STRUCTURAL ENGINEERING RESEARCH UNIT

# TÜBİTAK

# SERU / YMAÜ

## Structural Engineering Research Unit
## Yapı Mühendisliği Araştırma Ünitesi

**REPORT NO. 2003 / 2**

# Seismic Vulnerability Assessment Procedure for Low- to Medium-Rise Reinforced Concrete Buildings


**Güney Özcebe, M. Semih Yücemen, Ahmet Yakut and Volkan Aydoğan**

**Department of Civil Engineering**
**Middle East Technical University**
**06531 Ankara, Turkey**


**October 2003**

# Seismic Vulnerability Assessment Procedure for Low- to Medium-Rise Reinforced Concrete Buildings

Guney Ozcebe, M. Semih Yucemen, Ahmet Yakut
and
Volkan Aydogan

Department of Civil Engineering
Middle East Technical University
06531 Ankara, Turkey
October 2003

# FOREWORD

This report introduces a new methodology for the preliminary seismic vulnerability assessment on a regional scale of low- to medium-rise reinforced concrete buildings. The model is based on a novel utilization of the discriminant analysis technique of multivariate statistics. The basic estimation variables are chosen among those structural parameters leading to inferior seismic behavior. The earthquake damage data compiled for the 12 November 1999 Duzce earthquake are used to develop a discriminant function in terms of these estimation variables. The preliminary testing of the proposed model by using the seismic damage data associated with recent earthquakes that occurred in Turkey supports the predictive ability of the proposed model.

The databases used in the analyses presented in this report are available at the SERU website. Readers may access these databases by following the "Archives" link an the SERU[1] homepage.

This study is part of the pioneering work initiated towards the seismic vulnerability assessment of existing buildings in Turkey and jointly sponsored by the National Scientific and Technical Council of Turkey (TUBITAK) through Grant No. ICTAG YMAU-I574 and NATO Science for Peace Programme through Grant No. NATO SfP977231. This document is disseminated under the sponsorship of TUBITAK.

SERU and the members of the project team gratefully acknowledge the financial assistance provided by TUBITAK and NATO.

Guney Ozcebe

SERU Executive Director

21 October 2003

[1] A TUBITAK research unit established on the premises of the Civil Engineering Department, Middle East Technical University, Ankara Turkey.
Website: http://www.seru.metu.edu.tr

# TABLE OF CONTENTS

# LIST OF FIGURES

# LIST OF TABLES

# CHAPTER 1: INTRODUCTION

## 1.1. GENERAL

During the last decade, six major earthquakes struck different regions of Turkey and all six earthquakes caused significant damage, collapse of thousands of structures and killed nearly 30000 people. These earthquakes revealed that existing structures in Turkey have many deficiencies regarding their seismic performances. Hence, evaluation of the seismic resistance and the assessment of possible damage are quite imperative in order to take preventive measures and reduce the potential damage to civil engineering structures and loss of human lives during possible future earthquakes.

Turkey is a country of high earthquake risk, such that 89% of the population, 91% of land and 98% of the industry are located in seismically active zones. Since there are thousands of existing buildings in high seismicity regions and they are highly vulnerable during a strong ground motion, identification of the defective structures and mitigation of their hazard with detailed analysis and inspection are very difficult, expensive and time consuming. For this reason reliable, economical and yet simple methods are inevitably necessary to determine the vulnerability of existing structures.

Current approaches in seismic vulnerability evaluation methods follow three main stages. These stages are namely; walk-down evaluation, preliminary evaluation and final evaluation. Evaluation in the first stage does not require any analysis and it relies on the past performance of similar buildings. The goal of the walk-down evaluation is to determine the priority levels of buildings that require immediate intervention. The preliminary evaluation covers the buildings that are designated to be inadequate in the first stage. In this stage, the simplified analysis is performed based on variety of methods. The time needed for a preliminary evaluation of a particular building is about three to four hours. The final evaluation of the structure, mainly based on further detailed seismic performance analyses, is to be carried out by an experienced design engineer. In the final evaluation stage, buildings that cannot be classified in the first two stages are considered. The time needed for final evaluation of a particular structure can range from couple of days to several weeks.

Random variabilities and various sources of uncertainties are involved both in seismic demand and seismic capacity of structures. Hence, the assessment of potential seismic damage should be performed based on statistical and probabilistic concepts.

## 1.2. REVIEW OF PREVIOUS STUDIES

In this section, the most common seismic evaluation methodologies and relevant past studies are briefly explained. Although, the contents of these studies vary, the common objective is to interpret and classify the observed damage in the aftermath of earthquakes.

### 1.2.1. Seismic Vulnerability Evaluation Methods

**Evaluation Method in United States [3, 4, 5]**

Stepwise evaluation process, consisting three evaluation steps, is proposed by Applied Technology Council (ATC). The evaluation steps are namely Rapid Visual Screening, Evaluation in Detail and Engineering Evaluation. The stepwise evaluation may stop at any step in case the structure under assessment is found to be adequate at that step. If the safety level of the structure is not clear, the consideration of the next step is required.

In the first stage named as rapid visual screening, rapid and easy identification of the building that might suffer severe damage and lead to the loss of human lives during a strong ground motion is aimed. The procedure is principally based on scoring of a structure, which consists of various structural scores and performance modifier factors. The second stage, evaluation in detail, is performed in order to determine whether there are any weak links in the structure that can cause component or structural failure. In this evaluation stage, lateral load carrying capacity of the structure is appraised by utilizing shearing stress check and drift check. The shearing stress check is based on the quick estimation of shearing stress in columns and structural walls, while drift check is mainly based on quick estimation of story drift considering relative rigidity of frame elements. The final evaluation stage, named as engineering evaluation, is carried out by an experienced design engineer in case the building is marked as seismically deficient and inadequate in the first two stages. The current design code is utilized in order to perform the further detailed structural analyses of building from "as built drawings". Final evaluation of the structure may need several weeks for each structure.

**Evaluation Method in Japan [23, 24]**

Ohkubo [23] and Otani [24] describe Japanese standard for the evaluation of the seismic vulnerability of the existing low-rise reinforced concrete buildings. Japanese design code requires large dimensions of columns and shear walls in order to provide substantial lateral load resistance. Since the estimation of deformation capacities of such large columns and shear walls is more difficult than the estimation of their strength capacities, the screening procedure in Japanese standards is mainly based on examination of story shear of columns and structural walls. The Japanese seismic evaluation standard, named as "Standard and Commentary for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings" [18], follows three levels of screening procedures with different phases. These procedures are applicable for low-rise reinforced concrete buildings less than six stories with or without structural walls. For all of three screening levels, a structural index, $I_S$, is defined with three sub-indices; namely basic structural performance sub-index, $E_0$, structural design sub-index, $S_D$, and sub-index on time dependent deterioration of the building, T. Then, the structural index is compared with the required seismic performance index, $I_{S0}$, based on the estimation of equivalent static earthquake force. In case the structural index is greater than the required seismic performance index, the building is found to be adequate and no further analysis is required. On the other hand, lower value of structural index requires additional study as a next level of screening.

### 1.2.2. Relevant Past Studies

**Hassan and Sozen [17]**

Hassan and Sozen [17] proposed a simplified method for the classification of low-rise monolithic reinforced concrete structures in a given region according to their seismic vulnerability. The main objective of the proposed method is to identify buildings that have high probability of severe damage during a strong ground motion. The ranking process basically depends on the total floor area of the building and cross-sectional areas of columns, shear walls and masonry walls. "Wall index" and "Column index" are calculated for both directions of a structure and the indices corresponding to the weaker direction are plotted such that x and y axes will represent column and wall indices, respectively. The plot indicates the relative seismic vulnerability of the buildings with respect to each other in a given region.

**Gulkan and Sozen [16]**
Gulkan and Sozen [16] proposed a methodology in order to estimate the seismic vulnerability of reinforced concrete frame buildings with masonry infills. The method requires only total floor area, cross-sectional dimensions of columns and masonry infill walls. It is mainly based on defining the ranking on a two-dimensional plot using column and wall ratios. Column ratio is defined as the ratio of the sum of column areas in a given direction at the base level of the structure to the total floor area. Similarly, the wall ratio is simply the ratio of the effective masonry infill wall area in a given direction at the base level to the total floor area.

