A SEISMIC VULNERABILITY ANALYSIS PROCEDURE FOR URBAN LOSS ASSESSMENT

by

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To my beloved wife, Arzu and daughter, Pınar

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ABSTRACT

A SEISMIC VULNERABILITY ANALYSIS PROCEDURE FOR URBAN LOSS ASSESSMENT

The assessment of seismic vulnerability of buildings in seismically active urban areas is of great importance in terms of engineering, economical and social aspects. In this study an analytical procedure has been developed in order to obtain the vulnerability functions of existing buildings. The most important feature of the study is the building data based on information collected from real existing reinforced concrete buildings in the city of Bolu. The building information included over all geometry of the structure, as-built dimensions and configuration of the structural members, complete with reinforcement details of beams and columns, which are extracted from the design dossiers available in the Bolu municipality archives. All architectural details, loads and material properties of concrete and reinforcing steel are considered as given in the existing documents. The buildings are classified depending on their number of stories ranging from 2 to 7 story. Mathematical models of 120 buildings, 20 from each class, have been constructed in order to perform nonlinear response history analysis and obtain damage distributions by using 20 spectrum compatible ground motions. A statistical process has been applied to such damage distributions and parameters of the vulnerability functions are determined for each building class. The vulnerability functions are expressed in terms of spectral acceleration as well as in spectral displacement corresponding to the first mode period of the building under consideration.

In the first chapter of the study a brief explanation of the role of loss estimation studies in urban planning, urban disaster management and mitigation has been given. Main components of urban earthquake loss estimation are explained with particular emphasis given to the elements of building vulnerability relationships, for which the development of an analysis procedure constituted the objective of this thesis. In the second chapter, existing building vulnerability relationships have been evaluated and particular examples

of studies that have made important contribution to the development are briefly cited. In the third chapter, basic steps of a standard derivation of analytical vulnerability functions are identified and the importance of each step in the process has been explained. In the fourth chapter, the procedure developed in this thesis based on real building data has been explained in detail including the ground motion characterization, properties of building data, analytical methods used in the analysis, damage definition and quantification, statistical process and the determination of the vulnerability parameters. Numerical results are presented in the same chapter in the form of log-normal vulnerability curves with respect to representative seismic intensity parameter, spectral acceleration or spectral displacement. In the fifth chapter, the conclusions are presented and the contribution of the study to the existing knowledge in the field has been evaluated

ÖZET

KENTSEL DEPREM HASAR TAHMİNİ İÇİN BİR YÖNTEM

Yüksek sismik aktivitenin meydana geldiği bölgelerde bulunan büyük şehirlerdeki yapıların deprem tehlikesi hem mühendislik, hem sosyolojik hem de ekonomik olarak bütük önem taşımaktadır. Bu çalışmada kentsel deprem hasar tahmini için bir yöntem önerilmektedir. Bu çalışmada mevcut yapı soğunun depremde hasargörebilirliğinin belirlenemesi amacı ile analitik bir yöntem uygulanmıştır. Hasargörebilirlik çalışmaları deprem hareketinin özellikleri, yapı stoğu veri tabanı ve hasar düzeylerinin belirlenmesi olarak üç ana bileşene sahiptir. Bu çalışmanın en önemli özelliği söz konusu bileşenlerden bina envanterinin Bolu ili merkezindeki gerçek betonarme binalardan oluşmasıdır. Bina envanterine ait bilgiler Bolu ili belediye arşivinde mevcut olan söz konusu binalara ait projelere dayanann geometrisi, taşıyıcı elemanlarının yerleşimi ve daha da detaylı olarak betonarme detayları içermektedir. Söz konusu binalara etkiyen yükler, beton ve çelik sınıfları vediğer tasarım parametreleri mevcut projelerde belirtilen değerler olarka dikkate alınmıştır. Söz konusu çalışmada toplam 120 adet bina kat adetlerine dayalı olarak 2 ile 7 katlı, her sınıfta 20 adet olmak üzere, sınıflara ayrılmıştır. Binalarda meydana gelen hasar dağılımı deprem yönetmeliğinde tanımlanan spektruma uygun spektruma sahip 20 adet yer hareketi kullanarak elde edilmiştir. Analiz sonucu elde edilen hasar dağılımına uygulanan istatistiksel işlemler sonucunda her bir bina sınıfına iat hasargörebilirlik paramatreleri elde edilmiştir. Söz konusu hasar görebilirilik eğrileri her bir yapı sınıfının hakim periyoduna karşılık gelen spekral ivme ve spektral deplasmana gore çizilmiştir.

Çalışmanın birinci bölümünde deprem hasar kaybı tahmin çalışmalarının kenstel planlama, kentsel afet yönetimi ve kentsel afet zararlarını azaltma çalışmalarındaki rolü üzerinde durulmuştur. Ayrıca kensel deprem hasar kayıp çalışmalarının bileşenleri olan “sismik aktvitenin belilenmesi”, “yapı stoğu envanterinin hazırlanması”, “hasargörebilirlik parametrelerinin belirlenmesi” hakkında bilgiler verilmiştir. Yukarıda sıralanan bileşenler dikkate alınarak çalışmanın amacı açıklanmıştır. İkinci bölümde ise literatürde bulunan

temel hasargörebilirlik yöntemleri açıklanmış ve ilgili konuya önemli katkıda bulunan çalışmalardan kısaca söz edilmiştir. Üçüncü bölümde ise standart analitik hasargörebilirlik analizi yönteminin her bir bileşeni olan sismik aktivitenin belirlenmesi, bina stoğunun oluşturulması, kullanılan analiz yöntemi, hasar düzeylerinin tanımlanması ve hasargörebilirlik parametrelerinin belirlenmesi detaylı olarak açıklanmıştır. Dördüncü bölümde ise bina deprem hasargörebilirlik yöntemi için önerilen yöntem her bir bileşen için detaylı olarak açıklanmıştır. Elde edilen sonuçlar aynı bölüm kapsamında log-normal dağılımın uygulandığı hasargörebilirlik eğrileri spektral ivme ve deplasman şeklinde sunulmuştur. Beşinci bölümde ise elde edile sonuçlar ve literatürde mevcut hasargörebilirlik çalışmalarına katkısı değerlendirilmiştir.

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LIST OF SYMBOLS / ABBREVIATIONS

D_T	building damage index by Park and Ang
D_i	damage index for ith element
d_i	ith damage state
dE	unit energy per cycle
DI	Park and Ang damage index
DI_{ds}	threshold value damage state
E_i	total absorbed energy in the member i
$f_x(x)$	normal probability distribution function
m	standard deviation
M_y	yield moment of the element
P	probability of reaching and exceeding
s	standard normal variable
$S_a(T)$	Spectral Acceleration corresponding to period T
$S_d(T)$	Spectral Displacement corresponding to period T
S_{ae}	elastic spectral acceleration
$(S_{ae})_{m,ds}$	mean elastic spectral acceleration of damage state d_s
$\overline{X}$	mean of the population
x	random variable
Y_{mi}	median threshold value
α	confidence interval of mean of population
β	non-negative hysteretic energy parameter
β_i	lognormal standard deviation
β_{ds}.	natural logarithm of spectral acceleration data for damage state d_s
Φ	standard normal distribution function
λ	mean of natural logarithm of data
λ_i	energy absorbing contribution factor
θ_m	maximum rotation at the end of a member
θ_u	ultimate rotation capacity of a member

σ	standard deviation
ARMA	Auto Regressive Moving Avarage
ATC	Applied Technology Council
DI	Damage Index
DI_{HRC}	Homogenous reinforce concrete damage index
DMP	Damage Probability Matrix
EMS	European Microseismic Scale
FEMA	Federal Emergency Management Agency
FORM	First order reliability method
GIS	Geographical Information Systems
HRC	Homogenized Reinforced Concrete
IDA	Incremental Dynamic Analysis
$ISD_{max\%}$	Maximum inter-story drift ratio (in percent)
JPDF	Joint probability density function
LHS	Latin Hypercube Sampling
MIDR	Maximum inter-story drift ratio
MMI	Modified Mercalli Intensity
MRF	Moment resisting frame
MSK	Medvedev – Sponheuer – Karnik Scale
PGA	Peak ground acceleration
PGV	Peak ground velocity
PSI	Parameterless Scale of Intensity
SEAOC	Structural Engineers Association of California

1. INTRODUCTION

As one of the countries located in the most active seismic zones in the world, Turkey has suffered great damage from the earthquakes in recent decades. The level of damage was in the order of tens of thousands of casualties and billions of dollars of economic losses. It is a well-known fact that the major source of these losses is due to structural damage to the building stock. The observations after every earthquake have revealed that buildings designed and constructed without adequate seismic design and site inspection as well as the use of improper materials were the main reasons of structural damage. As a result of the rapid urbanization, the amount of those inferior buildings has increased in vast numbers, such that the decision makers are on the edge of a very difficult situation as how to deal with this huge vulnerable building stock. At this point seismic vulnerability studies may provide guidance for them to decide where to start to attack the problem.

The vulnerability assessment tools are developed in order to estimate the seismic risk prior to the earthquake occurrence. Results of such assessment studies have made it possible to prioritize the elements at risk and to decide on the vulnerable buildings whether to be demolished or to be retrofitted. On the other hand such studies help determine the common deficiencies of the seismic resistance of the building stock.

As to the social affects of earthquakes, from urban planning point of view, assessment of structural damage of building stock in a city would make a major contribution for the sustainable development. Vulnerability assessment studies can play an important role in rising public awareness as to the risk they are facing with, making the public take part in the mitigation actions and creating demand for the seismic resistant construction. As a side product, vulnerability assessment studies can make significant contribution to the insurance industry for the estimation of seismic risk. On the other hand, estimation of the most probable areas that would be affected in case of a future earthquake would help for organization and mobilization of rescue operations, medical interventions and temporary housing activities for an effective response. As a last point it must be emphasized that the vulnerability studies will help to shape up the future development of the cities as an efficient tool for a better urban planning.

1.1. Components of Urban Earthquake Loss Estimation

The main components of urban earthquake loss estimation process involve the following:

a) Estimation of the regional seismic hazard,
b) Identification and classification of urban elements at risk,
c) Use of appropriate vulnerability relationships.

Since each of those components is associated with large uncertainties, the process requires continuous development and refinement efforts for the creation of more reliable risk assessment tools.

As the first component, the seismic hazard analysis or in other words the estimation of seismic input is dealt with by considering the seismicity and the site conditions of the region under consideration. In addition to the development deterministic and probabilistic hazard analysis methodologies, the number available ground motion records is increasing worldwide by means of strong ground motion networks established in seismically active regions, which supplies reliable ground motion data.

The second component of the urban loss estimation, i.e., the identification and classification of urban elements at risk, mainly involves the building stock and urban infrastructure. The building inventory data is obtained through building census studies or street surveys, where the buildings are generally classified based on their construction materials (reinforced concrete, steel, masonry, timber), structural systems (infilled moment resisting frame, shear wall, dual system, braced system, unreinforced masonry, etc) and the number of stories. As a recent development in modern urban management, a number of cities have developed databases of their building stock and performed microzonation studies. It is evident that such databases would provide a better quality of building inventory and seismic hazard descriptions for loss estimation studies. As the quality of the data increases the assessment studies would become more reliable and the database would be more useful for future studies in terms of calibration and verification.

Finally, as the third component of urban seismic risk estimation, appropriately generated vulnerability relationships (fragility curves) are utilized to estimate the probability of various levels of damages being reached or exceeded for a given class of a building under a given earthquake level. Thus, vulnerability relationships reflect the damage probability of a certain type of structures with certain seismic resistance properties located in a given region.

The methods dealing with the development of vulnerability relationships are mainly divided into three classes; empirical, analytical and hybrid methods, the last of which is the combination of the first two. The detailed information will be given in Chapter 2 for each class with evaluation of related studies in the literature.

1.2. Elements of Building Vulnerability Relationships

The present study is mainly focused on the development of improved vulnerability relationship for building stocks. In the following paragraphs, the major elements of the development process are briefly explained and the effects of each element the resulting vulnerability relationships are evaluated.

The first element of the vulnerability relationship development is the identification of the building typologies to be considered. The building typologies used in empirical methods are identified as those observed in the post-earthquake damage surveys. Since such building typologies inevitably reflect the local characteristics of the damaged building stock in a given region, the resulting empirical vulnerability relationships should be used with caution in different conditions. In the development of analytical vulnerability relationships, generally, each building type is represented by a hypothetical structure that is assumed to reflect the seismic resistance properties of the building class under consideration both in geometrical and material aspects. In some cases the properties of those hypothetical structures are artificially generated by using Monte-Carlo simulation method or Latin hypercube sampling method. In view of the random distribution of actual structures possessing a great deal of uncertainties, the viability of such artificial building typology generation schemes is questionable. In the present study, an existing building stock is directly utilized in the development of vulnerability relationships in order avoid an inevitable bias involved in such artificial typology generation procedures.

The second element of the vulnerability relationship development is the identification of the representative seismic intensity parameter. The parameter chosen for the characterization of the ground motion intensity should be able to correlate with the observed or calculated damage of buildings. The ground motion intensity can be identified in different ways depending on the type of vulnerability relationship used. In empirical relationships, seismic intensity parameters such as Modified Mercalli Scale, MSK scale (Medvedev and Sponheuer,1969) or EMS98 scale (Grünthal,1998), are commonly used. Occasionally peak ground motion parameters such as Peak Ground Acceleration (PGA) or Peak Ground Velocity (PGV) have been used. The more recent intensity parameters are the spectral values of the ground motion such as Spectral Acceleration, $S_a(T)$, and Spectral Displacement, $S_d(T)$. The purpose of using spectral values is that the intensity parameter is directly related to the response of the building class. Some examples of vulnerability relationships based on different seismic intensity parameters are given in Figure 1.2 to Figure 1.1

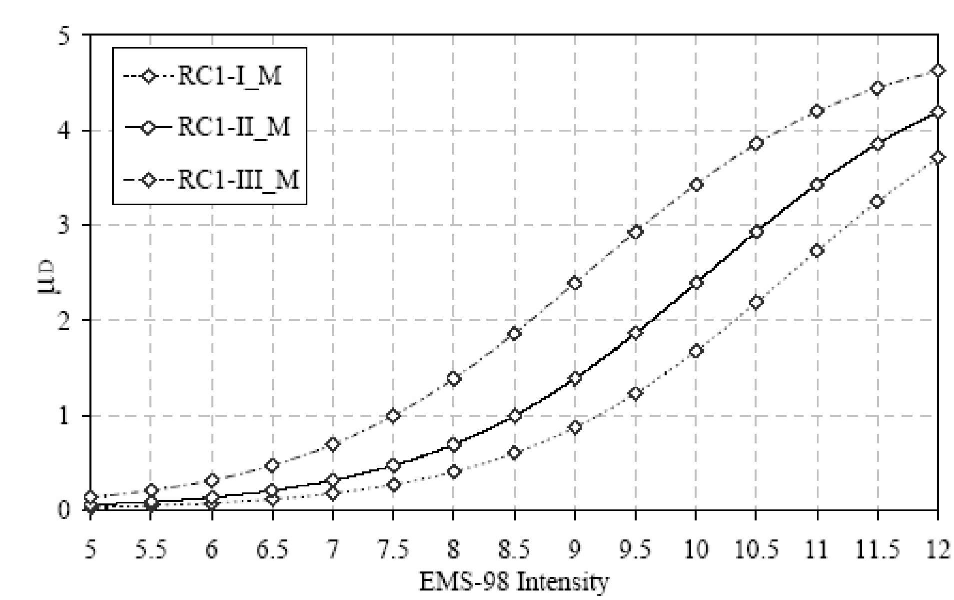

Figure 1.1. Fragility curves for RC structures with seismic intensity parameter defined as EMS-98 scale (Giovinazzi and Lagomarsino, 2006)

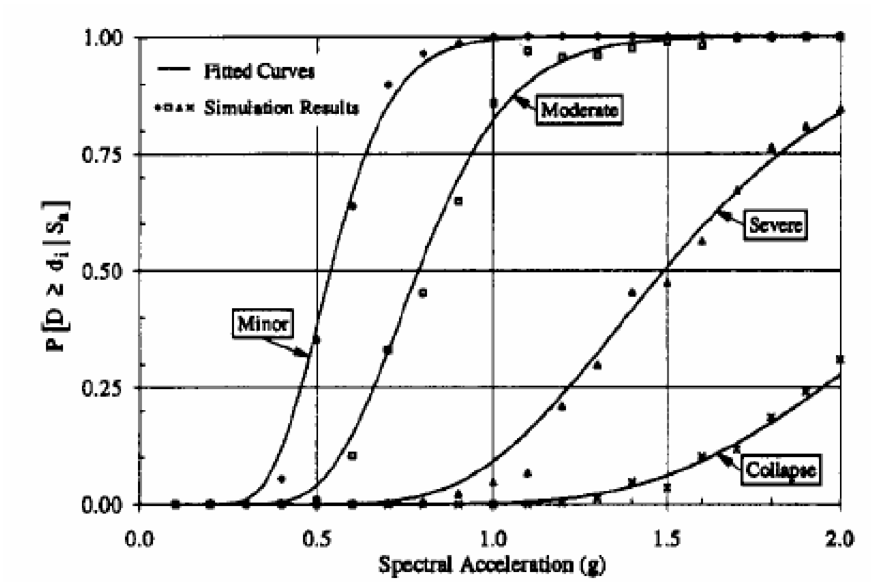

Figure 1.2. Fragility curves for high-rise frames with seismic intensity parameter defined as spectral acceleration (Singhal and Kiremidjian, 1996)

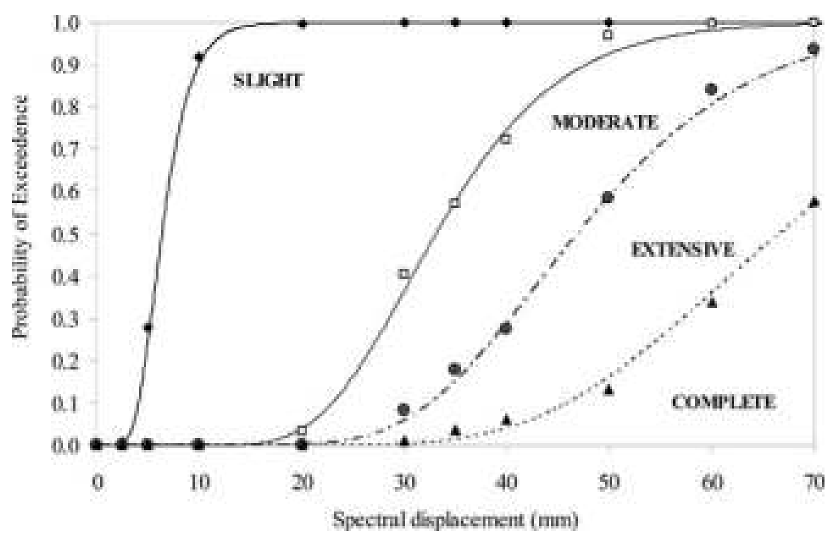

Figure 1.3. Fragility curves for flat-slab structures with seismic intensity parameter defined as spectral displacement (Erberik and Elnashai, 2005)

As the third element of the building vulnerability relationship development, identification of the damage levels plays a very important role. The damage level definition is directly related to the typology of building and the type of the vulnerability relationship used. In empirical relationships the damage levels are qualitatively identified, as in the case of EMS98 definitions. However such damage identification schemes are prone to be subjective, depending on the local conditions and construction practices. In case of analytical vulnerability relationships, identification of damage levels is done on the basis of damages estimated in structural members as a result of the analysis. This kind of damage identification must be correlated with the post earthquake damage surveys and laboratory tests. The damage level definitions are generally named as none, slight, moderate, heavy, extreme and collapse. One of the problems in damage definition is the estimation of the threshold values especially in the moderate level of damage.

The distribution of the damage parameter under a certain seismic intensity parameter is used in a statistical process in order to determine the probability of reaching and exceeding the predefined damage level threshold for a damage state. Generally a log-normal distribution is fitted to the results of the analysis and the parameters of the vulnerability function or so-called fragility curves are determined.

1.3. Objective of Present Study

The main objective of this study is to develop a realistic vulnerability functions through considering uncertainties associated with sampling of building data and seismic ground motion as well as identifying representative seismic demand parameters to quantify the real nonlinear seismic behavior and hence the damage of structures-The procedure is developed by using data of existing buildings of a typical Turkish city, Bolu.

The building inventory data to be used in this procedure is composed of real design data of a set of existing reinforced concrete building structures. The unique property of the data is that the as-built mathematical models of the structures can be constructed with their geometrical configuration, member dimensions, and all horizontal and vertical reinforcements given in the design drawings. The data is readily obtained from the archives of Bolu Municipality.

Thus, geometrical variability which has a significant effect on the structural response is taken into account by using real building data rather than using generic frames that are supposed to represent the building characteristics in a region as it is done in the majority of previous vulnerability studies. The material properties of the buildings are taken as they are indicated in the design drawings, in other words the material properties are determined in deterministic manner instead of assuming a mean and a standard deviation for both concrete and steel strengths. It may be argued that none of the existing studies on vulnerability assessment has that kind of a detailed structural data. It is believed that the data have the common characteristics of Turkish building stock both in terms of structural configuration and construction practice. Hence it is expected that the vulnerability relationships to be developed would represent the structural variability and seismic resistance properties of the typical reinforced concrete building structures in Turkey.

On the other hand recent studies revealed that (Rosetto and Elnashai, 2003, Shinozuka, 1997, Singhal and Kiremidjian, 1996) no satisfactory correlation can be established between structural damage and the peak ground motion parameters such as PGA or PGV. Therefore spectral values such as spectral acceleration (S_a) or spectral displacement (S_d) corresponding to predominant period of the building are used in this study since they have a direct relationship with the response of the structure under consideration.

The definition and quantification of the damage has also a significant effect on results. The damage measure to be used in the assessment should take into account the duration effect of the ground motion. Thus, level of damage in a member is better defined by considering the cyclic response of the member. In this line, the damage index proposed by Park and Ang (1985-a), which is a combination of ductility and the hysteretic response of the structural elements, is used in this study. Park and Ang damage index threshold values for different damage levels have been calibrated with past earthquake damage data and laboratory tests.

As mentioned above, the vulnerability curve parameters are estimated either by using observational data or analytical simulations. In analytical studies the important point~~s~~ is to estimate the seismic demand variation corresponding to all possible damage levels. In this study nonlinear response history analysis is used to evaluate the damage levels of the buildings contained in the database. All possible damage levels can be captured by selecting a wide range of ground motion intensity levels. But the use of selected ground motion databases may not result in all damage levels to be reached or the distribution of the results of damage levels may not be homogenous. In order to overcome this problem, a recently developed analysis methodology, namely "Incremental Dynamic Analysis" (IDA) (Vamvatsikos and Cornell, 2002), is used, in which the ground motion is scaled with respect to first mode spectral acceleration of the building under consideration with pre-defined spectral acceleration increments.

The results of the analysis are processed through the fundamental concepts of probability in order to estimate the parameters of the vulnerability curves. As in the other studies, the results are fitted to a log-normal distribution function. The assumptions in the statistical process have been verified by appropriate theorems of statistics. As a last step, the confidence levels of vulnerability curves have been determined in order to assess to what extent the curves are valid or whether any curve overlaps with others corresponding to other damage levels.

It is expected that the results of this study will make a significant contribution in terms of improved vulnerability relationships for use in future loss estimation studies in Turkey. All critical aspects of vulnerability analysis have been taken into account as detailed as possible in order to develop improved vulnerability relationships applicable to typical Turkish RC building.

2. EVALUATION OF EXISTING VULNERABILITY RELATIONSHIPS

2.1. Introduction

The seismic vulnerability of a structure can be described as its susceptibility to damage by ground shaking of a given intensity. The aim of a vulnerability assessment is to obtain the probability of a given level of damage to a given building type due to a seismic input. The various methods for vulnerability assessment that have been proposed in the past for use in loss estimation can be divided into three main categories: empirical, analytical or hybrid.

Regardless of the type of methodology, the common steps of any vulnerability assessment procedure are the estimation of seismic hazard, identification of a representative seismic intensity parameter and the damage level definition. The vulnerability relationships of the empirical methods are developed when sufficient building data based on post earthquake damage survey or those based on expert judgment are available .On the other hand the analytical approach is based on the analysis of representative building data with different geometrical and material characteristics, which are usually generated through simulation techniques. When limited damage distribution data is available from post earthquake damage surveys, a hybrid approach is used as a combination of empirical approach and the analytical approach, in which vulnerability relationships for non-existing damage levels of different building classes are determined by numerical simulation. The basic steps of the development process of vulnerability relationships are given in Figure 2.1.

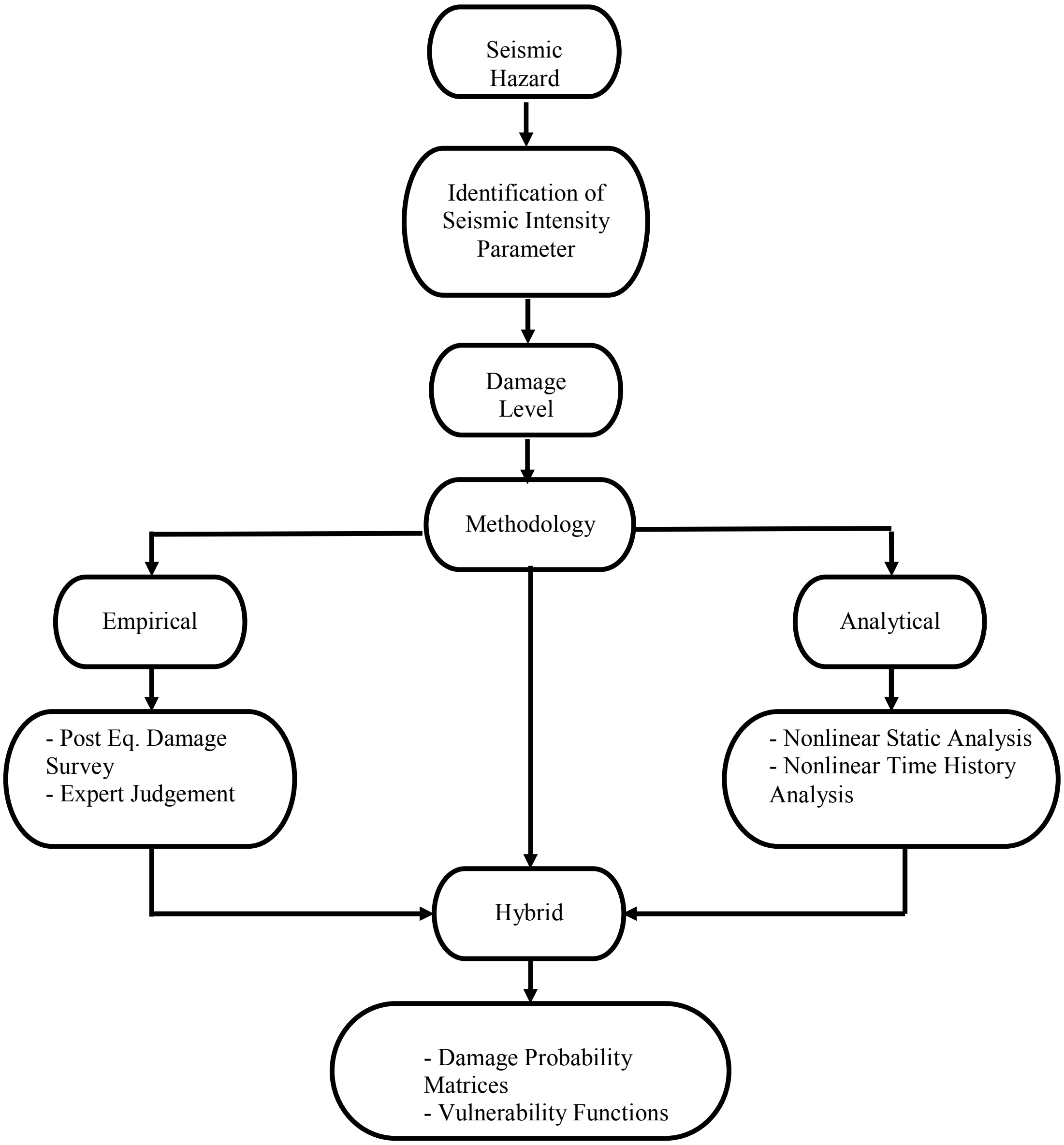

Figure 2.1. Basic steps of development process of seismic vulnerability relationships

2.2. Empirical Vulnerability Relationships

The seismic vulnerability assessment of buildings are first carried out in early 1970's through empirical vulnerability relationships developed and calibrated as a function of micro-seismic intensities. There are two main forms of empirical relationships based on damages observed after earthquakes-or assessed through expert judgment:

1) Damage probability matrices (*DPM*), which express in a discrete form the conditional probability of reaching or exceeding a prescribed damage level due to a given earthquake intensity
2) Vulnerability functions which are continuous functions expressing the probability of reaching or exceeding a given damage state as a continuous function of a parameter representing the earthquake intensity.

