time history analyses performed using the 200 simulated response-spectrum-compatible ground-motion acceleration time histories for each type of building. Using the seismic response data obtained from nonlinear time history analysis, the fragility curves of the buildings for slight, moderate, major, and collapse damage states according to HAZUS (1997) were developed by using the log-normal distribution assumption for the fragility curves.

Consequently, the results obtained from seismic fragility analysis provide useful information for seismic reliability of the retrofitted structures, and make it possible to evaluate the improvement of the similar mid-rise R/C buildings resulting from such retrofits by providing basic information for fragility enhancement.

1.2. Outline of the Thesis

Chapter 1-Introduction: Aim and objectives of the thesis are introduced.

Chapter 2-Literature review and background: A literature survey based on this thesis was given. For this, firstly, the use of steel bracing systems for seismic retrofitting of structures in the literature was explained. Secondly, the utilization of fragility analysis and available studies in the literature containing fragility analysis were summarized.

Chapter 3-Case study: This chapter provides a description of analytical models of the selected sample building and the retrofitted buildings with different types and distributions of eccentric steel bracings. Additionally, the methodology used in the fragility analysis was summarized and details of every step of fragility analysis were also given in this chapter. Therefore, the determination of the damage levels including nonlinear static analysis results of the original and retrofitted buildings, the properties of the ground motion acceleration records used were given in this chapter. Nonlinear time history analysis and development of the fragility curves from the seismic response data were also described in this chapter.

Chapter 4-Discussion of the results: Results obtained from the seismic assessment of the vulnerability of the sample building and the retrofitted building with eccentric steel braces through fragility analysis was presented. Discussion on the results of the fragility analysis and the fragility curves were given in this chapter.

Chapter 5-Conclusions: Conclusions based on results of this study as well as recommendation for future studies were presented.

CHAPTER 2

LITERATURE REVIEW AND BACKGROUND

2.1. Seismic Retrofitting Strategies

The performance evaluation of existing concrete structures shows that a considerable number of buildings need to be repaired or strengthened. The main reasons for the necessity to fortify existing structures are: the design and construction defects, modifications in codes of practice and standards, environmental effects, changes in their usage and loading conditions and the need to increase the number of floors (Rahai and Alinia, 2008).

There are many seismic retrofit techniques available, depending upon the various types and conditions of structures. Therefore, the selection of the type of intervention is a complex process, and is governed by technical as well as financial and sociological considerations. The following are some factors affecting the choice of various intervention techniques (Thermou and Elnashai, 2002).

- Cost versus importance of the structure,
- Available workmanship,
- Duration of work/disruption of use,
- Fulfillment of the performance goals of the owner,
- Functionally and aesthetically compatible and complementary to the existing building,
- Reversibility of the intervention,
- Level of quality control,
- Political and/or historical significance,

- Structural compatibility with the existing structural system,
- Irregularity of stiffness, strength and ductility,
- Adequacy of local stiffness, strength and ductility,
- Controlled damage to non-structural components,
- Sufficient capacity of foundation system, and
- Repair materials and technology available.

A retrofit strategy is a basic approach adopted to improve the probable seismic performance of the building or otherwise reduce the existing risk to an acceptable level (ATC 40, 1996). For the improvement of the seismic performance of the building, available technical retrofitting strategies consist of system complementation, system strengthening and stiffening, enhancing deformation capacity, and reducing seismic demand in the structure. System complementation means correcting some deficiencies that may lead to local failures such as lack of adequate chord or collector elements at the diaphragms, inadequate bearing length at precast element supports, or inadequate anchorage of structural components. Enhancing deformation capacity of the structure means improvement in building seismic performance by improving the deformation resistance ability of the structural components under earthquake loads. The methods used for enhancing deformation capacity of the available structure are; adding confinement to the existing structural elements, making local reductions in stiffness, modifying columns to alter failure mechanisms, providing supplemental support at locations subjected to deformation induced failure. Reducing earthquake demand is a newly developed strategy that includes reductions in the mass of the buildings and installation of base isolation or energy dissipation systems. System stiffening and strengthening are the most commonly used retrofitting strategies adopted for buildings with insufficient lateral force resisting capacity. Generally, typical systems for stiffening and strengthening include addition of new structural elements such as addition of shear walls, or addition of steel braces (ATC 40, 1996).

In order to strengthen concrete structures against lateral and seismic loading, the designers generally tend to lighten the total weight of structures, as well as strengthening them with shear walls, steel or concrete jackets or fiber reinforced polymer layers, external pre-stressing, and other popular means of bracings (Rahai and Alinia, 2008).

2.1.1. Addition of R/C Structural Shear Wall

The lateral load capacity of an existing structure may be increased by adding new structural elements to resist part or all of the seismic loads of the structure, leaving the old structure to resist only that part of the seismic action for which it is judged reliable (UNDP-UNIDO, 1983). Addition of shear walls is one of the most familiar structural levels of retrofitting technique to strengthen existing frame structures. This method is efficient for controlling lateral drifts and for reducing damage in frame components. Commonly, repair of an existing shear wall or infilling one of the bays in the frame structure is taken into account. The walls are commonly cast-in-situ but may be installed with shotcrete. Precast, prefabricated concrete elements can be employed to form new shear walls but their details are particularly critical and difficult to supply the desired performance. However, in some cases, shotcrete and precast concrete panels may be used so as to decrease time and cost.

Monolithic reinforced concrete shear walls can be situated either along the periphery of the building or inside of it. Adding walls along periphery is often easier as it does not upset the interior function of the room layout, although it will alter appearance and window layouts. In the first case, the main vertical end reinforcement for flexure and the web reinforcement for shear pass continuously along the entire height of the wall. The main problems are providing an adequate connection of the new shear walls with the floor and roof diaphragms and the foundations. The connection must transmit the shear forces between the existing floor or roof structure and the new shear wall. In the second case, shear walls placed inside of the building are connected with the floor structures by vertical longitudinal reinforcement passing through opened holes in the existing slab, concrete dowels or lugs formed by opened holes in the slab, diagonal bars placed in the dowels. The main longitudinal reinforcement situated at the wall ends passes continuously through the holes in the slab and must be spliced at convenient locations. However, most of the web reinforcement is interrupted at every slab level and it must be spliced by diagonal and vertical bars placed in the holes through the slab. The ends of the shear walls are also generally attached to existing building columns to provide gravity loads to counter uplift tendencies of the wall ends (UNDP-UNIDO, 1983).

Various research studies have been carried out for structural shear walls, and findings corresponding to detailed interventions have been reported (Altin et al., 1992; Pincheira and Jirsa, 1995; Lombard et al., 2000; Inukai and Kaminosono, 2000; Güneyisi and Altay, 2005). These investigations reveal that with the infilling process, details play an important role in the response of panels and the overall structure. The infilling process has a tendency to stiffen the structure such that the base shear can increase. The overturning influence and base shear are concentrated at the stiffer infill locations. As a result, strengthening of the foundation is typically required at these locations.

Jirsa and Kreger (1989) investigated one-story infill walls using four specimens. In their study, they used three one-bay, single-story, non-ductile R/C frames that were designed to represent 1950s construction techniques. These included wide spacing in the column shear reinforcement and compression splices that were insufficient to develop the required tensile yield strength. In their study, the first three walls varied in their opening locations. Longitudinal reinforcement was added adjacent to the existing columns to improve the continuity of the steel in the fourth specimen. The first three experiments had brittles failures due to the deficient column lap splices, even though the infill strengthened the frame. The fourth specimen enhanced both the strength and ductility of the frame.

2.1.2. Reinforced Concrete Jacketing

Reinforced concrete jacketing of a joint is performed in such a way that all the members connected at the joint collaborated together. It is generally appropriate only when both columns and beams framing to the joint are also jacketed. For adequate bond between the original and the new concrete and possibly for welding of the new reinforcement to the existing reinforcement, the concrete cover must be chipped away. It is necessary that sufficient thickness of the jacket be provided in order that the great number of reinforcement bars (longitudinal column and beam bars, ties, and stirrups) can be installed. Appropriate stirrups and closely spaced and adequately anchored ties are of great importance. The vertical and horizontal bars and ties must be assembled in a manner to form space reinforcement. It, together with the concrete jacket, must be able to transmit all the internal joint forces. Sufficient jacket stiffness is also necessary (UNDP-UNIDO, 1983).

Column retrofitting is often critical to the seismic performance of a structure. To prevent a story mechanism during an earthquake, columns should never be the weakest components in the building structure. The response of a column in a building structure is controlled by its combined axial load, flexure, and shear. Therefore, column jacketing may be used to increase strength so that columns are not damaged (Bracci et al. 1995). An example of a column jacketing is illustrated in Figure 2.1.

Recently, research has emphasized the applications of composite materials. In particular, carbon fiber reinforced polymer composite (CFRPC) material may be used for jackets when retrofitting columns. Because these jackets sufficiently confine the columns, column failure through the formation of a plastic hinge zone can be prevented (Harries et al. 1998).

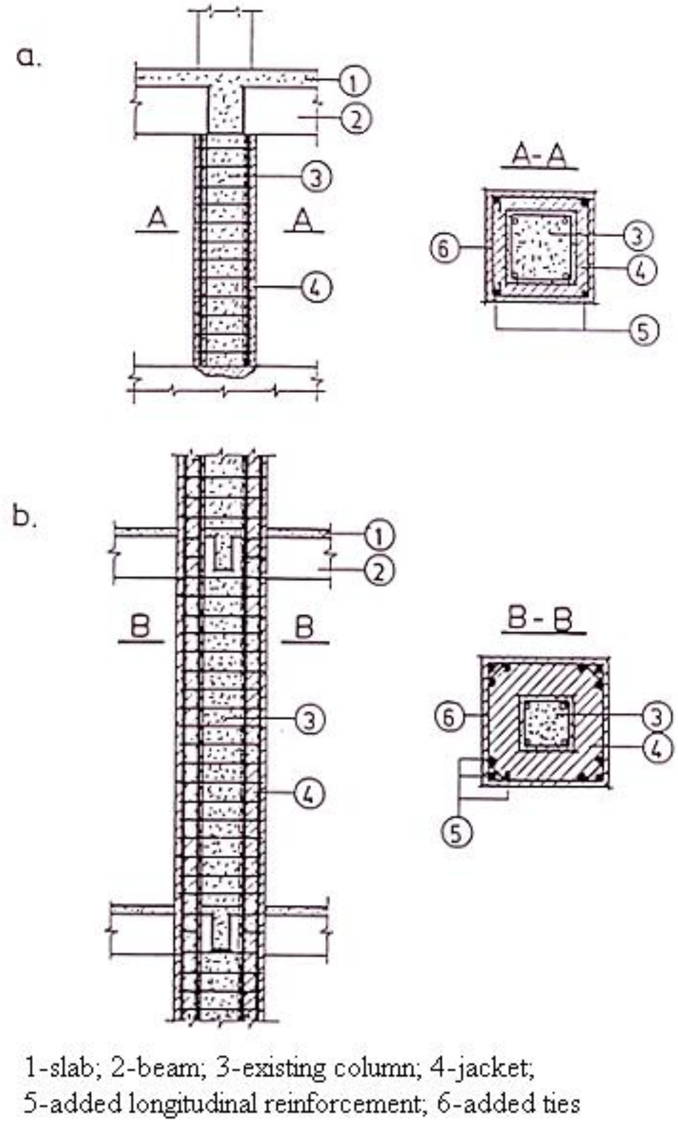


Figure 2.1 An example of a column jacketing (UNDP-UNIDO, 1983)

2.1.3. Steel Jacket

Steel jacketing has been used as a retrofit measure to enhance the flexural ductility, shear strength, or performance of lap splices in reinforced concrete columns. The steel jacket encasement provides enhanced confinement, which ultimately improves the ductility capacity through increased compressive strength and ultimate strain in the concrete (Chai, 1991). In the report of Building Construction under Seismic Conditions in the Balkan Region, steel profile jacketing technique are explained. For example, steel profile skeleton jacketing consists of four longitudinal angle profiles place one at each corner of the existing reinforced concrete column and connected together in a skeleton with transverse steel straps. They were welded to the angle profiles and can be either round bars or steel straps. Gaps and voids between the angle profiles and the surface of the existing column must be filled with non-shrinking cement grout or resin grout. In general, an improvement of the ductile behavior and an increase of the axial load capacity of the strengthened column are achieved. However, the stiffness remains relatively unchanged (UNDP-UNIDO, 1983).

2.1.4. Seismic Isolation

In recent times, numerous investigators have studied seismic isolation as a possible retrofit method (Gates et al., 1990; Constantinou et al., 1992; Tena-Colunga et al., 1997; Kawamura et al. 2000). The aim of this type of retrofit is to isolate the structure from the ground motion during earthquake events. The bearings are installed between the superstructure and its foundations. Since most bearings have excellent energy dissipation features, this method is most effectual for relatively stiff low-rise buildings with heavy loads.

2.1.5. Supplemental Energy Dissipation

Other commonly used techniques to add energy dissipation to a structure consist of installing frictional, hysteretic, viscoelastic, or magnetorheological (MR) dampers as components of the braced frames. Several researchers have studied supplemental energy dissipation techniques (Pekcan et al., 1995; Fu, 1996; Munshi, 1998; Kunisue et al., 2000; Yang et al., 2002; Güneyisi and Altay, 2008). On the other hand, FEMA 356 discusses some negative aspects in which while lateral displacements are reduced through the use of supplemental energy dissipation, the forces in the structure can increase not really if designed properly (FEMA 356, 2000).

2.1.6. Application of Braced Frames

As mentioned above, in order to provide lateral earthquake resistance of a frame structure, shear walls or steel bracings are generally adopted. Although, the use of steel braces in steel frames and the use of shear walls in reinforced concrete structures are preferred; in recent years there have been several studies for the use of steel bracings in reinforced concrete (R/C) structures, especially for the retrofitting purposes (Maheri and Sahebi, 1997; Abou-Elfath and Ghobarah, 2000; Ghobarah and Abou-Elfath, 2001; Maheri et al., 2003; Symth et al., 2004; Güneyisi and Altay 2005; Youssef et al. 2007; Mazzolani 2008). Moreover, the use of steel bracing systems for seismic retrofitting have some advantages such that; they add far less mass to the structure, they can be constructed with less disruption to the structure, they result in less loss of light, and they have a smaller effect on traffic patterns within the building (ATC 40, 1996).

In selection of the location of the bracing systems, it is significant that the effect of retrofitting on the flow of forces through the structure must be considered. Creating new undesirable weak structural members must be avoided. From a structural point of view, it may be desirable to brace as many bays of the frame as possible, so that increases in strength and stiffness are distributed uniformly. However, cost and functional considerations may bound the number of braced bays. Bracing only the perimeter frames have architectural advantages and it is not needed to change the use of interior space. An exterior steel bracing system is also beneficial with respect to torsional behavior of the structure under seismic loads. If exterior frames are strengthened only, then the increase in torsional resistance is provided and structural symmetry is obtained.

If exterior bracing systems were used, its impact on building aesthetics must be considered. The distribution of steel braces can make the difference between a system that enhances or detracts from the appearance point of view of the building. In addition to this, the bracing system should appear to be a natural component of the available structure. The property of steel braces such as configuration, detailing, and color can be used to give personality to the original building (Badoux and Jirsa, 1990).

The logical arrangement of steel bracings in plan and levels has a great influence on the response and on the lateral displacement of structures. In the case of braces with high slenderness ratios and while they are in tension, the system may experience excessive horizontal or vertical deformations before failure of the joints. On the other hand if the bracing members are in compression, lateral deflection may easily occur; and regarding the possibility of occurrence of plastic deformations, the structures' hysteresis curves become unstable. Bracings with medium slenderness ratios have a brittle behavior, and thus, when in compression, would not provide enough stiffness to contribute against lateral loads. Bracings members with small slenderness ratios can sustain compressive forces without undergoing buckling, and generally have a good dynamic behavior (Rahai and Alinia, 2008). Wakabayashi carried out a vast amount of research on highly slender bracings and concluded that these bracings do not have enough energy dissipation capacity under cyclic loadings with constant displacement cycles. Then, by decreasing the slenderness ratios, the hysteresis cyclic curves became more complex and due to the compressive buckling and plastic tensile elongations, plastic hinges were developed in them (Wakabayashi, 1970).

Steel bracing systems are classified as concentric steel bracings and eccentric steel bracings. Concentric steel bracing systems are used for the rehabilitation of nonductile R/C buildings in Mexico and Japan (Kawamata and Masaki, 1980; Canales and Briseno, 1992). Examples of different patterns of concentric steel bracing systems are illustrated in Figure 2.2. Within the last two decades, there have been many research works on improving the performance of concentric bracing systems. Astaneh et al. presented special details for joint connecting systems for steel bracings. The details, while let sufficient energy absorption in the connecting plate, prevented them from failure. They proposed a maximum slenderness ratio and recommended some guidelines for the design of bracing members and their connection systems. The research works continued until they proposed a new system of bracing which had stable load–displacement properties; and by aiming the concurrence compressive and tensile resistance of bracing members, their works led to a novel composite system. The new system consisted of a steel core covered by concrete coating (Asteneh et al., 1986).