**Ersoy and Tankut [13]**
In this study, a methodology for the seismic design of low-rise residential reinforced concrete buildings is proposed by Ersoy and Tankut [13]. Authors stated that no detailed structural analyses are required in case:

- The structure satisfies the minimum requirements for the dimensions of structural elements and the reinforcement ratios given in the "Specifications for Structures to be Built in Disaster Areas" [22].
- The minimum column areas, $\sum A_c$, and wall areas, $\sum A_w$, as suggested by the authors, are provided in both directions for the building under consideration.

**Pay [26]**
Pay [26] proposed a new methodology for the seismic vulnerability assessment of existing reinforced concrete buildings in Turkey. The proposed methodology was based on discriminant analysis technique and it was applied to the available seismic damage databases compiled from recent earthquakes in Turkey. Pay developed the method on the basis of the parameters that affect structural damage. The parameters included in the study were the number of stories, overhang ratio, soft story, redundancy, and square root of sum of squared moment of inertias. In the study, two parameters, overhang ratio and soft story, were found to be statistically insignificant. Therefore, the statistical analysis was based on the remaining three parameters.

**Askan [6]**
Askan [6] proposed three different stochastic approaches in order to estimate potential seismic damage to existing reinforced concrete buildings in Turkey. First, damage probability matrices for each seismic zone were derived by considering the available damage databases and expert opinion. As a second methodology, Askan proposed a reliability-based model, in which seismic demand and seismic capacity were taken as random variables. This model requires total floor area, cross-sectional areas of columns and walls on the ground floor, an approximate estimation of the fundamental period of the structure and local soil conditions. Askan [6] utilized a discriminant analysis technique as the third methodology in order to estimate the seismic vulnerability of reinforced concrete structures. The estimation parameters included in the study were number of stories, normalized square root of sum of squares of inertias (SRSSI), soft story, overhang, redundancy, density ratio and floor regularity factor. Finally, in order to compare the damage estimations obtained from these three methods, the results are expressed in terms of mean damage ratios and compared with each other. It is concluded that at any certain intensity level, the two empirical models, the damage probability matrices and the discriminant analysis method, gave mean damage ratios

that are very close to each other. However, the reliability-based model underestimated the damage rates at high intensities.

### 1.2.3. Objective and Scope of Study

The objective of this study is to develop a new statistical model to predict the seismic vulnerability of existing reinforced concrete buildings. In the development of a new methodology, particular emphasize is given on the identification of the structures that could suffer severe damage or collapse during a strong ground motion resulting in the loss of human lives. The basic aim of the analysis is to predict the preliminary seismic vulnerability of reinforced concrete buildings based on a number of engineering parameters, which are selected based on engineering judgment and observations.

The proposed method is based on one of the multivariate statistical techniques, called as the discriminant analysis. All statistical analyses for the execution of discriminant analysis is carried out by using the program SPSS 11.0 [28], which is a licensed software in METU and is commercially available. The method requires a number of engineering parameters that are correlated with structural damage and discriminates various damage states through a discriminant function, which is a linear function of the dominant damage inducing parameters. The study presented in this report includes only the low-rise to mid-rise reinforced concrete structures, with or without shear walls. This report is organized in the following manner:

In Chapter 1, general information on the purpose of the study is given and the objective of the study is explained. Later, the summaries of the most relevant studies related to the potential seismic damage prediction are presented.

In Chapter 2, main reasons of seismic damage in reinforced concrete buildings are briefly described. Then, the damage database used in the formation of the proposed methodology is introduced. Finally, the damage-inducing parameters that are the inputs of discriminant analysis are derived.

In Chapter 3, the proposed statistical models for two-damage states are presented and implemented for the estimation of potential seismic damage. The optimal classification technique for the two damage state groupings is also introduced.

In Chapter 4, the validity of the proposed methodology is checked by using the different damage databases compiled from recent earthquakes and the effectiveness of the proposed method is compared by the other methodologies proposed previously by different researchers.

Chapter 5 involves the modification of the proposed statistical method using site characteristics. Later, the application of the modified method to the scenario earthquake in Istanbul is presented. Chapter 6 includes the summary and main conclusions of the study.

The summary of the work presented in this report, along with the important findings, the conclusions and the recommendations for the future investigations are presented in Chapter 6.

# CHAPTER 2: DESCRIPTION OF THE SEISMIC DAMAGE DATABASE AND IDENTIFICATION OF THE DAMAGE INDUCING PARAMETERS

## 2.1. GENERAL

Earthquake force has one important and distinct property to be mentioned; that is its high uncertainty. All stages of earthquake vulnerability assessment, ranging from seismic hazard analysis to determination of a structure's response, include uncertainty.

Earthquake design principles, which limit the possible seismic vulnerability, are introduced with the development of modern earthquake codes. The general principle of earthquake resistant design according to "Specifications for Structures to be Built in Disaster Areas in Turkey" [22] is to prevent structural and non-structural elements of a building from any damage when it is subjected to a low-intensity earthquake; to limit the damage in structural and non-structural elements to repairable levels when it is subjected to a medium-intensity earthquake, and to prevent the overall or partial collapse of a building when it is subjected to a high-intensity earthquake, so that the loss of life is avoided.

In order to satisfy the general design principle stated above, there exist three main requirements of the earthquake resistant design concept:

- Strength requirements
- Ductility requirements
- Stiffness requirements

All three requirements of seismic resistant design are interrelated. These main requirements are to be satisfied by many provisions of the Code [22]. In addition, the seismic resistance of a structure may not be achieved by satisfying these requirements. The seismic performance of the structure is influenced by the characteristics of the seismic motion, properties of soil profile, and overall dynamic properties of the structure [11, 14].

In this chapter, reasons of damages to the reinforced concrete structures will be explained briefly and the structural data collected after 12 November Duzce Earthquake will be discussed.

## 2.2. REASONS OF DAMAGE IN REINFORCED CONCRETE STRUCTURES

The observations made after earthquakes have indicated that main causes of damage in reinforced concrete buildings can be classified into three groups:

- Improper configuration of architectural and structural systems
- Poor and inadequate detailing and proportioning
- Poor construction quality due to inadequate supervision

Almost 90 percent of the observed earthquake damage in reinforced concrete structures in Turkey during the past 30 years is due to one of the listed mistakes or combination of them [12, 14]. These deficiencies will be discussed briefly in the following paragraphs.

### 2.2.1. Building Configuration and Structural Systems

The deficiencies due to improper choice of architectural and structural systems can be listed as follows:

**Discontinuity in the Vertical Elements**
The load distribution between stories is not regular if the vertical load carrying members, i.e. columns or shear walls, are removed at some stories and supported by beams, or the structural walls of upper stories are supported by columns or beams underneath. This type of deficiency results in excessive loads in lower structural members and it can cause unexpected member failures.

**Sudden Changes in Stiffness and Strength**
In case sudden changes in stiffness or strength either in the elevation or in the plan of the structure exist, the distribution of the lateral loads entirely differs from those in uniform structures. Setbacks, changes in structural system over the height of building, changes in story height, unexpected participation of non-structural elements and changes in material properties are the main reasons of the sudden changes in stiffness or strength. Common problem of such discontinuities is the concentration of high inelastic deformations around the level of discontinuity.

**Short Columns**
Partial-height frame infills, which do not stand along the full height of the column, decrease the unsupported length of the column and lead to the formation of “short column”. The shear required to develop flexural yielding in the short column is considerably higher than the shear required to develop flexure yielding in full length of the column. Since this short column formations are generally not considered in design stage, short column fails in shear, which is quite a brittle failure, before flexural yielding and causes severe damage.

**Soft Story**
This deficiency is related to the lack of lateral stiffness resulting in very large deformations. In case lateral stiffness provided by the vertical members at any particular floor is appreciably less than the upper or lower neighboring floor, that story acts as a “soft story”. The main problem of this floor is excessive interstory drift. This excessive interstory drift causes damage both in structural and non-structural elements and leads to instability of structural system. If the structure is extremely massive and flexible, excessive second order moments resulting in heavy damage or collapse can be developed. Soft story formation is among the common defects of the existing building stock in Turkey.