However, there are various disadvantages associated with the use of empirical methods such as DPM's:

- A macro-seismic intensity scale is defined by considering the observed damage of the building stock and thus in a loss model both the ground motion input and the vulnerability is based on the observed damage due to earthquakes.
- The derivation of empirical vulnerability functions requires the collection of post-earthquake building damage statistics at sites with similar ground conditions for a wide range of ground motions. Additionally, large magnitude earthquakes occur relatively infrequently near densely populated areas, and results in accumulation of data around the low damage vs. ground motion part of the matrix and limits the statistical validity of the high damage/ground motion end of the matrix.
- Seismic hazard maps are now defined in terms of PGA (or spectral ordinates) and thus PGA needs to be related to intensity; however, the uncertainty in this equation is frequently ignored. When the vulnerability is to be defined directly in terms of PGA, where recordings of the level of the ground shaking at the site of damage are not available.
- When PGA is used in the derivation of empirically-defined vulnerability, the relationship between the frequency content of the ground motions and the period of vibration of the buildings is not taken into account.

2.3. Empirical Methods Based on Post Earthquake Damage Survey

Majority of empirical relationships is based on damage survey data of the past earthquakes. The results of empirical methods are either in the form of vulnerability curves or damage probability matrices where damage distribution data is sufficient depending on the earthquake severity.

In the first assessment methodology evaluated developed in the literature, the data is obtained from post earthquake field survey and the vulnerability of the buildings are estimated in terms of damage probability matrices which has been first proposed by Whitman et al. (1973) for the probabilistic prediction of damage of buildings from recent earthquakes and compiled Damage Probability Matrices for various structural topologies according to the damaged sustained in over 1600 buildings after the 1971 San Fernando earthquake.

One of the first European versions of a damage probability matrix was produced by Braga et al. (1982), which were based on the damage data of Italian buildings after the 1980 Irpinia earthquake, and this introduced the binomial distribution to describe the damage distributions of any class for different seismic intensities. The buildings were separated into three vulnerability classes (A, B and C) and a Damage Probability Matrix based on the MSK scale was evaluated for each class. This type of method has also been termed 'direct' by Corsanego and Petrini (1990) because there is a direct relationship between the building typology and observed damage.

Post earthquake damage survey is also used to obtain continuous vulnerability curves. After the use of Damage Probability Matrices for vulnerability assessment, continuous vulnerability functions based directly on the damage of buildings from past earthquakes were introduced. There is an obstacle for their derivation since the macro-seismic intensity is not a continuous variable. This problem was overcome by Spence et al. (1992) through the use of their "*Parameterless Scale of Intensity*" (*PSI*) to derive vulnerability functions based on the observed damage of buildings using the MSK damage scale. An example of empirical vulnerability curves is obtained by Coburn (RMS, 1999) depending on the damage distribution of mid-rise RC buildings in 1999 Kocaeli given in Figure 2.2.

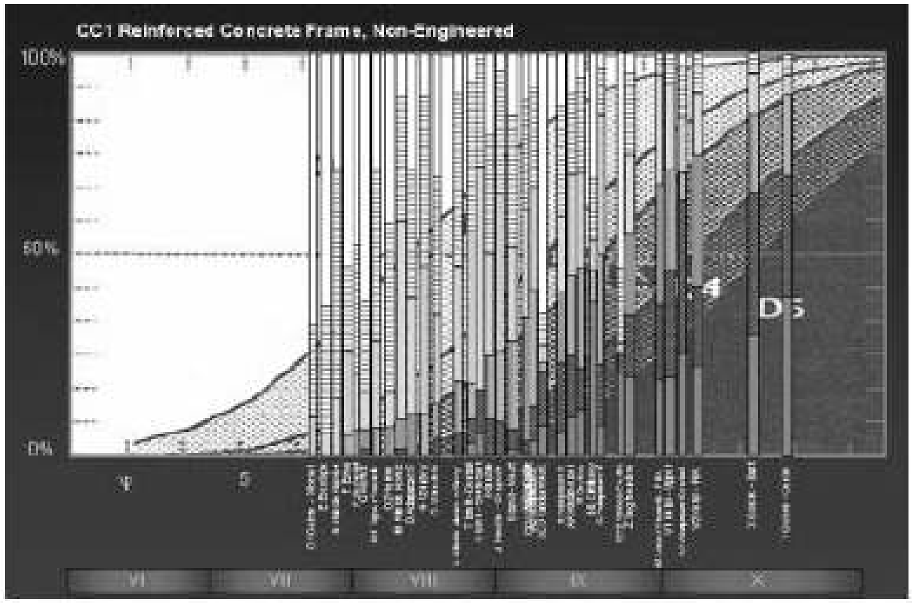

Figure 2.2 Empirical vulnerability relationships for mid-rise RC frame buildings after 1999 Kocaeli earthquake (Coburn, 1999)

The recently developed vulnerability functions do not use macro-seismic intensity or PGA to characterize the ground motion instead the ground motion parameter is selected as the spectral acceleration or spectral displacement at the fundamental elastic period of vibration (e.g., Rossetto and Elnashai, 2003; Shinozuka *et al.*,1997). The new methods have been an important development since it has a relationship between the frequency content of the ground motion and the fundamental period of vibration of the building stock is taken into consideration. The new vulnerability functions showed better correlation between the ground motion and the damage.

In an other study Rosetto and Elnashai (2003) have constructed new empirical fragility curves for reinforced concrete building populations based on the data of 99 post earthquake damage distributions observed in 19 earthquakes concerning a total of 340000 RC structures. Depending on the data available they have reinterpreted the heterogeneous data in terms of a new damage scale called "homogenized reinforced concrete (HRC) damage scale". They have further defined limit states are further defined in terms of a damage index, HRC damage index (DI_{HRC}) which provides a numerical reference scale for the experimental calibration with the structural response parameter of maximum inter-story drift ratio ($ISD_{max\%}$).

2.4. Empirical Methods Based on Expert Judgment

Damage Probability Matrices based on expert judgment and opinion were first introduced in ATC-13 (1985). More than 50 senior earthquake engineering experts were asked to provide low, best and high estimates of the damage factor (the ratio of loss to replacement cost, expressed as a percentage) for Modified Mercalli Intensities (MMI) from VI to XII for 36 different building classes.

Weighted means of the experts' estimates, based on the experience and confidence levels of the experts for each building class, were included in the averaging process, as described in Appendix G of ATC-13 (1985). A macro-seismic method has recently been proposed (Giovinazzi and Lagomarsino, 2004) that leads to the definition of damage probability functions based on the EMS-98 macro-seismic scale (Grünthal, 1998). An example of vulnerability curves obtained in this study is given in Figure 2.3.

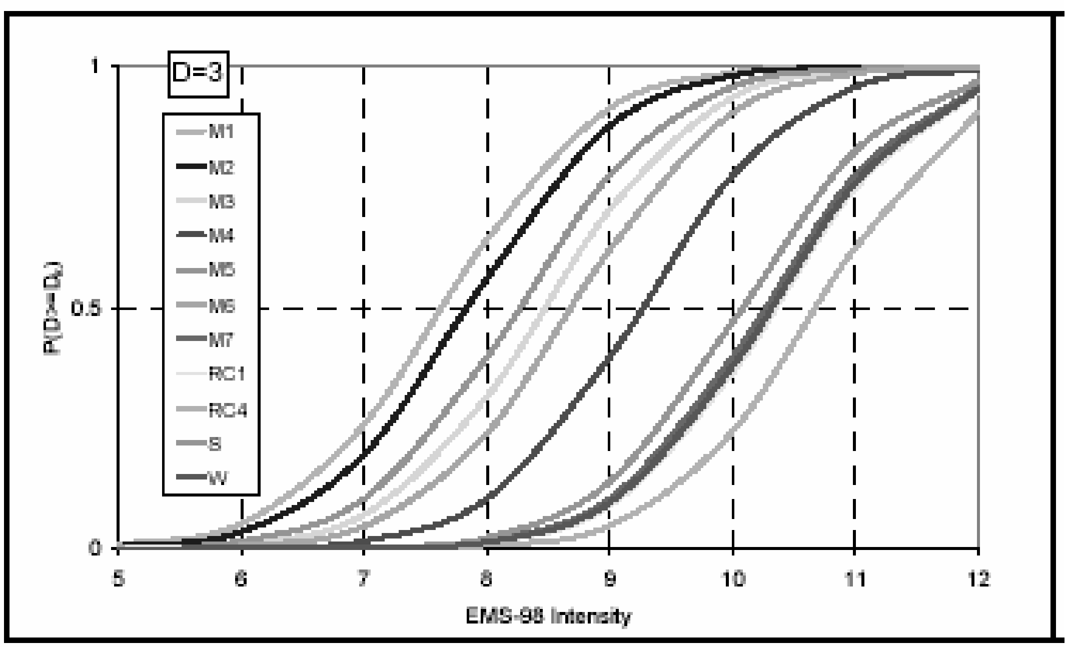

Figure 2.3. Vulnerability curves of RC Buildings for damage grade D3 (Giovinazzi and Lagomarsino, 2004)

The EMS-98 scale defines qualitative descriptions of "Few", "Many" and "Most" for five damage grades for the levels of intensity ranging from V to XII for six different classes of decreasing vulnerability. Damage matrices containing a qualitative description of the proportion of buildings that belongs to each damage grade for various levels of intensity.

2.5. Analytical Vulnerability Relationships

One of the methodologies in vulnerability assessment is the determination of the response parameters by performing analytical simulation of the structural data. This method is used when the structural data is limited or the engineering demand parameter that is used in vulnerability relationship is obtained by an analytical simulation.

Although vulnerability curves and damage probability matrices have traditionally been derived using observed damage data, recent proposals are based on the use of computational analyses to overcome some of the drawbacks of the methods. In Figure 2.4 below the basic components of the analytical calculation of vulnerability curves or damage probability matrices are summarized.

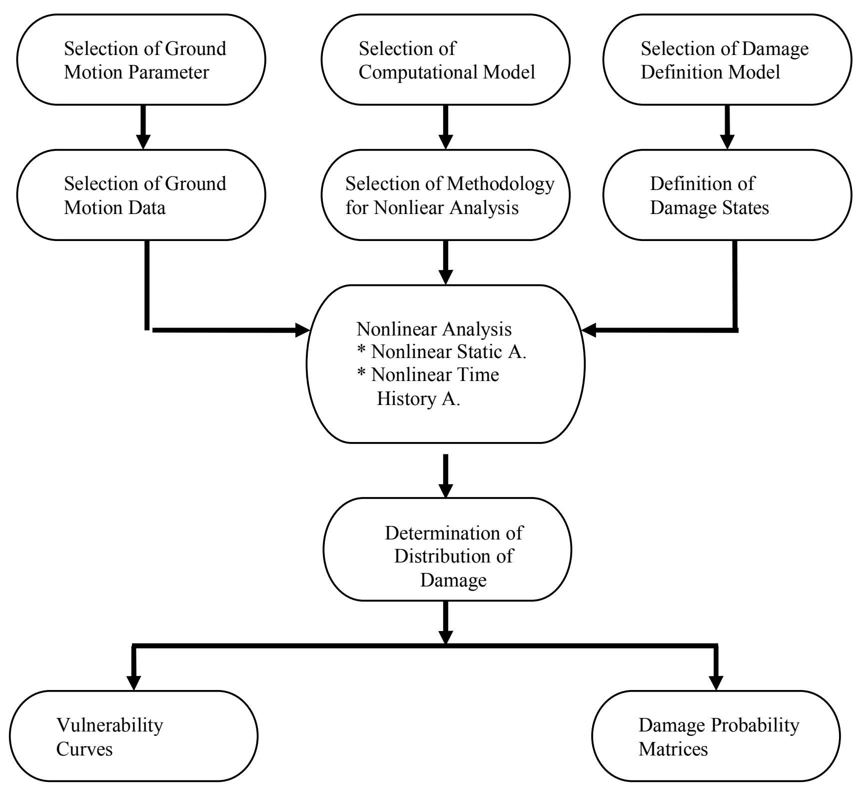

Figure 2.4. Flowchart for calculation of analytical vulnerability assessment

2.5.1. Analytical Methods Based on Nonlinear Static Analysis

By the recent developments in nonlinear analysis methods, nonlinear static procedure or so-called pushover analysis has been introduced as an analysis tool for vulnerability assessment. As the research in structural engineering increased, the limitations of the method, such as to be applicable on low and mid-rise buildings with first mode dominant response, have been overcome. More advanced methods of nonlinear static procedures are being used in vulnerability analysis for better representation of structural response.

Analytical vulnerability curves have frequently been used to support the empirical Damage Probability Matrices and vulnerability curves based on the observational damage data. But it must be pointed out that derivation of analytical vulnerability curves based on nonlinear static analysis may be computationally intensive and time consuming. Since the developed vulnerability curves reflect the characteristics of a certain structural type with certain seismic resistance characteristics, they can not be used for different areas or countries with diverse construction characteristics. The procedure must be performed for each area and structural type.

The analytical approach based on nonlinear static analysis has been used by Rosetto and Elnashai (2005) in which an analytical methodology for derivation of displacement-based vulnerability curves has been proposed representing an optimum solution compromising between reliability and computational efficiency. In the study under consideration an adaptive pushover analysis is employed within a capacity spectrum framework to assess the performance of a population of building models for increasing ground motion intensity. A new homogeneous reinforced concrete damage scale which is calibrated to maximum inter-story drift ratio for different structural systems is used to determine the damage state of the building at the performance point. The results of the assessments are used to construct response surfaces from which the damage statistics forming the basis of the vulnerability curves are generated through re-sampling.

In another analytical method based on nonlinear static analysis, Erberik and Elnashai (2004) focused on the derivation of fragility curves of medium-rise flat-slab buildings with masonry infill walls as a structural type that has not been evaluated. The limits states are used in the study are defined in terms of inter-story drift ratio and in order to determine the performance levels. The results of the analysis are evaluated by Latin Hypercube Sampling (LHS) method instead of Monte Carlo simulation since the amount of data was not as much as to be used in the latter method. Response statistics were assessed in terms of inter-story drift for determination of vulnerability function parameters and seismic intensity parameters were selected as spectral acceleration, $S_a(T)$, and spectral displacement, $S_d(T)$.

The last and the most well known methodology in this class is the HAZUS (1999) methodology in which the vulnerability relationships are obtained through analytical study in the capacity spectrum framework. The method can be classified in this section although the capacity of the buildings is determined by expert judgment, the damage state threshold values and their distribution in terms of drift ratio are determined by analytical studies. All the parameters regarding to estimation of capacity of the building classes, determination of demand and vulnerability curves are determined by the group of experts in the field of earthquake engineering. On the other hand values of damage state ratios are based on analytical studies of several researchers.

The vulnerability assessment component of the procedure is based on the Capacity Spectrum Method defined in ATC-40 (1996). The flow chart of the methodology is given in the (Figure 2.5) below.

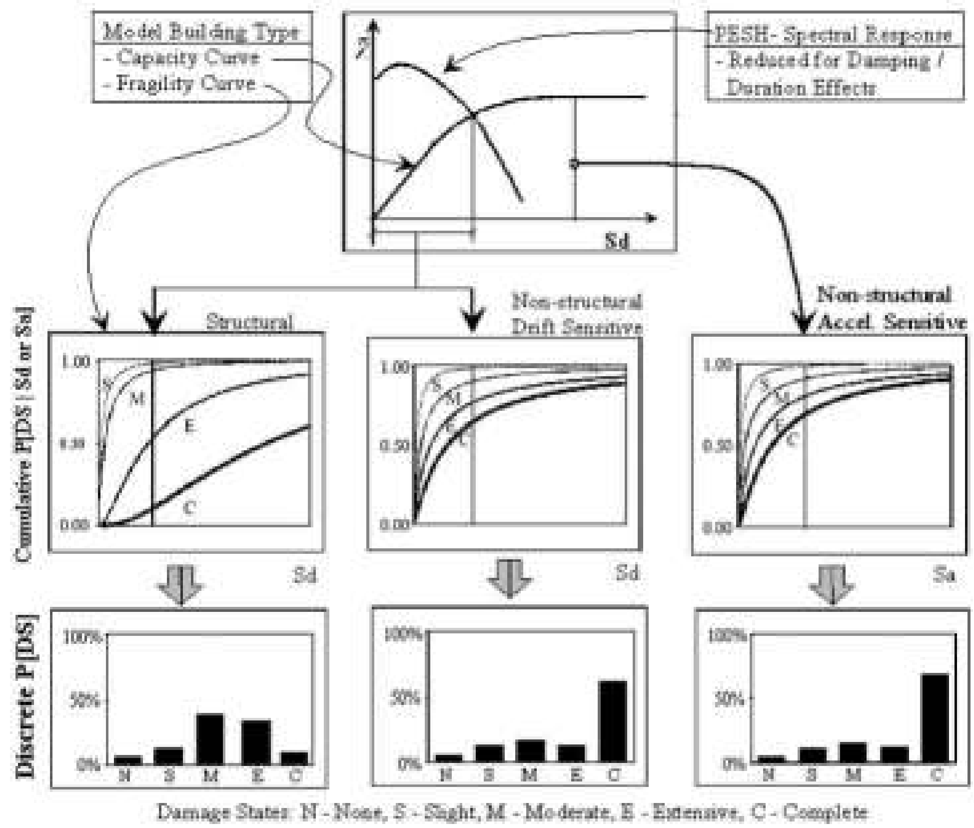

Figure 2.5. Illustration of the estimation of damage from ground shaking in HAZUS (FEMA, 1999)

A potential weakness of the method is that the capacity curves and vulnerability functions published in the HAZUS manual have been derived for buildings in the US having a limited range of storey heights; thus the application of this method to other parts of the world requires additional research to be carried out. Hence, capacity curves and vulnerability functions would need to be derived for the building stock under consideration. A faithful representation of the real structural behavior requires a great deal of information about the structure, including reinforcement details.

HAZUS has been adopted all over the world for the loss assessment of urban areas. The methodology in itself has not been adapted in any way, but the capacity curves and fragility functions have been calibrated to the building stock under consideration. Examples include the loss assessment of Turkey carried out by Bommer et al. (2002), the seismic risk assessment of Oslo documented by Molina and Lindholm (2005), the loss estimation of Taiwan (using a modified version of HAZUS called Haz-Taiwan) as discussed in Yeh *et al.* (2000).

2.5.2. Analytical Methods Based on Nonlinear Response History Analysis

The other analysis method used for vulnerability assessment is the time history analysis, which is widely used as the number of recorded ground motions and the computer capacities are increased. The major components of the method are:

1) characterization of the ground motion
2) characterization of the structural properties and
3) quantification of structural damage

Since all the components interacts each other, analysis results should be obtained in such a way that all variability of the components should be accounted properly. One of the basic advantages of this method is the opportunity to take into account the uncertainty of the characteristic of the grounds motion parameters such as peak ground acceleration, spectral acceleration, duration effect, site conditions, etc.

On the other hand, through non linear time history analysis the cyclic response of the elements can be taken into account in the estimation of the damage levels. Thus almost all the possible damage levels under a seismic action could be monitored and estimated via this method. But it must be indicated that as in the case of nonlinear static procedure nonlinear time history analysis is also time consuming and needs much computational effort.

One of the major studies of analytical approach performed by nonlinear response history analysis is the one done by Singhal and Kiremidjian (1996) in which the earthquake ground motion–damage relationships are presented in the form of probability distributions of damage at specified ground motion intensities. Since the study under consideration constitutes the basics of the procedure explained in Chapter 4 of this study, it will be dealt in detail in comparison to other studies cited above.

A systematic approach which does not depend on heuristic or on empirical data has been proposed in order to develop a damage–ground motion relationship. The probability of damage is estimated by quantifying the response of as structure subjected to a significant ensemble of ground motion with a wide range of parameter variations. The proposed method is used to develop fragility curves for the there categories of reinforced concrete frames grouped as low-rise with 1-3 stories, mid-rise with 4-7 stories and high-rise with 8 stories or more. The example frame structures have five bays in the longitudinal direction and one bay in the transverse direction. The sample building for each class of concrete frames is designed according to the 1990 SEAOC recommendations for special moment resisting frames. The compressive strength of concrete and the yield strength of steel are the only parameters treated as the strength random variables by considering a mean value and a standard deviation for both material strengths.

The ground motion modeling is performed through simulation technique called ARMA model which is useful for simulating ground motions for large scale structures where spatial variation of ground motion is important. The simulation technique is also capable of capturing non-stationarity in frequency content of the ground motion. The seismic intensity parameter used in the methodology is characterized by spectral acceleration, S_a, over period bands that corresponds to three frame classes under consideration.

The damage state of the structural members is defined in terms of Park and Ang Damage Index (1985-a). The so called index is used since it is simple and has been calibrated using data from various structures damaged during past earthquakes and laboratory tests. Park and Ang's (1985-b) global damage index is used to represent the performance of structural systems. It is defined as a weighted average of the local damage indices of each element.

The statistics of the Park and Ang (1985-a) damage index obtained at each acceleration value are used to obtain the parameters of lognormal probability distribution function at that ground motion level through Monte-Carlo simulation. From the simulations, the means, variances, and distribution functions of the output random variable, the quantitative measure of damage in this case, are estimated for an ensemble of time histories corresponding to a given level of ground motion. The probabilities of different damage states are evaluated from the probability distributions of the damage measure. As the last step, a log-normal distribution has been fitted to the distribution under consideration in order to obtain the fragility curves

The steps of the procedure that are used in the development of both fragility curves and damage probability matrices in this methodology is described in Figure 2.6.

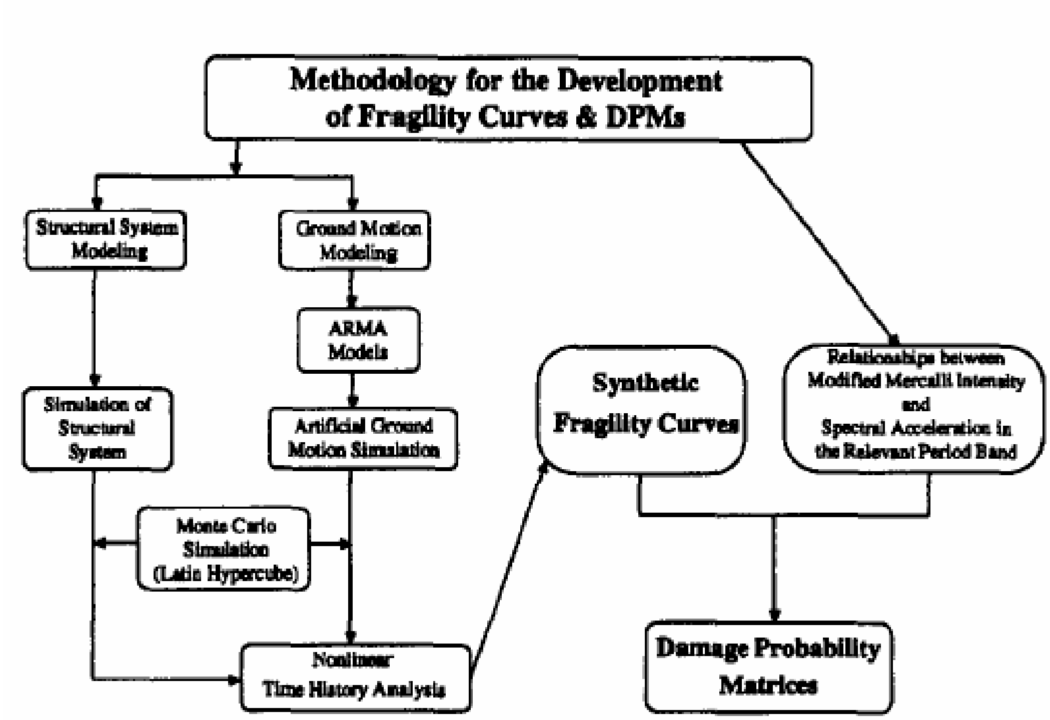

Figure 2.6. Steps of the methodology for construction of fragility curved and DMP (Singhal and Kiremidjian, 1996)

In a more recent analytical methodology, Jeong and Elnashai (2007) proposed an approach where a set of fragility relationships with known reliability is derived based on the fundamental response quantities such as stiffness, strength and ductility. An exact solution for a generalized single degree of freedom system is developed and employed to construct a response database of coefficients describing commonly used log-normal fragility relationships. Once the fundamental response quantities of a wide range of structural system are defined, the fragility relationships for various limit states can be constructed without recourse to further simulation.

In addition to the afore mentioned studies vulnerability assessment studies have been also performed by Turkish researchers in one of which Ay et al.(2006) performed a vulnerability analysis of low and mid-rise reinforced concrete RC structures. A set of ground motion is used in non linear response history analysis in which seismic intensity parameter is selected as peak ground velocity (PGV). The result of the time history analyses are obtained in terms of maximum inter-story drift ratio (MIDR). PGV and MIDR are used to get hazard versus demand relationship. As the result of the study they have concluded that the damage state probabilities reflect the inherent characteristics of the considered building class and the typical characteristics of Turkish low–rise and mid–rise RC frame buildings.

As another example study from Turkey, Akkar et al.(2005) obtained vulnerability relationships by analytical approach where building capacities are obtained from field data and their dynamic responses are calculated by time history analyses. Field data consist of 32 sample buildings representing the general characteristics of two to five stories substandard reinforced concrete buildings in Turkey. Lateral stiffness, strength, and deformation capacities of the sample buildings are determined by pushover analysis conducted in two principle directions. The inelastic dynamic structural characteristics of the buildings investigated are represented by a family of equivalent single degree of freedom systems and their seismic deformation demands are calculated under 82 ground motion records. Peak ground velocity is selected as the measure of the seismic intensity since maximum inelastic displacements are better correlated by PGV than peak ground acceleration, PGA.

The global drifts are used here to represent the damage limit states of the buildings because none of the buildings in the data set showed a soft story mechanism. The global yield drift ratio "θ_y" represents significant yielding of the system when the base shear is attained, whereas the ultimate global drift ratio "θ_u" corresponds to the state at which the building reaches its deformation capacity. The base shear coefficient "$\eta=V_y/W$" is the ratio of yield base shear capacity to the building weight. All of these parameters are presented in (Figure 2.7).

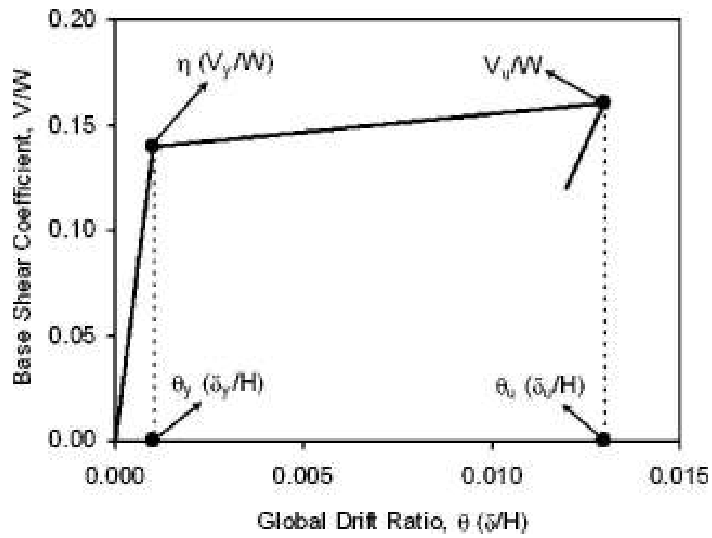

Figure 2.7. A typical bilinear capacity curve, (Akkar et el.,2005)

The statistical properties of the three basic parameters, "η", "θ_y" and "θ_u" are determined for each building class and their representative probability density functions are determined. A set of strong ground motion records were used to compute the empirical building fragility curves based on the building information given above. As the result of the study they have obtained the vulnerability curves for each type of the buildings and concluded that fragility functions taking into consideration the basic regional characteristics of an investigated building stock, based on field data, serve for reliable estimates of expected loses in similar buildings from strong ground shaking.