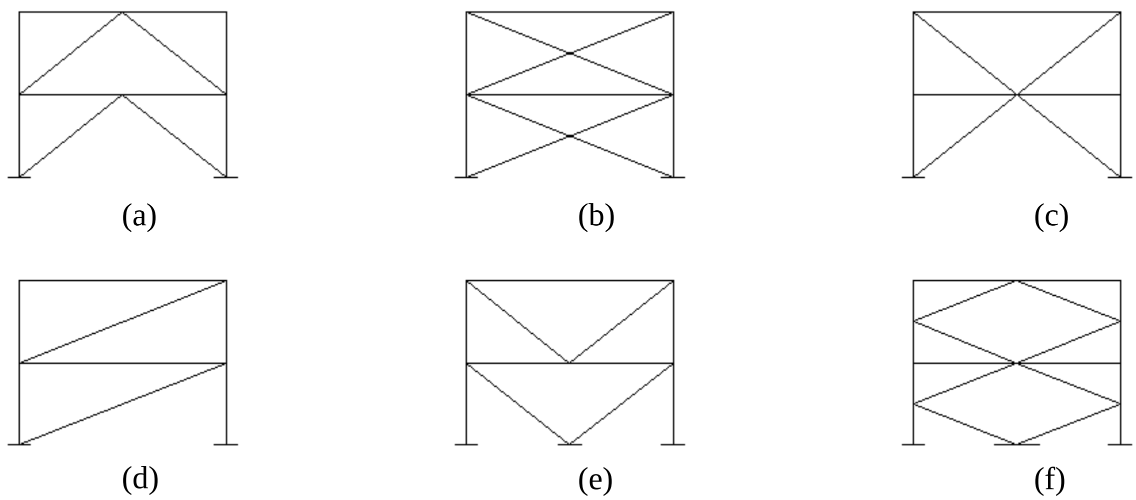
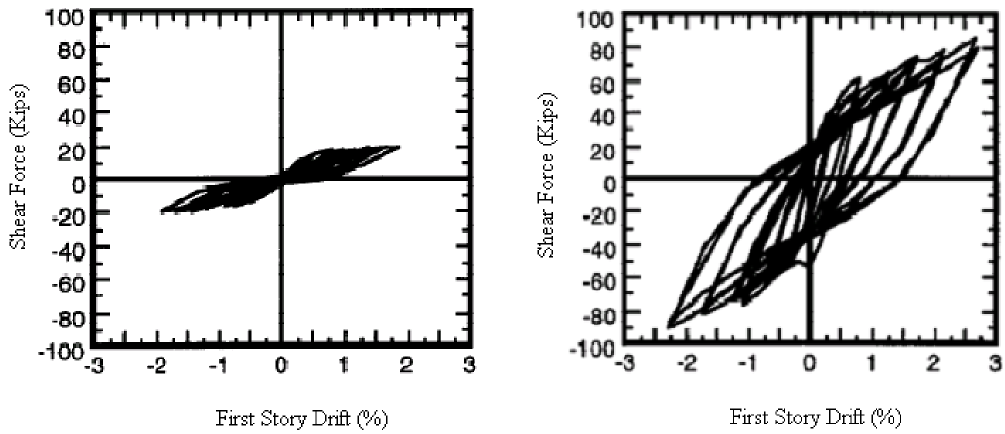


Figure 2.2 Different types of concentric steel bracing systems (a) opposite V-bracing, (b) X-bracing, (c) 2 story X-bracing, (d) diagonal bracing, (e) V-bracing, and (f) K-bracing

Goel and Masri investigated a weak slab-column building structure using a one-third scale, two-bay, two-story R/C slab-column frame specimen. They tested two different phases of the steel bracing on both the exterior and interior bays, respectively, and compared them with the original R/C frame. The concentric steel bracing system, namely inverted V-braced frame specimens were utilized. Figure 2.3 compares the hysteretic loops for the unretrofitted and retrofitted frame, showing the increase in strength, stiffness and energy dissipation due to retrofit. This observation was true for both retrofitted specimens. In particular, the results after applying the concrete-filled braces showed that the frame behaved in a very ductile manner through all fifteen cycles, with no failures (Goel and Masri, 1996).



(a) (b)

Figure 2.3 Hysteric loop of (a) the R/C frame and (b) braced frames (Goel and Masri, 1996)

Application of steel braces for seismic retrofitting of a reinforced concrete frame is more effective and easier than other methods. Steel braces are very important members for the earthquake resistance of a steel structure. Therefore, this property can also be used in the retrofitting of undamaged or damaged concrete building. Typical concentric type steel bracing applications are shown in Figures 2.4 and 2.5.

Figure 2.4 An example of inverted V type steel braces application for seismic retrofit (Coşkun, 2010)

Figure 2.5 An example of V type steel braces application for seismic retrofit (Coşkun, 2010)

The use of eccentric steel bracing in the rehabilitation of R/C structures has lagged behind concentric steel bracing applications due to the lack of sufficient research and information about the design, modeling and behavior of the combined concrete and steel system. To facilitate the application of eccentric bracing in rehabilitation, further research is needed in several areas such as testing of the R/C beam–steel link connection details and design as well as the development and implementation of link element models in analysis software (Ghobarah and Abou-Elfath, 2001). Different brace patterns are used in eccentrically braced steel frames. Examples of these patterns comprise V-bracing, K-bracing, X-bracing and Y-bracing are demonstrated in Figure 2.6.

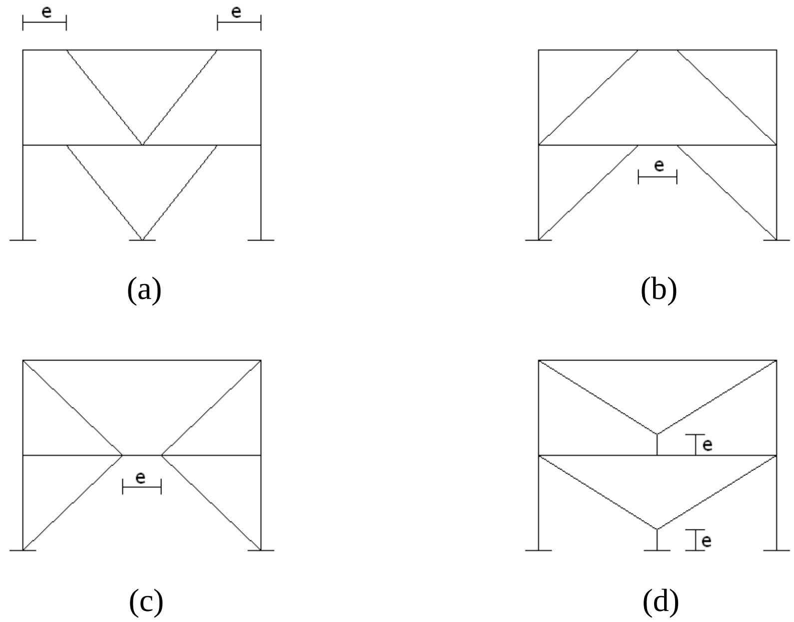


Figure 2.6 Different types of eccentric steel bracing systems (a) V-bracing, (b) K-bracing, (c) X-bracing, and (d) Y-bracing

One main advantage of eccentric braces is the possibility to select the cross section and the length of links and the cross section of braces almost independently of each other, thus following modulating stiffness and strength as required. In fact, the cross section of the link determines the story-shear plastic strength, whereas the link length and the brace cross-section quantify the stiffness of the bracing system (Mazzolani, 2008). Figure 2.7 illustrates the basic geometry of the R/C structure and typology of the eccentric brace system adopted for seismic retrofitting.

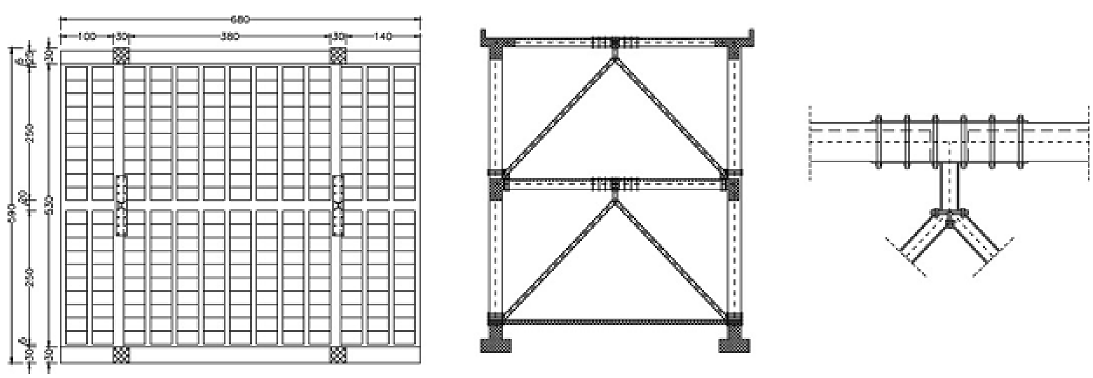


Figure 2.7 Basic geometry of the R/C structure and typology of the eccentric brace retrofitting system (Mazzolani, 2008)

2.2. Fragility Analysis

Recent natural disasters and malevolent man-made events have increased the interest in understanding extreme events and planning for their occurrence. Recent focus has been given to preparedness activities that improve response to and recovery from such extreme events. Indeed, the utility of risk-based decision making is not necessarily to articulate the ‘‘best’’ policy option, but rather to avoid the extreme, the worst, and the most disastrous options. To do so, a decision maker must be able to measure the outcomes of such extreme events and measure how risk management can control them (Barker and Haimes, 2009).

Low-rise and mid-rise R/C frame buildings constitute the major part of the building stock in Turkey and they are generally occupied for residential and commercial purposes. This type of construction has replaced the traditional building techniques especially in densely populated cities in recent years due to the rapid urban growth. However, the majority of this building stock is generally composed of low-engineered buildings which have not been adequately designed to resist earthquake forces. Recent devastating earthquakes in Turkey demonstrated that these types of buildings are highly vulnerable to seismic action. The replacement of these buildings with an earthquake resistant building stock will eventually occur, but not soon. Meanwhile it is inevitable to experience devastating earthquakes in the near future in Turkey, a country evidently under high seismic risk. Therefore, quantification of seismic risk posed by the existing building stock is an urgent issue for effective risk mitigation. There are two ingredients for quantifying seismic risk: identification of the seismic hazard and the assessment of the fragility of the existing building stock (Erberik, 2008).

Vulnerability curves relate strong-motion shaking severity to the probability of reaching or exceeding a specified performance limit state. Strong-motion shaking severity may be expressed by an intensity (I), peak ground parameters (a, v or d) or spectral ordinates (S_a, S_v or S_d) corresponding to an important structural period. The number of limit states used varies between three and five (Kwon and Elnashai, 2006).

Vulnerability curves play a critical role in regional seismic risk and loss estimation as they give the probability of attaining a certain damage state when a structure is subjected to a specified demand. Such loss estimations are essential for the important purposes of disaster planning and formulating risk reduction policies. The driving technical engines of a regional seismic risk and loss estimation system are (Elnashai, 2003):

- Seismic hazard maps (i.e. peak ground parameters or spectral ordinates).
- Vulnerability functions (i.e. relationships of conditional probability of reaching or exceeding a performance limit state given the measure of earthquake shaking).
- Inventory data (i.e. numbers, location and characteristics of the exposed system or elements of a system).
- Integration and visualization capabilities (i.e. data management framework, integration or seismic risk and graphical projection of the results).

Fragility curves, used for the assessment of seismic losses, are in increasing demand, both for pre-earthquake disaster planning and post-earthquake recovery and retrofitting programs. This is due to the difficulties associated with analyzing individual structures and the importance of obtaining a global view of anticipated damage or effects of intervention, before and after an earthquake, respectively. Analytically-derived, mechanics-based fragility relationships result in reduced bias and increased reliability of assessments compared to the fragilities based on post-earthquake observations (Rossetto and Elnashai, 2003a) or on expert opinion (e.g. HAZUS (1997)). Since analytical methods are based on statistical damage measures from analyses of structural models under increasing earthquake loads, employing an appropriate damage assessment method is central to deriving fragility curves (Jeonga and Elnashai, 2007).

Fragility is the conditional probability of attainment or exceedance of multiple damage states for a given intensity of ground excitation, as shown in Equation (2.1).

$$P(\text{fragility}) = P\,[LS|IM = x], \qquad (2.1)$$
$$P(LS) = P(C = D)$$

where C is the capacity, D is the demand, and IM represents the intensity measure of input ground seismic hazard with intensity level x. LS refers to the limit state.

There are multiple proposed fragility relationships for reinforced concrete structural systems that were developed using different methodologies and parameters for representation of seismic demand and damage. These relationships are derived using a few different types of approaches, which are summarized by Rossetto and Elnashai (2003b) as follows: (i) Empirical Fragility Curves based on field data. These are derived through statistical analysis of how real buildings performed in past earthquakes. Examples of such fragilities are those proposed by Miyakoshi et al. (1997) and Orsini (1999); (ii) Analytical Fragility Curves. This approach uses numerical techniques to simulate the behaviour of systems including variations in structural capacity and seismic demands. Studies done in this category include those by Singhal and Kiremidjian (1997) and Mosalem et al. (1997); (iii) Judgemental Fragility Curves. These are fragility curves that are based partially or wholly on expert opinion. With this approach a wide range of structure types are dealt with in the same manner treating the level of uncertainty uniformly; (iv) Hybrid Fragility Curves, which are constructed through a combination of more than one of the other three approaches.

The features and limitations of different types of fragility curves are summarized in Table 2.1 (Kwon and Elnashai, 2006).

Table 2.1 Categorization of vulnerability curves (Kwon and Elnashai, 2006)

Category	Characteristics	
Empirical vulnerability curve	Features	-Based on post earthquake survey -Most realistic
	Limitations	-Highly specific to a particular seismo-tectonic, geotechnical and built environment -The observational data used tend to be scarce and highly clustered in the low-damage, low-ground motion severity range -Include errors in structural damage classification -Damage due to multiple earthquakes may be aggregated
Judgmental vulnerability curve	Features	-Based on expert opinion -The curves can be easily made to include all the factors
	Limitations	-The reliability of the curves depends on the individual experience of the experts consulted -A consideration of local structural types, typical configurations, detailing and materials inherent in the expert vulnerability predictions
Analytical vulnerability curve	Features	-Based on damage distributions simulated from the analyses -Reduced bias and increased reliability of the vulnerability estimate for different structures
	Limitations	-Substantial computational effort involved and limitations in modeling capabilities -The choices of the analysis method, idealization, seismic hazard, and damage models influence the derived curves and have been seen to cause significant discrepancies in seismic risk assessments
Hybrid vulnerability curve	Features	-Compensate for the scarcity of observational data, subjectivity of judgmental data, and modeling deficiencies of analytical procedures -Modification of analytical or judgement based relationships with observational data and experimental results
	Limitations	-The consideration of multiple data sources is necessary for the correct determination of vulnerability curve reliability

CHAPTER 3

CASE STUDY

3.1. General Methodology

In the literature, there is no definitive method or strategy for development of fragility curves. For example, earthquake intensity parameters (peak ground acceleration (PGA), spectral acceleration (S_a), spectral velocity (S_v)) selected; earthquake damage levels (yield, failure, none, slight, moderate, major, collapse, etc.) considered; type of analysis used for obtaining seismic response data (nonlinear time history analysis, elastic spectral analysis, nonlinear static analysis) and the method used for the construction of fragility curves change from one study to another. This study employs accepted procedures whilst attempting to ensure that rational decisions are taken along the route to deriving fragility curves for the structural system considered.

In the methodology, the following three main parts were taken into account. Firstly, the earthquake intensity indicator, namely PGA was selected and 200 synthetic earthquake accelerations were generated. Secondly, the building configuration and material properties were defined and then design and analytical idealization of the structure was done. Thirdly, the damage states were selected as slight, moderate, major, and collapse and nonlinear static (pushover) analysis was performed and after that the criteria for identification of damage states were determined. After all, the nonlinear time history analyses were conducted and eventually the fragility curves were developed.

The methodology used for development of fragility curves is outlined in Figure 3.1 and a detailed description is given hereafter of the various steps depicted in Figure 3.1.