**Weak Story**
A weak story can be formed when the strength of lateral load carrying system of a particular floor is significantly smaller than that of the upper neighboring floors. The formation of weak story can result in the partial or total collapse of the structure.

**Pounding / Hammering**
Significant damage can be developed due to the pounding of two neighboring structures or two adjacent blocks of a structure. The seismic joints between the neighboring structures or the neighboring blocks of a structure should be sufficient and wide to prevent the collision caused by their different sway patterns.

**Redundancy**
Structural systems that have many lateral load-resisting elements are observed to perform well during earthquakes. Redundancy in the structure ensures redistribution of the earthquake-induced lateral forces within the structural system. That is, if any member in the lateral load carrying system fails for any reason, the remaining members of the structural system can provide necessary lateral load resistance to prevent total collapse of the system. Redundancy of a structure can be defined as the indication of the degree of the continuity of multiple frame lines to distribute lateral forces throughout the structural system.

**Torsional Irregularity**
Unless the center of rigidity and the center of mass of a structure coincide, the structure under ground excitation is subjected to a coupled effect called floor torsion. When there is an eccentricity between these two centers, additional excessive shear forces on the outermost vertical elements are created. In order to prevent floor torsion, the vertical load carrying elements and non-structural infills should be proportioned and arranged symmetrically in the plan of a structure.

**Overhangs**
In case the area of the ground floor is noticeably smaller than the area of typical intermediate story, the probability of inferior seismic response of the structure increases. The load carrying elements of the framed structures with higher amount of overhang areas are susceptible to considerably large disturbances due to the existence of elevated masses.

**Weak Column-Strong Beam**
When the beams are strong enough to force the hinging in columns, plastic hinges at column ends can result in story mechanism and high inelastic deformations in a story level. These inelastic deformations may lead to instability of the frame due to amplified second order moments. In order to avoid story mechanism, creating instability of whole structure, weak column-strong beam formations should be prevented.

### 2.2.2. Detailing

The basic principle of detailing is to provide the necessary strength and ductility at critical sections of structural members and at beam-column joints. Inadequate anchorage or splice length and inadequate confinement at these critical regions are the main reasons of damage related with detailing. Insufficient transverse reinforcement results in inadequate flexural rigidity. Since the members cannot dissipate excessive seismic energy without sufficient ductility, brittle failure occurs in the members and partial or total collapse of the structure could not be prevented. Another important criterion in detailing is to provide sufficient splice length to develop the yield strength of spliced bar at critical regions. In case the longitudinal bars in columns are spliced at the floor level without enough splice length, yielding cannot be reached and brittle failure occurs in the critical regions.

### 2.2.3. Construction

Although the earthquake resistant design principles are applied at the design stage, many unpredicted failures can be observed due to the mistakes made at construction stage. These mistakes can be grouped as [11]:

- Different member sizes than the ones in design drawings,
- Different reinforcement detailing than the detailing shown in design drawings,
- Lower material quality than the quality taken in design calculations,
- Addition of some structural or non-structural members, which are not considered in design stage,
- Addition of extra stories, which are not considered in design stage.

## 2.3. COLLECTION OF EARTHQUAKE DAMAGE DATA

After the 17 August Kocaeli Earthquake, having a moment magnitude of 7.4, and the 12 November Duzce Earthquake, with a moment magnitude of 7.2, two reconnaissance studies were carried out. The first field survey was performed by more than 20 researchers and engineers immediately after the Kocaeli Earthquake. In this survey, 152 among 854 low-rise to medium-rise monolithic reinforced concrete structures with one to six stories were studied in detail. Floor plans were generated and the observed damage was documented for each one of these buildings.

The second survey was carried out by the METU teams during the year 2000. In the second survey, 484 low-rise to medium-rise monolithic reinforced concrete buildings ranging from two to six stories were considered. Details of the second survey will be presented in the following sections. The statistical analyses in this study were carried out on the Duzce damage database consisting of these 484 reinforced concrete buildings examined during the second survey. It should be noted that all buildings forming the available database are located in the highest seismic zone of Turkey and they were subjected to strong ground motions of a level above that envisaged in the Specifications for Structures to be Built in Disaster Areas in Turkey [22] for the maximum design earthquakes. Moreover, post earthquake geotechnical surveys made in Duzce indicated that the city of Duzce is located on a rather uniform alluvial deposits composed of sand, silt and clay layers forming the top 20 meters of the soil profile.

### 2.3.1. Data Collection Sheets

In the second reconnaissance study, the information for the buildings was collected by filling out the three-page data collection sheets. The information collected using these data collection sheets can be grouped into four:

**General Information**

- Survey date
- Street address of the building
- GPS coordinates of the building
- The design and construction date of the building
- Name of survey team
- Damage level of the building

The damage states are classified as “none”, “light”, “moderate”, “severe” or “collapsed” to reflect the observed damage level of the structure. Damage levels are defined as follows:

None: No visual sign of damage to the structural and non-structural elements.

*Light*: Hairline flexural or inclined cracks in structural elements and flaking of plaster in non-structural elements.

*Moderate*: Inclined cracks in structural elements, spalling of concrete at beam-column joints, and flaking of large pieces of plaster in non-structural elements.

*Severe*: Wide inclined cracks in structural walls or columns, local structural failures, and wide or through cracks in non-structural walls.

*Collapsed*: Local or total collapse of the structure.

**Information on Architectural System**

- Number of stories
- Story heights
- Floor areas
- Irregularities both in plan and elevation as described in the Code [22]
- Location with respect to adjacent buildings
- Information on Structural System
- Floor plan of the ground story
- Dimensions of structural and non-structural elements
- Dimensions of upper stories
- Type of load carrying system
- Number of continuous frames in both directions
- Material of infill walls
- Material of structural walls in the basement (if exist)
- Floor system
- Material quality summaries (for concrete and reinforcement)

**Information on Workmanship and Quality**

- Some grading questions regarding the quality and workmanship of the structure to be answered by the surveyor

### 2.3.2. Classification of the Collected Data[†]

The Duzce damage database, which is used in this study, is classified according to the number of stories and observed damage states as shown in Table 2.1 and Figure 2.1. According to Figure 2.1, the distribution of the buildings according to damage levels is as follows:

None: 61 buildings

Light: 150 buildings

Moderate: 151 buildings

Severe: 58 buildings

Collapsed: 64 buildings

[†] This data base is available at http://www.seru.metu.edu.tr/archives/databases/duzce_database.xls.

Moreover, some general information related to the damage database is as follows:

- Distribution according to the existence of shear walls:

  No shear wall: 367 buildings

  Shear wall in one direction: 62 buildings

  Shear wall in both directions: 55 buildings

- The story heights vary from 2.20 m to 5.30 m. Most of the story heights are between 2.80 m and 3.00 m. In addition, 281 buildings have soft story formations.
- In general, buildings have inadequate number of continuous frames for resisting the lateral loads in either direction. Also, lateral reinforcement details are generally poor. Concrete qualities of buildings vary from 8 MPa to 35 MPa.
- 385 buildings have overhang formations.
- Number of stories varies from one to six.
- Most of the buildings have irregularities as defined in the Turkish Seismic Code [22].