2.5.3. Analytical Methods Based on Displacement Method

As one of the recent methodologies based on displacement based approach, Crowley et al. (2004) proposed a simple method that defines the capacity of a building class using a relationship between its deformation potential and fundamental period of vibration at different limit states and compares this with a displacement response spectrum. The uncertainty in the geometrical, material and limit state properties of a building class is considered and the first-order reliability method, FORM, is used to produce an approximate joint probability density function (JPDF) of displacement capacity and the period.

The proposed methodology by Crowley et al.(2004) uses mechanically-derived, as well as empirically proposed, formulae that describe the displacement capacity of the classes of buildings at three different limit states. These equations are given in terms of material and geometric properties, including the average height of the building in the class. The original concept is given in Figure 2.8 below, where the range of periods with displacement capacity below the displacement demand is obtained and transformed into range of heights using the aforementioned relationship between limit state period and height.

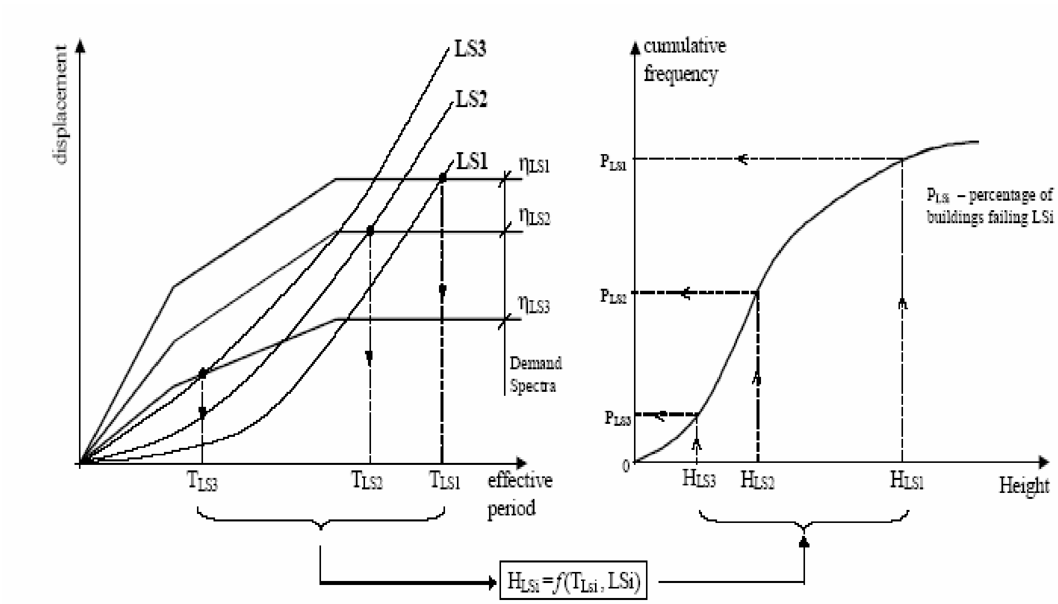

Figure 2.8. A deformation-based seismic vulnerability assessment procedure, (Crowley et. al,2004)

In the proposed methodology the buildings are classified as a group of buildings which share the same construction material, failure mechanism and number of story. The building classes considered within this methodology comprise any number of stories of the following;

1. Reinforced concrete beam-sway moment resisting frames
2. Reinforced concrete column-sway moment resisting frames
3. Reinforced concrete structural wall buildings
4. Un-reinforced masonry buildings exhibiting an out-of-plane failure mechanism
5. Un-reinforced masonry building exhibiting an in-plane failure mechanism

Damage definition is classified into four discrete bands of structural damage states as: none to slight, moderate, extensive or complete. A qualitative description of each damage band for reinforced concrete frames are given in Table 2.1 below.

Table 2.1. Damage level definitions and threshold values, (Crowley *et. al*,2004)

Structural Damage Band	Description
None to slight	Linear elastic response, flexural or shear type hairline cracks (<1.0 mm) in some members, no yielding in any critical section; hence limit state to damage band is structural yield point
Moderate	Member flexural strengths achieved, limited ductility developed, crack widths reach 1.0 mm, initiation of concrete spalling, limits to strains may be assumed as: ε_c = 0.004–0.005 ε_s = 0.010–0.015
Extensive	Significant repair required to building, wide flexural or shear cracks, buckling of longitudinal reinforcement may occur, limits to strains may be assumed as Inadequately confined members: ε_c = 0.005–0.010 ε_s = 0.015–0.030 Adequately confined members: ε_c = 0.010–0.020 ε_s = 0.040–0.060
Complete	Repair of building not feasible either physically or economically, demolition after earthquake required, could be due to shear failure of vertical elements or excess displacement -

The determination of the displacement demand is done by displacement response spectrum as the seismic input for the building class under consideration. Vulnerability curves for each limit state may easily be generated with the method outlined in this methodology once the JPDF of limit state displacement capacity and period has been calculated, as has been illustrated in Figure 2.9 below. The probability of failing the limit state is simply found from the volume of the JPDF below a given level of displacement response, S_d.

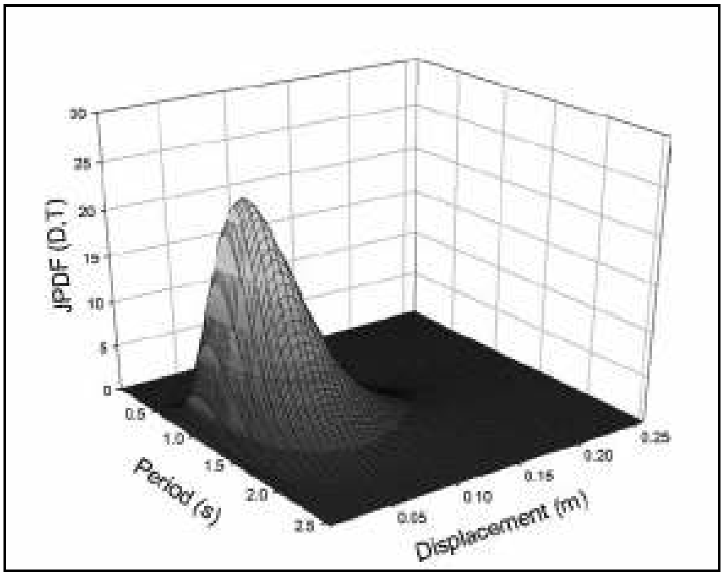

Figure 2.9. The JPDF of capacity for a four storey column-sway RC building class, (Crowley *et. al*,2004)

The computational efficiency of the methodology will also permit the performance of systematic sensitivity studies to establish the relative influence of each input parameter on the results of an earthquake loss model.

2.6. Hybrid Vulnerability Relationships

In some cases the data available may not be enough and the use of empirical relationships may lead misleading results for the construction of vulnerability relationships. In the hybrid approach vulnerability relationship is constructed by using empirical approach depending on the observed damage after an earthquake and analytical approach is used in representing the missing damage survey data.

Before the introduction of hybrid approach in the vulnerability assessment applications, Damage Probability Matrices based on intensity were available for the assessment of seismic risk on a large scale. The use of observed damage data to predict the future effects of earthquakes also has the advantage that when the damage probability matrices are applied to regions with similar characteristics, a realistic indication of the expected damage should result and many uncertainties are inherently accounted for.

As an example for this approach Kappos et al.(2004) proposed a methodology for the derivation of seismic vulnerability curves based on hybrid approach which combines statistical data with appropriately post processed nonlinear dynamic analyses that permit extrapolation of statistical data to earthquake intensities for which no data are available. The proposed so-called “hybrid” approach starts from available damage statistics and estimates of damage at the intensities for which no data are available using analytical nonlinear simulation. In the procedure of construction of fragility curves a certain empirical relationship has been used between intensity (*I*) and peak ground acceleration (*PGA*). As the result of the “hybrid” procedure which combines the statistical data with appropriately processed results of nonlinear analysis, vulnerability parameters of the each building class has been determined.

In another hybrid method application as a case study, Kappos *et al*.(2002) proposed and applied a “hybrid” vulnerability assessment methodology for the Volos metropolitan area in Greece, in order to determine the seismic risk and economic loss of the region. The hybrid methodology used has both empirical and analytical approaches. This approach to seismic vulnerability assessment has been developed in recognition of the fact that reliable statistical data for seismic damage are quite limited and typically correspond to a very small number of intensities. The basic steps of the hybrid methodology for deriving damage probability matrices (DPMs) based on statistical and time history analysis of appropriate models are summarized as;

1. Construct the columns of each DPM for which statistical data from past earthquakes are available, using standard empirical procedure.
2. Construct the remaining parts of DPM on the basis of the results of the inelastic time history analysis of models simulating as closely as practicable the behavior of each building class.
3. For intensities greater than 10, judgment should be used to construct the corresponding columns of the DPM since the empirical data is extremely scarce and analytical data tends to be unreliable.

2.7. Evaluation of the Existing Vulnerability Assessment Methodologies

As stated above both empirical and analytical methods have their own advantages and disadvantages in vulnerability assessment procedure. Depending on the data available at hand one must decide the best methodology to use for reliable results. The methodology must consider the properties of the building stock both in capacity and demand aspects. The seismic input must reflect the hazard at the site under consideration. All the resources of uncertainties such as ground motion definition, building inventory, capacity and demand estimation must be explicitly accounted for as much as possible. Only by considering the above mention features an optimal and ideal vulnerability assessment can be achieved. Unfortunately it is unlike that a single methodology which satisfies these requirements. For example many analytical and mechanical models, though theoretically superior, require a large amount of detailed data, but the benefit of collecting such data is often not proven through validation of the methodology with empirical methods based on the observed damage data. On the other hand, the derivation of vulnerability curves from the observed data does not always consider the all the characteristics of the building stock. Furthermore, many methodologies do not explicitly model the various sources of uncertainty; this can be a problem when, for example, the uncertainty in the seismic demand needs to be removed from the vulnerability assessment calculations.

As a result of the evaluation, it appears that the ideal approach for the future needs to be a combination of the positive aspects of different vulnerability assessment methodologies. In other words, reliable vulnerability assessment of a given region is likely to need for the employment of at least two different approaches, which should complement or verify each other.

3. STANDARD DERIVATION OF ANALYTICAL VULNERABILITY FUNCTIONS

3.1. Introduction

Since this study is focused on derivation of vulnerability functions through analytical analysis, the components of the process are explained in general concept. Each component is evaluated by consideration of its effect on the vulnerability analysis and uncertainty associated with it. In some cases in order to clarify the issue some examples considering the available applications are given.

In general the components of derivation of analytical vulnerability functions can be listed as;

- Sampling and generation of building structural data
- Sampling and generation of earthquake ground motion data
- Identification of representative seismic intensity parameter
- Identification of representative seismic demand parameter
- Quantification of damage states in terms of representative seismic demand parameter
- Estimation of variation of representative seismic demand parameter with respect to representative seismic intensity parameter
- Statistics of damage state thresholds with respect to representative seismic intensity parameter
- Evaluation of vulnerability functions

3.2. Characterization of Building Structural Data

The analytical derivation of the vulnerability functions is based on numerical simulation of the structural response under seismic action. The building inventory data used in the analysis is of great importance on the analysis results. In case of reinforced concrete structures, structural response is basically depends on the geometrical configuration and material strength properties, both of which have significant effect on the seismic response. In general the distribution of geometrical configuration is neglected and representative examples for each class of buildings are considered in the analysis. Structural data generation is mainly based on consideration of uncertainty in material strength by assuming a mean and a standard deviation for concrete and steel strength. The building structural data is generally determined through consideration of representative examples with material properties having a certain distribution.

In addition to the geometrical and material variability, the building structural data is classified into other groups with common properties. These properties are determined by considering other features of buildings that have effect on seismic response. These properties can be listed as:

- structural system (frame, infilled frame, shear wall, dual, etc)
- number of story (or considering height as low, mid, high-rise)
- construction year (considering compliance of code provisions as low-code, high-code, etc.)

As an example, the building structural data which are determined by representative reinforced concrete frame systems for low, mid and high-rise buildings and material strength properties are assumed to have certain mean values in the study of Singhal and Kiremidjian (1996). On the other hand Rosetto and Elnashai (2005) have also designed a sample building and generate the building structural data through simulation of material strengths. Akkar *et al*. (2005) have used geometrical properties of real buildings while material strength has been determined as in the studies mentioned above.

3.3. Characterization of Seismic Input

In the analytical derivation of vulnerability functions the seismic input can be in the form of a response spectrum or acceleration time history. The response spectrum to be used in the analysis can be a codified one or it can be determined through a seismic hazard analysis which reflects the seismicity and local site conditions of a certain region.

In case of using acceleration time history in the analysis, ground motion can be determined basically two ways. Firstly, ground motion data base can be determined by using the recorded motions which are obtained by the help of strong ground motion network established in seismically active regions. But it must be kept in mind that ground motions selected from the ground motion catalogues should have similar properties in terms of faulting type, fault distance and site conditions in comparison to the region under consideration. Additionally the ground motion data should cover a wide range of intensity parameter for the homogenous distribution for building damage.

Although there are a huge number of recorded ground motions, it may not be possible to find a set of ground motion having all the properties match with properties of the region. In this case another technique is used as simulation of ground motion in order to have sufficient number of ground motions with the desired properties. The simulation is performed by using different methods such as stochastic method, ARMA method or methods to obtain spectrum compatible time histories.

The seismic input is determined by simulation of ground motion concerning the seismic properties of the region through ARMA model by Singhal and Kiremidjian (1996) to be used in nonlinear response history analysis in their study. On the other hand Erberik and Elashai (2004) used spectrum compatible time histories in their study in order to estimate the demand distribution. The site specific response spectrum is used by Rossetto and Elnashai (2005) for demand estimation in the framework of capacity spectrum method.

3.4. Identification of Representative Seismic Intensity Parameter

The identification of intensity parameter to be used in vulnerability analysis is directly related to the analysis tool used in demand estimation of the building data. Additionally, the demand parameter used in the analysis should be proper for identification of building response. There must be a satisfactory relationship between the intensity parameter and damage state of the building stock under consideration. The intensity parameters used commonly in analytical studies are peak values of the ground motion, namely peak ground acceleration, PGA, and peak ground velocity, PGV. Recent studies and lessons learned from past earthquakes revealed that PGA is usually insufficient in explaining the spatial damage distribution during a severe earthquake. Furthermore, the acceleration does not reflect the duration of ground motion which is directly related to the accumulation of damage in structures. So the correlation of PGA as an intensity parameter is not so strong with the damage of the structure. In some studies PGV is used as an intensity parameter since PGV could better describe deformation demands beyond the elastic range. PGV and PGA do not necessarily occur during the same ground vibration cycle. PGV primarily influences the seismic spectral response of medium period systems.

3.5. Identification of Representative Seismic Demand Parameter

The seismic demand parameter to be used in analytical vulnerability assessment is strongly related to building type and analysis method used in the process. The demand parameter should be determined through the analysis. Additionally distribution of the results should cover all the possible damage levels in terms of selected parameter for the structures under consideration.

The widely used seismic demand parameters are inter-story drift ratio, building drift, ductility ratio, final softening index, since they are easy to determine through analysis and their values have a meaning for correlation to level of damage in engineering point of view. On the other hand the seismic demand parameter can be selected as the damage measures developed in recent years in different aspects. The damage measures are basically classified in two groups as deformation based and energy based. But these damage measures have a common deficiency as they do not reflect the complex cyclic response of the reinforced concrete members. In order to overcome the problem, combined indices which are the combination of deformation and cyclic energy have been introduced by different researchers. In the case of nonlinear response history analysis where the cyclic response of the structural members have significant effect on the overall damage level of the structure, combined damage indices should be used. Additionally the combined damage indices determined at member level should be converted to over all damage indices. Generally this conversion is done by using weighting functions based on cyclic energy absorptions of each member.

As a general evaluation of the global damage indices, it can be said that the use of weighted averages of local damage indices to assess the global damage level of a structure has two limitations. Firstly, the global damage index can only be as reliable as the local values from which it is derived and as the second limitation, there is no obvious way of determining the weighting that should be given to different structural elements or different levels of damage. On the other hand, the softening damage indices provide a promising method of assessing a global state of a structure, but they provide no identification of the distribution of damage.

The overall review of the development of seismic damage indices for concrete structures suggests that all of the existing approaches have some limitations. Nevertheless, with the aid of some further development and validation studies, damage indices have potential to become valuable tools in retrofit decision-making and structural assessment.

But it can be said that the importance of cyclic loading effects on the damage induced in concrete structures is widely recognized. The most promising local damage indices are therefore those with a cumulative form, in which repeated cyclic at a fixed amplitude causes an increase in damage. There has been a concentration on bending behavior in the formulation and validation of the indices, so that their ability to represent shear damage remains uncertain.

On the other hand, the global damage indices formulated directly from local indices require the use of an appropriate combination method. Existing approaches generally consist of a pre-determined, weighted average of the local indices, but it could be argued that a more flexible, application-specific approach would be more appropriate. Global softening indices have ability to describe the overall damage state of the structure, but give very little information on damage distribution.

Singhal and Kiremidjian (1996) have used Park and Ang damage index (1985-a) for the seismic demand parameter as the result of the nonlinear response historyanalysis. Akkar et al. (2005) used building drift as the seismic demand parameter in their analytical vulnerability assessment study. Erberik and Elnashai (2004) used maximum inter-story drift ratio as the seismic demand parameter in vulnerability assessment of flat-slab structures. As a last example Ay *et al*. (2006) used softening index as the result of nonlinear static analysis in their study.

3.6. Quantification of Damage in Vulnerability Analysis

One of the challenging issues in the analytical vulnerability analysis of existing buildings is the definition and quantification of damage since it has a significant effect on the obtained results. The selected damage parameter is strongly related to the type of the building under consideration, property of the data available, method used and whether the assessment is done before or after the earthquake.

The construction of the relationship between damage level definitions and damage level thresholds is evaluated by many researchers in different aspects. Damage level definitions are related to seismic demand parameters such as interstory drift, building drift by means of post earthquake damage surveys or through analytical process. As an example Ghobarah (2004) has proposed limit values for determination of building damage state in terms of drift values for different structural types such as moment resisting ductile and non-ductile frame systems and dual systems. Depending on a comprehensive volume of experimental research and post earthquake observation on the behavior of systems under consideration damage levels in terms of inter-story drift ratio is given in Table 3.1.

Table 3.1. Drift ratio limits associated with various damage levels,(Ghobarah,2004)

State of Damage	Ductile MRF	Non-ductile MRF	MRF with infills	Ductile walls	Squat walls
No damage	<0.20	<0.10	<0.10	<0.20	<010
Repairable Damage					
(a) Light	0.40	0.20	0.20	0.40	0.20
(b) Moderate	<0.10	<0.50	<0.40	<0.80	<0.40
Irreparable Damage (>yield)	>1.00	>0.50	>0.40	>0.80	>0.40
Severe damage- Life Safety-Partial Collapse	1.80	0.80	0.70	1.5	0.70
Collapse	>3.00	>1.00	>0.80	>2.50	>0.80

In another study of Ghobarah *et al*. (1997) performed nonlinear time history analysis to frame structures in order to assess the level of damage. As the result of the analyses element hinges and overall stability of the structures are examined. During the process Ghobarah used a damage index which is first defined by Park and Ang (1985-a). From the results of analyses five levels of performance (damage) for ductile moment resisting frame are defined using the following drift limits given in Table 3.2.

Table 3.2. Damage Levels and corresponding Drift ratio Limits, (Ghobarah,1997)

Damage (Performance) Level	**Drift Ratio**
Elastic Limit	0.7%
Minor Damage Limit	2.0%
Repair Limit	4.0%-4.5%
Collapse Prevention	5.0%-5.5% (Damage Index value of 0.6)
Collapse Limit	Damage Index value grater than 1

In the study of Rossetto and Elnashai (2003) in which a new damage scale namely "Homogenized Reinforced Concrete Damage Scale" (HRC) has been introduced based on the post earthquake damage survey of a large database. HRC is subdivided into 7 "Damage States" and damage states are further defined in terms of "Damage Index (DI_{HRC})" as given in Table 3.3.below.

Table 3.3. Damage states defined in terms of HRC by Rossetto and Elnashai (2003)

DI_{HRC}	**Damage State**
0-10	None
10-20	Slight
20-40	Light
40-70	Moderate
70-90	Extensive
90-100	Partial Collapse
>100	Collapse

The distribution of DI_{HRC}- ISD_{max} pairs is used in nonlinear regression analysis in order to determine a relationship between DI_{HRC} and inter-story drift ratio. Depending on the obtained relationships the threshold values of $ISD_{max\%}$ defining the HRC-scale damage limit states for general RC structures (all), on-ductile MRF, infilled frames and shear-wall structures are listed in Table 3.4 below.

Table 3.4. $ISD_{max\%}$ limits for HRC scale by Rossetto and Elnashai (2003)

HRC damage state	All	Non-ductile MRF	Infilled MRF	Shear walls
None	0.00	0.00	0.00	0.00
Slight	0.13	0.32	0.05	0.26
Light	0.19	0.43	0.08	0.34
Moderate	0.56	1.02	0.30	0.72
Extensive	1.63	2.41	1.15	1.54
Partial Collapse	3.34	4.27	2.80	2.56
Collapse	>4.78	>5.68	>4.36	>3.31

In another study performed by Singhal and Kiremidjian (1996) a damage assessment methodology for the reinforced concrete structures has been introduced. In order to define the damage levels for the members local damage index defined by Park and Ang been used and the global damage index has been determined by using the weighted average of the local damage indices in which the weighting function is proportional with the energy dissipated in each element. The range of the Park and Ang damage index for different damage states have been established to reflect damage to the reinforced concrete structure is given in Table 3.5 below.

Table 3.5. Damage index thresholds and corresponding damage states (Singhal and Kiremidjian, 1996)

Damage State	Park and Ang Damage Index
None	0.0-0.10
Minor D.	0.10-0.20
Moderate D.	0.20-0.50
Extensive D.	0.50-1.0
Collapse	>1.0

As an example of damage level threshold values defined for Turkish type of RC buildings, in the study performed by Akkar *et al*. (2005) displacement vulnerability functions for low and mid rise reinforced concrete structures have been determined by using Turkish buildings and damage levels are defined in terms of global drift ratio. The median threshold values for each performance levels and associated damage limits in terms of global drift for each building class is given in Table 3.6 below.

Table 3.6. Performance levels and corresponding global drift thresholds for each story, (Akkar *et al*., 2005)

Number of Story	**Immediate Occupancy (Light Damage)**	**Life Safety (Moderate Damage)**	**Collapse Prevention (Severe Damage)**
2	0.0011	0.0085	0.0160
3	0.0011	0.0080	0.0150
4	0.0012	0.0081	0.0150
5	0.0011	0.0065	0.0120

As it can be deduced from the tables above, the damage level quantification and determination of threshold values for each damage level is strictly depended on the structural type and analysis methodology. But one thing is certain that in case of reinforced concrete structures the damage definition should be defined in terms of damage measures in which cyclic response is taken into account. One of the promising points is that as the data concerning the post earthquake damage survey and laboratory test are increasing better correlation can be achieved between damage levels and damage measures.

3.7. Estimation of Damage Distribution in Vulnerability Analysis

Estimation of variation of representative seismic demand parameter with respect to representative seismic intensity parameter in analytical vulnerability analysis can be done through the analysis method used in the process. As mention in the previous sections the analysis methods can be nonlinear static or nonlinear time history analysis. The building data used in the vulnerability assessment is analyzed through one of the analysis methods and the distribution of damage measures is determined with respect to seismic intensity parameter. Both the demand and intensity parameter are closely related to the type of the structure and method of analysis.

But it must be kept in mind that the distribution of the damage measures should cover all the possible damage states. In case of both nonlinear static and dynamic analysis this can be achieved by selecting a proper seismic demand parameter that could be determined for the whole range of damage levels and the ground motions used in the response history analysis should have wide range of intensity parameter used in the vulnerability analysis. But in some cases the results of the damage distribution could not be so homogenous that each damage level has similar amount of data. In order to overcome this problem, depending on the amount of the data, Monte Carlo simulation or Latin Hypercube Sampling methods are used to homogenize the results.

Another approach can be increasing the seismic intensity parameter in a regular way to have homogeneous damage distribution in the results. This can be done in case of non linear response history analysis where a monotonically scaled ground motions are used in vulnerability analysis. As the result of the time history analysis performed by scaled motions the damage distribution would be more homogeneous in comparison to the above mention applications. A sample of this approach has been applied in this study in the framework of Incremental Dynamic Analysis (IDA) proposed by Vamvatsikos and Cornell (2002) details of which is given in Section 4.5.

3.8. Statistical Evaluation of Damage Distribution in Vulnerability Analysis

The distribution of damage state threshold with respect to representative seismic intensity parameter is used in statistical process in which vulnerability parameters are determined. In general statistical process is done under the assumption that the damage distribution is log-normally distributed. And a log-normal probability function is fitted to the analysis results after determination of mean value and standard deviation of the lognormal distribution.

There are two ways of verification of the assumption done in the statistical process. The first one is calculation of probability of reaching or exceeding a damage level defined by a damage measure threshold for each value in the damage distribution. And by plotting the analytically derived vulnerability curve and the discrete solutions on the same graph will provide to evaluate the assumption to be valid or not. On the other hand the same result can be obtained using some methods available in statistics such as “The Kolmogorov-Smirnov test” or “Chi-square Goodness of Fit Test”. The verification of log-normal distribution of the results is also done both ways in the procedure of this study details of which is given in Section 4.7.

4. A NONLINEAR VULNERABILITY ANALYSIS PROCEDURE USED IN URBAN LOSS ESTIMATION

4.1. Introduction

In this section, steps of the analytical procedure used for the vulnerability assessment of existing RC structures are summarized and brief explanations are given for each step as shown in the flow chart is given in Figure 4.1. In general a vulnerability assessment procedure based on an analytical approach can be mainly divided into four steps;

1. Selection of seismic input parameters and ground motions
2. Preparation of structural data
3. Performing structural analysis
4. Statistical process of analyses results.

In the first step of the methodology the type of the seismic input should be determined. It can be in the form of an acceleration response spectrum or a ground motion data set. In the case of latter case ground motion data set can either be selected from strong ground motion catalogues, or obtained by simulation techniques. The characteristics of the ground motion data in terms of frequency content, effective duration and peak values should be suitable for a response history analysis to cover all possible building responses. On the other hand sufficient number of ground motions should be selected to consider the ground motion uncertainty as much as possible.

The selection, classification and preparation of the building data is the second step of the methodology. The building data may be classified depending on their structural system such as moment resisting frame systems, shear-wall systems and dual systems together with their number of stories. A further classification can be made based on the construction date of buildings to indicate the code requirements applied.

In the third step the analysis method is selected, which is highly dependent upon the estimation of demand parameter to be used in vulnerability assessment. In addition to the selection of proper analysis method, mathematical models of the structures should be constructed in such a way that nonlinear behavior of the structure under seismic action could be properly simulated so that the analysis method should capture the nonlinear response of the structures properly.

As the last step of the procedure, the distribution of the demand parameters for each damage state with respect to pre-defined damage threshold values should be evaluated in a statistical process in order to estimate the parameters of vulnerability curves. It should be emphasized that the assumptions made for the distribution of results must be checked by statistical rules in order to avoid misleading results in vulnerability curves.

In the procedure used for vulnerability assessment in this study, the seismic input is selected as a set of ground motion from the strong ground motion catalogues and all are scaled to match the codified response spectrum. In order to estimate all possible damage states nonlinear "Incremental Dynamic Analysis" (IDA) method (Vamvatsikos and Cornell, 2002) is selected as the analysis tool. Elastic spectral acceleration corresponding to the first mode period of the structure has been selected as the "engineering demand parameter" which is also used in the scaling procedure in Incremental Dynamic Analysis. Definition of damage levels and corresponding threshold values are based on "Park and Ang damage Index" (Park and Ang, 1985-a). Statistical analysis of the distribution of damage index for each elastic spectral acceleration level and for each damage level has been performed. As a result of the statistical analysis, parameters for the construction of vulnerability curves for each damage state and each building class is determined. The basic steps are also shown in Figure 4.1 as a flow chart to summarize the methodology.