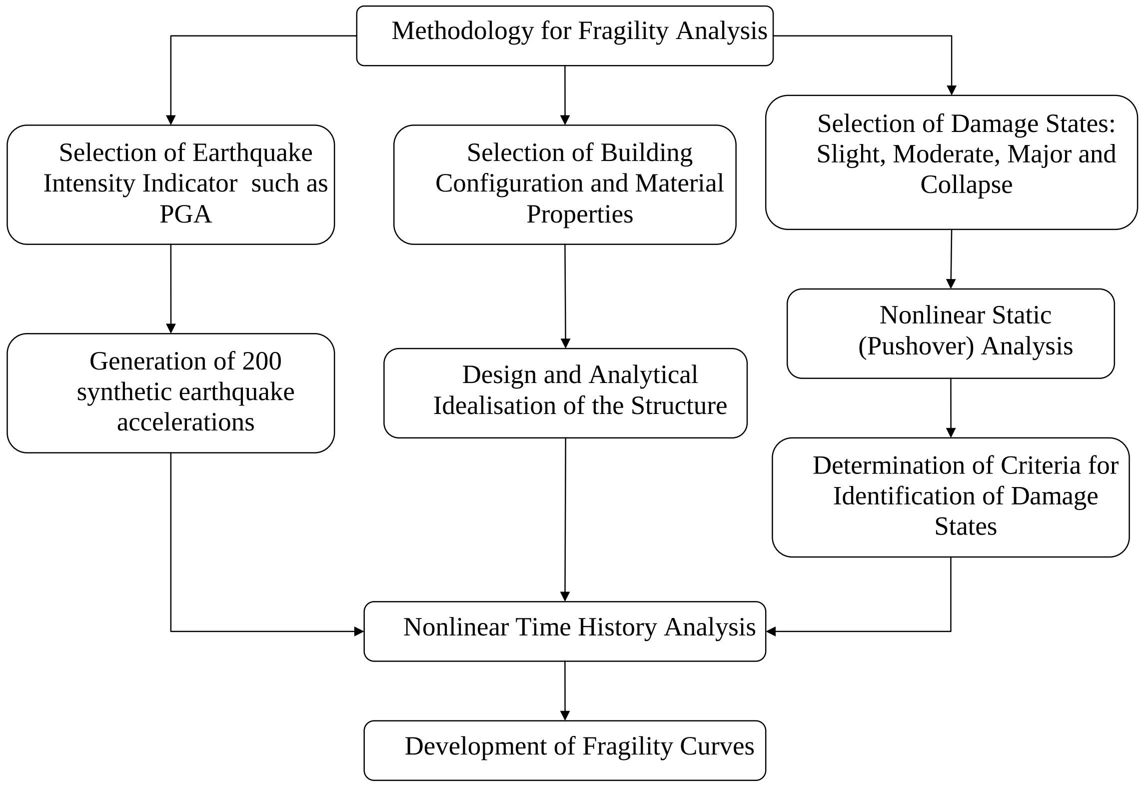


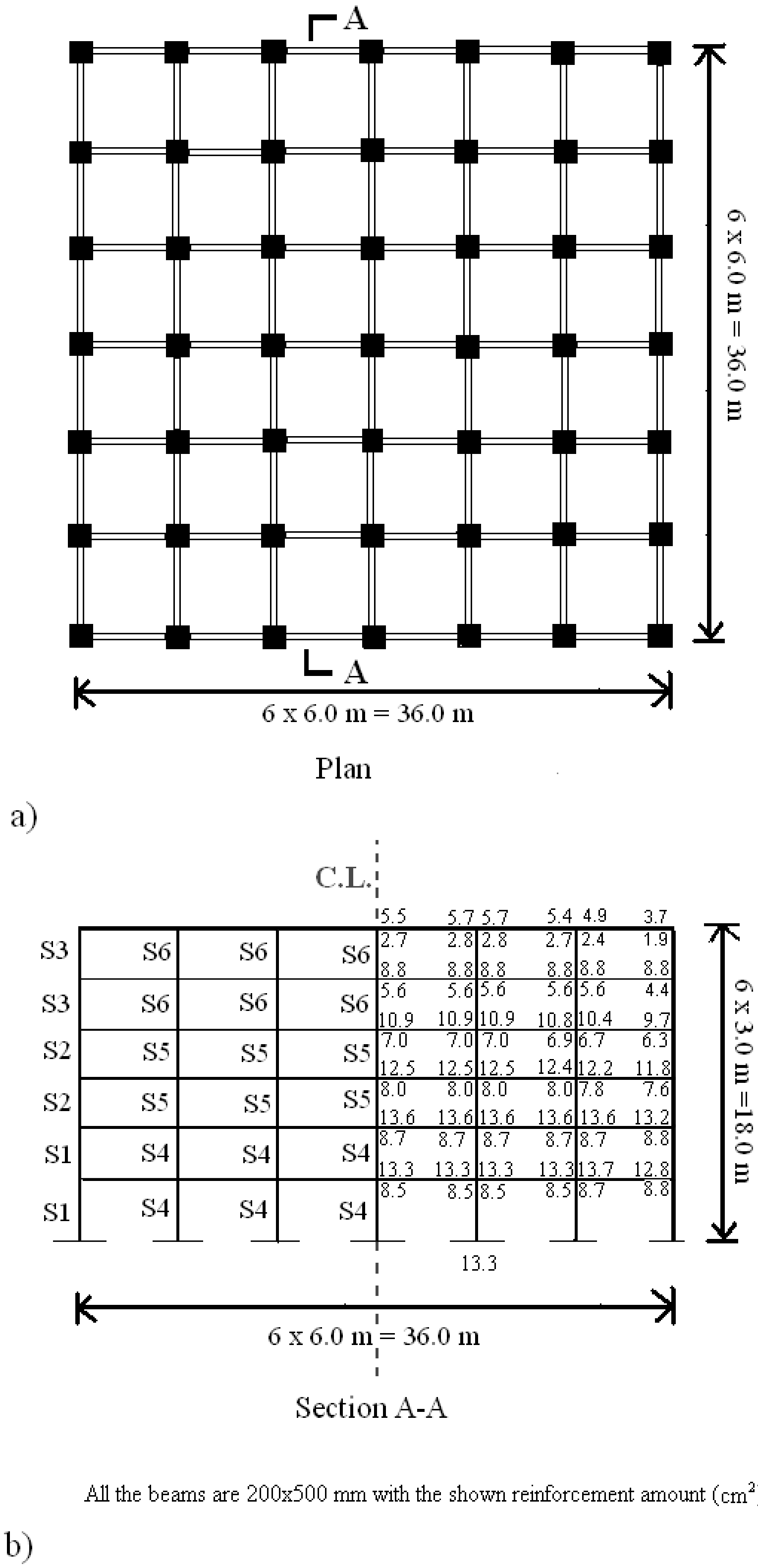
Figure 3.1 The flowchart of the methodology utilized in the development of fragility curves

3.2. Description of Original Building

In this study, a six storey reinforced concrete (R/C) office building was selected as a sample building, which is designed according to the 1975 Turkish Seismic Code (ABYYHY, 1975). This code was revised and updated in 1998 (ABYYHY, 1998) and 2007 (ABYYHY, 2007). It is known that there are considerable R/C building stocks in Turkey. Most of them were constructed before 1998. In recent earthquakes in Turkey, most of the buildings were subjected to damage due to the weakness of structural system. For this reason, in the design of sample building, 1975 version of seismic code was taken into account.

The building consists of typical beam-column R/C frames. It has no shear walls and it is located in a high seismicity region in Turkey. It has a local site class of Z1 according to the description in the current Turkish Seismic Code (ABYYHY, 2007). In the design phase of the building for both gravity and seismic loads, the regulations of 1975 Turkish Seismic Code was used. The gravity load is composed of dead load and live load. The weight of the structural members and the masonry infill walls are included in the dead load. The live load used was 2.0 kN/m^2, which is typical for an office building. Any other loads, such as wind load, snow load were not considered. The soil-structure interaction was not taken into account and the base of the columns at the ground floor is assumed to be fixed.

The building has a height of 18 m above ground, each storey height is 3 m and it has plan dimensions of 36 m by 36 m. It is a typical symmetrical building, which may be chosen for simplicity in the analysis. Typical floor plan and elevation of the building are shown in Figure 3.2. The compressive strength of concrete is 16 MPa with the corresponding Young's Modulus of 27000 MPa and the characteristic yield strength of steel is 220 MPa for both longitudinal and transverse reinforcement. The slab thickness at each floor is 12 cm. There are four types of columns are employed at the building. The columns S1 and S5 are 40 cm x 40 cm in cross section, and 8 ø16 mm longitudinal reinforcements are used. The columns S2 and S6 are 35 cm x 35 cm in cross section, and 8 ø14 mm longitudinal reinforcements are used. The columns S3 are 30 cm x 30 cm in cross section, and 8 ø12 mm longitudinal reinforcements are used. The columns S4 are 45 cm x 45 cm in cross section, and 8 ø18 mm longitudinal reinforcements are used. Dimensions of the columns and amount and arrangement of longitudinal reinforcement are provided in Figure 3.3. All the beams at the building are 20 cm x 50 cm and the amount of top and bottom reinforcements are also given in Figure 3.2 as a unit cm^2 from the analysis results. The stirrups used in the structural elements are 8 mm in diameter with 20 cm spacing, as common practices. The six-story building was modeled as a series of planar frames connected at each floor level by rigid diaphragms. Therefore, only two-dimensional analysis was performed and the original frame is denoted by Frame O. For better understanding of building, three dimensional form of the building is also given in Figure 3.4.



All the beams are 200x500 mm with the shown reinforcement amount (cm^2)

Figure 3.2 A layout for (a) first floor plan and (b) elevation of the original building

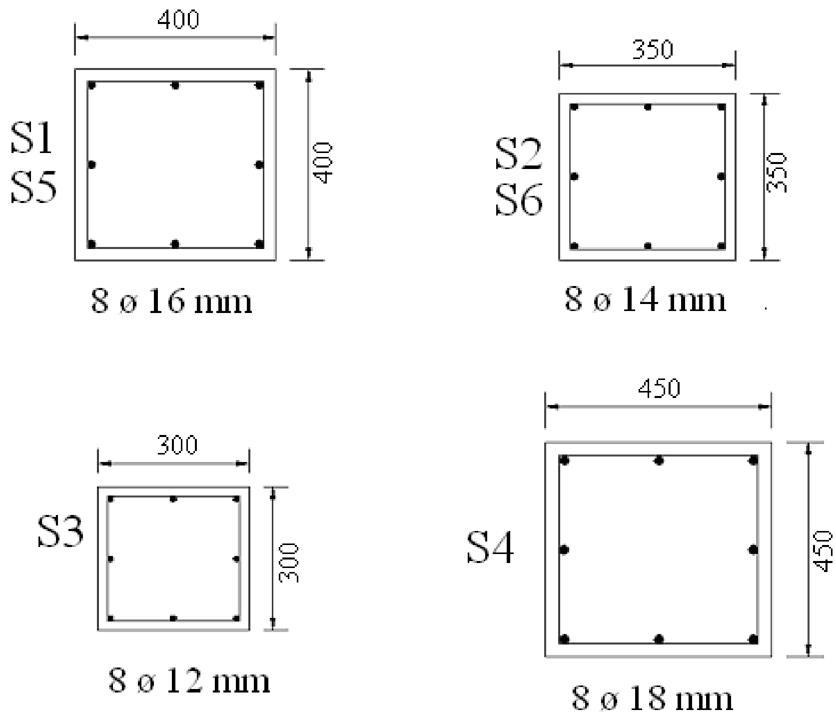


Figure 3.3 Sections and reinforcement details of columns (in mm)

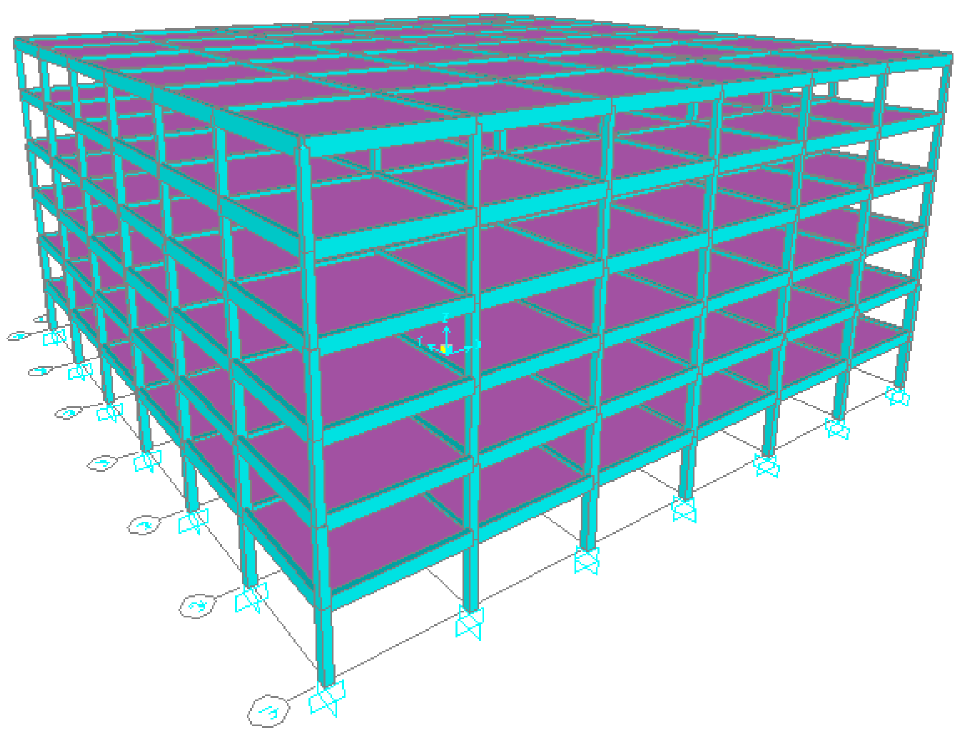


Figure 3.4 3D view of the building

3.3 Description of Retrofitted Building

The mid-rise R/C office building described was retrofitted by twelve different retrofitting cases to improve the seismic performance of the building. In retrofitting approach, three different retrofitting groups were defined on the basis of the type of the eccentric steel bracing systems, namely D, K, and V braces and in each group, four different distributions of these braces were applied in order to investigate the effect of distribution of braces over the height of the building.

In the first group of the retrofitted cases, eccentric D-braces were applied to exterior frames of the original building, as shown in Figure 3.5. According to the distribution of steel braces in the frame, the frames were named as Frame D1, D2, D3, and D4. Except model D1, the steel braces were applied only four bays of exterior frames. The steel brace members used in all frames have the same length, cross section, and material properties. They have square tube sections with 17 cm width and 4 mm thickness and the slenderness ratio of the braces is 79.

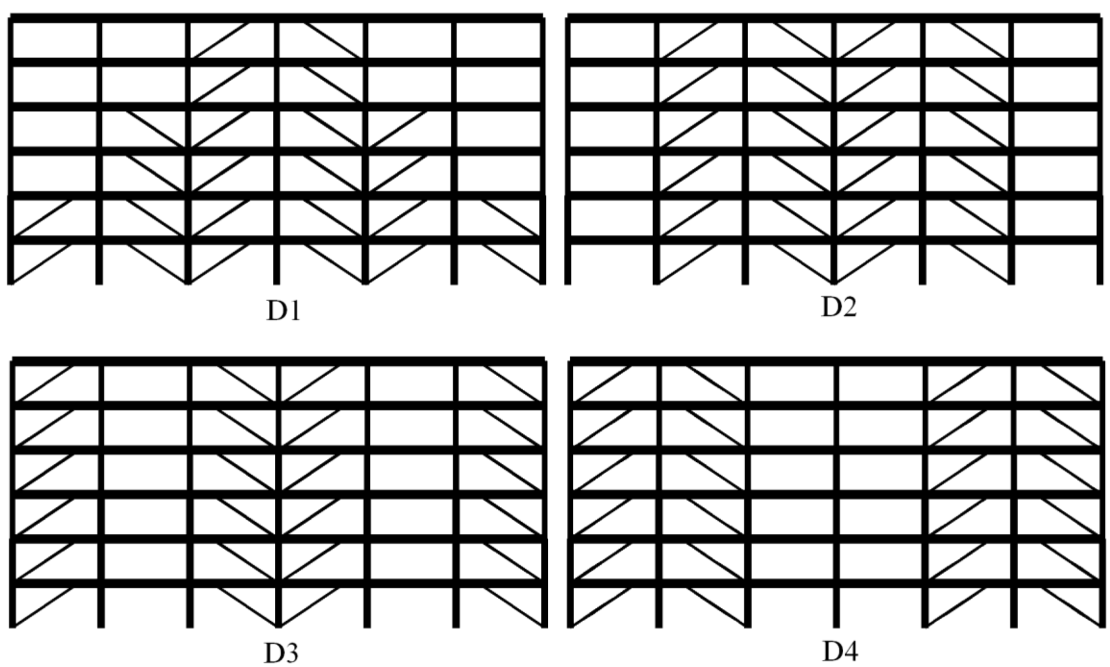
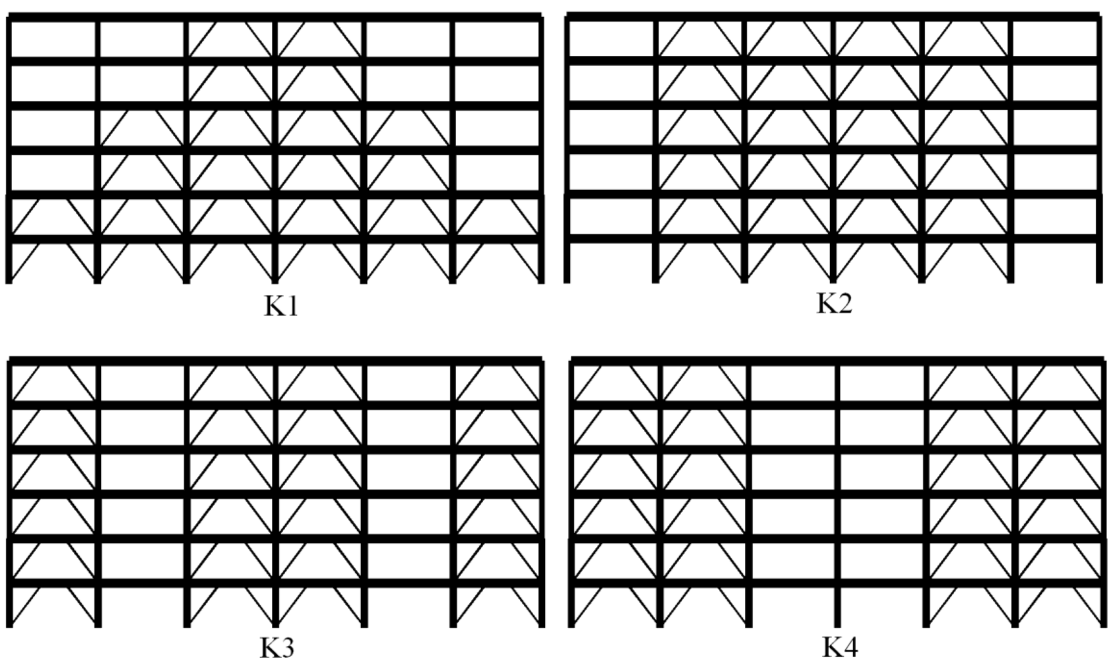


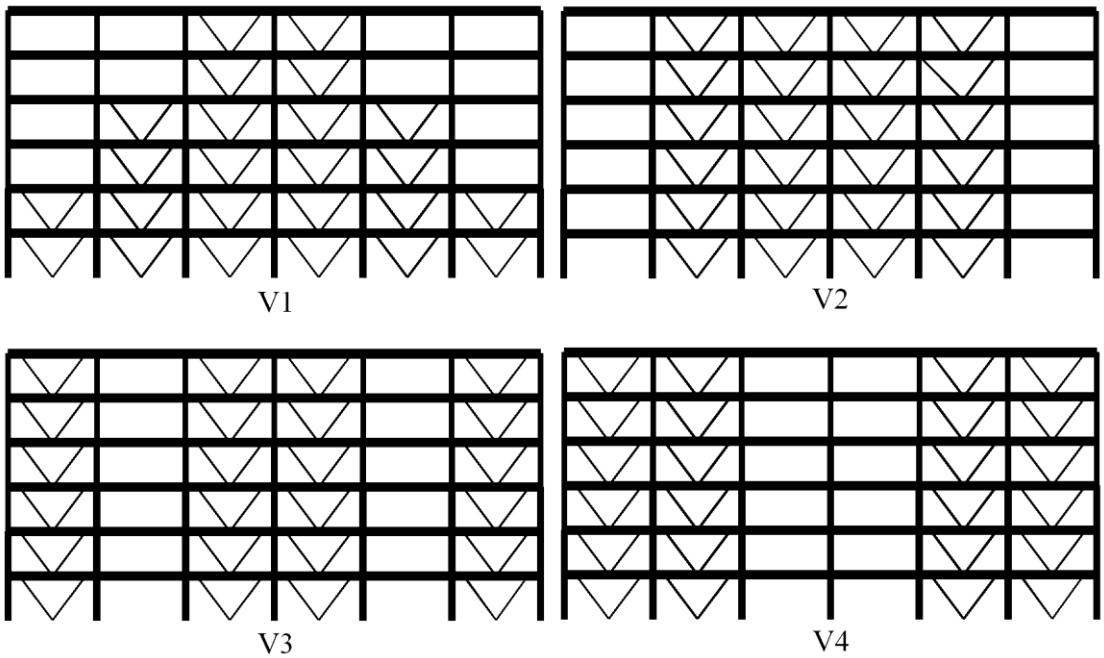
Figure 3.5 Layout of D braced frames

Similar to first group of the retrofitted case, in the second and third groups of the retrofitted cases, except models K1 and V1, the K and V type steel braces were applied only four bays of exterior frames of the original building, as shown in Figure 3.6. Also similar to D braced frames, these K and V braced frames were named as Frame K1, K2, K3, K4 and Frame V1, V2, V3, V4 with respect to brace type and distribution in the frame, respectively. In the K and V type steel braces, all the bracing members have square tube sections with 12 cm width and 4 mm thickness. The slenderness ratio is 79 like D brace members.