**Table 2.1.** Classification of the Duzce Damage Database According to Number of Stories and Damage States

| **Number of Stories** | **Number of Buildings** | **Observed Damage States** | | | | |
|---|---|---|---|---|---|---|
| | | None | Light | Moderate | Severe | Collapsed |
| **2** | **23** | 7 | 13 | 3 | 0 | 0 |
| **3** | **122** | 18 | 62 | 29 | 13 | 0 |
| **4** | **157** | 17 | 44 | 59 | 22 | 15 |
| **5** | **168** | 17 | 31 | 56 | 21 | 43 |
| **6** | **14** | 2 | 0 | 4 | 2 | 6 |
| **Total** | **484** | **61** | **150** | **151** | **58** | **64** |

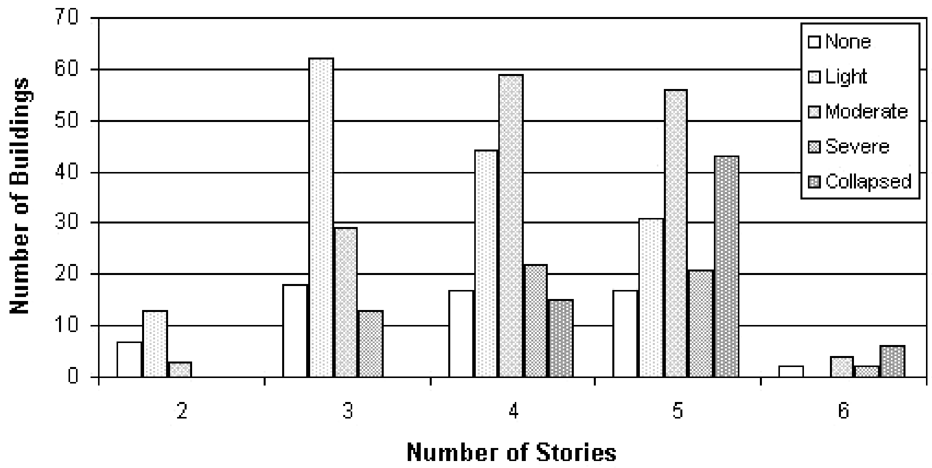



**Figure 2.1.** Classification of the Duzce Damage Database According to Number of Stories and Damage States

## 2.4. INTERPRETATION OF THE SEISMIC DAMAGE DATA

The effects of different parameters on seismic damage vary significantly. From the Duzce damage database, parameters related to number of stories, lateral rigidity, lateral strength, soft story, redundancy and overhang area can be obtained. These parameters are used as the basic damage inducing variables for the preliminary seismic assessment model described in Chapter 3.

Structural members, i.e. columns, shear walls and masonry walls, are defined as follows:

- *Column*: a vertical reinforced concrete member with a maximum cross-sectional dimension to minimum cross-sectional dimension ratio less than seven.
- *Shear Wall*: a vertical reinforced concrete wall with a maximum cross-sectional dimension to minimum cross-sectional dimension ratio greater than or equal to seven.
- *Masonry Wall*: an unreinforced masonry infill wall with no window or door openings and supported by columns or shear walls at its both ends.

In addition to the above definitions, all structural walls around the elevator shafts, whether they satisfy the required height to width ratio or not, are considered as shear walls. The separation of shear walls into the areas, both in x and y directions, is shown in Figure 2.2.a. For the vertical elements with unusual cross-sections like in Figure 2.2.b, Equation 2.1 given below is used to determine whether the member is to be classified as shear wall or column. In other words, if Equation 2.1 is satisfied, the member is classified as a shear wall; otherwise it is treated as a column.

$$\frac{l_1 + l_2 + l_3}{t} \geq 7 \quad (2.1)$$

where $l_1$, $l_2$ and $l_3$ are the corresponding lengths of horizontal, inclined and vertical parts in Figure 2.2.b.

The rotated members, whose axes do not coincide with x and y axes, are separated into two members as shown in Figure 2.2.c according to Equations 2.2.a, 2.2.b, 2.3.a and 2.3.b. For both members, the thickness is taken as the original thickness of the rotated member.

$$l_{x1} = l_x' \quad (2.2.a)$$

$$l_{y1} = t \quad (2.2.b)$$

$$l_{x2} = t \quad (2.3.a)$$

$$l_{y2} = l_y' \quad (2.3.b)$$

where;

l: length of the rotated member shown in Figure 2.2.c

t: thickness of the rotated member shown in Figure 2.2.c

$l_{x1}$: length of the first member shown in Figure 2.2.c

$l_{y1}$: thickness of the first member shown in Figure 2.2.c

$l_{y2}$: length of the second member shown in Figure 2.2.c

$l_{x2}$: thickness of the second member shown in Figure 2.2.c

$l_x'$: length of x-component of the rotated member

$l_y'$: length of y-component of the rotated member

The moment of inertias of the vertical members with respect to their centroids are calculated at the ground level according to Equations 2.4.a and 2.4.b.

$$I_x = \frac{1}{12} \cdot l_x \cdot l_y^3 \tag{2.4.a}$$

$$I_y = \frac{1}{12} \cdot l_y \cdot l_x^3 \tag{2.4.b}$$

where;

$l_x$: length of the member in x-direction as shown in Figure 2.2.d

$l_y$: length of the member in y-direction as shown in Figure 2.2.d

$I_x$: moment of inertia with respect to x-axis

$I_y$: moment of inertia with respect to y-axis

The areas of the vertical members, i.e. columns, shear walls and masonry walls, are calculated at the ground level using Equations 2.5, 2.6 and 2.7, respectively.

$$(A_{col})_x = k_x \cdot A_{col} \tag{2.5.a}$$

$$(A_{col})_y = k_y \cdot A_{col} \tag{2.5.b}$$

where;

$A_{col}$: area of the column

$(A_{col})_x$: area of the column in x-direction

$(A_{col})_y$: area of the column in y-direction

$k_x$ and $k_y$: constants which define corresponding areas in x and y directions, respectively. These values shall be taken as follows:

$k_x = 1/2$ for square/circular columns

$k_x = 2/3$ for rectangular columns $l_x > l_y$

$k_x = 1/3$ for rectangular columns $l_x < l_y$

$k_y = 1 - k_x$

$$(A_{sw})_x = k_x \cdot A_{sw} \tag{2.6.a}$$

$$(A_{sw})_y = k_y \cdot A_{sw} \tag{2.6.b}$$

where;