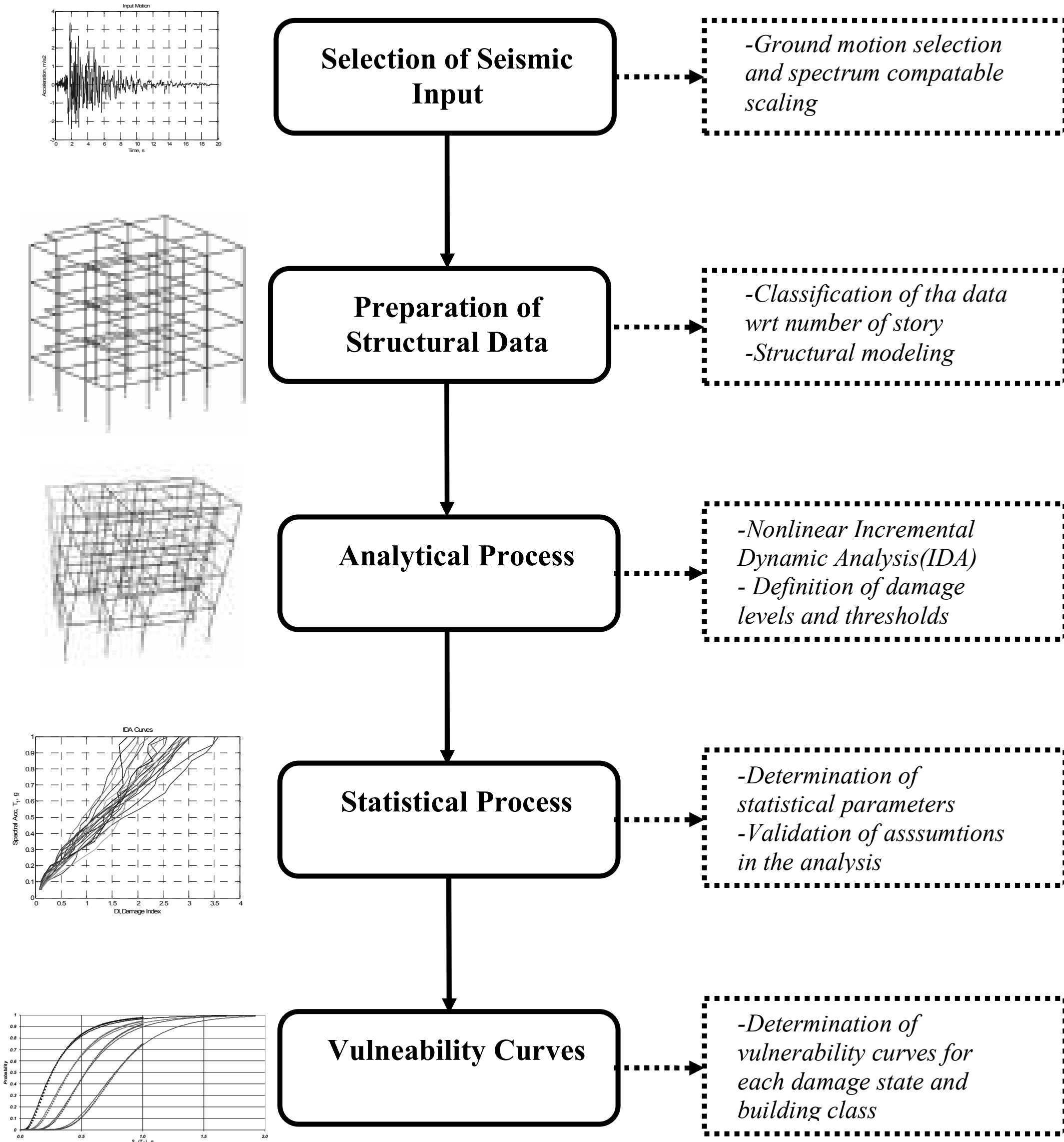

Figure 4.1. Basic Steps of procedure used in the vulnerability assessment

4.2. Characteristics of the Ground Motions

Selection of ground motions to be used in the vulnerability analysis is a key issue since the distribution of the response parameter is directly related to the input ground motion. The ground motion data set used in the vulnerability analysis is selected from the earthquake catalogues available. In the selection of recorded motions, it has been kept in mind to limit the minimum fault distance to 10 km in order to avoid near field effects. On the other hand soil conditions, faulting mechanisms and magnitudes are selected so as to correspond to average values rather than extreme. The properties of the selected ground motions are given in Table 4.1 below.

Table 4.1. Properties of the selected ground motions

No:	Earthquake Name	Mag.	Station	Dist.(km)	Site Class	PGA (g)
1	Chalfant Valley	6.2	54428 Zack Brothers Ranch	18.7	D	0.447
2	Chalfant Valley	6.2	54429 Zack Brothers Ranch	18.7	D	0.400
3	Loma Prieta 1989	6.9	APEEL 2 - Redwood City	47.9	D	0.220
4	Loma Prieta 1989	6.9	1686 Fremont - Emerson Court	43.4	B	0.192
5	Mammoth Lakes 1980	6.0	54214 Long Valley dam	19.7	A	0.484
6	Mammoth Lakes 1980	5.7	54214 Long Valley dam	14.4	A	0.245
7	Mammoth Lakes 1980	6.0	54301 Mammoth Lakes H. S.	14.2	D	0.390
8	Morgan Hill 1984	6.2	47380 Gilroy Array #2	15.1	C	0.212
9	Morgan Hill 1984	6.2	57382 Gilroy Array #4	12.8	C	0.348
10	Northridge 1994	6.7	90074 La Habra - Briarcliff	61.6	C	0.206
11	Northridge 1994	6.7	24575 Elizabeth Lake	37.2	C	0.155
12	Northridge 1994	6.7	24611 LA - Temple &amp	32.3	B	0.184
13	Northridge 1994	6.7	90061 Big Tujunga, Angeles Nat	24.0	B	0.245
14	Northridge 1994	6.7	90021 LA - N Westmoreland	29.0	B	0.401
15	Whittier Narrows 1987	6.0	Brea Dam (Downstream)	23.3	D	0.313
16	Whittier Narrows 1987	6.0	108 Carbon Canyon Dam	26.8	A	0.221
17	Whittier Narrows 1987	6.0	90034 LA - Fletcher Dr	14.4	C	0.213
18	Whittier Narrows 1987	6.0	90063 Glendale - Las Palmas	19.0	C	0.296
19	Whittier Narrows 1987	6.0	90021 LA - N Westmoreland	16.6	B	0.214
20	Whittier Narrows 1987	6.0	24461 Alhambra, Fremont Sch	13.2	B	0.333

Although number of recorded ground motions is increased in recent years by means of strong ground motion networks established in earthquake prone areas, selection of a number of recorded ground motions with proper characteristics in terms of fault distance, magnitude and local site conditions is not so easy. It is evident that as the number of ground motion records used in the analysis increases results will be more reliable and represent the mean value of the response parameters in a better way. But it must be kept in mind that number of ground motions should be limited to a certain level for the sake of computational effort and time. In order to obtain ground motion sets with certain characteristics recorded ground motions can be scaled by considering a certain ground motion parameter or to be compatible with a spectrum.

In this study, following the selection of the recorded ground motions, a scaling scheme has been applied in order to have spectrum compatible ground motions. Through the scaling process an iterative procedure is applied until response spectra of scaled ground motions match with the target response spectrum to a satisfactory level in all period ranges. The iterative procedure has been applied by using a program coded in MATLAB. (Celep, 2007).

The target response spectrum is selected as the acceleration response spectrum given in the Turkish Seismic Code 2007 for seismic zone 1 and site class Z3. The scaling is performed in the frequency domain in which recorded motion is simply scaled up or down uniformly to best match the target spectrum within a period range of interest. The procedure is based on minimizing the differences between the scaled motion’s response spectrum and target spectrum.

Acceleration response spectrum of each scaled time history is compared with the code spectrum in Figure 4.2. It can be observed that the results are satisfactory, as since the scaled motions’ spectra fit quite well with the code spectrum.

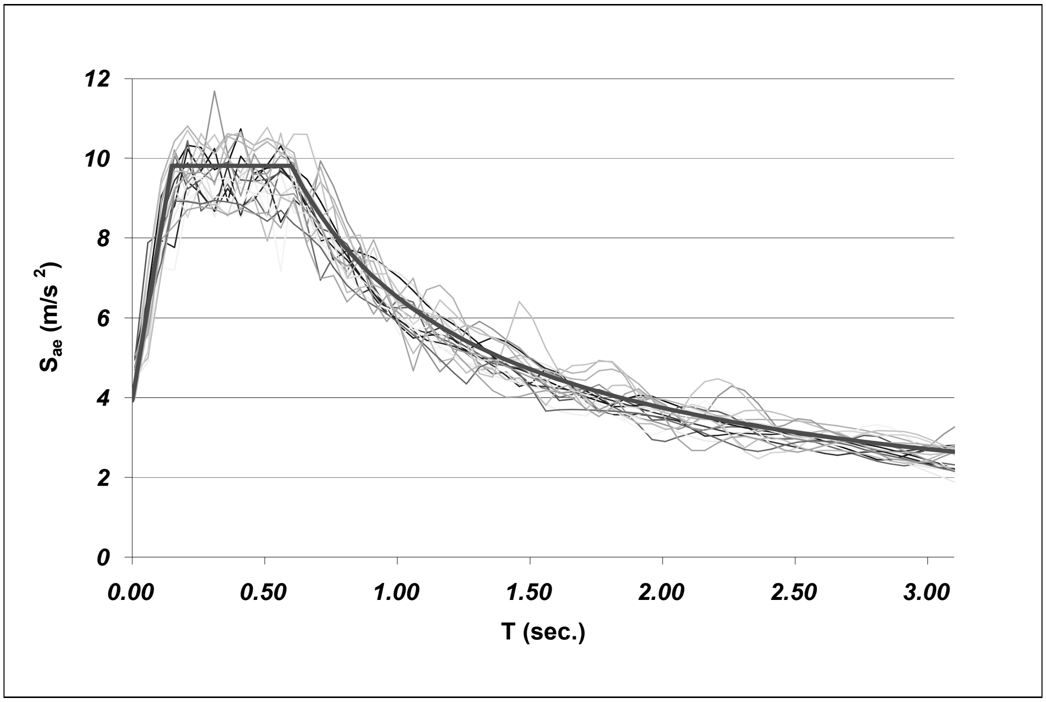

Figure 4.2. Acceleration response spectra of the scaled records and code spectrum

4.3. Properties of Structural Data

Building data set used in the vulnerability analysis is composed of 120 typical reinforced concrete residential buildings randomly selected from the building stock of the city of Bolu in Western Turkey which has been struck by the Duzce earthquake occurred on 12th November 1999 with a magnitude of M_w=7.2. Buildings are classified into six groups each containing 20 buildings according to their number of stories ranging from two to seven stories. All buildings are framed structures with monolithic beam-column connections. In selection of the building data set it has been kept in mind that none of the buildings have suffered damage in 1999 earthquakes. It can be urged that buildings represent the local engineering capabilities and construction practices of a typical Anatolian city. The unique property of the building data is that all of the data is composed of real structures. The design parameters such as concrete and steel strength, loads, framework plans of all stories representing the dimensions and configuration of the structural members are determined based on the design drawings available at the archives of the Bolu municipality. All buildings have been also checked whether there is any actual difference with respect to the existing drawings. The differences observed during this survey, if any, have been reflected to the data base. By the help of a group of civil engineers in the city of Bolu, computer models of the buildings under consideration have been prepared using design drawings. The models also contained the longitudinal and transverse reinforcement as given in the existing reinforcement details. A typical framing plan and a 3-D view of a sample building are shown in Figure 4.3 and Figure 4.4 respectively.

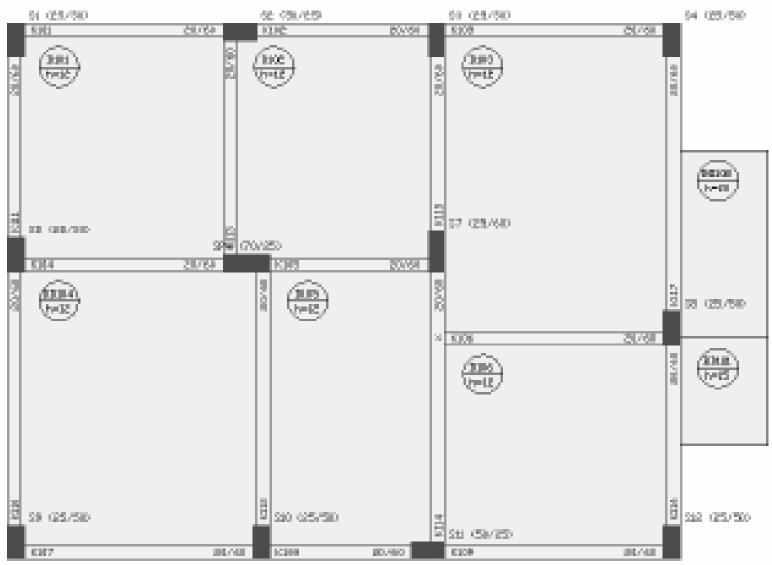

Figure 4.3. Typical framing plan of a sample building

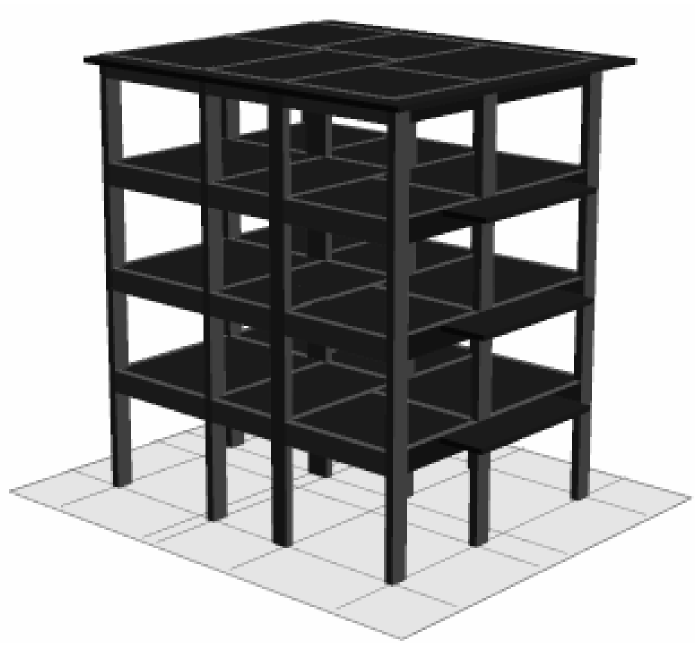

Figure 4.4. 3-D view of a sample building

4.4. Characteristics of Structural Modeling

All buildings are modeled in RUAOMOKO software platform, developed by Carr at University of Cuntenburry, New Zealand (Carr, 2004) and officially licensed to Bogazici University Department of Earthquake Engineering, The beams and columns are modeled as frame elements fifty percent of their gross section stiffness considers as cracked section stiffness. The masses of stories are assumed to be lumped at story levels with rigid diaphragm modeling. Nonlinear behavior of members is represented by plastic hinges, i.e., inelastic deformations are assumed to be lumped at member ends. Plastic hinge properties are determined by section analyses considering section dimensions, material strengths and reinforcement configuration. The plastic hinge is modeled as a rotational spring in a finite region of the member. The modeling assumptions of the plastic hinges as made in the software are given in Figure 4.5 and Figure 4.6 below.

Nonlinear time history analysis is performed in RUAUMOKO by using unconditionally stable Newmark Average Acceleration Method. The damping of the structures is taken into account as Rayleigh damping, in which damping matrix is assumed to be proportional with mass and stiffness matrices.

Levels of structural damage are identified by Park and Ang (1985-a) damage indices. Parameters of "*member damage index*" used in calculations are given as an input for the software and member damage index has been calculated for both ends of each structural member. The "*member damage index"* is defined as the linear combination of ductility level of the member and the total hysteretic energy at the end of the response history as given in equation (4.1)

$$DI = \frac{\theta_m}{\theta_u} + \frac{\beta}{M_y \theta_u} \int dE \qquad (4.1)$$

The damage state of the structure under seismic loading is defined by the *"global damage index"* which is determined by the local Park and Ang (1985-b) damage indices obtained for each structural member. The global damage index for the whole structure is defined as a weighted average of the local damage indices calculated for each element. The weighting function is assumed proportional to the energy dissipated in the respective element. Thus global damage index is given as;

$$D_T = \sum \lambda_i D_i \qquad \lambda_i = \frac{E_i}{\sum E_i} \tag{4.2}$$

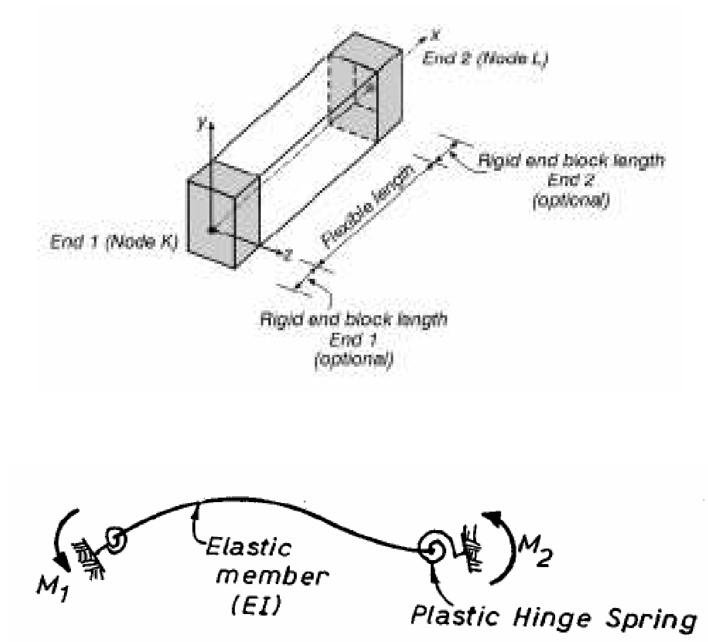

Figure 4.5. Frame element with rigid end blocks and hinges (Carr, 2004)

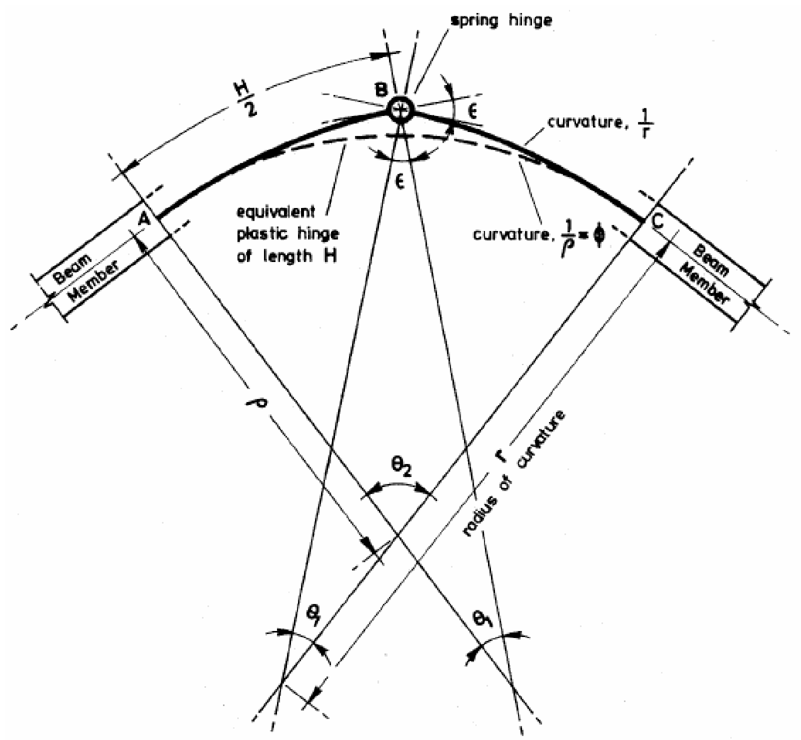

Figure 4.6. Giberson hinge model (Carr, 2004)

4.5. Analysis Procedure for Estimation of Damage Distribution

In order to capture all the possible damage states that the structure could experience under a seismic excitation, “Incremental Dynamic Analysis” (IDA) procedure proposed by Vamvatsikos and Cornell (2002) has been used. IDA is a parametric analysis tool which involves subjecting a structural model to one or more ground motion records, each scaled up to multiple levels of intensity, thus producing one or more curves of response parameterized versus intensity level. IDA has numerous advantages such as; understanding of the range of response or demands versus the range of the potential levels of a ground motion, better understanding the structural implications of severe ground motion levels, better understanding the changes of the nature of the structural responses as the intensity measure increase, producing dynamic capacity of the global structural system.

As a the first step of the analysis methodology, after selection of a single record a scale factor which is non-negative scalar that produces a multiple of the ground motion is determined. Although the scale factor is the simplest way to characterize the scaled motion, it is not convenient for engineering purposes since it offers no information of the real intensity parameter of the scaled record and its effect on a given structure. For more practical purposes, a measure which can be determined as the ground motion intensity should be involved to the scaling process in order to have a better representation of the increasing effect of the seismic input. Although many quantities have been proposed to characterize the "intensity" of a ground motion record, it may not always be apparent how to scale such intensity measures as moment magnitude, duration, or modified Mercalli Intensity which must be treated as non-scalable. Common examples of intensity measures can be listed as the Peak Ground Acceleration (*PGA*), Peak Ground Velocity, the $\xi = 5$ per cent damped Spectral Acceleration at the structure's first-mode period ($S_a(T_1, 5\%)$). Once the ground motion has been scaled monotonically by a scale factor the next step is the determination of the "damage measure" or "structural state variable" which represents the response of the system under seismic loading. In other words a "Damage Measure" is an outcome of the corresponding nonlinear dynamic analysis. Some possible alternatives could be the maximum base shear, peak rotations, peak storey ductilities, various damage indices (e.g., a global cumulative hysteretic energy, global Park–Ang index), peak roof drift, peak inter-storey drifts, or their maximum, the maximum peak inter-storey drift.

As a result of the nonlinear incremental dynamic analysis using one ground motion, a curve representing the response of the structure in terms of selected damage measure is plotted under increasing ground motion intensity. The same procedure can be repeated each ground motion in the database and a set of IDA curves is obtained.

In this study a set of spectrum compatible ground motion, details of which are given in the previous section, has been used. As the representative seismic intensity parameter is selected as the first mode spectral acceleration ($S_a(T_1)$) corresponding to the first mode period of the structure. Scale factor for spectral acceleration is changed from 0.05g to 1.0g with 0.05g increments.

The stages of the "Incremental Dynamic Analysis" procedure applied in the methodology are given below:

1. Select the ground motion to be used in the analysis ($\ddot{u}_g(t)$)
2. Define intensity measure, (Sa(T1)), incremented at each analysis step as Sai(T1)=0.05g, 0.10g, 0.150g, 0.21g, 0.25g, 0.30g, 0.40g, 0.50g, 0.60g, 0.70g, 0.80g, 0.85g, 0.90g, 0.95g, 1.00g.
3. Determine the scaled time history for each step such that Sai(T1) is a given value in Step 2.

$$\ddot{u}_{ig}(t) = \lambda_i \ddot{u}_g(t) \tag{4.3}$$

4. Determine the damage measure (In this case damage measure is Park and Ang damage index
5. Perform nonlinear time history analysis for each scaled ground motion
6. Determine the damage measure for each spectral acceleration increment
7. Repeat the above steps for the next ground motion.

By using the steps described above, nonlinear time history analyses are performed for each structure, for each ground motion and for each spectral acceleration level.(120 buildings x 20 ground motions x 20 spectral acceleration levels = 48000 nonlinear time history analyses.).The estimation of variation of representative seismic demand parameter, Park and Ang Damage Index, with respect to representative seismic intensity parameter, $S_a(T_1)$ is done through parametric nonlinear response history analysis named "Incremental Dynamic Analysis". As a result of the analyses IDA curves for each building class are plotted. The curves, which are given in Figure 4.7 to Figure 4.12 represent the relationship between representative ground motion intensity measure, first-mode spectral acceleration, with the representative seismic demand parameter defined by the global Park and Ang damage index.

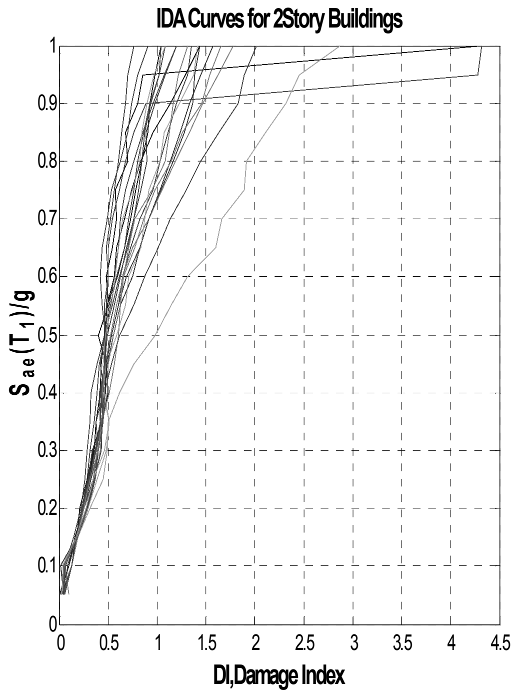

Figure 4.7. IDA curves for 2 story buildings

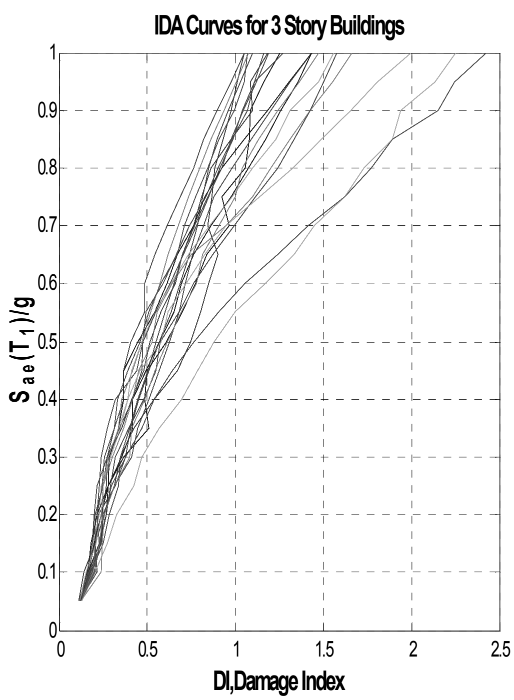

Figure 4.8. IDA curves for 3 story buildings

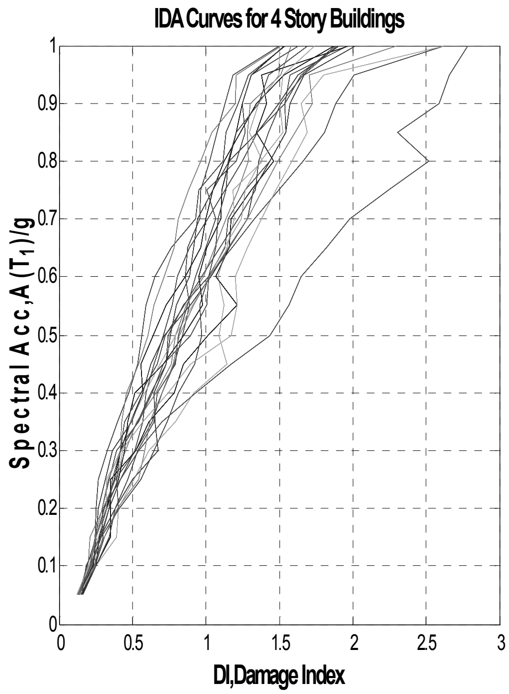

Figure 4.9. IDA curves for 4 story buildings

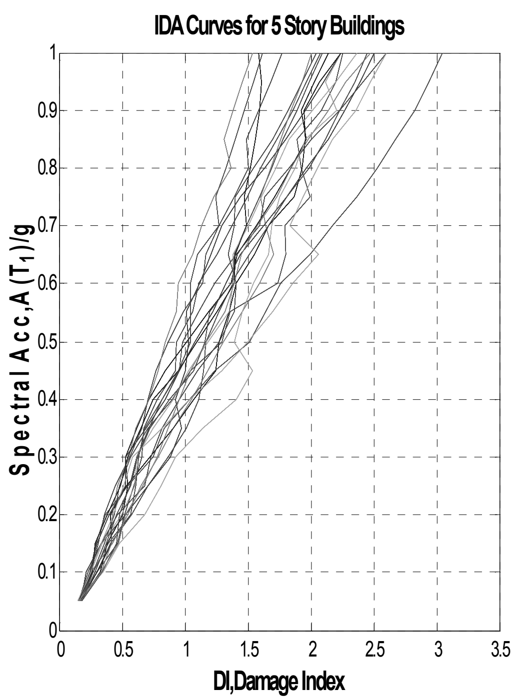

Figure 4.10. IDA curves for 5 story buildings

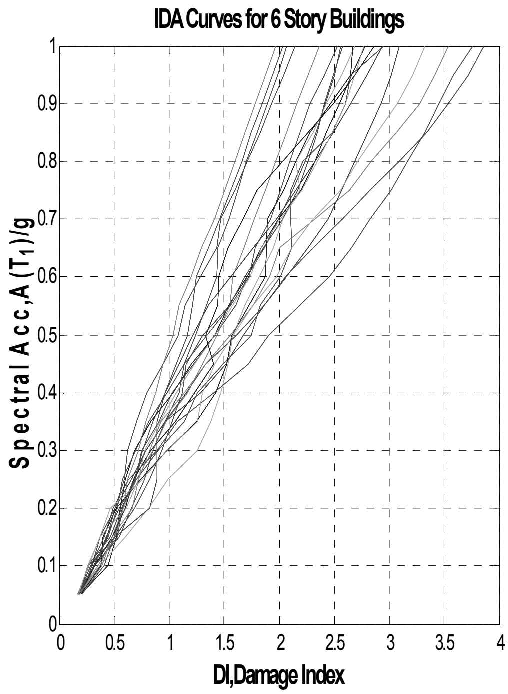

Figure 4.11. IDA curves for 6 story buildings

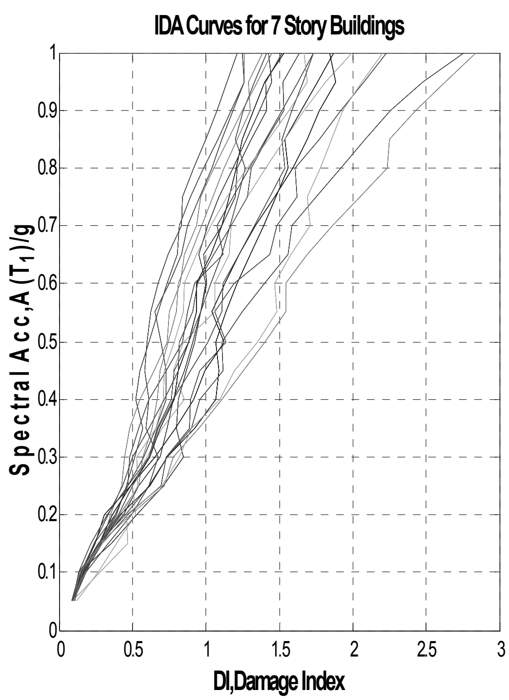

Figure 4.12. IDA curves for 7 story buildings

As it can be observed from the figures, the global damage level of the structure is proportional with the first mode spectral acceleration. Although the damage index values greater than 1.0 have no physically meaning, but the distribution of spectral accelerations for each damage level defined by a damage index threshold value will be used in the statistical analysis for the determination of parameters of the vulnerability curves.