In all retrofitting cases, the effective length factors of eccentric braces assumed to be 1.0 for in-plane and out-of-plane buckling. The steel type of the retrofitting braces is St 52. The Young's modulus of steel is E = 200,000 MPa and the yield stress of steel is f_y = 360 MPa.



(a)



(b)

Figure 3.6 (a) Layout of K braced frames; (b) Layout of V braced frames

3.4. Linear Dynamic Analysis of Original and Retrofitted Buildings

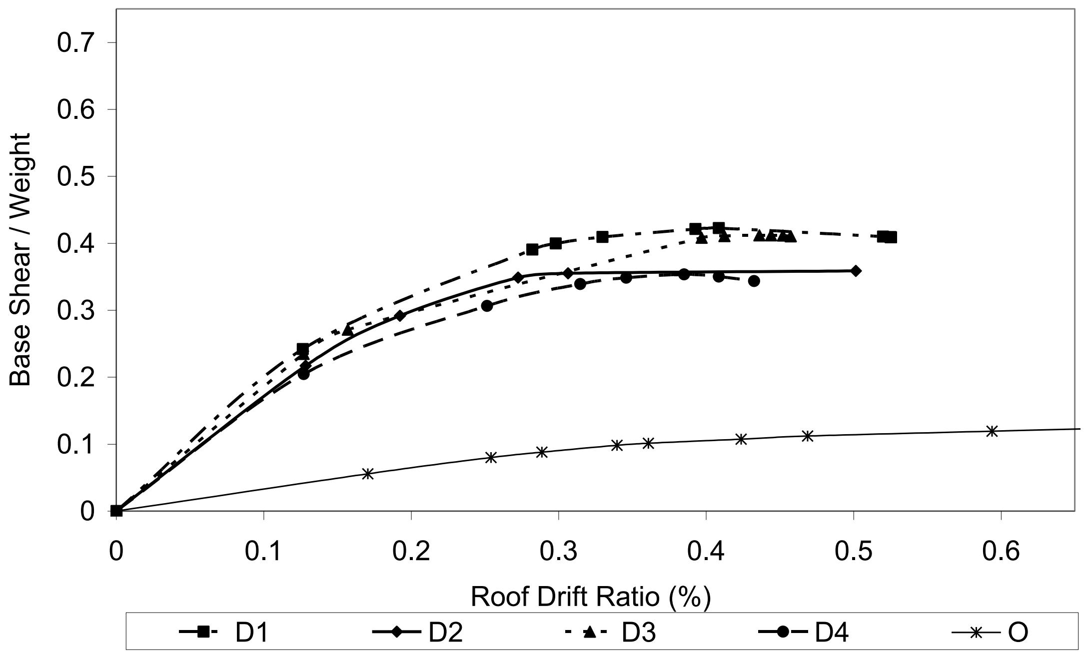
To have a general idea about the structural response characteristics of selected original and retrofitted frames, first the linear dynamic analysis (eigen value analysis) were applied. According to the results of these analyses, the periods of free vibration of the first three modes of the original and retrofitted frames were obtained and listed in Table 3.1. As shown in Table 3.1, the braced frames had shorter fundamental period and therefore relatively higher stiffness than the original frame. The first period T_1 of the original frame was almost twice of the first periods of the braced frames. When braced frames were compared with each other, it was pointed out that V braced frames had the shortest fundamental period, and therefore the greatest stiffness. When the other two namely; K and D braced frames were compared, it is observed that the D braced frames had slightly shorter fundamental period than the K braced frames.

Table 3.1 Periods of free vibration of original and braced frames

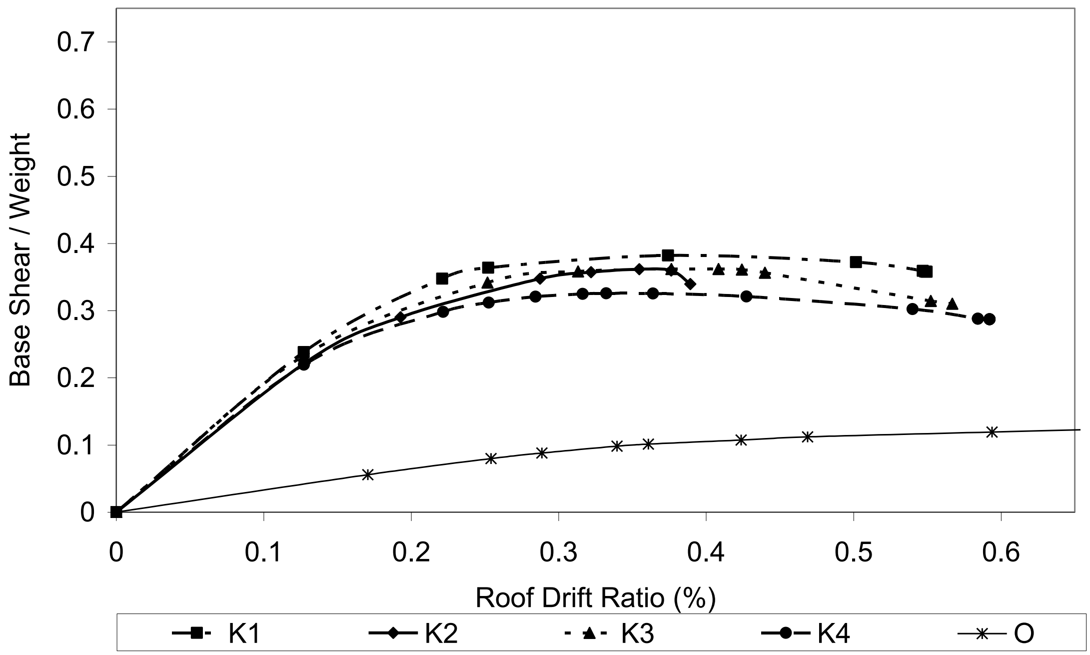
Building Type		First Period , T_1 (sec.)	Second Period, T_2 (sec.)	Third Period, T_3 (sec.)
Original	O	0.752	0.276	0.158
D-Braced	D1	0.464	0.197	0.132
	D2	0.477	0.179	0.124
	D3	0.484	0.177	0.113
	D4	0.482	0.181	0.125
K-Braced	K1	0.483	0.191	0.114
	K2	0.497	0.171	0.105
	K3	0.501	0.174	0.106
	K4	0.502	0.174	0.106
V-Braced	V1	0.398	0.166	0.101
	V2	0.412	0.147	0.091
	V3	0.430	0.149	0.093
	V4	0.424	0.148	0.090

3.5. Nonlinear Static Analysis of Original and Retrofitted Buildings

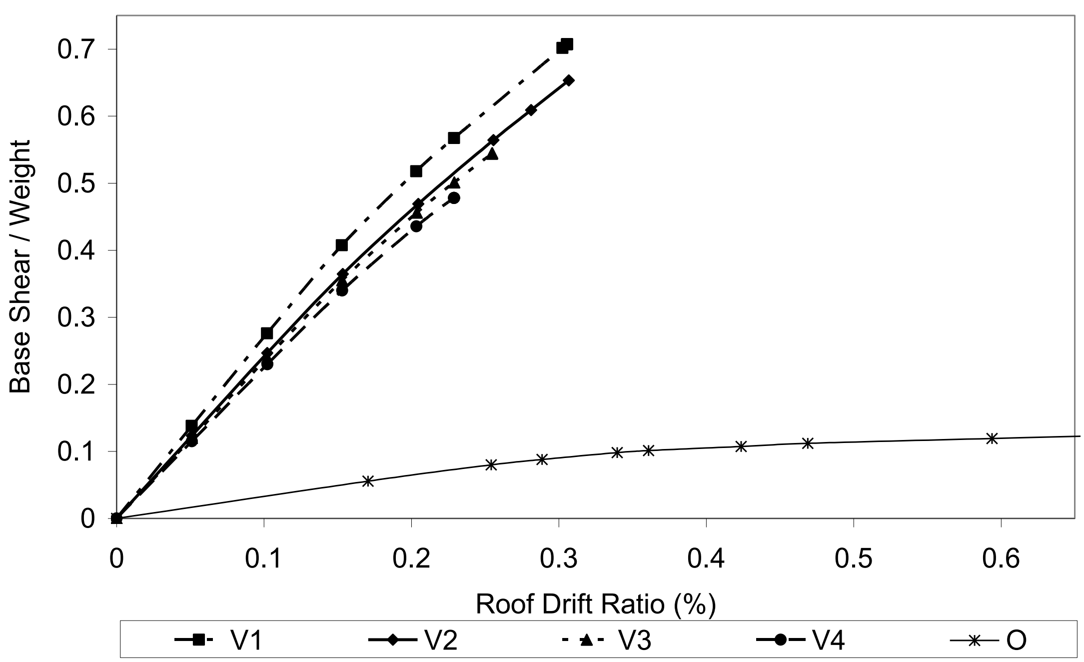
After the eigen value analysis, the nonlinear static (pushover) analysis were applied to the original frame and braced frames to evaluate the stiffness and strength characteristics of them. In the nonlinear static analysis, an inverted triangular lateral load distribution was used. When modeling the beam and the column members, they were modeled as nonlinear frame elements with lumped plasticity by defining plastic hinges at both ends of the beams and columns. Two types of hinges were used. Firstly, PMM (axial force biaxial moment) hinges were defined for columns and secondly, M3 (uniaxial moment) hinges were defined for beams. The moment-curvature analyses of beams and the PMM interaction diagram of the columns were obtained by using XTRACT Cross-Sectional Analysis Program (2009). When performing the moment curvature analysis and the PMM interaction diagrams, section properties and constant axial loads on the structural members were considered. The axial forces were assumed to be zero on the beams and they were assumed to be constant and equal to the dead load plus 30 % of the live load on the columns. In the PMM and the moment-curvature analysis, the Mander model (Mander et al., 1998) for confined concrete and the typical steel stress-strain model with parabolic strain hardening for steel were implemented. P-Delta effects were also taken into account in the analysis. Shear failures were not observed in any cases of nonlinear static analysis. This is because the assumed compressive strength of concrete of the original building was sufficient to prevent shear failures. The lateral load-roof drift ratio of the original and the braced frames obtained from nonlinear static analysis are given in Figure 3.7.



(a)



(b)



(c)

Figure 3.7 Capacity curves of the original and the retrofitted (a) D braced, (b) K braced, and (c) V braced frames

In the results of the nonlinear static analysis, it was observed that the original frame, namely Frame O sustained a lateral yield load of approximately 0.06W and an ultimate lateral load of 0.12W, where W represents the weight of the building. For strengthening the original frame, using of D type eccentric steel braces such as Frame D1, D2, D3, and D4 increased the yield load to 0.23W, 0.22W, 0.24W, and 0.20W; and the ultimate lateral load to 0.41W, 0.36W, 0.41W, and 0.34W, respectively. The use of K type braces in retrofitting, namely K1, K2, K3, and K4 increased the yield load to 0.24W, 0.22W, 0.23W, and 0.23W and the ultimate load to 0.38W, 0.36W, 0.36W, and 0.35W, respectively. The capacity curves obtained for V type braces that are V1, V2, V3, and V4 showed that the yield load reached 0.28W, 0.25W, 0.24W, and 0.23W, and the ultimate lateral load carrying capacity to 0.71W, 0.65W, 0.54W and 0.48W, respectively. The results of the capacity curves indicated that the greatest ultimate lateral load carrying capacity was reached for the V1 braced frame. For a given value of lateral load, the drift of braced frames was reduced as compared to the drift of the original frame. For example, for a lateral load of 0.10W, the roof drift ratio of Frame O was equal to 0.36 % and it was equal to 0.05, 0.06, 0.05, and 0.06 % for frames D1, D2, D3, and D4 respectively.

3.6. Determination of Damage Levels

The behavior and failure modes of such reinforced concrete moment resisting frame structures are mainly governed by deformation, so in this study, similar to the studies in the literature (Kircher et al., 1997; Erberik and Elnashai, 2004; Symth et al., 2004), the inter storey - drift was used as a parameter for expression of damage. In actual fact, the levels of damage are a continuum, but in the current study, the full range was divided into four separate damage levels for practical purposes. The limit values of these damage levels were determined according to HAZUS (1997). These damage levels are slight, moderate, major, and collapse. In defining the limit values of these damage levels, the capacity curve of the original frame obtained from pushover analysis was used. According to the HAZUS (1997), first of all, the local limit states of the structural members (such as beams or columns) are considered in terms of the first crack, yield or failure observed during the pushover analysis. Afterwards, these local limit states of members are used to obtain the global limit state of the structure. For example, when the first structural component is yielded on its load deformation curve, and also cracks in some columns and beams can be observed, this point on the capacity curve is defined as slight damage limit. Moderate damage limit is defined as about 5 % of structural components have each reached the yield point on their respective load deformation curves. Major damage limit is defined by a similar to moderate damage limit, except that about 25 % of structural components have each reached the yield point and some have reached the failure point on their load deformation curves. Lastly when the inter storey - drift value on the load deformation curve is at a point, which at least 50 % of structural components have reached failure, or the structure has lost its stability, this point is called as collapse damage limit. The capacity curve of the original frame showing the limit values of slight, moderate, major, and collapse damage states are given in Figure 3.8 and the limit values of inter storey - drift ratios determined for each damage level are presented in Table 3.2.

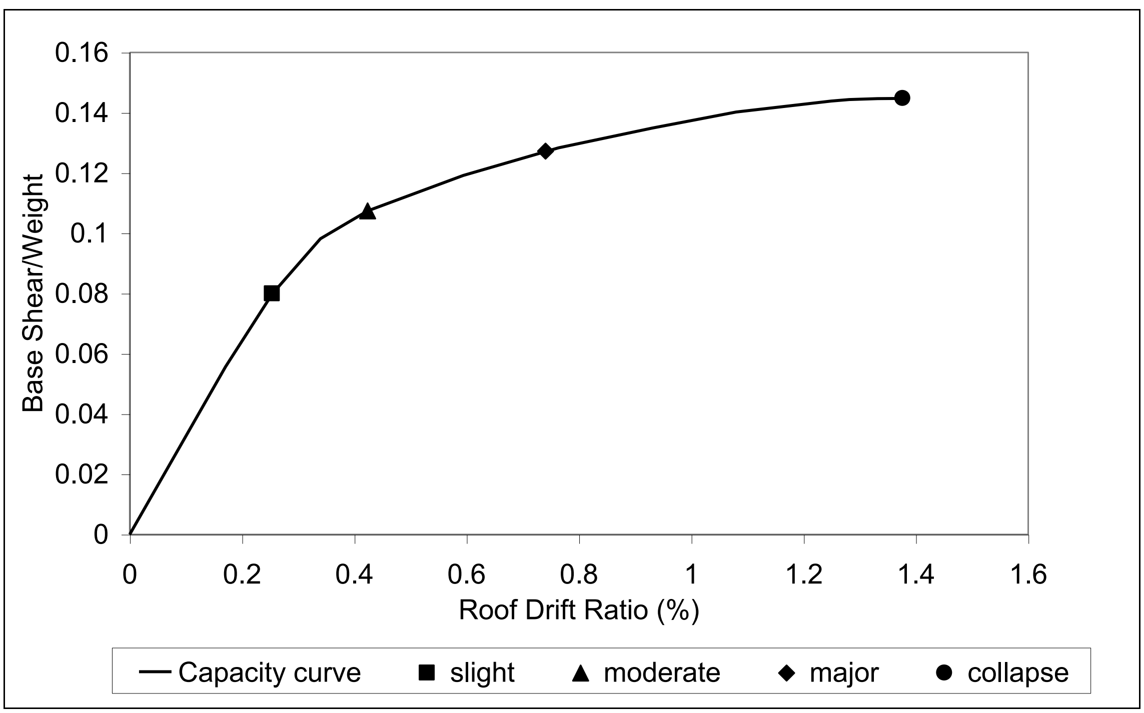


Figure 3.8 Limit values of structural damage levels determined according to the HAZUS (1997)

Table 3.2 Defined damage levels

No	Damage Levels	Inter-Storey Drift Ratio (%)
1	Slight	>0.25
2	Moderate	>0.42
3	Major	>0.74
4	Collapse	>1.38

3.7. Modeling of Input Ground Motions

In seismic fragility analysis, the selection of representative set of earthquake motions is an important step. These motions are at different levels of ground motion intensity which represents the variability in earthquakes. Synthetic ground motions were generated compatible with target response spectra according to the approach suggested by Deodatis (1999) and later modified by Saxena et al. (2000). The response spectrum used in this study was taken from the current Turkish Seismic Code (ABYYHY, 2007) for Z1 type soil conditions.