$A_{sw}$: area of the shear wall

$(A_{sw})_x$: area of the shear wall in x-direction

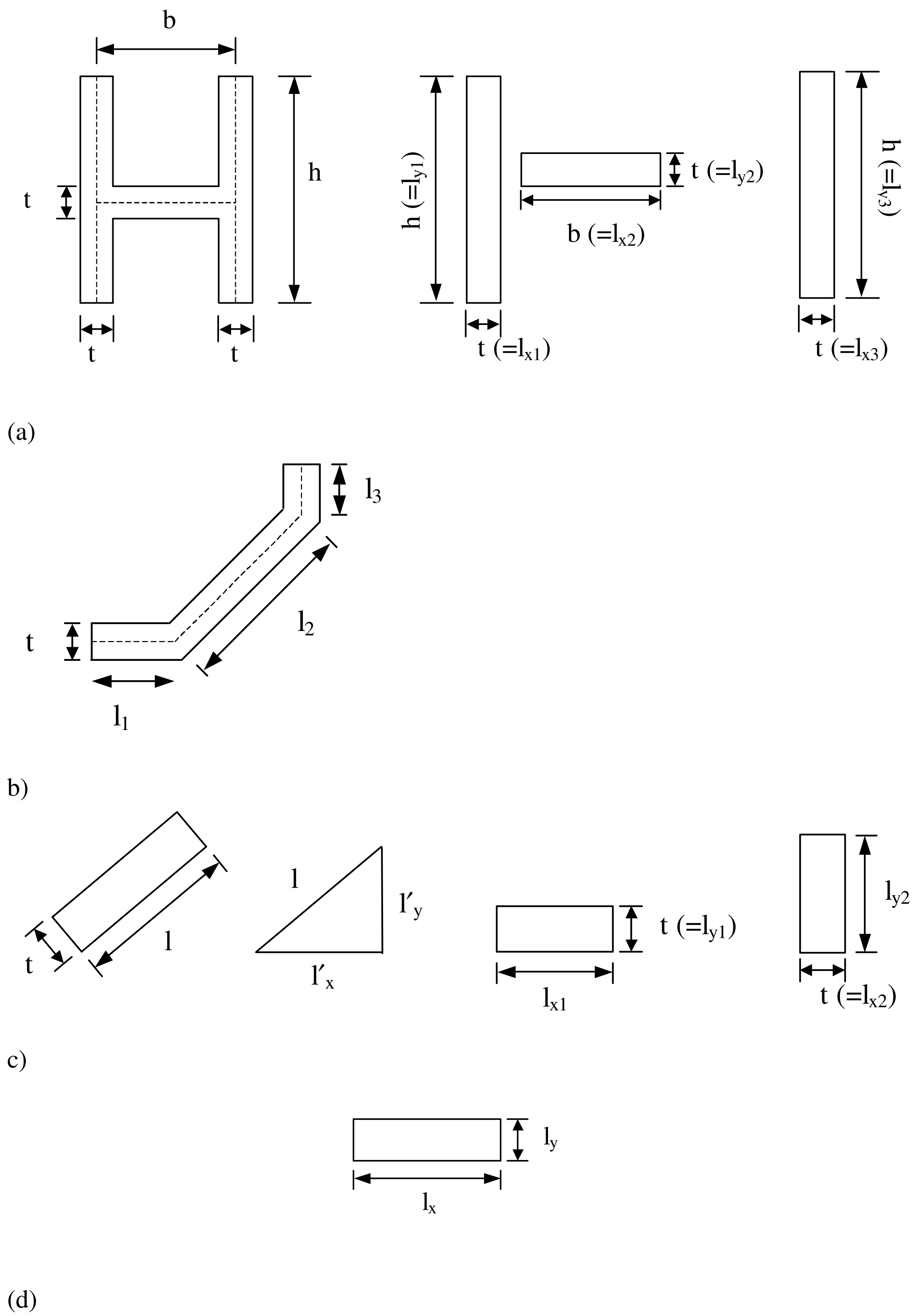



**Figure 2.2.** Definitions of the Geometrical Parameters for Cross-Sections

$(A_{sw})_y$: area of the shear wall in y-direction

$k_x$ and $k_y$: constants which define corresponding areas in x and y directions, respectively

$k_x = 1$ for shear wall in the direction of x-axis

$k_x = 0$ for shear wall in the direction of y-axis

$k_y = 1 - k_x$

$$(A_{mw})_x = k_x \cdot A_{mw} \tag{2.7.a}$$

$$(A_{mw})_y = k_y \cdot A_{mw} \tag{2.7.b}$$

where;

$A_{mw}$: area of the masonry wall

$(A_{mw})_x$: area of the masonry wall in x-direction

$(A_{mw})_y$: area of the masonry wall in y-direction

$k_x$ and $k_y$: constants which define corresponding areas in x and y directions, respectively

$k_x = 1$ for masonry wall in the direction of x-axis

$k_x = 0$ for masonry wall in the direction of y-axis

$k_y = 1 - k_x$

The total area is defined as the total area of individual floor systems above the ground level at which calculations of moments of inertias and the calculation of member areas are performed.

### 2.4.1. Definition of the Parameters Considered in Vulnerability Evaluation

In determining the estimation variables to be used in the analysis, the basic assumption is that all of the buildings involved in each inventory are exposed to a specific earthquake. Post earthquake geotechnical surveys conducted in Duzce indicated that Duzce is founded on uniform alluvial deposits composed of sand, silt and clay layers. Moreover, the mean distance from the province to the fault is approximately 8 km. Thus, it can be assumed that each building stock in itself has faced almost the same ground motion properties. Hence, the damage will be evaluated only on the basis of structural responses rather than including the excitation parameters.

Since the methodology is constructed on the structural response of buildings, it is advantageous to consider the deficiencies that are present in architectural and structural systems of the damaged buildings involved in the damage database. Thus, six parameters reflecting these deficiencies were defined. These parameters are; number of stories, minimum normalized lateral stiffness index, minimum normalized lateral strength index, normalized redundancy score, soft story index and overhang ratio [7]. In the next chapter, these six parameters will be used to develop a statistical methodology for the preliminary seismic vulnerability assessment of the existing reinforced concrete structures. These parameters are described below.

**Number of Stories**

The observations made after recent earthquakes in Turkey on the reinforced concrete buildings indicated that number of stories is the most dominating damage inducing parameter and it is defined as the total number of individual floor systems above the ground level where the calculation of moments of inertias and the cross-sectional areas of the vertical elements are performed. Generally, higher level of damage is expected with an increase in number of stories. Number of stories is denoted by "*ns*" in the proceeding formulations.

**Minimum Normalized Lateral Stiffness Index**

This index is the indication of the lateral rigidity of the ground story, which is usually the most critical story. If the height, boundary conditions of the individual columns and the properties of materials are kept constant, this index would also represent the stiffness of the ground story. The minimum lateral stiffness index is denoted by "*mnlstf*" in the formulations. This index considers the moment of inertias of the columns and shear walls at the ground story and is calculated from the following:

$$mnlstf = \min(I_{nx}, I_{ny}) \quad (2.8)$$

$$I_{nx} = \frac{I_x}{\sum A_f} \times 1000 \quad (2.9.a)$$

$$I_{ny} = \frac{I_y}{\sum A_f} \times 1000 \quad (2.9.b)$$

where;

$$I_x = \sum (I_{col})_x + \sum (I_{sw})_x \quad (2.10.a)$$

$$I_y = \sum (I_{col})_y + \sum (I_{sw})_y \quad (2.10.b)$$

where;

$(I_{col})_x$, $(I_{col})_y$: moment of inertias of columns about x and y axes, respectively. They are to be calculated according to Equations 2.4.a and 2.4.b, respectively.

$(I_{sw})_x$, $(I_{sw})_y$: moment of inertias of shear walls about x and y axes, respectively. They are to be calculated according to Equations 2.4.a and 2.4.b, respectively.

$\Sigma(I_{col})_x$, $\Sigma(I_{col})_y$: summation of the moment of inertias of all columns about their centroidal x and y axes, respectively.

$\Sigma(I_{sw})_x$, $\Sigma(I_{sw})_y$: summation of the moment of inertias of all shear walls about their centroidal x and y axes, respectively.

$I_{nx}$, $I_{ny}$: total normalized moment of inertia of all members about x and y axes, respectively.

$\Sigma A_f$: total floor area above ground level.

**Minimum Normalized Lateral Strength Index**

The minimum normalized lateral strength index is used as an indication of the base shear capacity of the most critical story. In the calculation of minimum normalized lateral strength index, the effects of partition walls at the ground floor are considered. The contribution of the partition walls are assumed to be 10 percent of the contribution of the shear walls based on the results of the tests conducted at the Structural Mechanics

Laboratory at Middle East Technical University [2, 21]. In other words, masonry walls are assumed to carry 10 percent of the shear force that can be carried by a structural wall having the same cross-sectional area. This index is represented by "*mnlstr*" in the formulations and it is to be calculated by using the following equations:

$$mnlstr = \min(A_{nx}, A_{ny}) \tag{2.11}$$

$$A_{nx} = \frac{A_x}{\sum A_f} \times 1000 \tag{2.12.a}$$

$$A_{ny} = \frac{A_y}{\sum A_f} \times 1000 \tag{2.12.b}$$

where;

$$A_x = \sum (A_{col})_x + \sum (A_{sw})_x + 0.1 \sum (A_{mw})_x \tag{2.13.a}$$

$$A_y = \sum (A_{col})_y + \sum (A_{sw})_y + 0.1 \sum (A_{mw})_y \tag{2.13.b}$$

where;

$(A_{col})_x$, $(A_{col})_y$: cross-sectional areas of columns in x and y directions, respectively. They are to be calculated according to Equations 2.5.a and 2.5.b, respectively.

$(A_{sw})_x$, $(A_{sw})_y$: cross-sectional areas of shear walls in x and y directions, respectively. They are to be calculated according to Equations 2.6.a and 2.6.b, respectively.

$(A_{mw})_x$, $(A_{mw})_y$: cross-sectional areas of masonry walls in x and y directions, respectively. They are to be calculated according to Equations 2.7.a and 2.7.b, respectively.

$\Sigma(A_{col})_x$, $\Sigma(A_{col})_y$: summation of the cross-sectional areas of all columns in x and y directions, respectively.

$\Sigma(A_{sw})_x$, $\Sigma(A_{sw})_y$: summation of the cross-sectional areas of all shear walls in x and y directions, respectively.

$\Sigma(A_{mw})_x$, $\Sigma(A_{mw})_y$: summation of the cross-sectional areas of all masonry walls in x and y directions, respectively.

$A_{nx}$, $A_{ny}$: total normalized cross-sectional areas of all members in x and y directions, respectively.

$\Sigma A_f$: total floor area above ground level.