4.6. Estimation of Damage Level by Park and Ang Damage Index

With the distribution of the representative seismic demand parameter, Park and Ang Damage Index, with respect to representative seismic intensity parameter, $S_a(T_1)$, determined, the next step is the quantification of damage state thresholds in terms of representative seismic demand parameter. Since Park and Ang damage index is accepted as one of the good indicator of damage in calibration studies performed by various researchers, it has been selected as the damage quantification parameter for the damage level threshold values. In the literature a range of damage level threshold values are given based on post earthquake surveys and laboratory tests.

Proposed Park and Ang (1985-b) indices corresponding to typical damage levels and thresholds are given as (Ghobarah, 1997):

$DI < 0.05$	Elastic Limit
$0.05 < DI < 0.14$	Minor Damage
$0.14 < DI < 0.40$	Repairable Damage
$0.40 < DI < 0.60$	Unrepairable Damage
$0.60 < DI < 1.00$	Extensive Damage
$DI > 1.00$	Collapse

Karim and Yamazaki (2001) have proposed damage level thresholds in terms of Park and Ang (1985-b) index as given below.

0.0<DI<0.14	No damage
0.14<DI<0.40	Slight damage
0.40<DI<0.60	Moderate damage
0.60<DI<1.00	Extensive damage
1.00<DI	Complete damage

Singhal and Kiremidjian (1996) have also proposed damage level thresholds in terms of Park and Ang (1985-b) index as given in Table 4.2 which have been used in their vulnerability assessment study. The description of damage states of reinforced concrete members defined in terms of Park and Ang Damage Index (1985-b) can be given as:

Minor Damage: Minor cracks throughout buildings, partial crushing of concrete in columns

Moderate Damage**:** Extensive large cracks throughout building, spalling of concrete

Severe Damage**:** Extensive crashing of concrete, disclosure of buckled reinforcements

Collapse: Total or partial collapse of building

Table 4.2. Damage level thresholds in terms of Park and Ang index (Singhal and Kiremidjian,1996)

Damage State	**Range of Park and Ang Index**
Minor	0.1 - 0.2
Moderate	0.2 - 0.5
Severe	0.5 - 1.0
Collapse	> 1.0

4.7. Statistical Evaluation of Damage Distribution

In the previous section analyses results are obtained in the form of IDA curves which graphically represents the distribution of $S_a(T_1)$ as the representative seismic intensity parameter with respect to Park and Ang Damage Index as the representative seismic demand parameter. In this section the distribution under consideration will be used for calculation of probability of reaching or exceeding a certain level of damage for a certain intensity parameter through statistical process.

The statistical process starts with an assumption that the analyses results are log-normally distributed. The probability of reaching and exceeding a certain level of damage of each analysis result is calculated for each level of damage state and spectral acceleration. The detailed calculations of probability of discrete solutions are given in the proceeding sections. In addition to discrete solutions, the analytical vulnerability functions for each damage level and building class with respect to spectral acceleration is obtained by using the mean and log-normal standard deviation of same distribution. The analytically obtained continuous functions are graphically compared with the discrete results in order to evaluate the match, or in other words the validity of assumption of log-normal distribution of the results. On the other hand, assumption is also verified by means of statistical tests available in literature. Once the assumption is valid, the confidence intervals of the analytical vulnerability curves are determined in order to evaluate the damage level intervals are separated satisfactorily.

4.7.1. Probabilistic Evaluation of Analysis Results

The probability of reaching or exceeding a damage level for a given spectral acceleration level is calculated through distribution of the damage parameter for each spectral acceleration. The steps of the calculation repeated for each spectral acceleration values are given below:

1. Determination of probability density function of the damage measure (DI) for a given spectral acceleration value, Sa(T1).

2. Transformation of the damage measure variable to log-normal distribution variable
3. Determination of probability density function of log-normal variable
4. Calculation of log-normal cumulative function of the distribution
5. Calculation of probability of reaching or exceeding of a certain damage level defined by a threshold value

The probability density functions of "Damage Index" values obtained from nonlinear response history analysis for each spectral acceleration level for each building class are calculated and graphically represented on IDA curves. Examples of probability density distributions are plotted for selected spectral acceleration levels such as $S_{ae}(T_1)$ = 0.1 g, 0.2g, 0.5g and 1.0g. Obtained distributions are used in calculation of discrete probability for each damage level. The aforementioned distributions of DI values are shown on the IDA curves for each damage level and building class from Figure 4.13 to Figure 4.18.

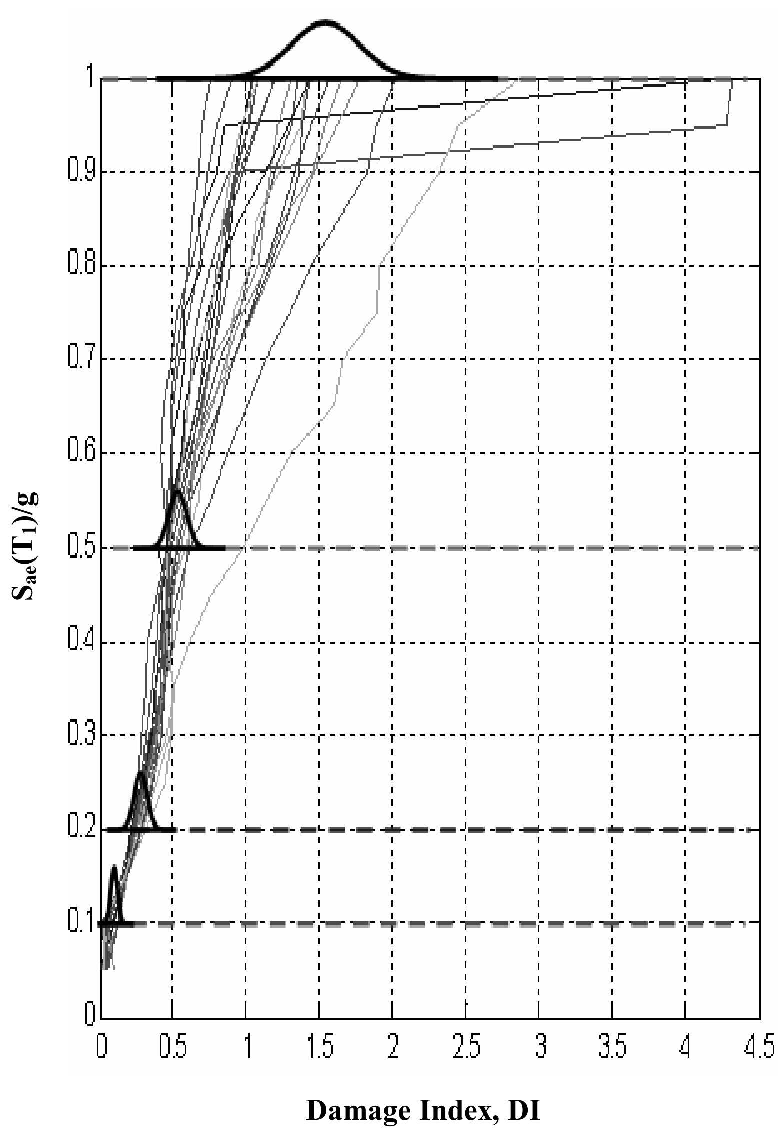

Figure 4.13. Probability density functions of DI distributions for different spectral acceleration levels for 2 story buildings

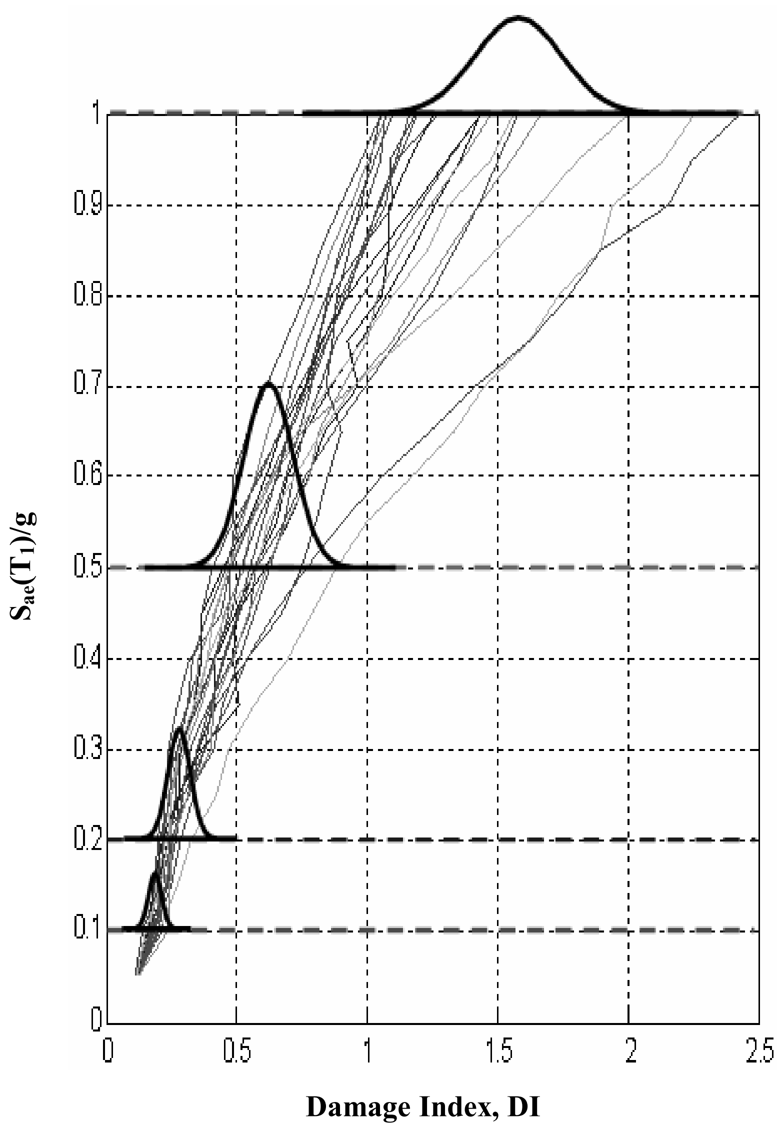

Figure 4.14**.** Probability density functions of DI distributions for different spectral acceleration levels for 3 story buildings

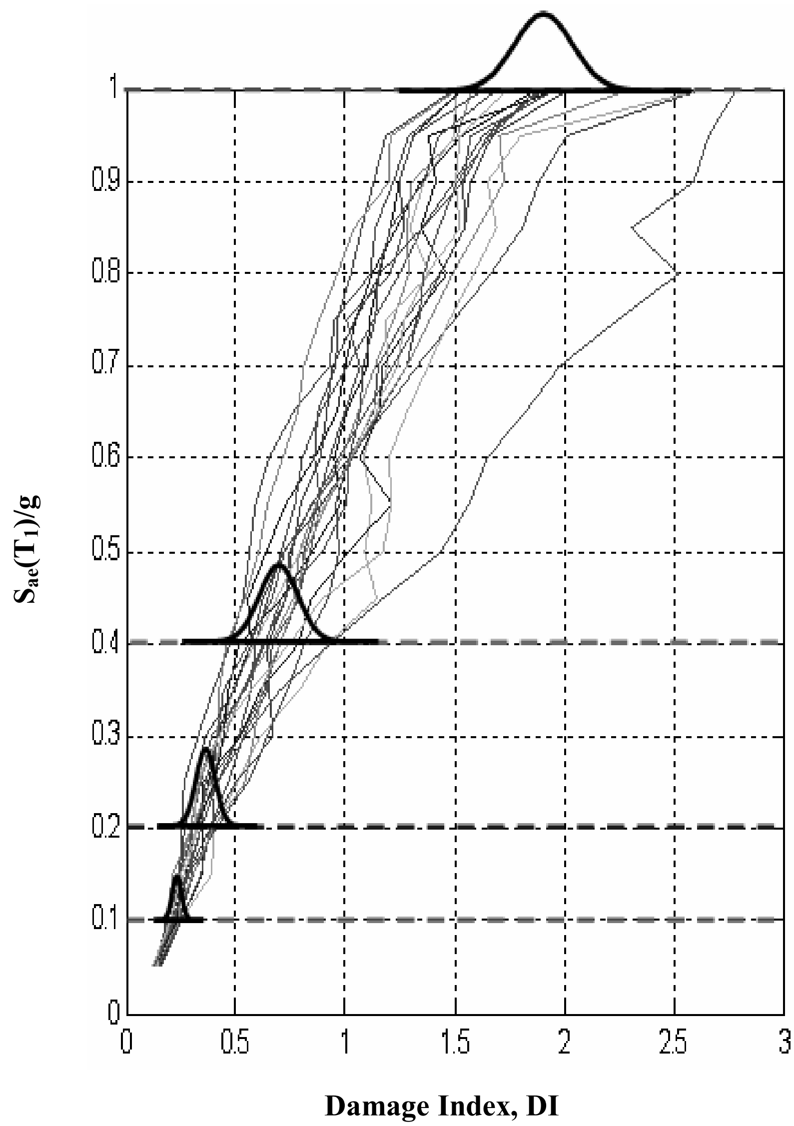

Figure 4.15. Probability density functions of DI distributions for different spectral acceleration levels for 4 story buildings

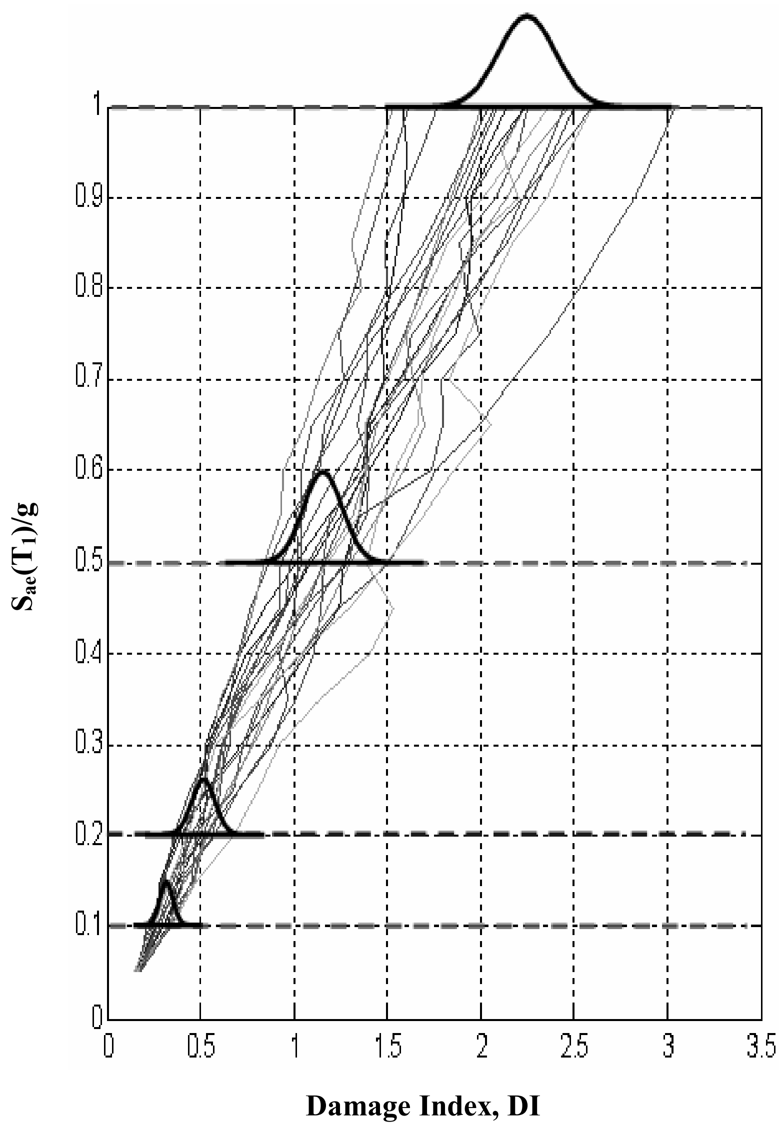

Figure 4.16. Probability density functions of DI distributions for different spectral acceleration levels for 5 story buildings

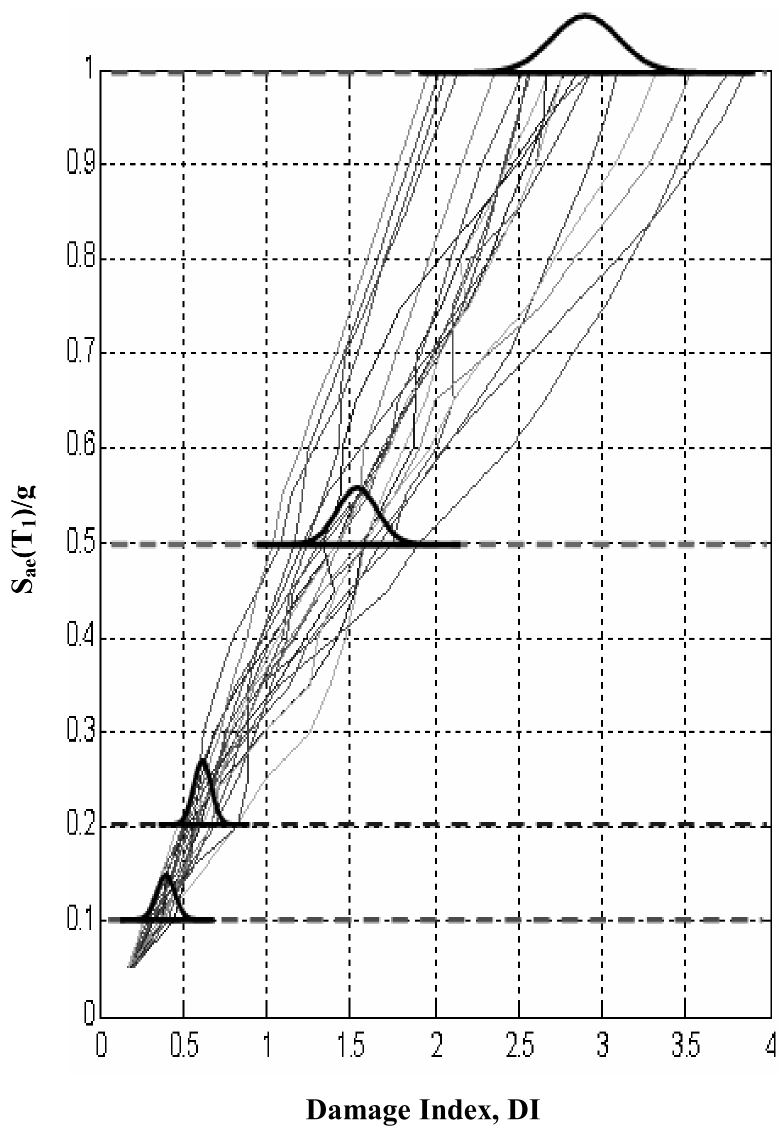

Figure 4.17. Probability density functions of DI distributions for different spectral acceleration levels for 6 story buildings

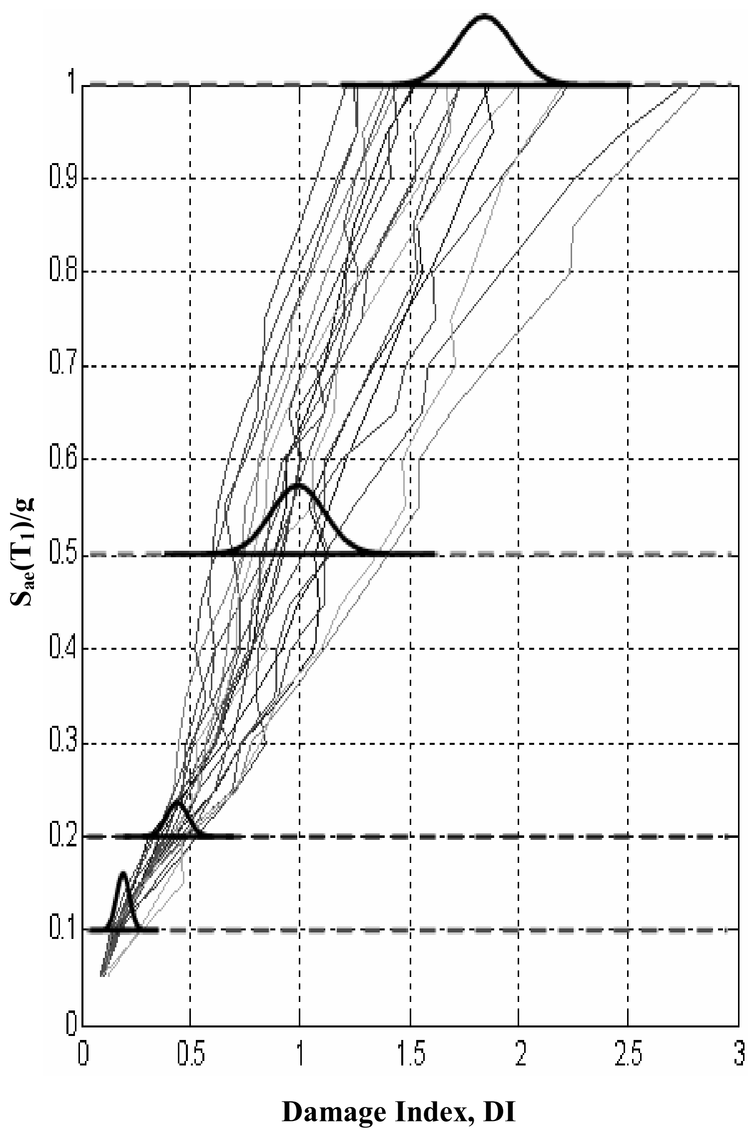

Figure 4.18. Probability density functions of DI distributions for different spectral acceleration levels for 7 story buildings

As the first step, probability density function of distribution of representative seismic demand parameter, DI, as the random variable is calculated for each $S_a(T_1)$ level. In order to calculate lognormal probability density function of the variable, a transformation is applied from random variable "DI" to lognormal variable "s". The transformation process is summarized and the relationships used are given below.

The probability distribution of the natural logarithm of the random variable "*x*" (in this case DI) is called the "log-normal distribution" and the probability density function of "*x*" is given in equation (4.5) as:

$$f_X(x) = \frac{1}{\sqrt{2\pi}\zeta x}\exp\left[-\frac{1}{2}\left(\frac{\ln x-\lambda}{\zeta}\right)^2\right] \quad (0 < x < +\infty) \tag{4.4}$$

The parameters of the distribution are λ, the mean of the natural logarithm and ζ, the standard deviation of "*x*". By the logarithmic transformation of variables, the probability of the log-normal variables can be determined by the standard normal probability table. The probability of the random variable to be in the interval of (*a,b*) can be calculated by the equation (4.5) as:

$$P(a < X \leq b) = \int_a^b \frac{1}{\sqrt{2\pi}\zeta x}\exp\{(-\frac{1}{2}\left[(\ln x-\lambda)/\zeta\right]^2\}dx \tag{4.5}$$

By applying the transformation of $s = \dfrac{\ln x-\lambda}{\zeta}$ and $dx = x\zeta ds$, and introducing the log-normal probability function "*Φ*", the above equation can be re-written as in equation (4.6);

$$P(a < X \leq b) = \frac{1}{\sqrt{2\pi}}\int_{(\ln a-\lambda)/\zeta}^{(\ln b-\lambda)/\zeta} e^{-s^2/2}ds = \Phi(\frac{\ln b-\lambda}{\zeta}) - \Phi(\frac{\ln a-\lambda}{\zeta}) \tag{4.6}$$

In the next step of the calculations, the probability density function is used in order to determine the log-normal cumulative distribution function for each damage level by simply addition of the discrete probabilities corresponding to each log-normal variable "*s*". A sample for graphical representation of both probability density and cumulative probability functions of a log-normal variable is shown in Figure 4.19. These calculations are repeated for the distribution of DI values of each spectral acceleration value and damage levels.

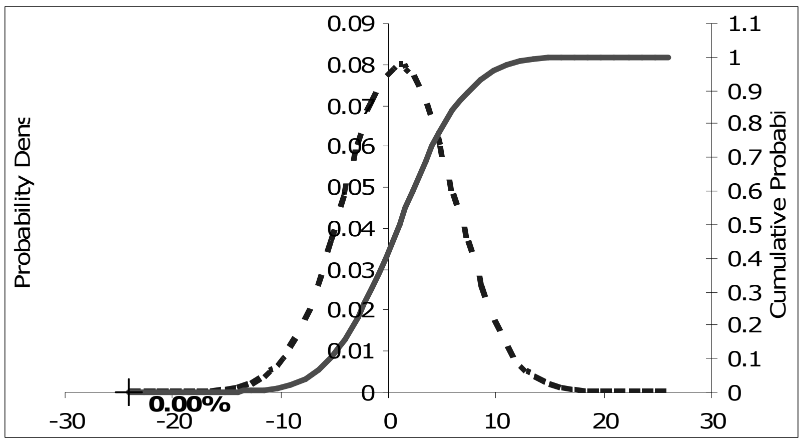

Figure 4.19. Sample probability density function (dashed) and cumulative probability function (straight) of log-normal variable

In order to clarify the above mentioned steps for probability calculation, analysis results of 4 story buildings for two levels of spectral acceleration $S_a(T_1)$=0.2g and $S_a(T_1)$=0.5g are used for detailed calculations. By using the distribution in Figure 4.19, calculation of probability of reaching and exceeding each damage levels for $S_{ae}(T_1)$=0.20 g is given below by graphical representation from Figure 4.20 to Figure 4.23.