In this study, PGA values were considered from 0.01 g to 1.0 g to consider a wide range of ground motion shaking levels when simulating the input acceleration time histories. By using the algorithm written in MATLAB program by Güneyisi (2007), a total of 200 ground-motion acceleration time histories were generated to establish a complete set of fragility curves. The design code spectrum and the elastic spectrums of some of the artificially generated ground motion records are compared with each other given in Figure 3.9. To have an idea about the synthetic ground motions, the artificially generated earthquake ground motions having 0.35 PGA and 0.70 PGA were given as an example in Figures 3.10 (a) and 3.10 (b), respectively.

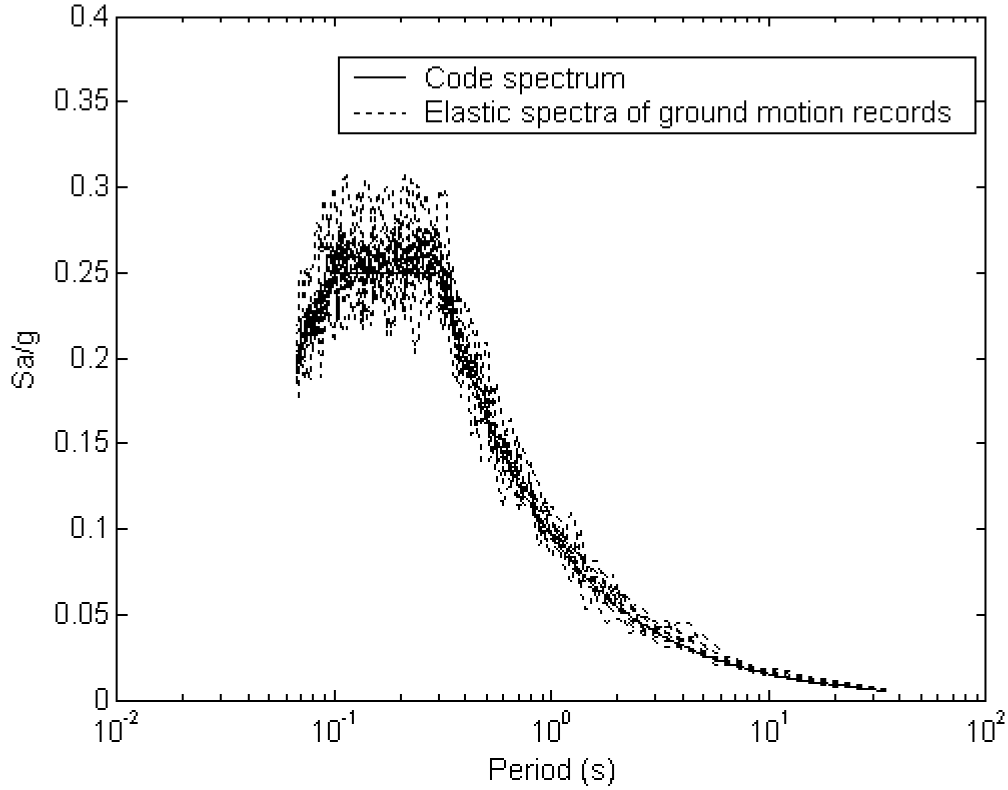
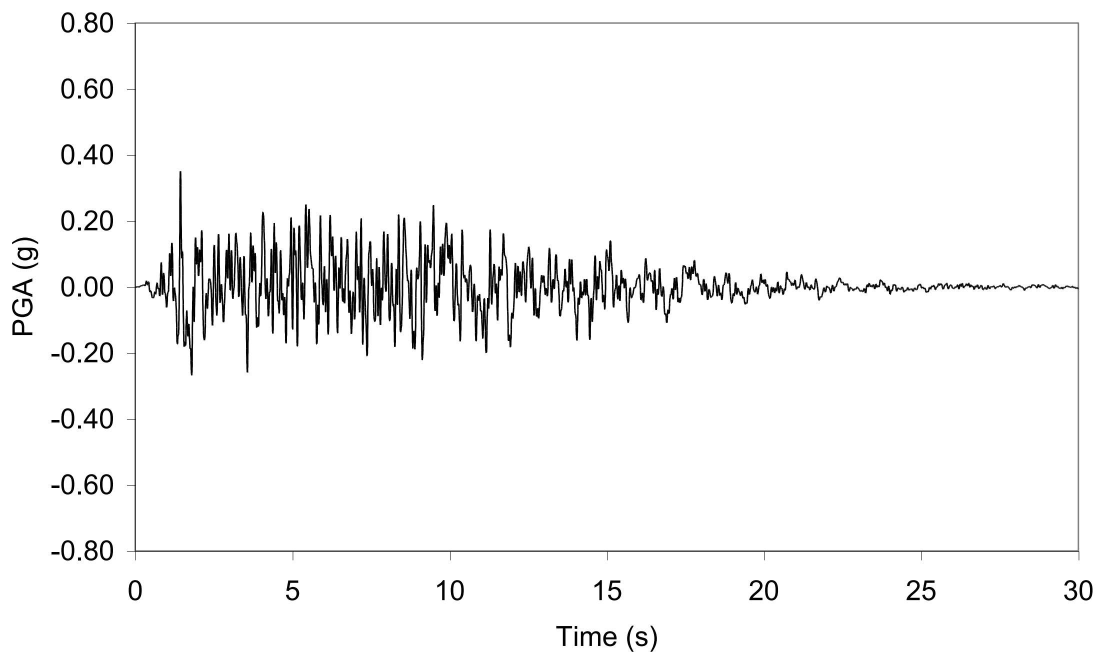
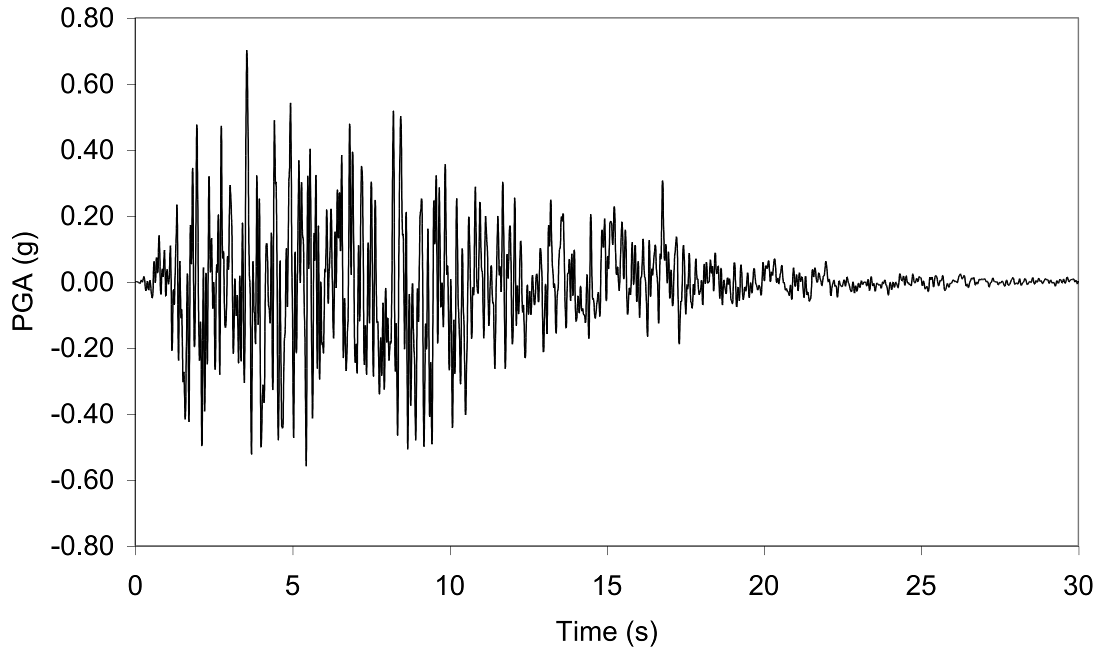


Figure 3.9 Comparison of the elastic spectra with the code spectrum



(a)



(b)

Figure 3.10 Artificially generated earthquake ground motion records having (a) 0.35 PGA and (b) 0.70 PGA

3.8. Nonlinear Time History Analysis

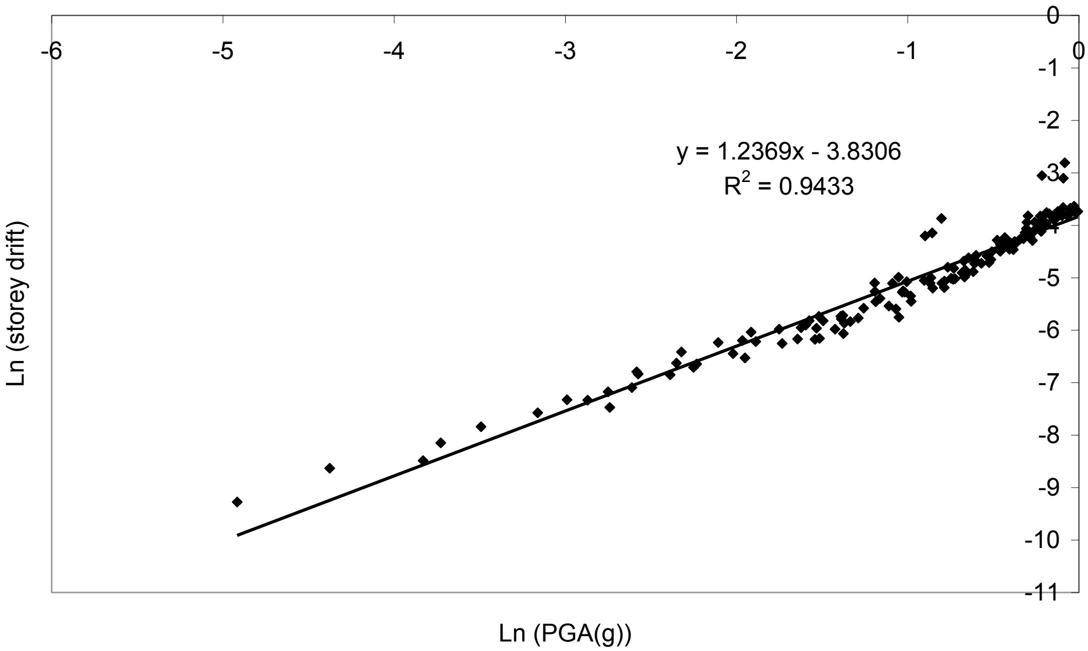
For fragility analysis, the seismic response of the original and retrofitted structures was evaluated by nonlinear time history analyses. The beam and the column elements were modeled as nonlinear frame elements with lumped plasticity. This lumped plasticity is defined by plastic hinges at both ends of the beams and columns, as in the nonlinear static analysis. In the nonlinear time history analysis, 200 simulated response spectrum compatible ground motion acceleration time histories were utilized to perform the 200 nonlinear time history analyses for original building and each type of retrofitted conditions of building. From the results of each of these 200 nonlinear time history analyses, 200 values of maximum inter storey – drifts were obtained for original building and each type of retrofitted conditions of building. The fragility curves were created by using these values of maximum inter storey – drifts.

3.9. Generation of Analytical Fragility Curves

In the seismic risk assessment of the buildings, the fragility curves are very important. By using fragility curves, the potential seismic performance of any type of building can be evaluated. The relative vulnerability of original and retrofitted states of mid-rise R/C buildings may be presented through a comparison of their vulnerability functions or known as fragility curves. Fragility curves show the probability of meeting or exceeding a level of damage under a given input ground motion intensity parameter, which is a typical conditional probability. This conditional probability can be expressed as:

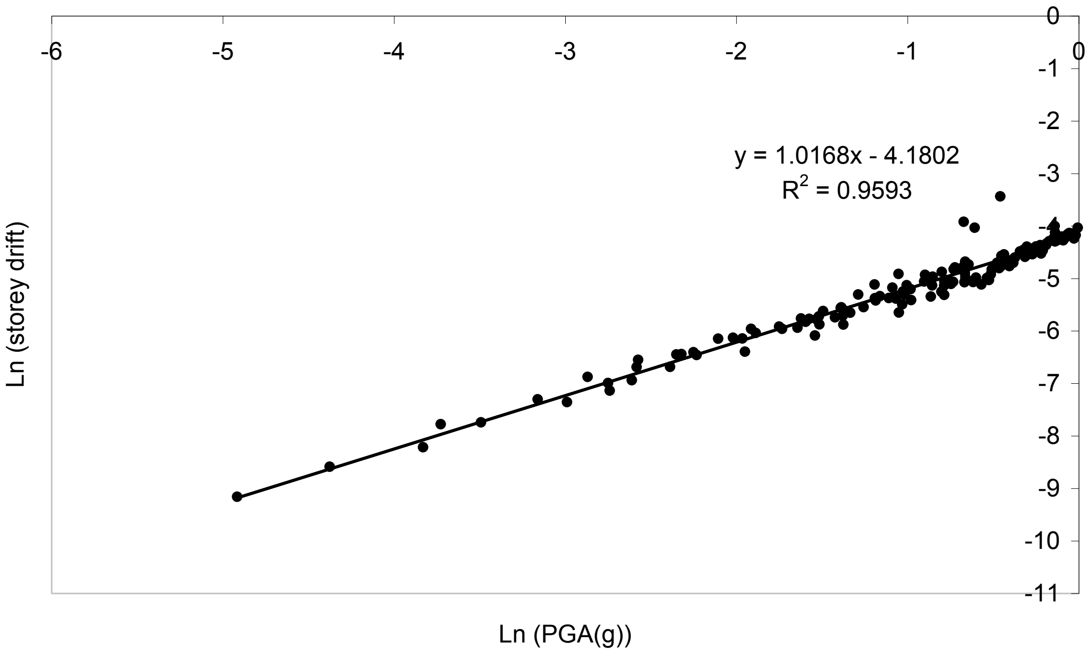
$$P \le D = \Phi\left[\frac{1}{\beta}\ln\left(\frac{X}{\mu}\right)\right] \tag{3.1}$$

where Φ is the standard normal cumulative distribution function, X is the lognormal distributed ground motion intensity parameter, and μ is the median value of ground motion index at which the building reaches the threshold of damage state D, defined using allowable drift ratios and β is the standard deviation of the natural logarithm of ground motion index of damage level. For fragility analysis, the seismic intensity is mostly defined as peak ground acceleration (PGA), spectral acceleration (S_a), velocity (S_v), or spectral displacement (S_d) in the current methods. In this study, PGA was used as the seismic intensity parameter. The median and standard deviation of the PGA values for each damage level were obtained by performing linear regression analysis. The regression of the probabilistic seismic demand models for the original frame, and D1, K1, and V1 braced frames are illustrated in Figure 3.11. It was shown in the figure that the correlation coefficients, R^2 were in the range of 0.94 to 0.97, which indicates fairly good linearity.