**Normalized Redundancy Score**

Redundancy is the indication of the degree of the continuity of multiple frame lines to distribute lateral forces throughout the structural system. The normalized redundancy ratio (*nrr*) of a frame structure is calculated by using Equation 2.14.

$$nrr = \frac{A_{tr} \cdot (f_x - 1) \cdot (f_y - 1)}{A_{gf}} \tag{2.14}$$

where;

$f_x$, $f_y$: number of continuous frame lines in the critical story in x and y directions, respectively.

$A_{tr}$: the tributary area for a typical column. $A_{tr}$ shall be taken as 25 $m^2$ if $f_x$ and $f_y$ are both greater than or equal to 3. In all other cases, $A_{tr}$ shall be taken as 12.5 $m^2$. The reason for this additional penalty on such buildings is that buildings having significant irregularities in plan and/or buildings having just one frame in either direction are considered more vulnerable than the others.

$A_{gf}$: the area of the critical story (usually the ground story).

The definition of normalized redundancy score, "*nrs*", based on "*nrr*" values is shown in Table 2.2 below.

**Table 2.2.** Definition of the Normalized Redundancy Score

| | **Normalized Redundancy Score (*nrs*)** |
|---|---|
| *nrr* > 1.0 | 3 |
| 1.0 > *nrr* > 0.5 | 2 |
| *nrr* < 0.5 | 1 |

**Soft Story Index**

As mentioned in Section 2.2.1, the existence of soft story is one of the major causes of damage to reinforced concrete buildings in Turkey. Lower amount of partition walls in the ground story is one of the main reasons for soft story formations. Since the effects of masonry walls were included in the calculation of "*mnlstr*", the soft story index is simply defined as the ratio of the height of first story, $H_1$, to the height of the second story, $H_2$. It should be noted that the soft story index is derived on the basis of the ratio of the stiffness of the first story to that of second story, where the stiffness of a story is related to the story height with its third power. Equation 2.15 expresses soft story index, "*ssi*", in a mathematical format as:

$$ssi = \frac{H_1}{H_2} \tag{2.15}$$

where;

$H_1$: height of ground story (i.e. first story)

$H_2$: height of second story

**Overhang Ratio**

This parameter includes the deficiencies due to overhanging portions of the building. The overhang area is defined as the floor area beyond the outermost frame lines on all sides of a structure, such as heavy balconies. The summation of the overhang area of each floor, $A_{oh}$, divided by the area of ground floor, $A_{gf}$, is defined as the overhang ratio, as shown in Equation 2.16. The overhang ratio is designated by "*or*" in the formulations.

$$or = \frac{A_{oh}}{A_{gf}} \tag{2.16}$$

where;

$A_{oh}$: area of overhang portion of a structure in the first story

$A_{gf}$: area of ground floor

# CHAPTER 3: PROPOSED STATISTICAL MODEL FOR PRELIMINARY SEISMIC VULNERABILITY ASSESSMENT

## 3.1. INTRODUCTION

In this chapter, the Duzce damage database compiled after 17 August 1999 and 12 November 1999 earthquakes is examined by utilizing statistical techniques. The main objective of the statistical analysis used in this study is to develop a model for preliminary seismic vulnerability assessment of low- to mid-rise reinforced concrete buildings based on a number of engineering parameters that were described in Chapter 2. In the following sections, firstly, the methodologies and the statistical tools are briefly described. Then, the statistical models are developed and the applications of these models to the available damage databases are presented.

## 3.2. STATISTICAL BACKGROUND

The effects of different parameters on seismic damage vary. In order to make a more rational and systematic evaluation of damage inducing parameters in the seismic vulnerability assessment, discriminant analysis technique is adopted.

In the most general sense earthquake damage to structures can be grouped in five damage states as: none (N), light (L), moderate (M), severe (S) and collapsed (C). Since the difference between severe damage and collapsed damage states is insignificant for life safety purposes, it is quite acceptable to combine the severe damage and collapsed cases into one group. In a similar manner, none and light damage states can be combined into one group based on the fact that the distinction between these two damage states is not critical from the point of view of vulnerability analysis. Therefore, there will be three damage state groupings, namely (N+L), (M) and (S+C).

It is possible to evaluate structures at different performance levels according to different objectives. If the main objective is to identify the buildings that are severely damaged or collapsed, the first three damage states (i.e. N, L and M) can be considered as one group and the severely damaged state and collapsed cases as the other group, reducing the distinct damage state groupings into two. Since the main objective is the identification of severely damaged and collapsed buildings for life safety purposes, this classification can be referred as "Life Safety Performance Level" (LSPL). If the main concern is to identify the structures which suffer no damage or light damage during an earthquake, the first two damage states (N and L) can be considered as one group and remaining damage states (M, S and C) as the other group, reducing the distinct damage state groupings again into two. This identification is named as "Immediate Occupancy Performance Level" since the main concern is to identify the buildings that can be occupied immediately after a large magnitude earthquake. Table 3.1 shows the classification of damage states, LSPL and IOPL, and the corresponding damage indicators.

The main objectives of the discriminant analysis technique utilized in this study are as follows:

- To identify the parameters that discriminate "best" among the damage states by utilizing the available damage database,

- To develop a function for computing a new variable or a damage score that will represent differences among different damage states,
- To develop a rule for classifying future observations into one of the damage states with minimum error.

**Table 3.1.** Classification of Damage States

| Damage States | Actual Indicator | LSPL Indicator | IOPL Indicator |
|---|---|---|---|
| **None** | 0 | 0 | 0 |
| **Light** | 0 | 0 | 0 |
| **Moderate** | 1 | 0 | 1 |
| **Severe** | 2 | 1 | 1 |
| **Collapsed** | 2 | 1 | 1 |

Discriminant analysis is one of the statistical techniques known as Multivariate Analysis of Variance, which tests group differences due to several dependent variables simultaneously with the investigation of number of cases. In this study, "groups" are the damage states of the buildings, "cases" are the buildings involved in the Duzce damage database and "variables" are the selected damage inducing parameters described in Chapter 2. These variables are known as "discriminator variables". The first step in discriminant analysis is to select the set of these variables that provides best discrimination among the groups. After the selection of the best discriminator variables, the linear combinations of these discriminator variables, known as "discriminant functions", are derived. The values obtained from these discriminant functions, called as "discriminant scores", are used for future classifications of the buildings to different damage states. The discriminant function is estimated in such a way that the ratio of the between-groups sum of squares to the within-group sum of squares for the discriminant scores is maximized [27]. Number of discriminating variables that are used in the analysis are the minimum of (g-1) and (p), where g is the number of groups and p is the number of variables.

## 3.3. STATISTICAL ANALYSIS OF DAMAGE DATA

The damage data, containing 484 low-rise to medium-rise monolithic reinforced concrete structures, compiled after the 17 August 1999 and the 12 November 1999 earthquakes described in Chapter 2 form the database for the statistical analyses performed in this chapter. The statistical analysis will be carried out of two different stages. In the first step, "Life Safety Performance Level" will be considered with two damage stages. And in the second stage, "Immediate Occupancy Performance Level" is taken into consideration. In all cases, the implementations of discriminant analysis are carried out by using the SPSS 11.0 software [28], which is licensed to METU and is commercially available.

### 3.3.1. Two Damage State Grouping

The analyses are conducted for both “Life Safety Performance Level” and “Immediate Occupancy Performance Level”. In two damage state grouping, only one discriminant function is obtained since the number of groups (g) is equal to two.

**Life Safety Performance Level**
In “Life Safety Performance Level”, the main objective is to identify the buildings, which have a tendency to suffer severe damage or to collapse under the design earthquake. If the severely damaged and collapsed structures are discriminated from the entire database, the life safety objective is satisfied. Therefore, the damage indicator “1” is used for severely damaged or collapsed buildings and “0” is used for the group consisting of other damage states; namely none, lightly and moderately damaged cases.