DS1 : Minor Damage (DI_{DS1} =0.10)

P(DI $\geq$ DI_{DS1}) = 26.11 %

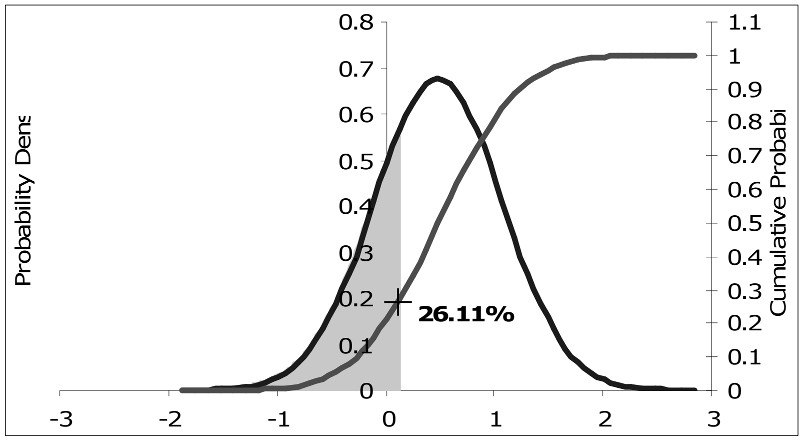

Figure 4.20. Probability calculation of DS1 for $S_{ae}(T_1)$ = 0.20 g

DS2: Moderate Damage (DI_{DS2} =0.20)

P(DI $\geq$ DI_{DS2}) = 13.14 %

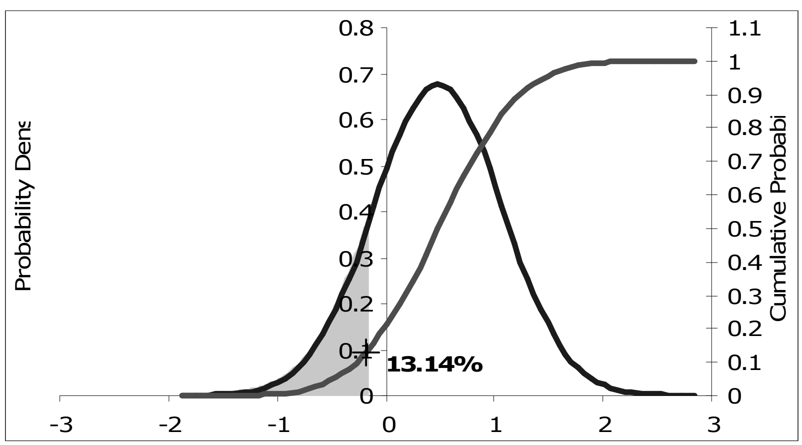

Figure 4.21. Probability calculation of DS2 for $S_{ae}(T_1)$ = 0.20 g

DS3: Severe Damage (DI_{DS3} =0.50)

P(DI $\geq$ DI_{DS3}) = 0.52 %

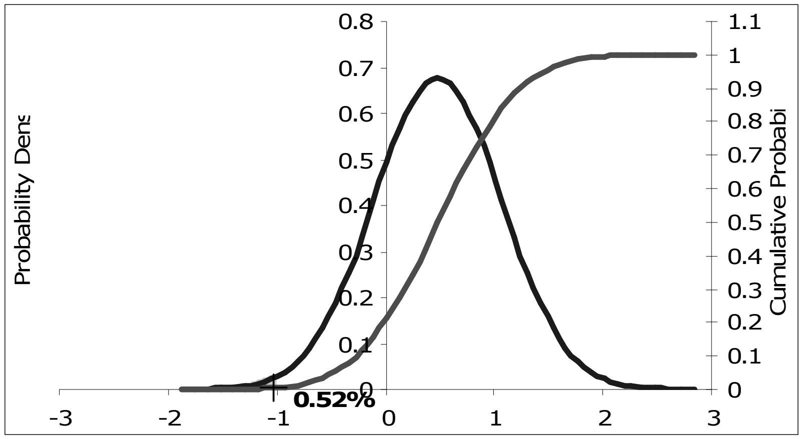

Figure 4.22. Probability calculation of DS3 for $S_{ae}(T_1)$ = 0.20 g

DS4 : Collapse Damage (DI_{DS4} =1.0)

P(DI $\geq$ DI_{DS4}) = 0.09 %

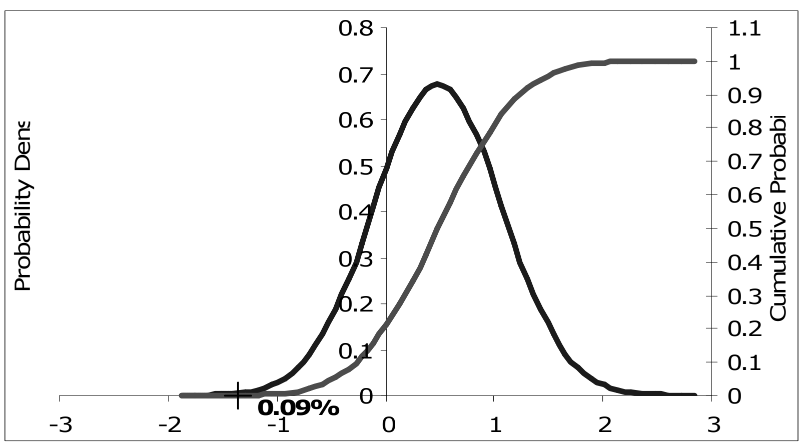

Figure 4.23. Probability calculation of DS4 for $S_{ae}(T_1)$ = 0.20 g

The same calculations are repeated for distribution of $S_{ae}(T_1)$=0.50 g. Determination of the probability of reaching and exceeding each damage levels is given below by graphical representation from Figure 4.24 to Figure 4.27.

DS1 : Minor Damage (DI_{DS1} = 0.10)

P(DI $\geq$ DI_{DS1}) = 65.54 %

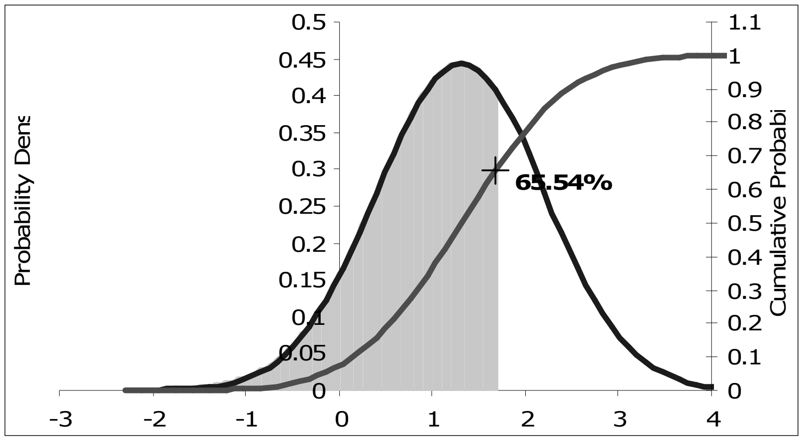

Figure 4.24. Probability calculation of DS1 for $S_{ae}(T_1)$ = 0.50 g

DS2: Moderate Damage (DI_{DS2} = 0.20)

P(DI $\geq$ DI_{DS2}) = 56.36 %

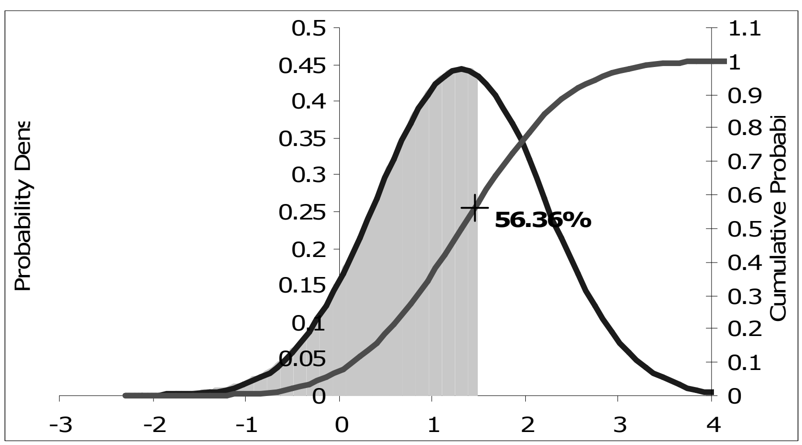

Figure 4.25. Probability calculation of DS2 for $S_{ae}(T_1)$ = 0.50 g

DS3: Severe Damage (DI_{DS3} =0.50)

P(DI $\geq$ DI_{DS3}) = 16.85 %

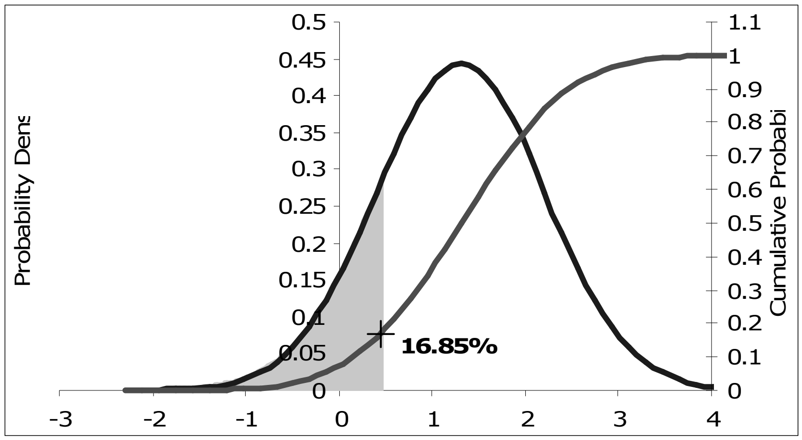

Figure 4.26. Probability calculation of DS3 for $S_{ae}(T_1)$ = 0.50 g

DS4: Collapse Damage (DI_{DS4} =1.00)

P(DI $\geq$ DI_{DS4}) = 5.48 %

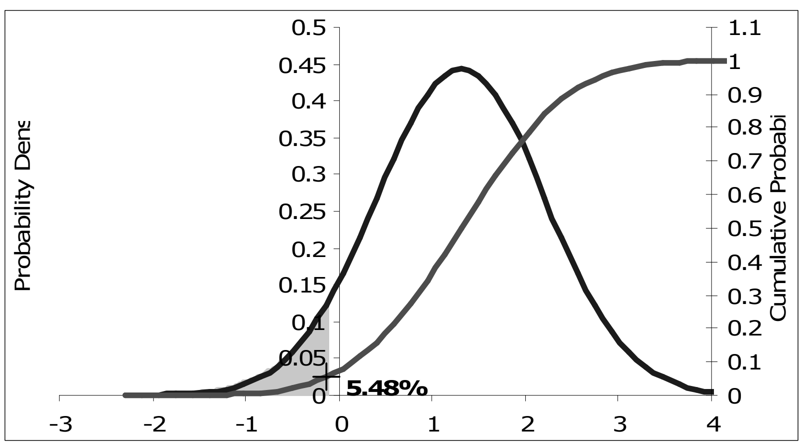

Figure 4.27. Probability calculation of DS4 for $S_{ae}(T_1)$ = 0.50 g

The processes summarized for $S_{ae}(T_1)$=0.20 g and 0.50 g is repeated for each spectral acceleration level for each damage state and for each building class. As the result of this process discrete probabilities obtained from the analysis have been obtained. The discrete probabilities are plotted on the graph in order to see the scatter of the results and evaluate the trend of the graphs for each damage level. Since the discrete solutions are ranging from 0.05g spectral acceleration level to 1.0g spectral acceleration level, number of obtained solutions is limited and could not give a satisfactory proof for trend of distributions. In this line, spectral acceleration interval is changed to 0.01 g level and by using the calculated results the missing values are determined by interpolation. It must be indicated that since the upper limit of spectral acceleration is1.0g, no calculation has been done for grater values by using extrapolation techniques. The graphs of probability distribution of interpolated values for each damage state and building class with respect to spectral acceleration corresponding to first mode period are given from Figure 4.28 to Figure 4.33.

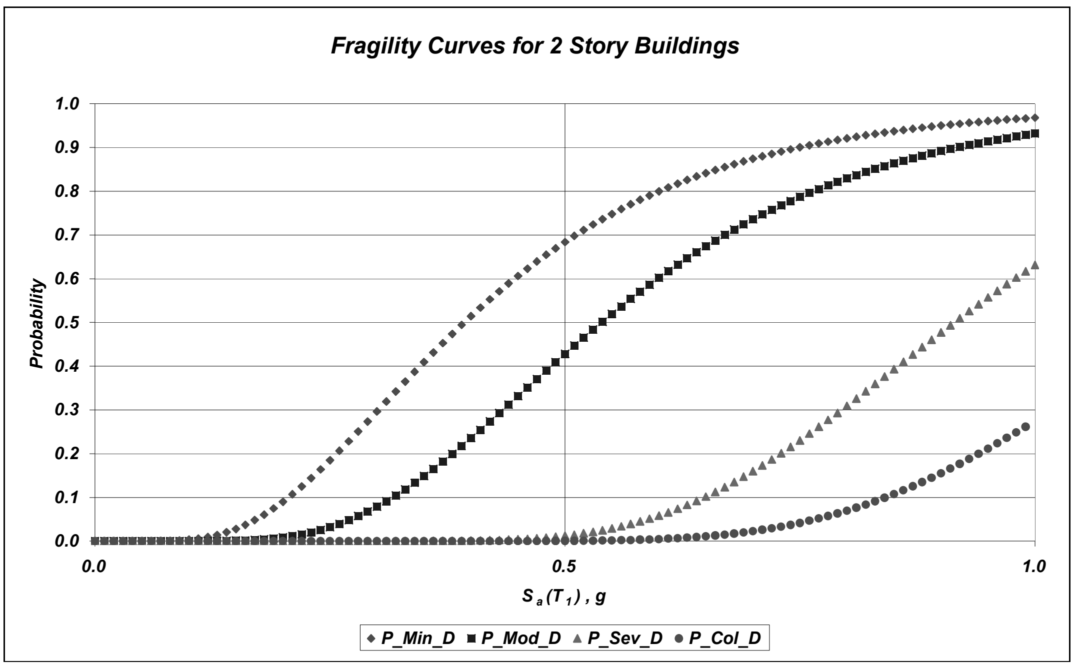

Figure 4.28. Probability distributions of discrete solutions for 2 story buildings

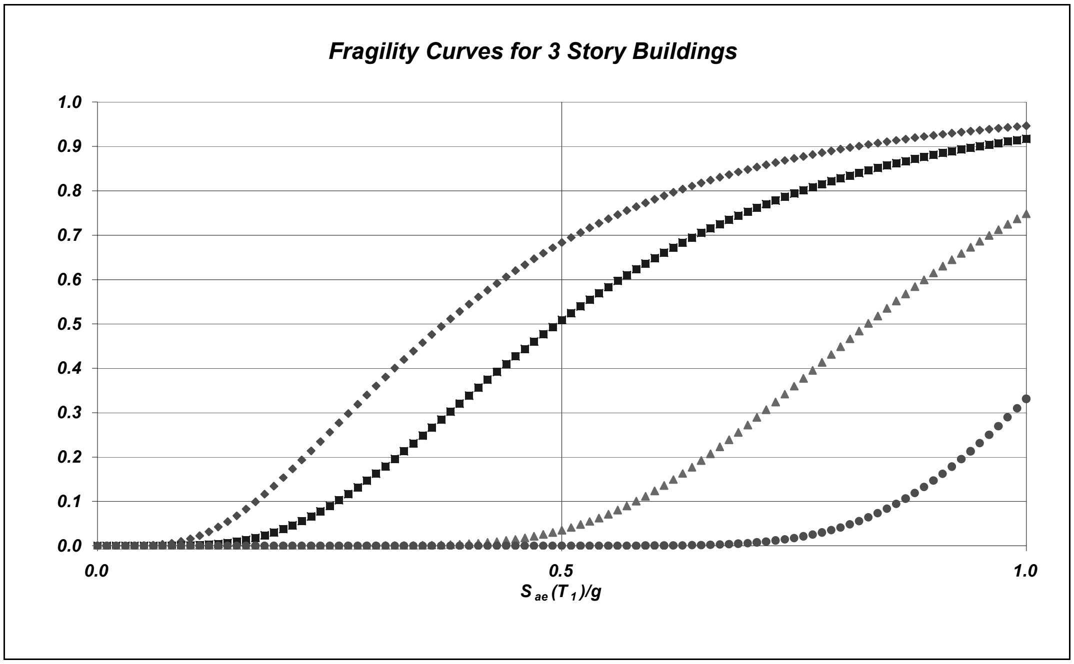

Figure 4.29. Probability distributions of discrete solutions for 3 story buildings

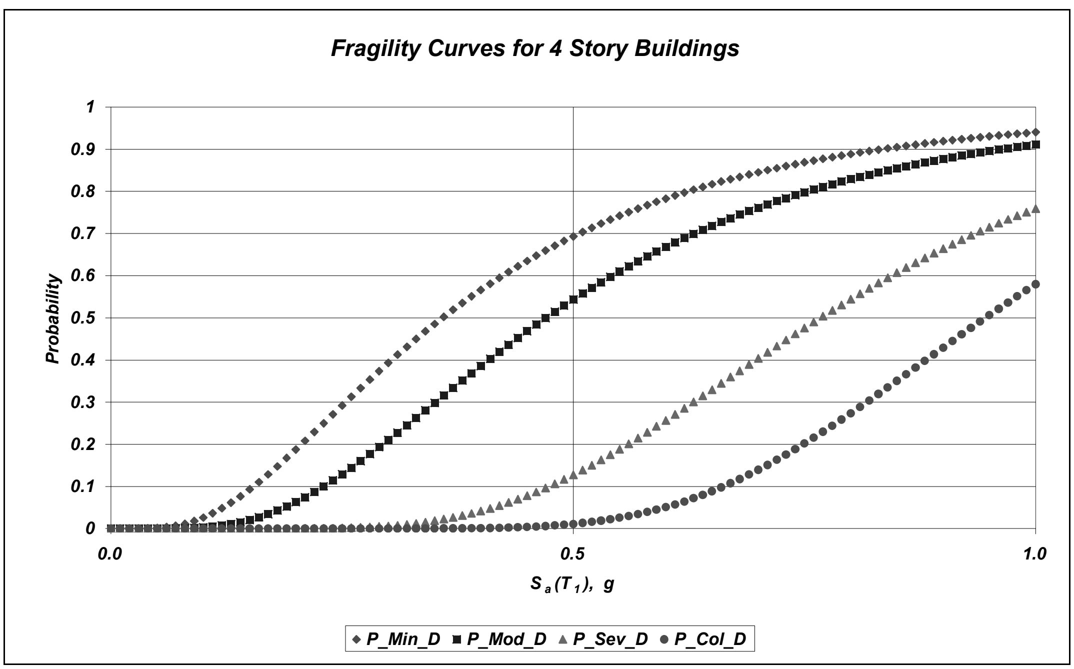

Figure 4.30. Probability distributions of discrete solutions for 4 story buildings

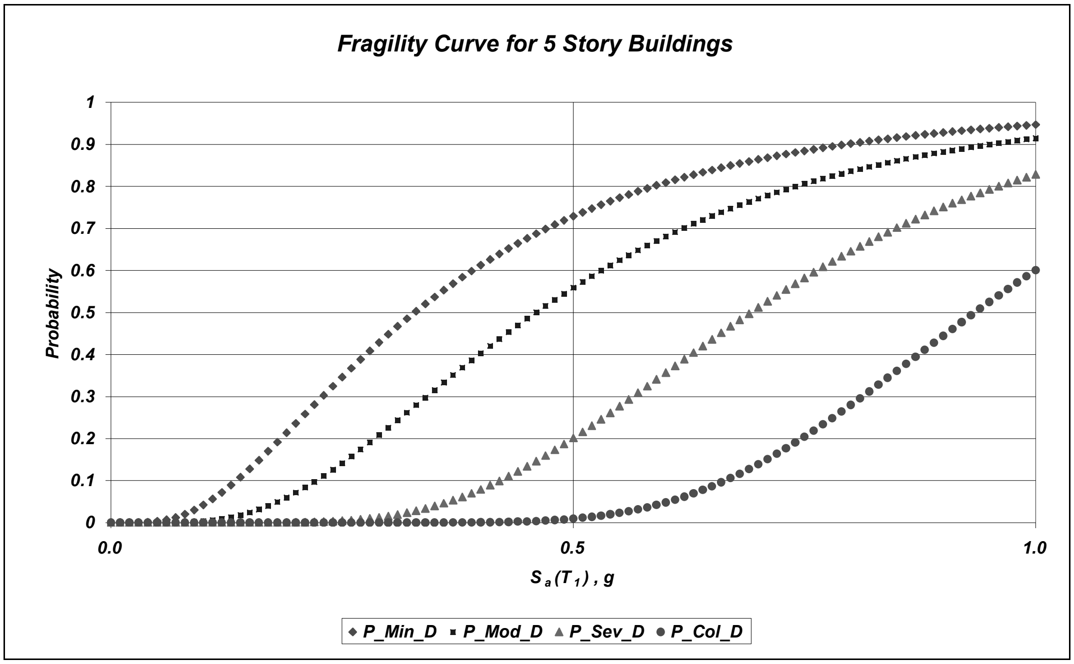

Figure 4.31. Probability distributions of discrete solutions for 5 story buildings

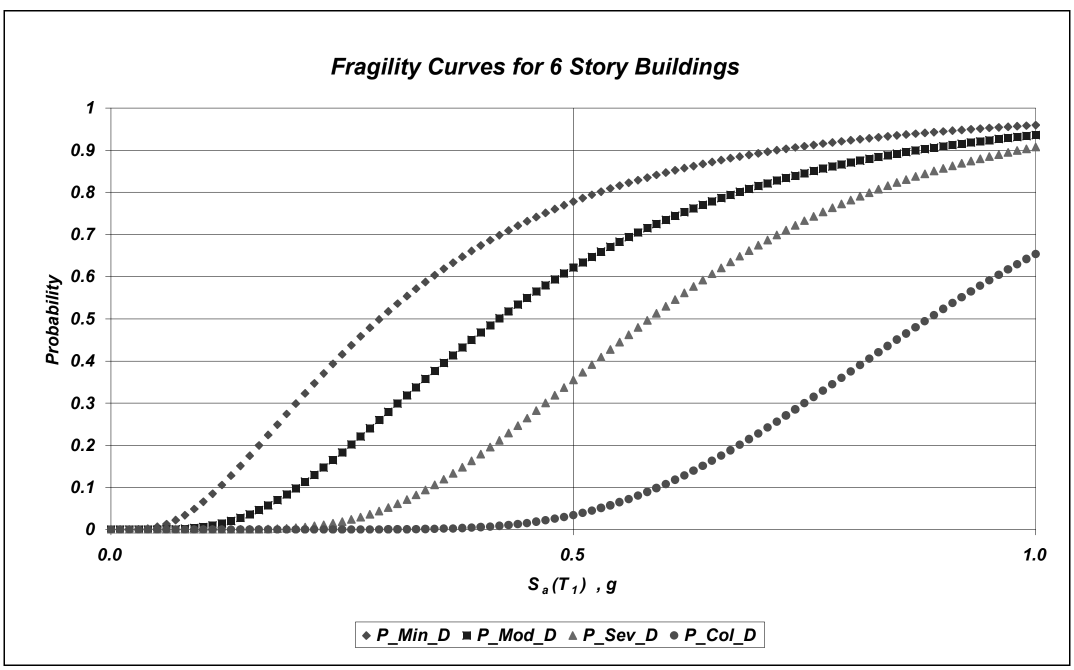

Figure 4.32. Probability distributions of discrete solutions for 6 story buildings

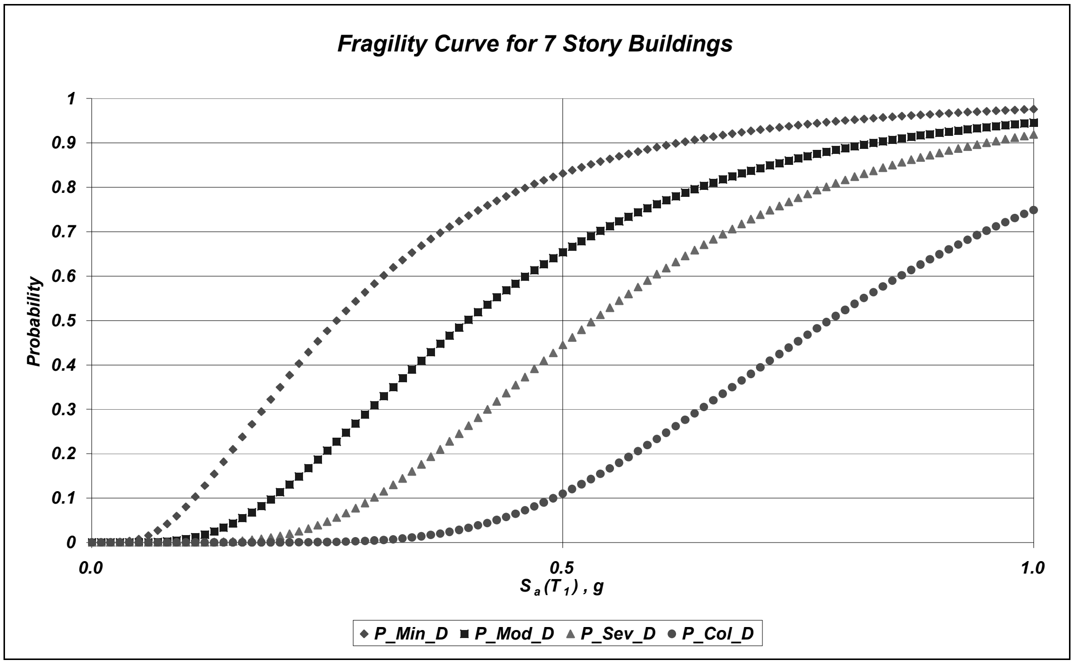

Figure 4.33. Probability distributions of discrete solutions for 7 story buildings

4.7.2. Determination of the Vulnerability Curve Parameters

Once the distribution of the representative seismic intensity parameter, $S_a(T_1)$, with respect to representative seismic demand parameter, DI, is determined, statistical parameters can be estimated for the construction of the vulnerability curves by using the distribution for each damage state and each building class. Utilizing this distribution a lognormal probability distribution function can be fitted to represent vulnerability curves, which simply relate the probability of reaching or exceeding of multiple damage states to ground motion parameter severity and can therefore regarded as graphical representation of the seismic risk. A vulnerability curve for a particular damage state is obtained by computing the conditional probabilities of reaching or exceeding that damage state at various levels of ground motion. In other words, a plot of computed conditional probabilities versus the ground motion parameter describes the vulnerability curve for that damage state. The vulnerability curve is generally expressed by a log-normal cumulative distribution function as given in equation (4.7):

$$P(D \geq d_{\mathrm{i}} \mid Y) = \Phi\left[\frac{1}{\beta_{\mathrm{i}}} \ln\left(\frac{Y}{Y_{\mathrm{mi}}}\right)\right] \tag{4.7}$$

where "*P"* is the probability of the damage parameter "*D"* reaching or exceeding the value of "d_{i}" for the ith damage state for a given ground motion characterized by an earthquake parameter "*Y"*, whereas "Y_{mi}" is the median threshold value. "Φ" represents the standard normal cumulative distribution probability function. "β_i" is the lognormal standard deviation.

In this study the ground motion parameter is spectral acceleration corresponding to the first mode of the structure, $S_{ae}(T_1)$, damage parameter is the Park and Ang (1985-a) damage index, *DI.* The general description of the vulnerability curve can be re-written by using the selected parameters as given in equation (4.8):

$$P(DI \geq DI_{\mathrm{ds}} \mid S_{\mathrm{ae}}) = \Phi\left[\frac{1}{\beta_{\mathrm{ds}}} \ln\left(\frac{S_{\mathrm{ae}}}{(S_{\mathrm{ae}})_{\mathrm{m,ds}}}\right)\right] \tag{4.8}$$

which would represent the probability of Park and Ang damage indices (*DI*) reaching or exceeding the threshold value of each damage state (DI_{ds}) for the ground motion characterized by the spectral acceleration (S_{ae}) corresponding to the elastic strength demand of a building class at its fundamental period.

The steps of the vulnerability curve construction process can be summarized as;

1. Determine of the $S_{ae}(T_1)$ vs *DI* for each damage state
2. Assuming that the distribution is log-normally distributed, determine the median, $(S_{ae})_{m,ds}$, and standard deviation of the natural logarithm of spectral acceleration data, β_{ds}.
3. Apply Chi-square goodness of fit test for the validation of the lognormal distribution assumption of the distribution
4. Construct the vulnerability curve by using equation (4.8).

As stated in the previous sections, validation of log-normal distribution assumption for the analysis results has been done by plotting the discrete solution probabilities obtained from the analysis on the same graph with the analytically obtained the vulnerability curves in order to see the degree of match with each other.