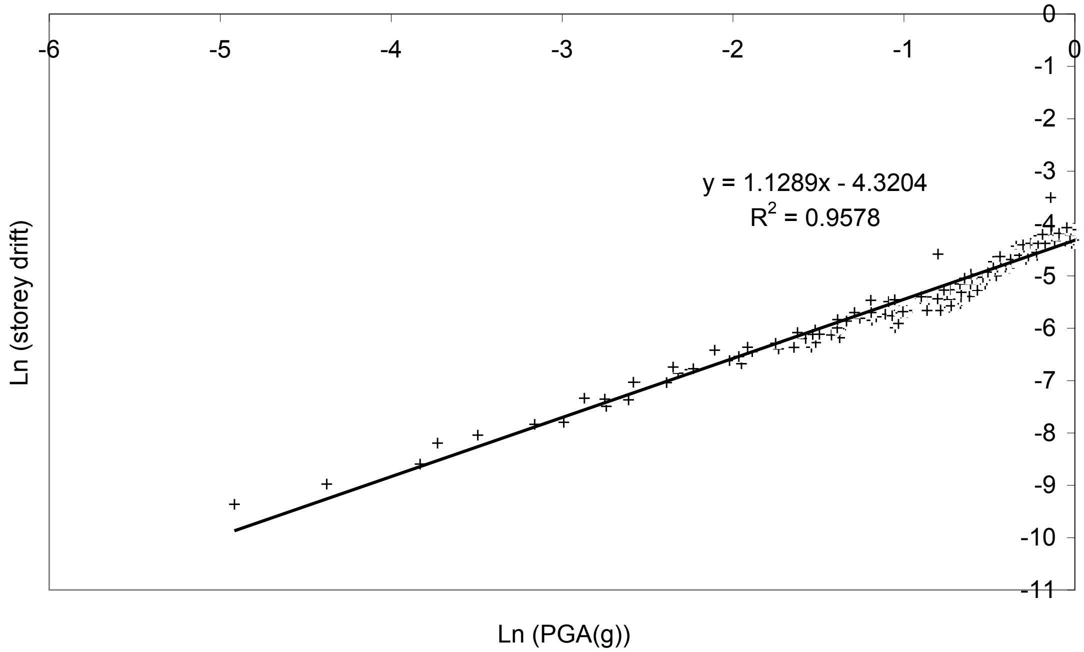
y = 1.2369x - 3.8306
R^2 = 0.9433
Ln (storey drift)
Ln (PGA(g))

(a)

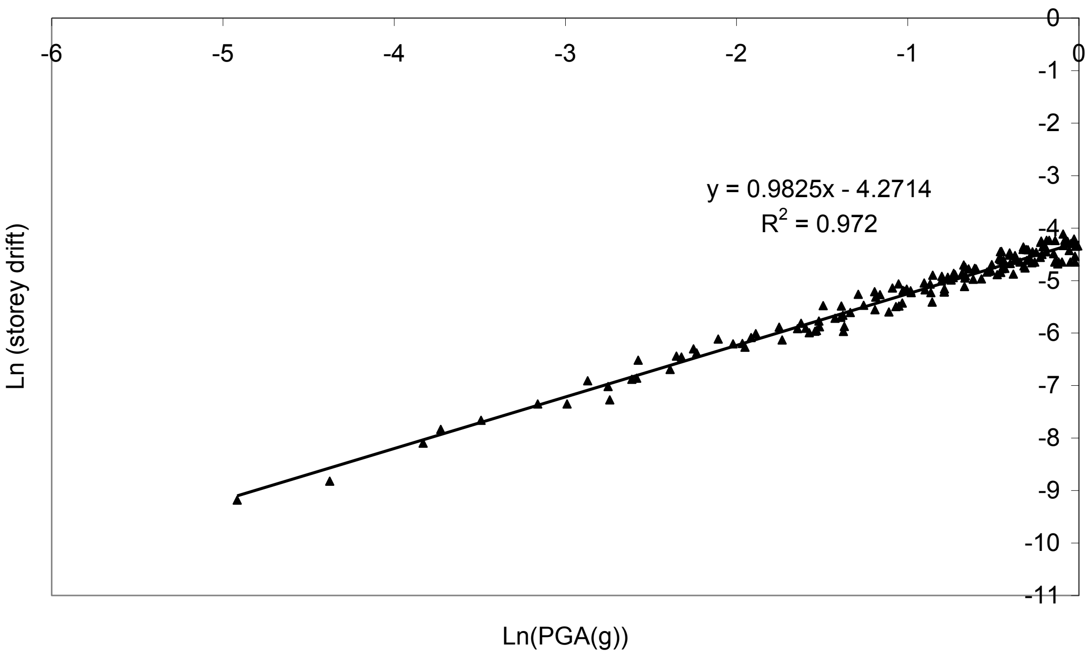


y = 1.0168x - 4.1802
R^2 = 0.9593
Ln (storey drift)
Ln (PGA(g))

(b)



(c)



(d)

Figure 3.11 Regression of the probabilistic seismic demand model of (a) original frame, (b) D1 braced frame, (c) K1 braced frame, and (d) V1 braced frame

The fragility curve parameters: median and standard deviation of log-normally distributed ground motion indices under artificial earthquake accelerations for the original and braced frames for each damage level under consideration are given in Table 3.3.

Table 3.3 Fragility curve parameters

Building Type		Damage Level							
		Slight		Moderate		Major		Collapse	
		μ	β	μ	β	μ	β	μ	β
Original	O	0.069	0.347	0.149	0.478	0.282	0.250	0.534	0.306
K-Braced	K1	0.083	0.503	0.175	0.579	0.360	0.697	0.719	0.666
	K2	0.075	0.366	0.173	0.492	0.349	0.735	0.711	0.714
	K3	0.072	0.367	0.172	0.492	0.342	0.742	0.701	0.714
	K4	0.073	0.366	0.170	0.492	0.339	0.731	0.693	0.714
Average of K		0.075	0.400	0.172	0.513	0.347	0.726	0.706	0.702
V-Braced	V1	0.085	0.658	0.209	0.703	0.420	0.565	0.881	0.595
	V2	0.084	0.667	0.198	0.647	0.417	0.538	0.875	0.620
	V3	0.082	0.650	0.193	0.687	0.413	0.469	0.872	0.607
	V4	0.081	0.650	0.187	0.687	0.403	0.494	0.868	0.607
Average of V		0.083	0.656	0.196	0.681	0.413	0.516	0.874	0.607
D-Braced	D1	0.081	0.456	0.191	0.536	0.390	0.455	0.796	0.687
	D2	0.080	0.477	0.186	0.545	0.379	0.413	0.781	0.674
	D3	0.078	0.477	0.183	0.547	0.375	0.394	0.790	0.675
	D4	0.076	0.370	0.180	0.507	0.369	0.404	0.780	0.696
Average of D		0.078	0.445	0.185	0.533	0.378	0.416	0.786	0.683

CHAPTER 4

DISCUSSION OF THE RESULTS

In this study, the fragility curves developed for the original and the retrofitted frames were utilized for evaluation of possible damage to structures that had similar characteristics to those of the original one and evaluation of possible damage to structures retrofitted with eccentric steel braces. The fragility curves developed for the original building for four damage levels, namely slight, moderate, major, and collapse are given in Figure 4.1. Moreover, the set of fragility curves developed for the D braced, K braced, and V braced frames are shown in Figures 4.2, 4.3, and 4.4, respectively.

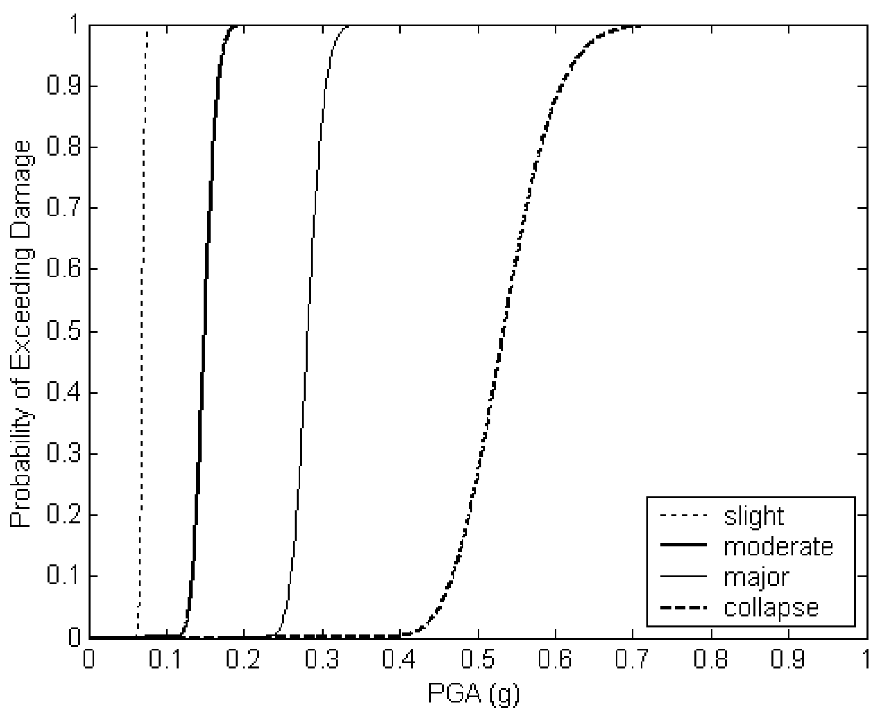
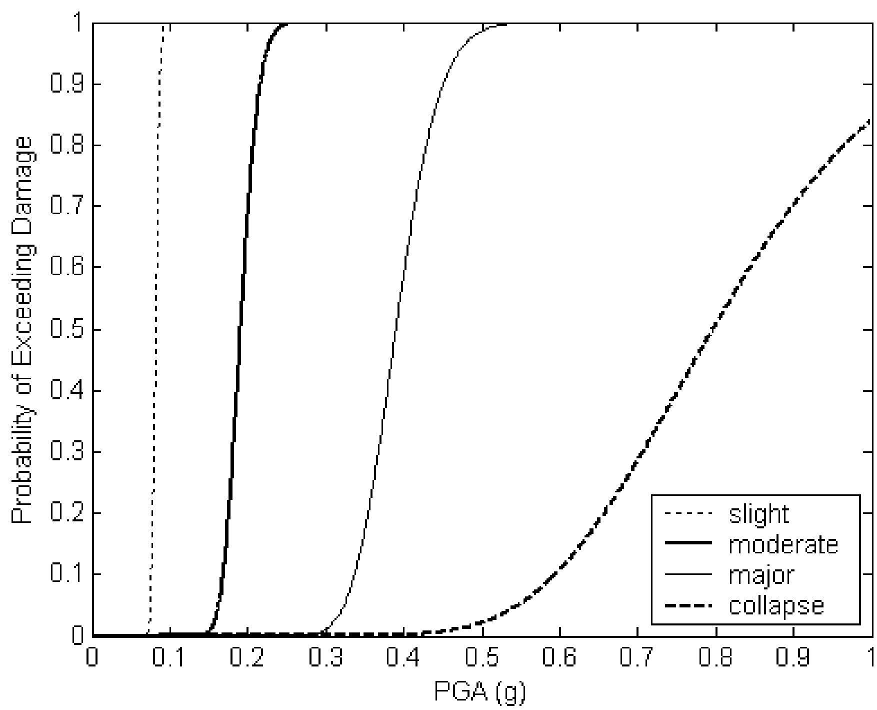
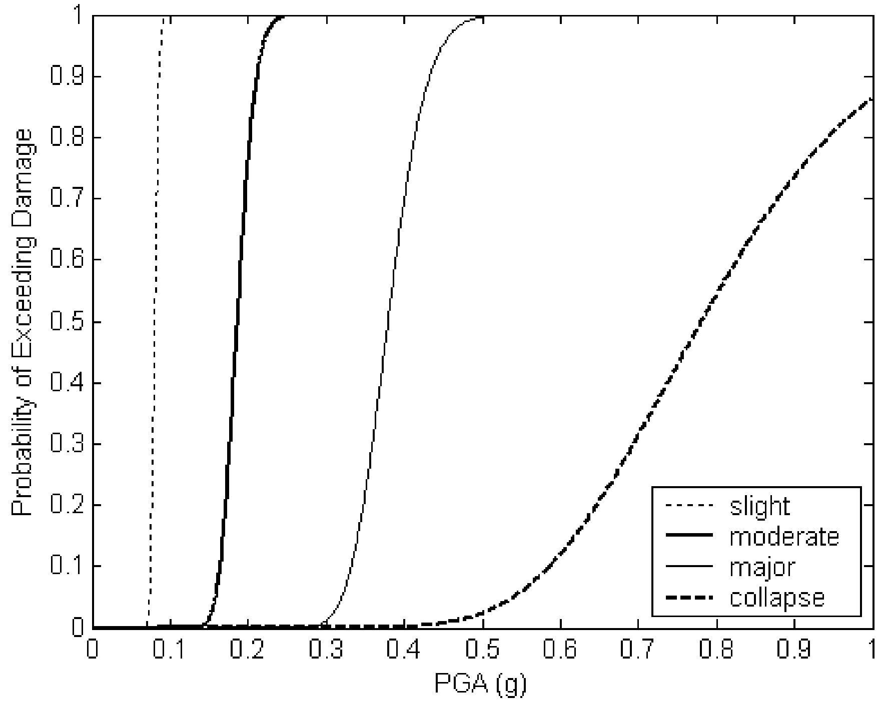


Figure 4.1 Fragility curves developed for the original building

When the developed fragility curves were examined, it was found out that, for all levels of damage, the shapes of the fragility curves of the original and retrofitted buildings were similar but with varying values. This indicated that, firstly, the median fragility values after retrofit were greater than the corresponding values before retrofit, and secondly if the fragility curves were used to estimate the average number of buildings suffering from a certain state of damage, the number of damaged buildings after retrofit was smaller than those before retrofit, when the building was subjected to earthquake. Consequently, for each damage level, the obtained physical improvement in the seismic fragility as a result of addition of steel braces became apparent in terms of enhanced fragility curves shifting those to the right according to the original building.

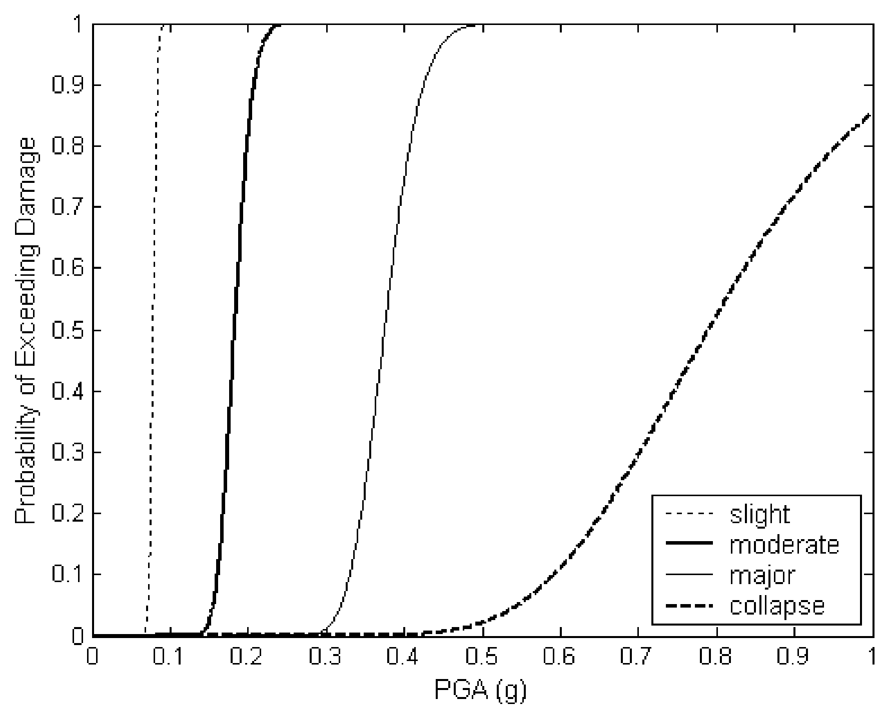


(a)



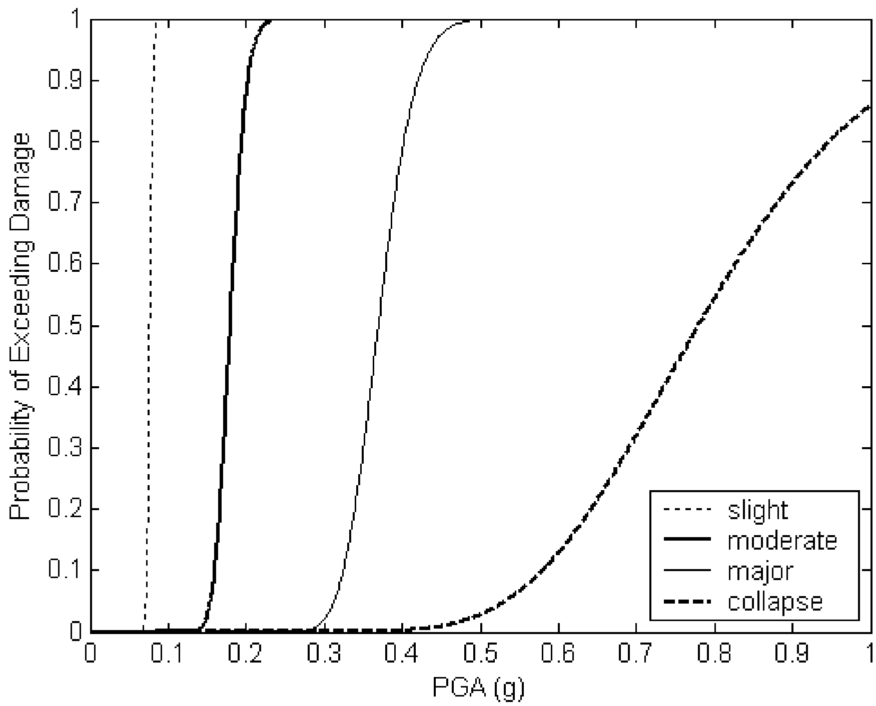
Probability of Exceeding Damage
PGA (g)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
slight
moderate
major
collapse

(b)



Probability of Exceeding Damage
PGA (g)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
slight
moderate
major
collapse

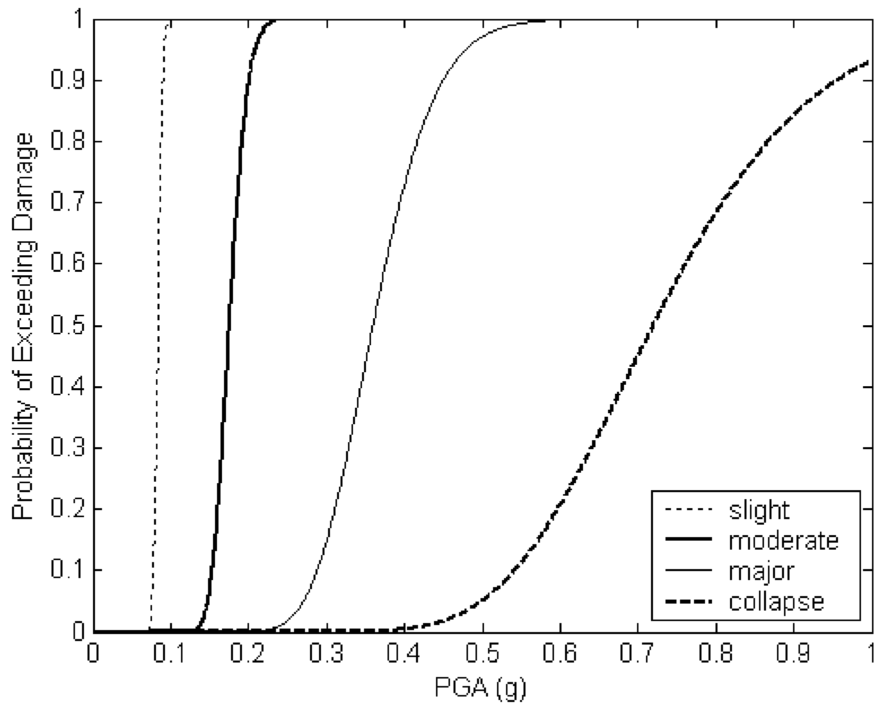
(c)