The unstandardized estimate of discriminant function based on previously derived six damage-inducing parameters is obtained by utilizing the SPSS software and the Duzce damage database. The resulting function is as follows:

$$DS = 0.620 \cdot ns - 0.246 \cdot mnlstf - 0.182 \cdot mnlstr - 0.699 \cdot nrs + 3.269 \cdot ssi + 2.728 \cdot or - 4.905 \quad (3.1)$$

where;

*DS:* discriminant score

*ns:* number of stories

*mnlstf:* minimum normalized lateral stiffness index

*mnlstr:* minimum normalized lateral strength index

*nrs:* normalized redundancy score

*ssi:* soft story index

*or:* overhang ratio

The discriminant score is a composite index, formed by the linear combination of the damage inducing parameters, or discriminating variables. The discriminant score is used for the estimation of the damage state of the structure. The function in Equation 3.1 is named as unstandardized discriminant function, because unstandardized (raw) data are used for computing this discriminant function. Unstandardized coefficients are used in computing the discriminant scores from Equation 3.1. However, these unstandardized coefficients do not contain any information about the absolute contribution of a variable in determining the discriminant score. If the relative importance of each variable is required, the discriminant functions based on standardized coefficients should be considered. In order to obtain the standardized coefficients of variables, each unstandardized coefficient is multiplied by the pooled standard deviation of the corresponding variable [20]. The corresponding standardized discriminant function is as follows:

$$DS_s = 0.563 \cdot ns - 0.082 \cdot mnlstf - 0.161 \cdot mnlstr - 0.502 \cdot nrs + 0.443 \cdot ssi + 0.201 \cdot or \quad (3.2)$$

Here, $DS_s$ denotes the standardized discriminant score. Standardized coefficients are normally used in the determination of the relative importance of discriminator variables that form the discriminant function. The larger the absolute value of the standardized coefficient of a variable in standardized discriminant function, the greater is the relative importance of the corresponding variable. Therefore, it can be concluded from Equation

3.2 that “number of stories”, with a standardized coefficient of 0.563, is the most dominant variable in forming the discriminant function.

Though the standardized coefficients are used to evaluate the contribution of the variables, such an explanation may not be valid in case the variables are highly correlated among themselves. If two variables share nearly same discriminating information, i.e. they are highly correlated; they must share their contributions to the discriminant score even if that joint contribution is important. Thus, their standardized coefficients may be smaller than the coefficients in case only one of them is used. This is because the standardized coefficients take the simultaneous contributions of all the other variables into consideration [27].

In addition to the standardized coefficients, a better statistical parameter that indicates the association between the discriminant functions and the discriminating variables is the structure coefficient. In fact, the structure coefficient of a single discriminator variable is the correlation coefficient between the discriminant score and the discriminator variable. The value of structure coefficients changes between -1 and 1. If the absolute value of the structure coefficient is closer to 1, there is more association between the variable and the discriminant function and vice versa. Table 3.2 shows the structure coefficients of the parameters obtained for the Duzce damage database. As observed from the Table 3.2, “number of stories” is the most dominating parameter with the maximum structure coefficient of 0.738.

**Table 3.2.** Structure Coefficients Obtained for the Duzce Damage Database for LSPL

| Variables | Structure Coefficients |
|---|---|
| ***ns*** | +0.738 |
| ***mnlstf*** | -0.076 |
| ***mnlstr*** | -0.503 |
| ***nrs*** | -0.555 |
| ***ssi*** | +0.418 |
| ***or*** | +0.167 |

The calculated discriminant score of each building based on the unstandardized coefficients is compared with the discriminant scores of the mean values of the group, i.e. discriminant scores of group centroids. Each building is assigned to the closest group according to its discriminant score. A summary of the classification results for the Duzce damage database, known as “classification matrix”, is shown in Table 3.3. As it can be observed from these results, 69.0 % of original grouped cases are correctly classified. The correct classification rate for severely damaged or collapsed structures is 71.3%.

**Table 3.3.** Classification Results for the Duzce Damage Database for LSPL

<table>
<tr><td colspan="3" rowspan="2"></td><td colspan="2">Predicted Group Membership</td><td rowspan="2">Total</td></tr>
<tr><td>0</td><td>1</td></tr>
<tr><td rowspan="4">Original Group Membership</td><td rowspan="2">Count</td><td>0</td><td>247</td><td>115</td><td>362</td></tr>
<tr><td>1</td><td>35</td><td>87</td><td>122</td></tr>
<tr><td rowspan="2">%</td><td>0</td><td>68.2</td><td>31.8</td><td>100.0</td></tr>
<tr><td>1</td><td>28.7</td><td>71.3</td><td>100.0</td></tr>
<tr><td colspan="6">69.0% of original grouped cases correctly classified</td></tr>
</table>

**Immediate Occupancy Performance Level**

In "Immediate Occupancy Performance Level", the main concern is to identify the buildings that are suitable for habitation immediately after the design earthquake. In such buildings, the damage should be limited and repairable. In case none or lightly damaged buildings are discriminated from the entire database, the immediate occupancy objective is satisfied. Therefore, the damage indicator "0" is used for none or lightly damaged buildings and "1" is used for the other damage states, namely for moderately and severely damaged and collapsed cases. For the Duzce damage database and using the previously defined six damage-inducing parameters, the unstandardized and standardized discriminant functions are obtained from the SPSS software and presented below:

$$DS = 0.808 \cdot ns - 0.334 \cdot mnlstf - 0.107 \cdot mnlstr - 0.687 \cdot nrs + 0.508 \cdot ssi + 3.884 \cdot or - 2.868 \quad (3.3)$$

$$DS_s = 0.720 \cdot ns - 0.112 \cdot mnlstf - 0.095 \cdot mnlstr - 0.488 \cdot nrs + 0.070 \cdot ssi + 0.285 \cdot or \quad (3.4)$$

The standardized discriminant function in Equation 3.4 implies that "number of stories", with a standardized coefficient of 0.720, is the most dominating parameter among the six damage-inducing parameters taking place in the discriminant function.

Table 3.4 shows the structure coefficients for the Duzce damage database for IOPL. As observed from the table, "number of stories" is the most dominating parameter with the maximum structure coefficient of 0.789.

**Table 3.4.** Structure Coefficients Obtained for the Duzce Damage Database for IOPL

| Variables | Structure Coefficients |
|---|---|
| **ns** | +0.789 |
| **mnlstf** | -0.085 |
| **mnlstr** | -0.481 |
| **nrs** | -0.594 |
| **ssi** | +0.092 |
| **or** | +0.284 |

A summary of the classification results for the Duzce damage database is shown in Table 4.5. 72.5 % of the original grouped cases are correctly classified whereas the correct classification rate for none or lightly damaged buildings is 69.2 %.

**Table 3.5.** Classification Results for the Duzce Damage Database for IOPL

| | | | **Predicted Group Membership** | | **Total** |
|---|---|---|---|---|---|
| | | | 0 | 1 | |
| **Original Group Membership** | Count | 0 | 146 | 65 | 211 |
| | | 1 | 68 | 205 | 273 |
| | % | 0 | 69.2 | 30.8 | 100.0 |
| | | 1 | 24.9 | 75.1 | 100.0 |
| 72.5% of original grouped cases correctly classified | | | | | |

## 3.4. CLASSIFICATION FOR FUTURE OBSERVATIONS

In this section, a model is proposed for the preliminary seismic vulnerability assessment of existing reinforced concrete buildings. The proposed model is based on the derived discriminant functions with six damage-inducing parameters, namely “ns”, ”mnlstf”, “mnlstr”, “nrs”, “ssi” and “or”. Firstly, the application of the two-damage state groupings namely: LSPL and IOPL groupings will be taken into consideration. Then, the optimal classification technique for the two damage state grouping will be presented.

### 3.4.1. Two Damage State Grouping

Damage state (score) of a structure is decided by calculating the discriminant score obtained from the unstandardized discriminant functions derived previously. This score is compared with the mean values of the discriminant scores of the groups, and then the structure is assigned to the closest group. Hence, it is necessary to divide the discriminant space into two regions for the future classification of the existing structures with respect to the two damage state groups. The value of the discriminant function that divides the region into two is called the cutoff value. Generally, the cutoff value that minimizes the number of misclassifications for the sample data is the average of the mean discriminant scores for the two groups [27]. The cutoff value is to be calculated from the following equation:

$$CV = \frac{\overline{DS_0} + \overline{DS_1}}{2} \tag{3.5}$$

where; CV is the cutoff value and $\overline{DS_i}$ is the mean value of the discriminant score for the $i^{th}$ group

**Life Safety Performance Level**

In the case of Life Safety Performance Level, the unstandardized discriminant function, given in Equation 3.1, is used for future classification of existing structures with respect to the two damage states, namely as “severe damage or collapse” (S+C) and “no severe damage” (N+L+M).