On the other hand after having constructed the vulnerability curves which represent the median value of the distribution, 95 per cent confidence limits of the mean value are calculated for each damage level and building class.

4.7.3. Verification of Log-Normal Distribution Assumption of Results

Before the determination of the statistical parameters of $S_{ae}(T_1)$ values which reaches or exceeds damage index values of 0.10, 0.20,0.50 and 1.0 for minor, moderate, severe, complete damage states respectively, the log-normal distribution assumption is been checked by using the well-known statistical tool, “Chi-Square Goodness of Fit Test”, for the determination of the distributional adequacy of the data at hand. Since the number of data available is large enough the aforementioned tool can be used. The chi-square test is used to test if a sample of data belong a population with a specific distribution. An attractive feature of the chi-square goodness-of-fit test is that it can be applied to any distribution for which you can calculate the cumulative distribution function. The chi-square goodness-of-fit test is applied to binned data (i.e., data put into classes). This is actually not a restriction since for non-binned data you can simply calculate a histogram or frequency table before generating the chi-square test. If the test is applied on a log - normal distribution, the test statistic is small and the hypothesis is accepted under the assumed percent level of significance which is 5 per cent. Since the chi-square test statistics is a number between zero and unity, the smaller the statistics the more acceptable the data as log-normal distributed. The results of the statistics done for each damage level and building class are obtained in the range of 0.05 to 0.15 which yields that the distribution of the analyses results to log-normal is valid.

4.7.4. Determination of the Statistical Parameters of Vulnerability Curves

Following verification process, the statistical parameters, the median, $(S_{ae})_{m,ds}$, and standard deviation of the natural logarithm of spectral acceleration data, β_{ds}, of the log-normally distributed $S_{ae}(T_1)$ values are calculated. These parameter are given for each damage state and building class in Table 4.1. Also the graphs of the vulnerability curves with respect to $S_{ae}(T_1)$ and $S_{de}(T_1)$. Spectral displacement corresponding to first mode period of the structure is calculated by using the average fundamental period of the building class under consideration. The vulnerability curves for each building class with respect to spectral acceleration and spectral displacement are given in Figure 4.34 to Figure 4.45. On the other hand probabilities of the discrete solutions obtained are plotted by the dots on the same graph in order to see the match with the continuous curve. As it can be seen from the graphs, the discrete solutions show a quite satisfactory match with the analytical results.

Table 4.3. Statistical parameters of the log-normal distribution for each damage state and building class

	Damage State							
Building Class	**Minor** ($DI_{min} = 0.1$)		**Moderate** ($DI_{mod} = 0.2$)		**Severe** ($DI_{sev} = 0.5$)		**Collapse** ($DI_{col} = 1.0$)	
	$(S_{ae})_{m,min}$	β_{min}	$(S_{ae})_{m,mod}$	β_{mod}	$(S_{ae})_{m,sev}$	β_{sev}	$(S_{ae})_{m,col}$	β_{col}
2 story	0.405	0.572	0.550	0.456	0.905	0.305	1.210	0.270
3 story	0.385	0.657	0.515	0.535	0.833	0.322	1.100	0.195
4 story	0.355	0.748	0.485	0.588	0.775	0.418	0.975	0.316
5 story	0.325	0.757	0.455	0.623	0.685	0.428	0.925	0.292
6 story	0.285	0.778	0.415	0.628	0.585	0.448	0.875	0.344
7 story	0.255	0.748	0.395	0.628	0.535	0.496	0.775	0.401

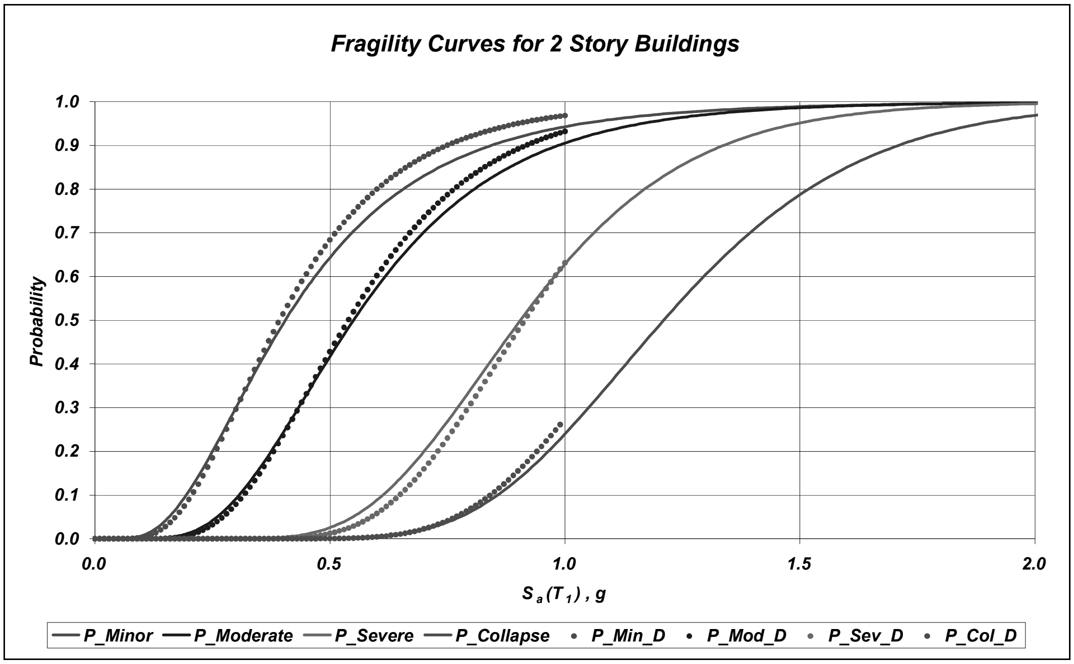

Figure 4.34. Vulnerability curves wrt $S_{ae}(T_1)$ for 2 story buildings
(dots indicate the discrete solution from the analysis)

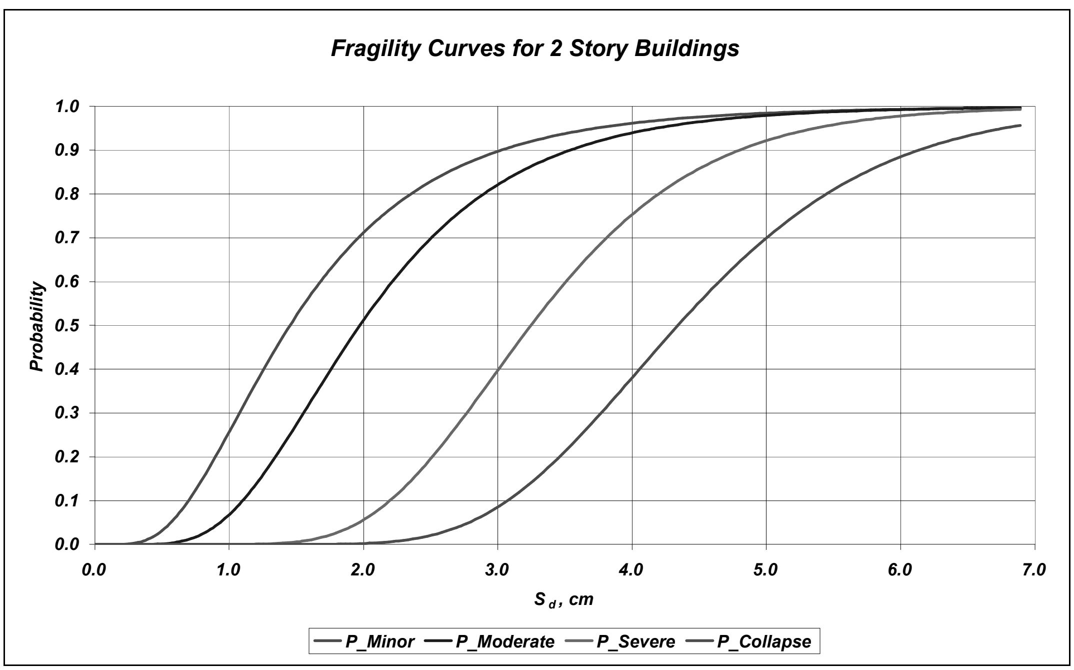

Figure 4.35. Vulnerability curves wrt $S_{de}(T_1)$ for 2 story buildings

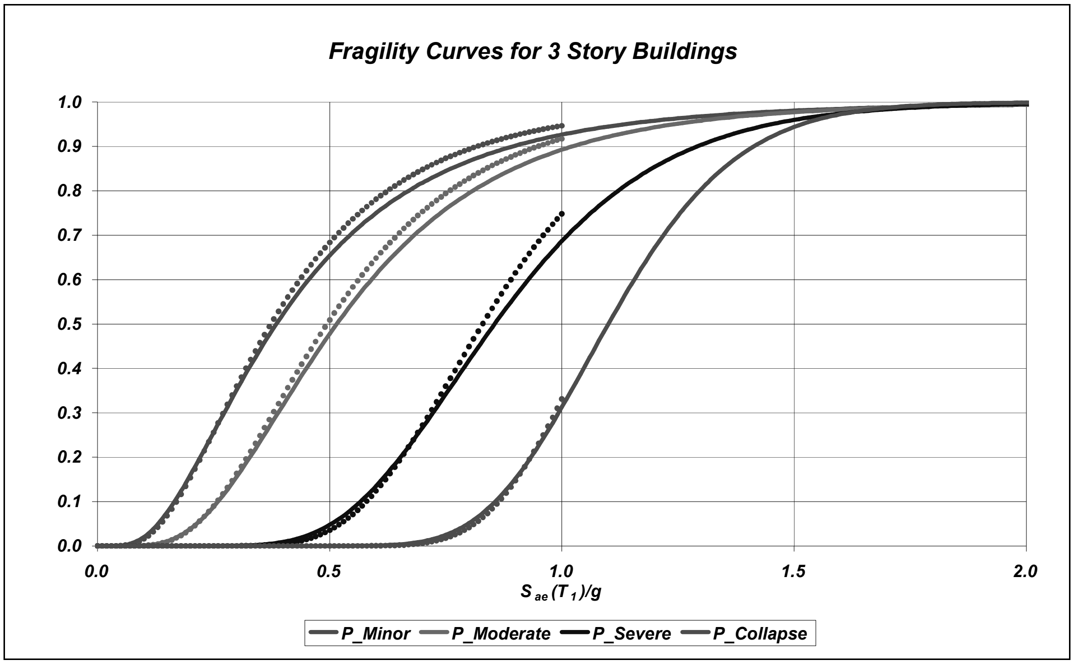

Figure 4.36. Vulnerability curves wrt $S_{ae}(T_1)$ for 3 story buildings

(dots indicate the discrete solution from the analysis)

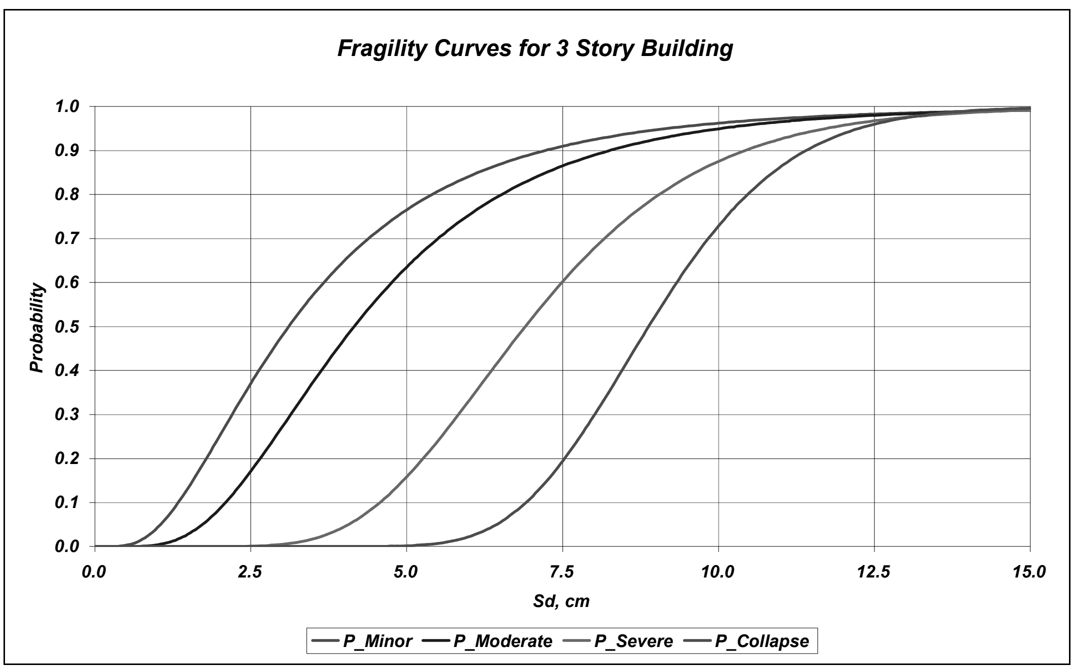

Figure 4.37. Vulnerability curves wrt $S_{de}(T_1)$ for 3 story buildings

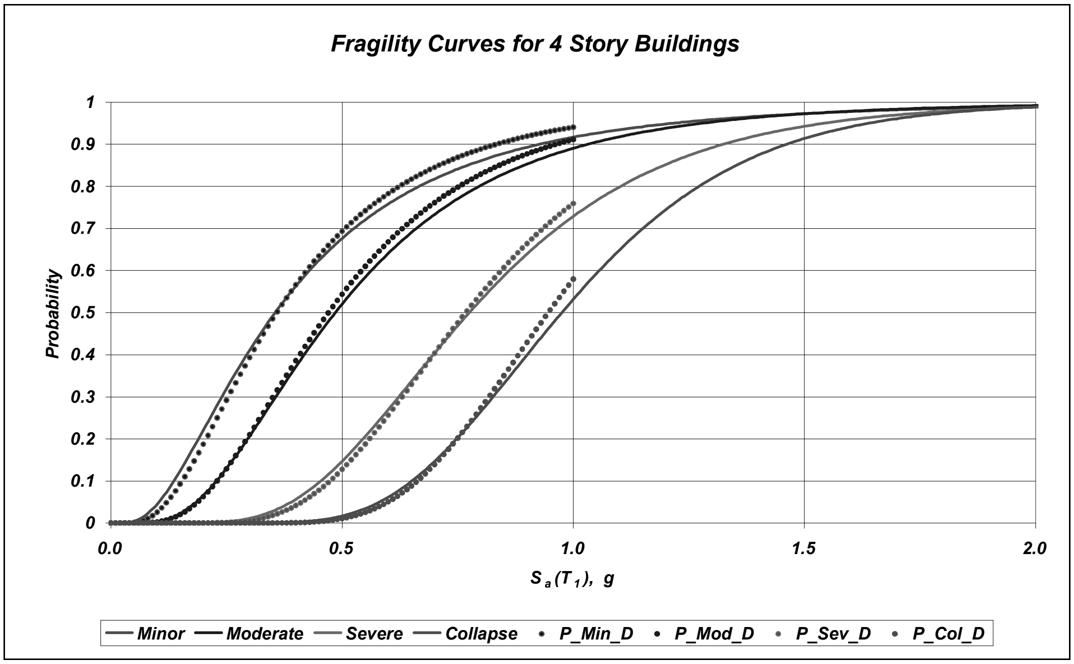

Figure 4.38. Vulnerability curves wrt $S_{ae}(T_1)$ for 4 story buildings

(dots indicate the discrete solution from the analysis)

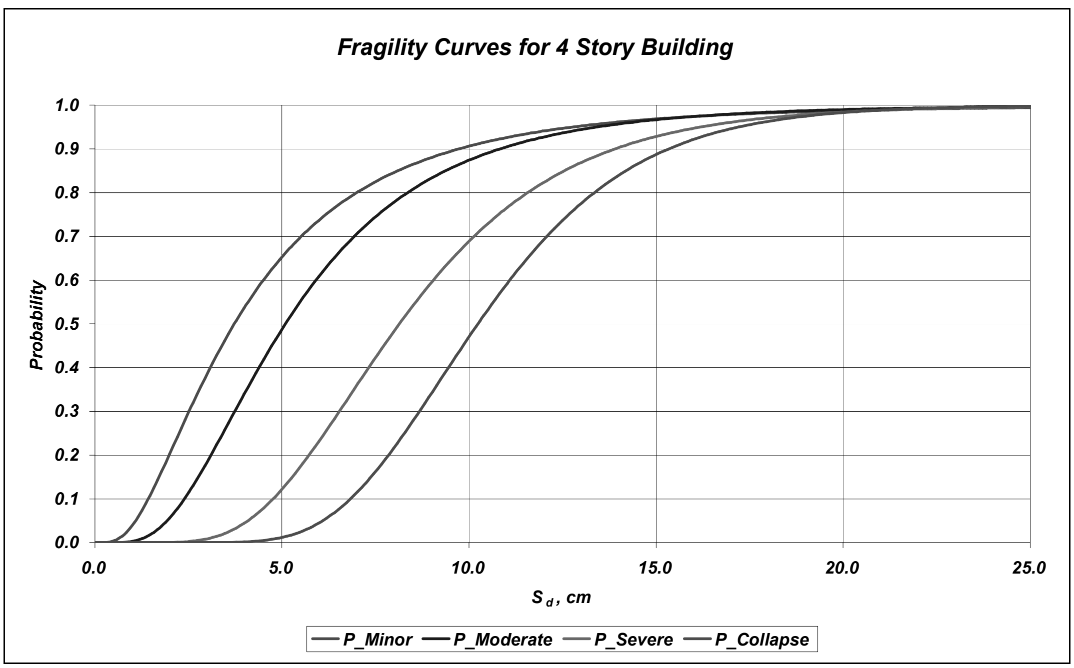

Figure 4.39. Vulnerability curves wrt $S_{de}(T_1)$ for 4 story buildings

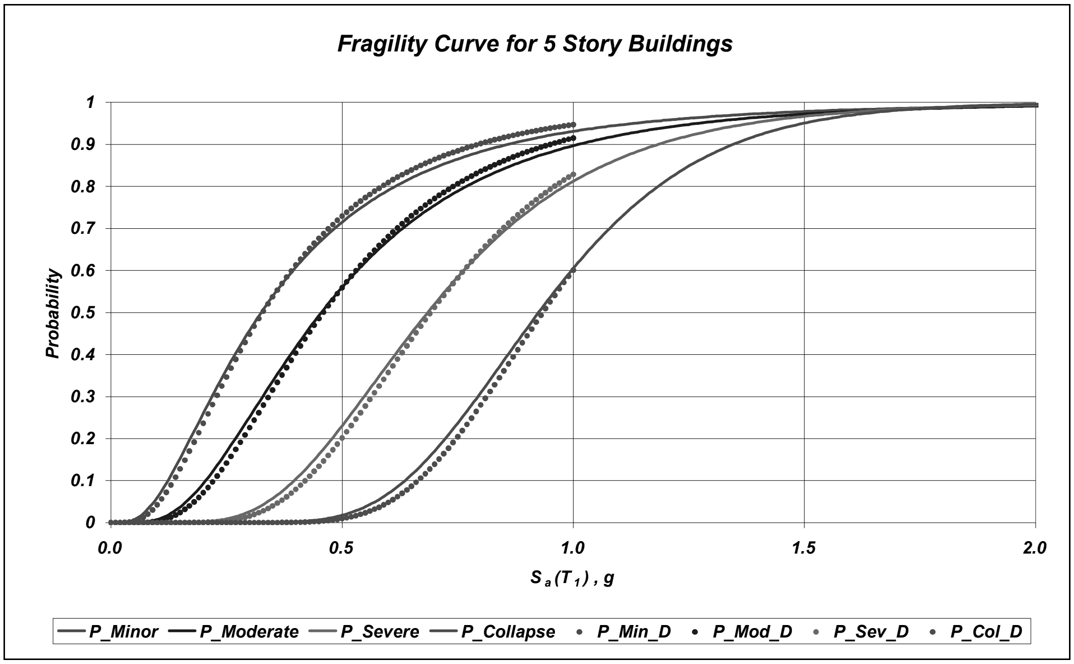

Figure 4.40. Vulnerability curves wrt $S_{ae}(T_1)$ for 5 story buildings
(dots indicate the discrete solution from the analysis)

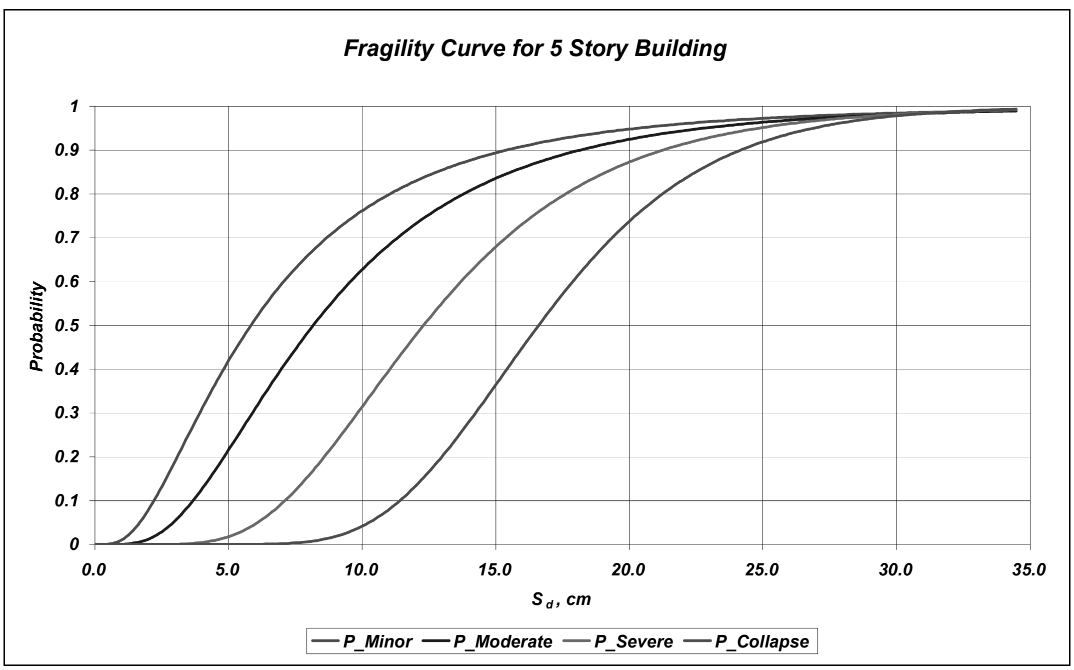

Figure 4.41. Vulnerability curves wrt $S_{de}(T_1)$ for 5 story buildings

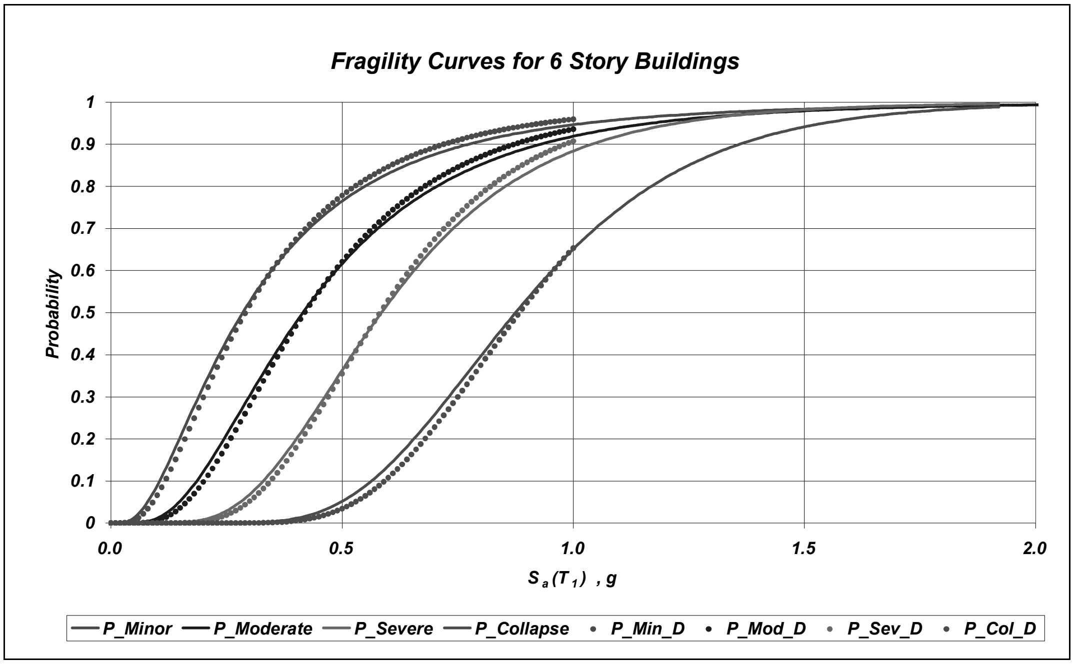

Figure 4.42. Vulnerability curves wrt $S_{ae}(T_1)$ for 6 story buildings

(dots indicate the discrete solution from the analysis)

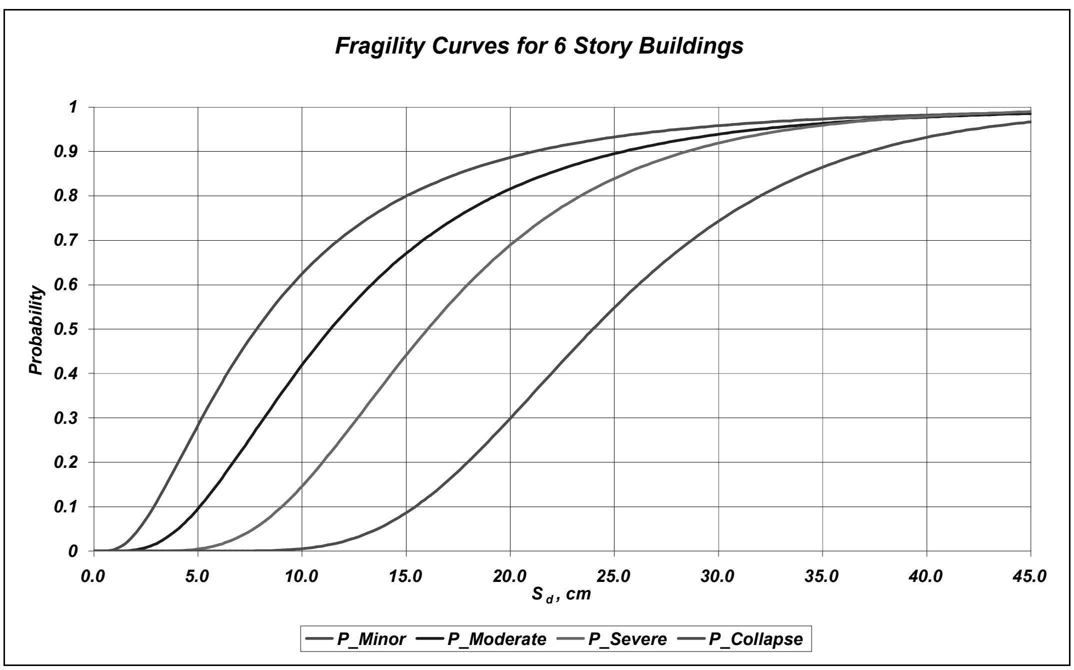

Figure 4.43. Vulnerability curves wrt $S_{de}(T_1)$ for 6 story buildings

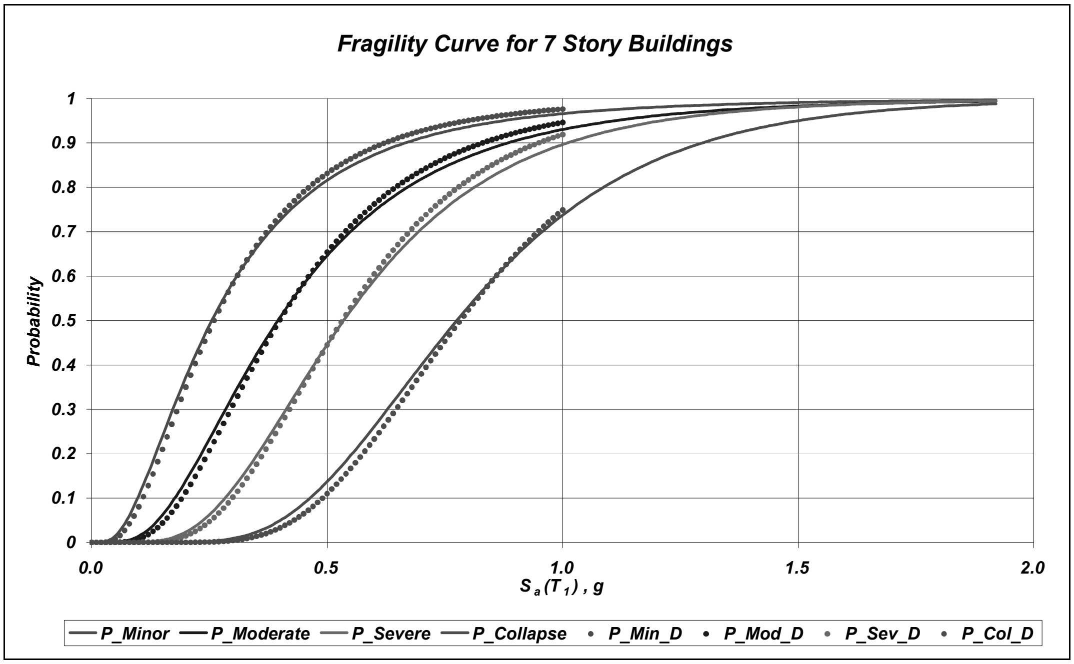

Figure 4.44. Vulnerability curves wrt $S_{ae}(T_1)$ for 7 story buildings

(dots indicate the discrete solution from the analysis)

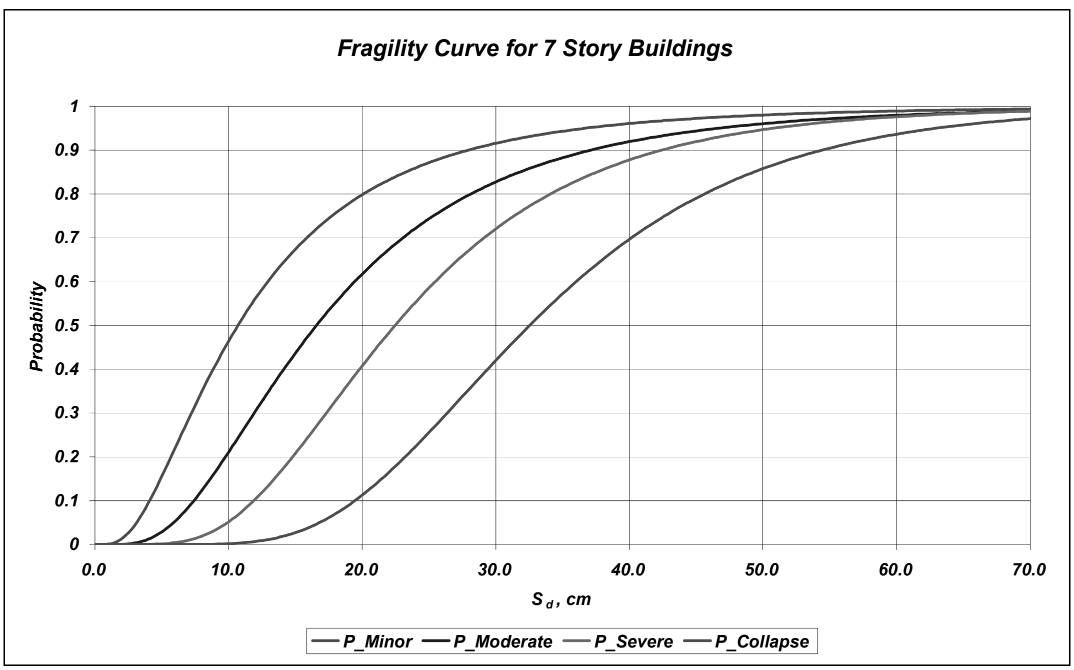

Figure 4.45. Vulnerability curves wrt $S_{de}(T_1)$ for 7 story buildings

4.7.5. Determination of the Confidence Intervals of Vulnerability Curves

Since the constructed vulnerability curves represent the mean values of the data at hand, it is essential to investigate to what extend or to what level of confidence the proposed curves are valid. In order to determine the confidence intervals of each vulnerability curves the statistical parameters of each distribution such as mean and standard deviation, is used for 95 per cent confidence interval.