(d)

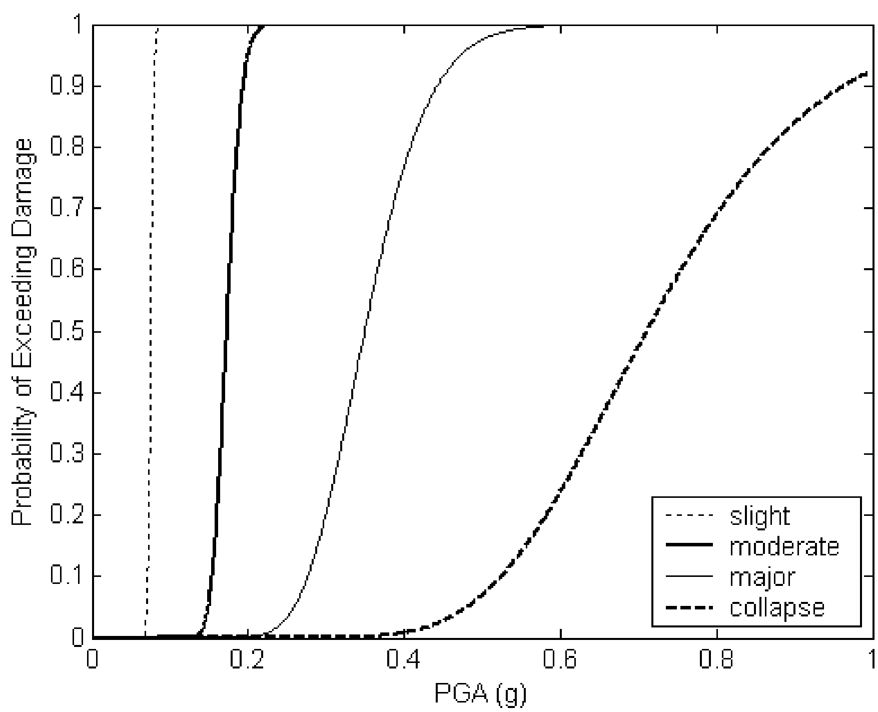
Figure 4.2 Fragility curves developed for the retrofitted buildings by using different distributions of D braces: (a) D1, (b) D2, (c) D3, and (d) D4

Another remarkable point on the fragility curves developed for the original and the braced frames (From Figures 4.1 to 4.4) was that the fragility curves became flatter as damage level shifts from slight to collapse. This showed that the structure was more sensitive to the changes under low PGA values than high PGA values, in other words the small variations in low PGA values resulted in noticeable differences in the probability of exceedance of damage levels. Similar observations were also reported by Erberik and Elnashai (2004) and Guneyisi and Altay (2008) regarding the flatness of the fragility curves developed for different type of structures.



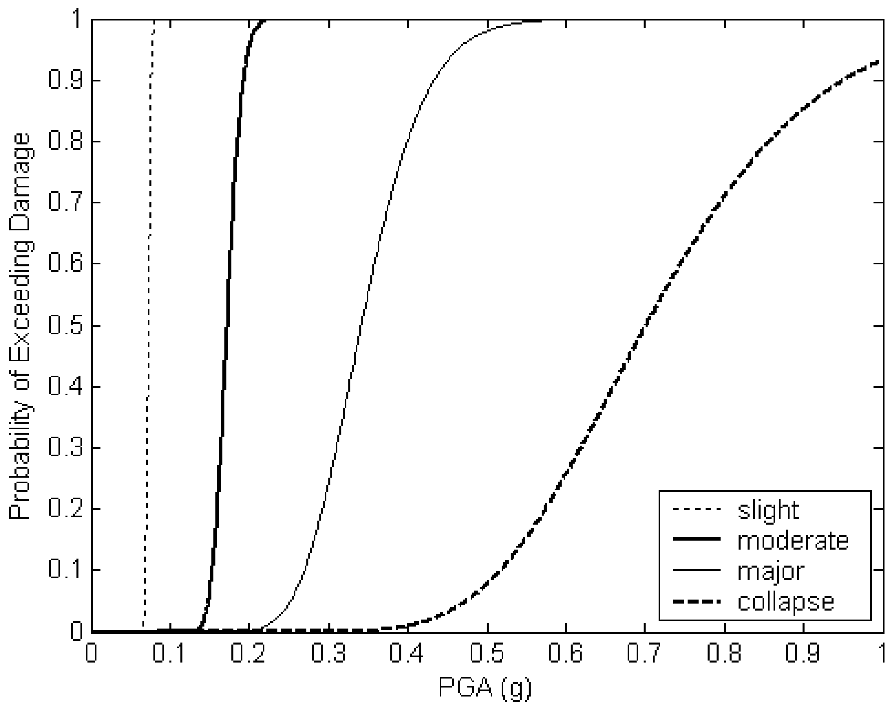
Probability of Exceeding Damage
PGA (g)
slight
moderate
major
collapse

(a)

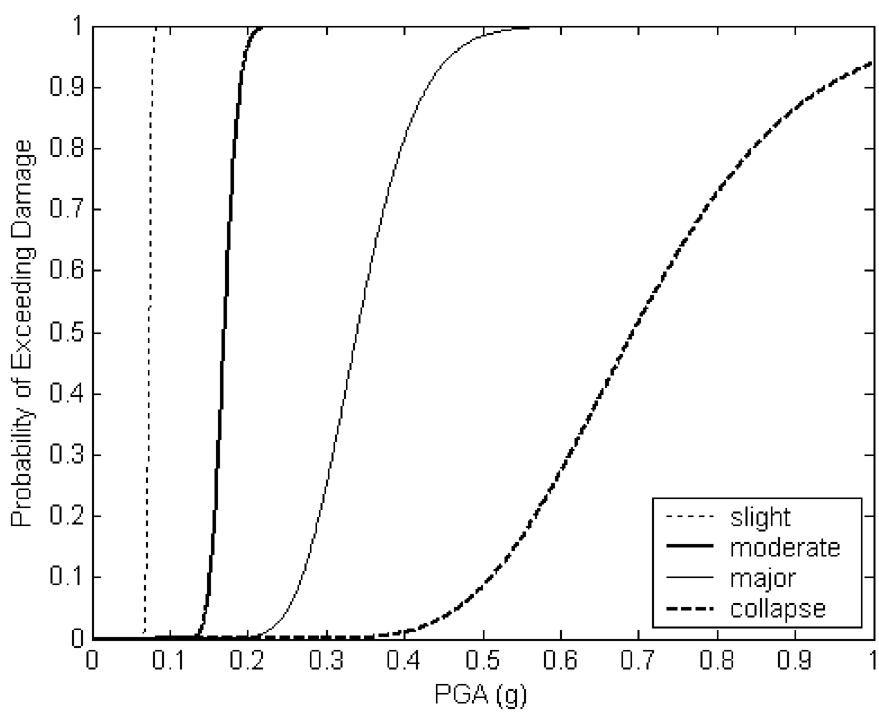


Probability of Exceeding Damage
PGA (g)
slight
moderate
major
collapse

(b)



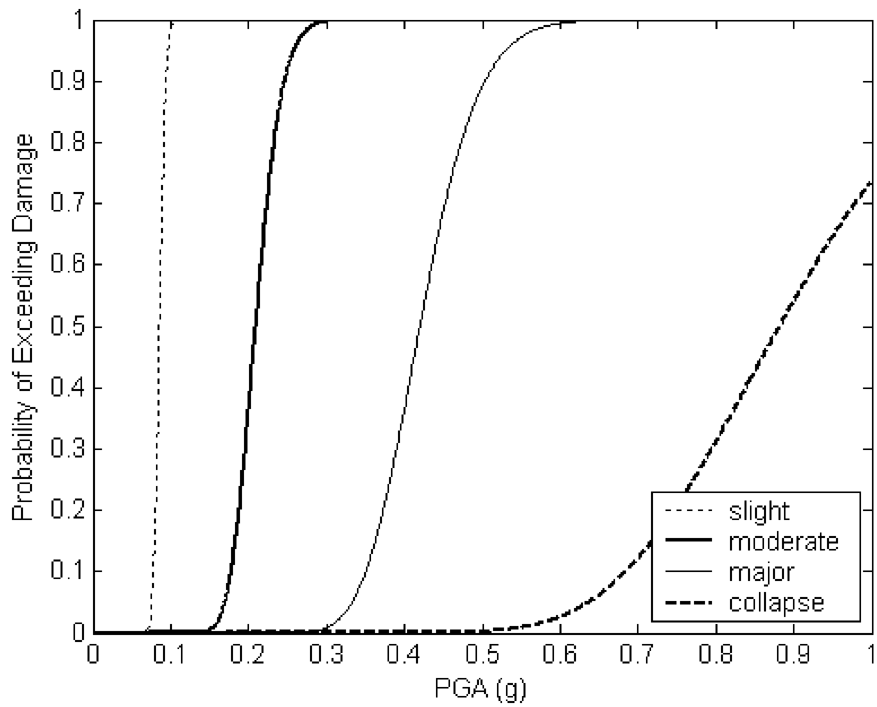
(c)



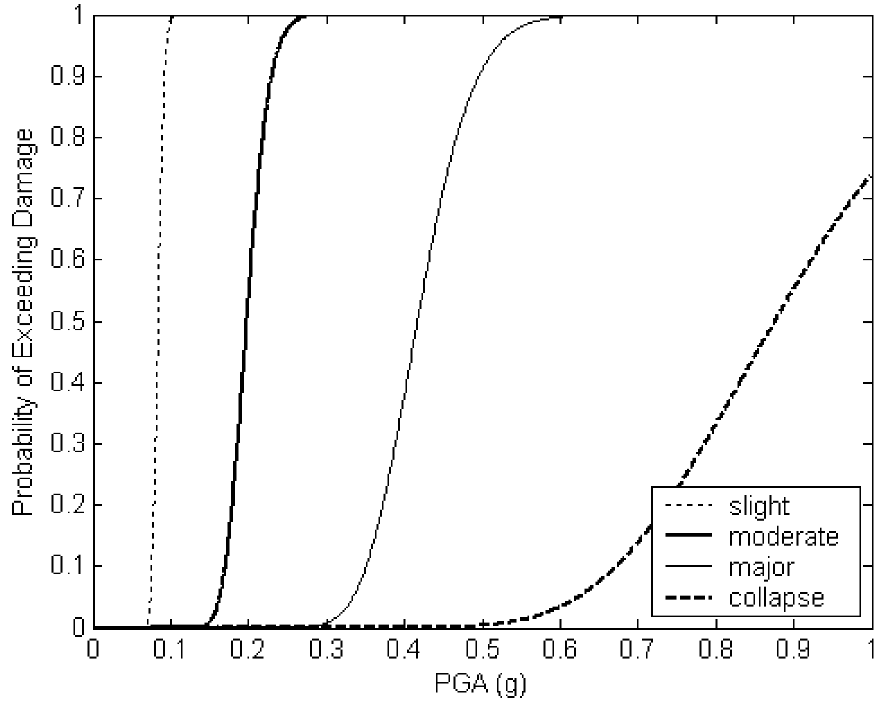
(d)

Figure 4.3 Fragility curves developed for the retrofitted buildings by using different distributions of K braces: (a) K1, (b) K2, (c) K3, and (d) K4

The effectiveness of using steel braces on seismic performance of the original building was also examined by means of median values. The median values of fragility curves constructed for the braced frames were compared in each damage level. For better understanding the effect of distribution of steel braces, the median value of probability of exceedance is presented with a histogram given in Figure 4.5.

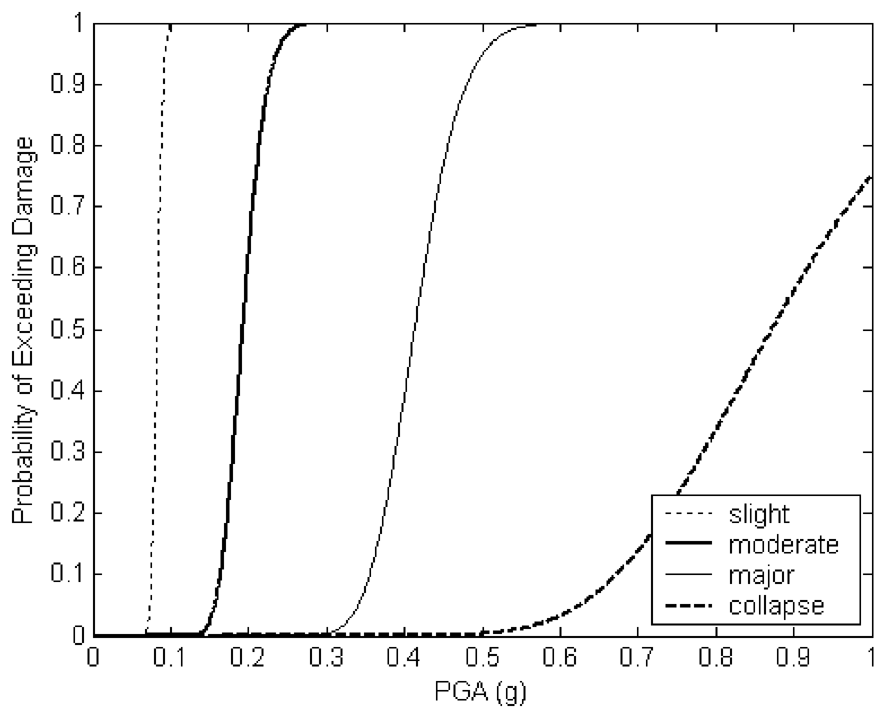


(a)



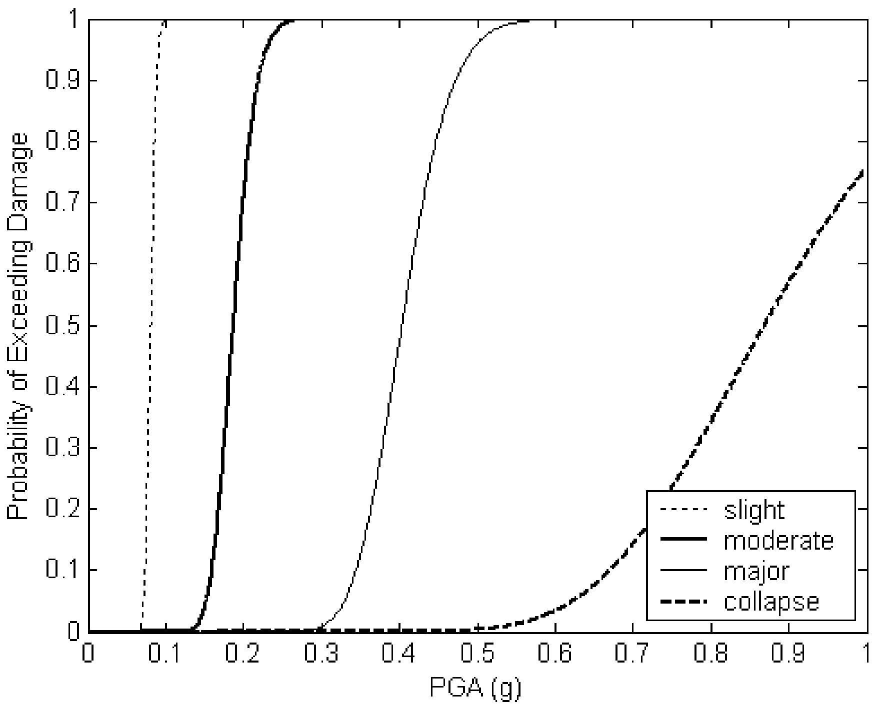
Probability of Exceeding Damage
PGA (g)
slight
moderate
major
collapse

(b)



Probability of Exceeding Damage
PGA (g)
slight
moderate
major
collapse

(c)



(d)

Figure 4.4 Fragility curves developed for the retrofitted buildings by using different distributions of V braces: (a) V1, (b) V2, (c) V3, and (d) V4

As seen in Figure 4.5, for the slight damage level, the median peak ground accelerations ranged from 0.072 to 0.085 which were obtained for the K3 braced frame and V1 braced frame, respectively. For the moderate damage level, the median values varied from 0.170 (for the K4 braced frame) to 0.209 (for the V1 braced frame). For major and collapse damage levels, the minimum median values were obtained as 0.339 and 0.693 for the K4 braced frame, and the maximum median values were achieved as 0.420 and 0.881 for the V1 braced frames, respectively. Another thing that may be observed regarding fragility curves was the slight changes in the fragility curves according to braced frame distributions. The changes in the median values with respect to distributions reached maximum of 15%, 12%, and 7%, among the K braced, V braced, and D braced frame groups, respectively. If the figure was analyzed carefully, it was pointed out that in most cases, the first type of distributions had the maximum median value; however the fourth type of distributions had the minimum. Second and third distribution types interchangeably gave better results.

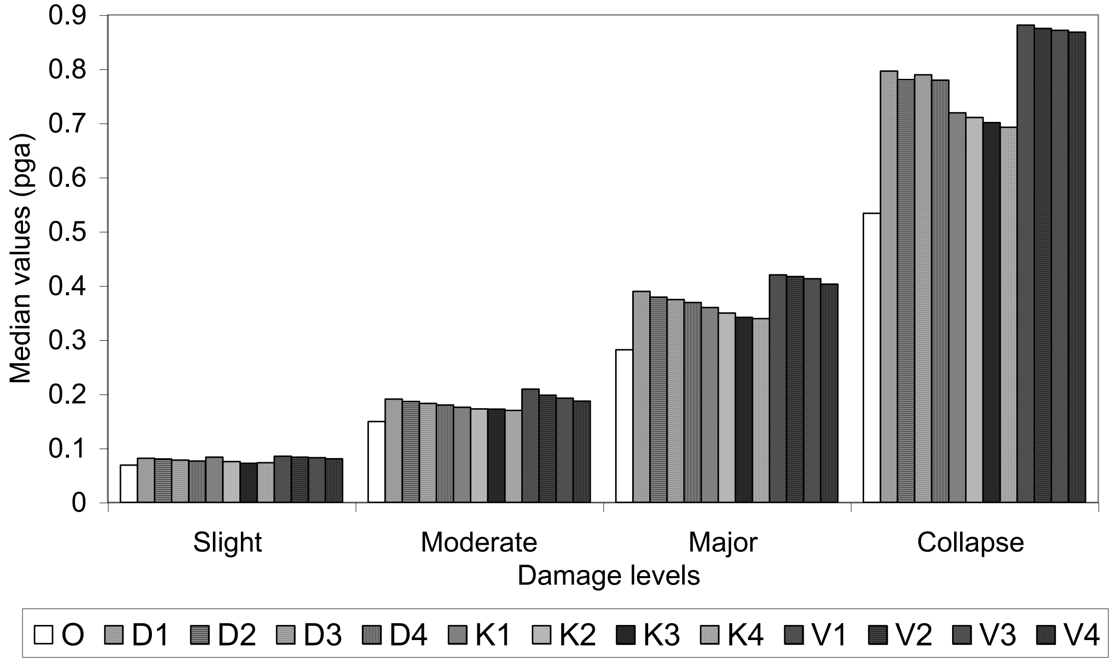
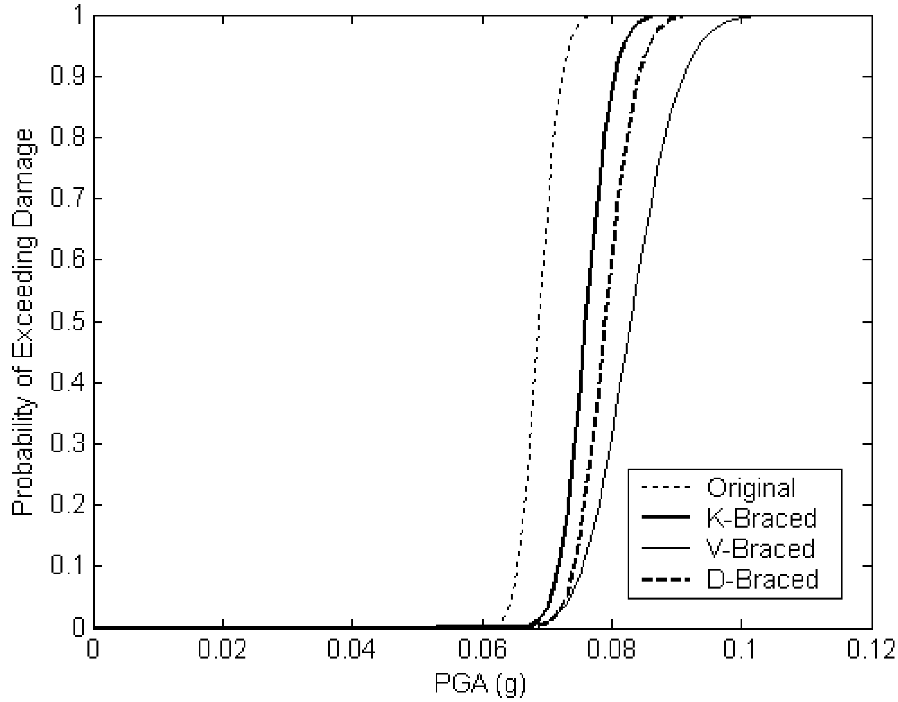
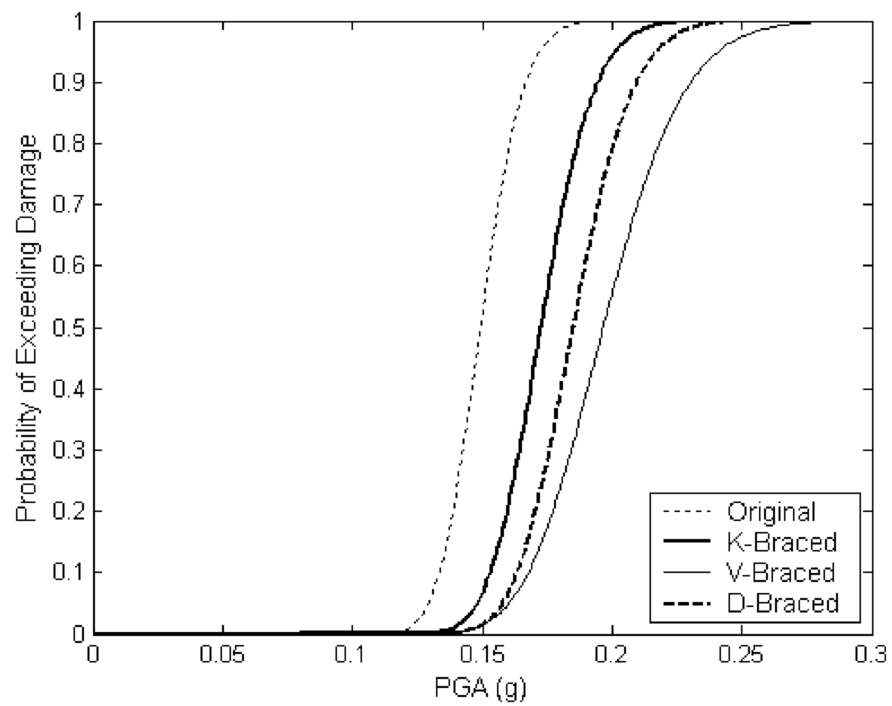


Figure 4.5 Comparison of median values of the original and braced frames

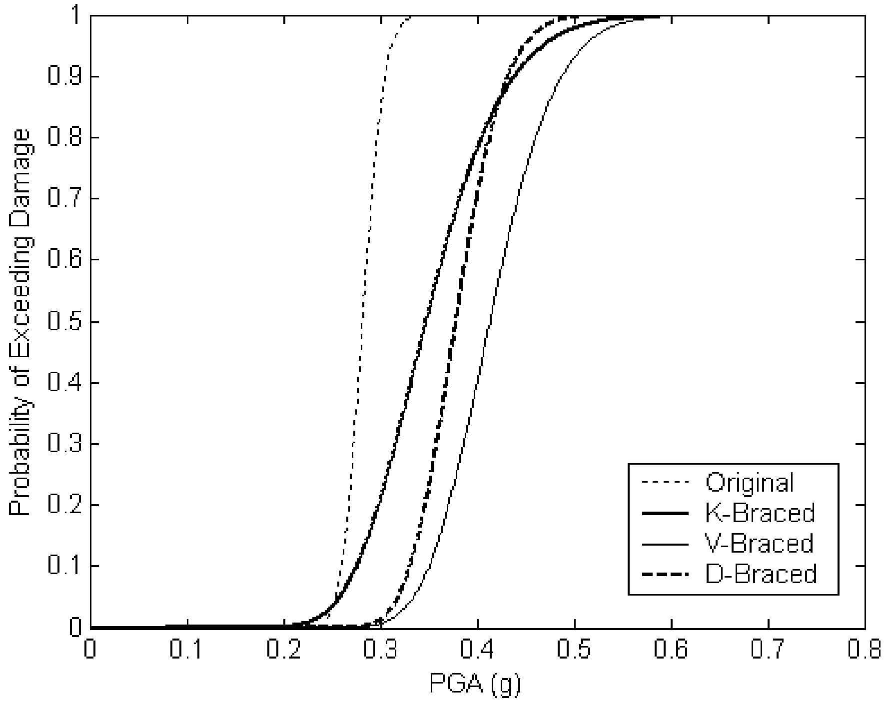
As mentioned above, the differences resulting from the distribution type of steel braces were small in each braced frame groups, so the types of braces were compared by calculation of the average values of medians and standard deviations of the fragility curve parameters which is given in Table 3.3. By using these average values of medians and standard deviations, the fragility curves were drawn separately for each damage level to determine the alone effect of type of braces. Figure 4.6 depicts comparison of the fragility curves of the original and braced frames for different damage levels, namely slight, moderate, major, and collapse.


Probability of Exceeding Damage
PGA (g)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.02
0.04
0.06
0.08
0.1
0.12
Original
K-Braced
V-Braced
D-Braced

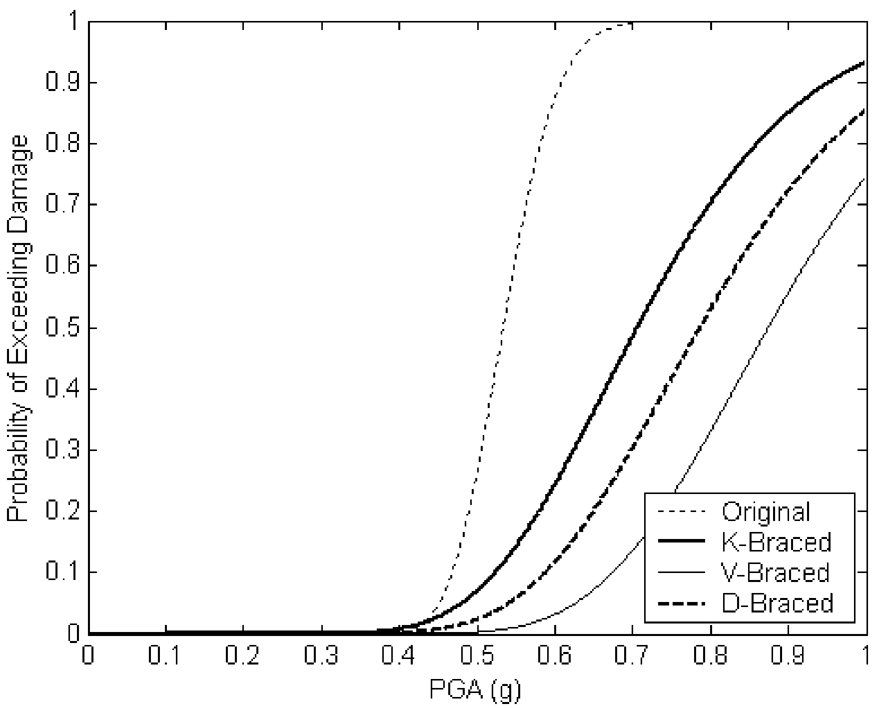
(a)


Probability of Exceeding Damage
PGA (g)
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
0.05
0.1
0.15
0.2
0.25
0.3
Original
K-Braced
V-Braced
D-Braced

(b)



(c)



(d)

Figure 4.6 Comparison of the fragility curves of the original and braced frames for (a) slight, (b) moderate, (c) major, and (d) collapse damage levels

From Figure 4.6, it was obvious that the eccentric steel braces performed well on the seismic performance of the building. To give an example, for complete damage, the buildings were up to 1.47, 1.32, and 1.63 less fragile after retrofitting by D, K, and V braces, respectively; compared to the case before retrofitting in terms of median values. It was also observed from the figure that in enhancing the seismic performance of the building the effectiveness of steel braces was increasing with the increasing degree of damage. As a result of comparison of type of frames with each other, the V braced frames had the greatest effect on the seismic performance of the building.

The amount of fragility enhancement over the original structure for each retrofitting case at each state of damage was computed and plotted as a function of the state of damage. In the computation of this, the original and the retrofitted frames, namely K braced, V braced and D braced frames and corresponding sets of fragility curves before and after retrofitting were considered. In Figure 4.7, by interpolation of an analytical function, the "enhancement curve" was plotted through curve fitting for each type of steel braces used.

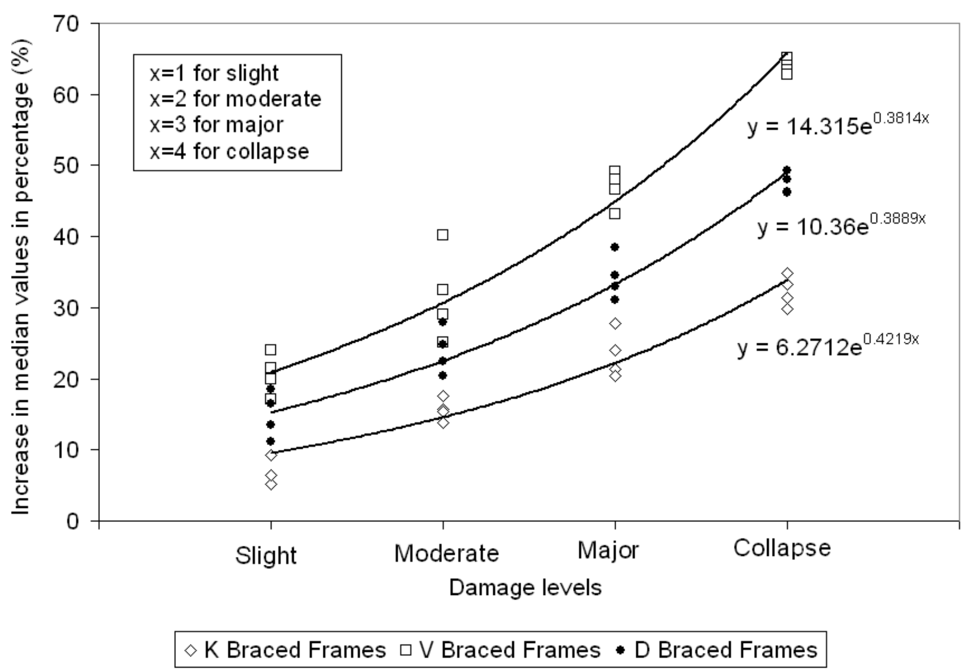


Figure 4.7 Fragility enhancement curves with respect to K, V, and D braces

As shown in Figure 4.7, the V type bracing had the maximum effect on the fragility enhancement. After that the D type and K type bracings were followed. For the V type bracing, the first curve showed 21, 31, 45, and 66% improvements for each damage state described on the x-axis. For the D and K type bracing, the other two enhancement curves showed 15, 23, 33, and 49% and 10, 15, 22, and 35% improvements in the median values for the slight, moderate, major, and collapse damage levels, respectively.

CHAPTER 5

CONCLUSIONS

In this thesis, the effects of type and distribution of eccentric steel braces on seismic fragility of mid-rise reinforced concrete (R/C) buildings was investigated considering different condition of damage states, namely slight, moderate, major, and collapse. For this, the fragility analyses of a sample mid-rise R/C building before and after retrofitting were conducted through nonlinear time history analysis by using a set artificially generated earthquake ground motions compatible with the design spectrum. These analyses were performed to determine the effectiveness of different type (D, K, and V) and distribution of eccentric steel braces on seismic vulnerability of building. Based on the results obtained in this numerical study and subsequent analysis, the following conclusions could be drawn:

1. The nonlinear time history analysis performed on the existing building and the three types of braced frames indicated that each artificially generated ground motion record exhibited its own features, demonstrated by frequency content, duration, sequence of peaks and their amplitude. The dispersion in the results of different ground motions depended on the characteristics of both the structure and the record.

2. This study revealed that the maximum lateral load carrying capacity was reached when the V type bracing was used for retrofitting proposes, however, it was nearly the same for D-braced and K-braced frames. Moreover, all retrofitted buildings exhibited lower displacement when subjected to earthquake ground motions as compared to the behavior of the existing building. Similar to

the results of lateral load carrying capacity, the V-braced frames had the lowest displacement among the three retrofitting cases. The displacements of the D-braced and K-braced frames were almost similar, with a little more displacement of K-braced frame.

3. Analysis of the results indicated that the developed fragility curves for the existing frame and the retrofitted frames had similar shapes, but with varying values of PGA. These varying values of PGA indicated the physical improvement of the seismic vulnerability due to addition of retrofitting braces by shifting those to the right associated with the existing building.

4. From the results of this study, it was pointed out that the selected retrofit systems for the strengthening of the existing R/C building resulted in supportive effects on the seismic performance of the building. The created fragility curves after retrofitting with steel braces showed improvement (less fragile) compared to those before retrofit by as much as 1.8 times (V1 braced frame) based on median PGA values.

5. When the median PGA values of the retrofitted cases were compared with the existing building, it was found that the effectiveness of the retrofitting in enhancing seismic performance increased with the increase in damage states of the building. However, the use of different retrofitting strategies for seismic rehabilitation brought out different performance. All types of retrofitting cases had relatively higher stiffness and shorter fundamental period than the existing building. Especially, the V-braced retrofitting had the greater stiffness as compared to the D-braced and K-braced retrofitting strategies.

6. According to the fragility analysis, distributions of the eccentric steel braces slightly affected the seismic reliability of the braced frames. Among distributions, the first distributions (K1, V1, or D1) generally give the greatest seismic reliability while the fourth distributions (K4, V4, or D4) give the least,

which could be verified from median PGA values. These results were consistent with the results of nonlinear static analyses.

7. Fragility enhancement curves with respect to K, V, and D braces demonstrated that the V type bracing had the maximum effect on the fragility enhancement. After that the D type and K type bracings were followed. For the V type bracing, 21, 31, 45, and 66% improvements in the median values were achieved for the slight, moderate, major, and collapse damage levels, respectively. For the D and K type bracing, 15, 23, 33, and 49% and 10, 15, 22, and 35% improvements in the median values were attained for each damage states, respectively.

8. The computed analytical fragility curves corresponding to stated damage levels could be used to evaluate the mid-rise reinforced concrete building design, retrofit and performance of the building in past seismic events. Furthermore, the composed fragility curves might be useful in determining the potential losses resulting from earthquakes and to assess the effectiveness of the eccentric steel braces in retrofitting.

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