A brief explanation of determination of confidence interval of mean of a distribution and its meaning for the population would be useful in order to clarify the process. Although there are different relationships for the determination of the mean, it is more meaningful to determine this parameter in terms of a confidence intervals in order to have an idea that how accurate is the estimation of the parameter. There are methods to evaluate this issue called the "confidence interval determination".

In order to determine interval estimation for a parameter "θ" of a population, an interval of $\hat{\theta} \pm k$ is introduced. This interval indicates the probability of any variable to be in the range of proposed interval. Depending on the predefined confidence level, range of the selected variable, $\hat{\theta}$, is determined with respect tothe mean value of the distribution. As an example the defining a probability, $P(\hat{\theta} - k < \theta < \hat{\theta} + k.) = 0.95$, the interval value "$k$" can be determined. This means that the probability of the selected variables in the population to contain mean value is 95 per cent. The determined level is called the "confidence interval" and the probability is called the "factor of safety".

Once the mean, "m", and the standard deviation, "σ", of a distribution with data number "n", is known, the confidence interval for the mean of the population, $\bar{X}$, can be determined by using a simple transformation is given in equation (4.9) as;

$$\frac{X - m}{\sigma / \sqrt{n}} \tag{4.9}$$

which yields a normal distribution denoted as $N(0,1)$.

For a defined confidence probability, α, the confidence interval of the mean of the population, $m \pm k$, can be determined by using equation (4.10).

$$P[\overline{x} - k_{\alpha/2}(\sigma / \sqrt{n} < m < \overline{x} + k_{\alpha/2}(\sigma / \sqrt{n})] = 1 - \alpha \tag{4.10}$$

The above relation is presented in (Figure 4.46) below.

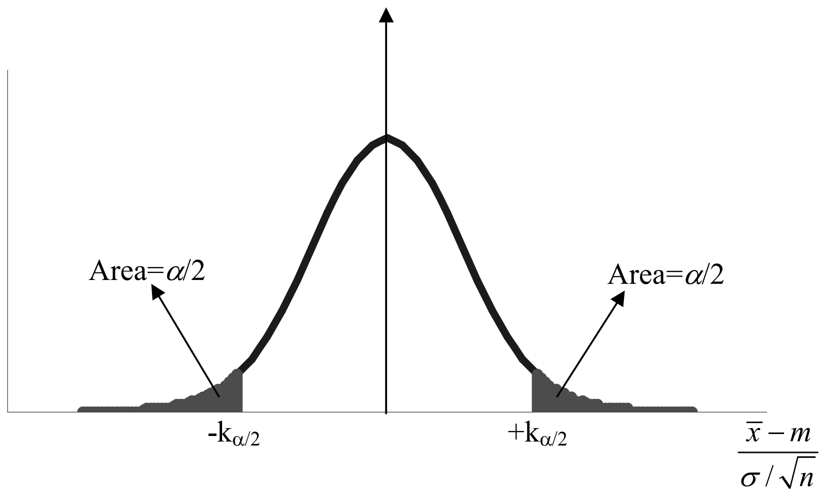

Figure 4.46. Graphical representation of the determination of the confidence interval of the mean

Depending on the procedure defined above, the 95 per cent confidence interval of mean of each damage state for each building class is determined. The obtained intervals are reflected to the plots of the curves. It can be seen that the intervals do not intersect each other very much, at least in the major portion of the spectral acceleration levels which is an indicator of proper separation of each damage state levels in by the given he threshold values. In the higher values of the spectral acceleration levels some some portions of the curves overlap each other where structures undergo to high nonlinear deformations in which damage state can rapidly change from one to another. The vulnerability curves with the defined confidence levels are given in Figure 4.47 to Figure 4.52.

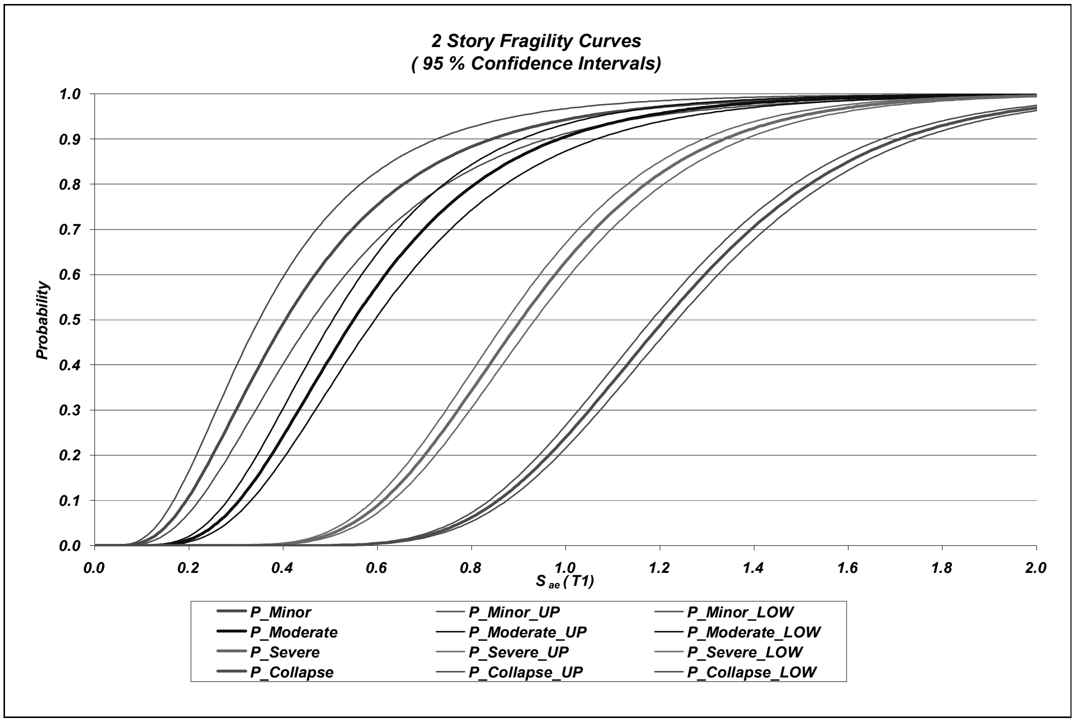

Figure 4.47. Vulnerability curves for 2 story buildings wrt $S_{ae}(T_1)$ with 95 % confidence intervals

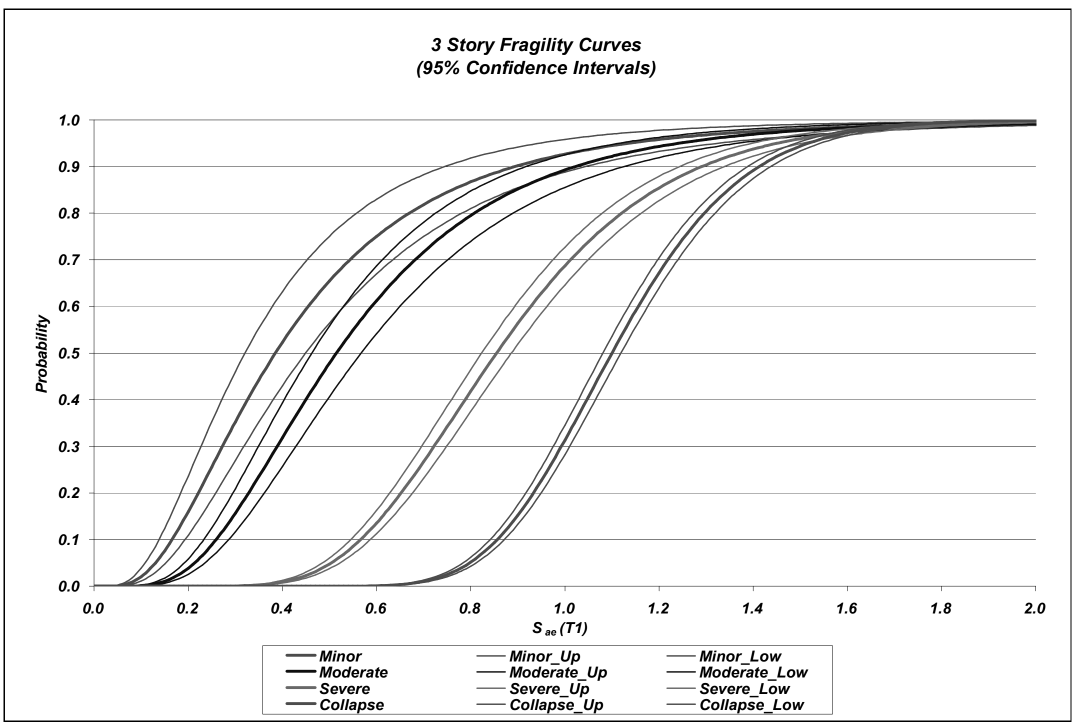

Figure 4.48. Vulnerability curves for 3 story buildings wrt $S_{ae}(T_1)$ with 95 % confidence intervals

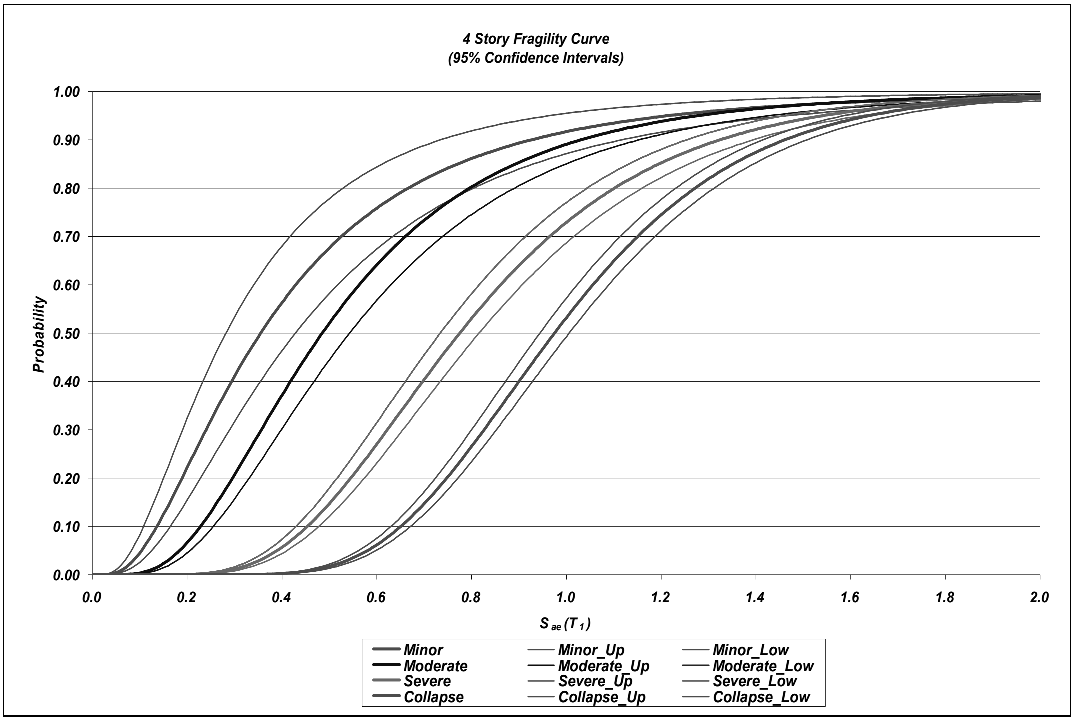

Figure 4.49. Vulnerability curves for 4 story buildings wrt $S_{ae}(T_1)$ with 95 % confidence intervals

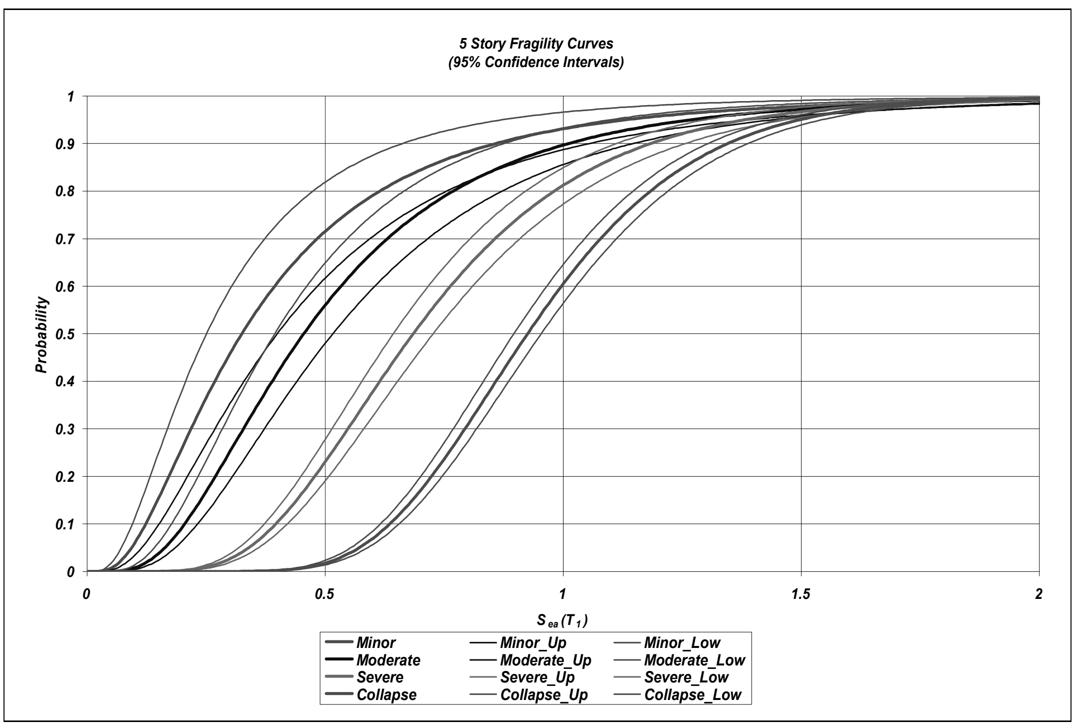

Figure 4.50. Vulnerability curves for 5 story buildings wrt $S_{ae}(T_1)$ with 95 % confidence intervals

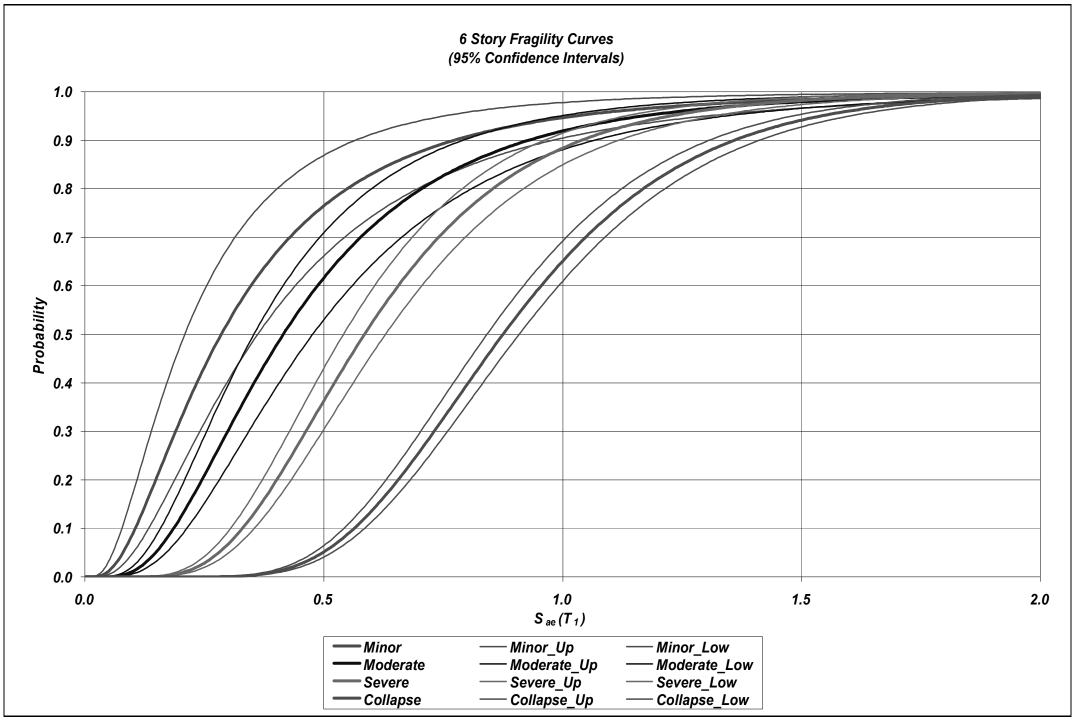

Figure 4.51. Vulnerability curves for 6 story buildings wrt $S_{ae}(T_1)$ with 95 % confidence intervals

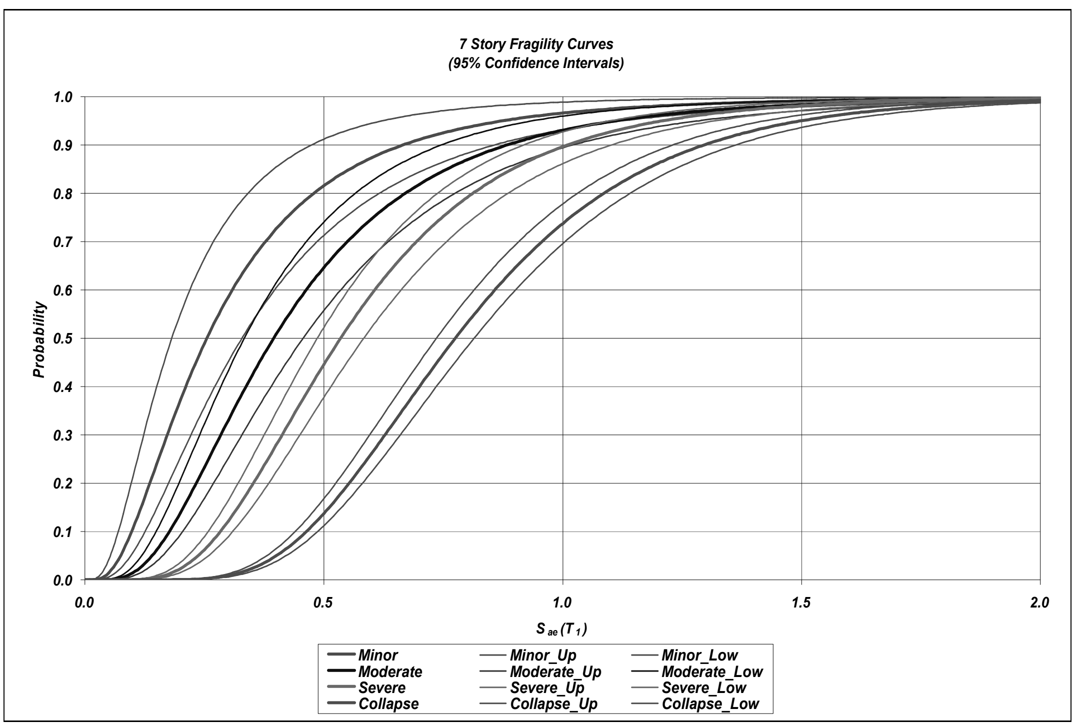

Figure 4.52. Vulnerability curves for 7 story buildings wrt $S_{ae}(T_1)$ with 95 % confidence intervals

5. CONCLUSIONS

The traditional approach adopted in developing vulnerability relationships is generally based on artificial simulation of the building data. On the contrary, a set of real reinforced concrete building design data is utilized in the present study, which represents the unique feature of the research effort undertaken. The data compiled from the city of Bolu in mid-western Turkey involves full structural analysis information including longitudinal and transverse reinforcement details to enable comprehensive nonlinear analyses of such buildings. Accordingly, as a result of an exhaustive analytical study involving 120 real buildings randomly selected from Bolu, typical fragility curves are generated for various damage states.

All selected buildings are framed structures, which are classified into six groups each containing 20 buildings ranging from two to seven stories. Thus, buildings are precisely classified with respect to their own number of stories, respresenting a major refinement compared to the traditional rough classification as low, mid and high-rise buildings, which corresponds to a relatively larger scatter in damage evaluation.

The building data can be considered as representative of local engineering capacities and construction practices of a typical Anatolian city resulting in typical reinforced concrete structures with inherent seismic characteristics of Turkish type building construction.

Levels of structural damage are identified in terms of Park and Ang (1985-a) damage index, and its threshold values are used to define the damage states. Using this damage index, it has been possible to evaluate the duration effect and the cyclic behavior of RC members under a given ground motion, since the damage in Park and Ang index is defined as a combination of ductility and cumulative hysteretic energy dissipation. On the other hand, the threshold values of the selected damage index have been calibrated for different damage levels by post earthquake damage surveys and significant number of laboratory tests performed on the reinforced concrete members.

Selected 20 ground motion records have been scaled with respect to fundamental-mode spectral acceleration of each building with 0.05g increments up to 1.00g. Nonlinear response history analyses are performed in the form of Incremental Dynamic Analysis (IDA) for 20 spectral acceleration levels of 20 ground motion records applied to 120 buildings, altogether making up 48000 computer runs. Analysis output is compiled in the form of IDA curves showing the variation of Park and Ang damage index with respect to spectral acceleration levels. Discrete data points are further increased for an enhanced resolution of statistical results by interpolating the IDA results for 0.01g increments.

Results of the statistical study are presented in the form of log-normal fragility curves, which are successfully fitted to the discrete output data obtained from IDA for specified damage state thresholds. The curves represent the cumulative probability of reaching or exceeding damage states, namely sight, moderate, severe and collapse damage states. The ground motion parameter is taken as the spectral acceleration of the building concerned in its fundamental mode. Complementary results are also presented where the ground motion parameter is taken as spectral displacement representing the elastic displacement demand of the building considered.

Based on unique features of real building data and the refined nonlinear analysis method combined with a realistic damage quantification procedure, the results of the present study are expected to be evaluated as a significant contribution to the existing knowledge base in urban loss estimation.

Although the analytical vulnerability assessment procedure proposed in the study cover a wide range of uncertainty of all the components of the methodology, three are still points to be evaluated in more detail. One of them is the classification of the building inventory. In this study the structural data is classified depending on the number of stories since the story number has an important effect on the structural response and so to the results. The classical classification of structures as low, mid and high-rise may be to rough for vulnerability analysis. The structures in the same category with different story numbers may have significantly different response and this may lead to a scatter in the results. Additionally the classification of the building data as low, mid and high-rise would not be consistent with precision of the other components of the study. It is recommended that building inventory to be classified based on number of stories for better representation of seismic vulnerability of existing structures.

It should be indicated that during the representation of seismic input where a spectrum compatible scaling procedure has been applied, some of the spectral acceleration levels existing in the actual records has been truncated as the result of the process. By doing so some of the structural response corresponding to that level of spectral acceleration is omitted. As a self criticism of the study, the motion input part could be modified in order to evaluate the effect of scaling procedure.

Although there are a lot of advances in the components of vulnerability assessment as mentioned above, there are still some points to be evaluated. One of them is to increase the number of types of the structures in the building inventory such as shear wall, dual systems in addition to frame systems, since the very recent structures are constructed in this manner and the newly constructed buildings become existing buildings after they have constructed. New methods should be introduced for the vulnerability assessment of the high-code buildings design in recent years not only in structural damage aspect but also in non-structural and economical aspects.

On the other hand the vulnerability of the structures which are under thread of near field effect of the ground motion must be evaluated and vulnerability assessment procedures must be developed. As all the components of the vulnerability assessment have a certain degree of uncertainty, it will be very useful to investigate degree of the effect of uncertainty in material and structural geometry on the results. In the case of analytical methods to be used in the assessment, the relationship between the ground motion parameters such as spectral acceleration and engineering parameters should be studied in detail for different structural types and different damage levels. This kind of a study would provide threshold values of damage levels in terms of well known engineering demand parameters such as building drift, inter-story drift etc.

As to the verification of the results obtained in the study, vulnerability relationships could be compared with the damage observed in the filed after 1999 Duzce earthquake in city of Bolu but there are limitations to archive this purpose. One of them is that the observed damage is available in terms of “intensity” or “PGA” while the proposed vulnerability relationships are in terms of spectral values. Since there is no relationship to correlate intensity to spectral values for that region, this comparison can not be made. Once a relationship is constructed another problem arises that the damage definitions concerning the observed data and analytical study is different. So it is very hard to correlate the calculated damage distribution with the observed survey for city of Bolu.

As a last point, in order to apply the vulnerability relationships for another city in Turkey, an additional vulnerability study should be performed in order to correlate the obtained results with two different approaches, since none of the approaches reveal satisfactory results. For a city like Istanbul with huge number of vulnerable building stock, the results of this study could be used when another complementary vulnerability assessment study performed at the same time in order to evaluate the effect of the components in vulnerability procedure on the obtained results. On the other hand this kind of a study could play a guide for improving the quality and properties of data at hand as well as the methodology used.

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