The mean discriminant scores are obtained as –0.247 and 0.734 for the “no severe damage” and “severe damage or collapse” damage state groups, respectively, from the output of SPSS software. According to Equation 3.5, the cutoff value for LSPL is (-0.247+0.734)/2 = 0.244. Hence, for any reinforced concrete building with known “ns”, “mnlstf”, “mnlstr”, “nrs”, “ssi” and “or”, the value of unstandardized discriminant score is to be calculated from Equation 3.1. If the calculated discriminant score is greater than the 0.244, the building is expected to suffer severe damage or collapse during a large magnitude earthquake and may cause loss of life, whereas if its discriminant score is less than 0.244, no severe damage is expected.

**Immediate Occupancy Performance Level**

In the case of Immediate Occupancy Performance Level, the unstandardized discriminant function given in Equation 3.3 is to be used for future classification of existing reinforced concrete structures with respect to the two damage state grouping, namely as “suitable for immediate occupancy” (N+L) and “not suitable for immediate occupancy” (M+S+C).

In this case, the mean discriminant scores for first and second groups are –0.541 and 0.418, respectively. From Equation 3.5, the cutoff value for IOPL is (-0.541+0.418)/2 = -0.062. Hence, for any reinforced concrete building with known discriminant parameters, the unstandardized discriminant score is to be calculated from Equation 3.3. If the calculated discriminant score is greater than -0.062, the building is expected to experience moderate or severe damage or collapse during a large magnitude earthquake, whereas if its discriminant score is smaller than -0.062, the building will be rated as "suitable for immediate occupancy".

**Optimal Classification for the Two Damage State Grouping**

In the preliminary vulnerability assessment of existing buildings, the main objective is to satisfy the life safety. Although the correct classification rates for the Duzce damage database are reasonably high, the proposed method should be modified in order to increase the efficiency of the method, especially in classifying the damage states related to life safety. In addition to the need for improved efficiency, there is 35/484×100 = 7.2% incorrect classification rate, which may result into life loss according to "Life Safety Performance Level". Also, as it was stated in the preceding sections, "number of stories" is the most dominant and the most discriminating damage inducing parameter in both performance levels. Therefore, the proposed method should be modified to account for these observations in an optimal way, and achieve the following objectives:

- To increase the efficiency of the proposed method,
- To increase the discriminating contributions of the parameters other than "number of stories",
- To increase the life safety performance correct classification rate above a certain level. For this purpose, the maximum acceptable misclassification rate resulting in life loss is restricted to 5 %,

Since the "number of stories" is the most dominating parameter, it is aimed to establish a functional relationship between cutoff values and the "number of stories". In the determination of the number of story dependent cutoff function, two constraints are imposed at each story level. These constraints are;

- the minimum acceptable correct classification rate for each sample of individual story group is restricted to be 70 % and,
- the maximum tolerable misclassification rate causing loss of life for each sample of individual story group is restricted to 5 %.

After obtaining the cutoff points for each story groups, a third-order polynomial cutoff function is assigned to each performance level using the "least squares fitting method". The cutoff functions for different performance level are given in Table 3.6.

**Table 3.6.** Cutoff Functions in terms of "Number of Stories" for LSPL and IOPL

| Performance Level | Cutoff Function |
|---|---|
| **LSPL** | $-0.090ns^3+1.498ns^2-7.518ns+11.885$ |
| **IOPL** | $-0.085ns^3+1.416ns^2-6.951ns+9.979$ |

As it was stated in the preceding sections, buildings may be evaluated according to either "Life Safety Performance Level" (LSPL) or "Immediate Occupancy Performance Level" (IOPL). At the end of the analysis for LSPL, the structure is classified as either "unsafe" (S or C) or "safe" (N, L or M). The damage indicator "1" is used for "unsafe" rating and the damage indicator "0" is used for "safe" rating. On the other hand, after the IOPL analysis, the structure is classified as either "safe" (N or L) or "unsafe" (M, S or C) with the corresponding damage indicator variables "0" and "1", respectively.

In the optimal classification method, in order to classify an existing structure, the structure is analyzed both for "Life Safety Performance Level" and "Immediate Occupancy Performance Level", simultaneously. The decision for the vulnerability of structure will be concluded at the end of these two analyses. If both performance levels assign indicator variable "0" for the structure; i.e. "safe" for both LSPL and IOPL cases, the structure is considered to be "safe". In the same way, if the structure is marked as "1" in both performance levels; i.e. "unsafe" for both LSPL and IOPL cases, it is rated as "unsafe". In all other cases, the structure is classified as "requires further evaluation". In other words, if there is any contradiction between the estimations of the two performance levels, the building is left for further detailed seismic performance analyses and further evaluations. The decision table for the results of simultaneous analyses of both performance levels is shown in Table 3.7.

**Table 3.7.** Decision Table for Classification According to the Optimal Classification Method

| **Damage Indicator** | | **Classification According to the Optimal Method** | **Indicator in the Optimal Method** |
|---|---|---|---|
| **LSPL** | **IOPL** | | |
| 0 | 0 | SAFE (None or Light Damage) | 0 |
| 1 | 1 | UNSAFE (Severe Damage or Collapse) | 1 |
| 1 | 0 | Requires Further Evaluation | 2 |
| 0 | 1 | Requires Further Evaluation | 2 |

The classification results of the Duzce damage database according to the optimal classification method are shown in Table 3.8. The correct classification rate in determining the severely damaged and collapsed structures is increased to 80.3 %, and the total misclassification that corresponds to the life loss is reduced to 13/484×100 = 2.7%. The percent of buildings that is left for further evaluations is (37+11+51)/484×100 = 20.5 %.

**Table 3.8.** Classification Results for the Duzce Damage Database According to the Optimal Classification Method

<table>
<tr><th colspan="3" rowspan="2"></th><th colspan="3">Predicted Group Membership</th><th rowspan="2">Total</th></tr>
<tr><th>0</th><th>1</th><th>2</th></tr>
<tr><td rowspan="6">Original Group Membership</td><td rowspan="3">Count</td><td>None or Light</td><td>130</td><td>44</td><td>37</td><td>211</td></tr>
<tr><td>Severe or Collapsed</td><td>13</td><td>98</td><td>11</td><td>122</td></tr>
<tr><td>Moderate</td><td>37</td><td>63</td><td>51</td><td>151</td></tr>
<tr><td rowspan="3">%</td><td>SAFE</td><td>61.6</td><td>20.8</td><td></td><td>100.0</td></tr>
<tr><td>UNSAFE</td><td>10.7</td><td>80.3</td><td></td><td>100.0</td></tr>
<tr><td>Requires Further Evaluation</td><td></td><td></td><td>20.5</td><td>100.0</td></tr>
</table>

# CHAPTER 4: VALIDATION OF THE PROPOSED STATISTICAL MODEL AND COMPARISONS WITH OTHER METHODOLOGIES

## 4.1. GENERAL

It should be noted that the discriminant functions derived in the previous sections are based on the Duzce damage database. It is assumed that all of the 484 buildings in the damage database were subjected to the same earthquake excitation under similar soil conditions. In other words, each building stock in the database in itself has faced "almost" the same ground motion characteristics, thus the damage will be evaluated only on the basis of structural responses rather than including the excitation parameters.

It is desirable to check the efficiency of the proposed model by examining the correct classification rates in cases of different damage databases compiled from recent earthquakes in Turkey. Therefore, proposed classification models are applied to the damage databases compiled from the 1992 Erzincan, 2002 Afyon and 2003 Bingol earthquakes. Later, the methodologies proposed by Hassan and Sozen [17] and Ersoy and Tankut [13] are applied to these damage databases in order to compare the efficiency of the proposed statistical model in predicting the seismic damage to reinforced concrete buildings.

## 4.2. VERIFICATION OF THE PROPOSED STATISTICAL MODEL

In this section, the damage databases compiled from the 1992 Erzincan, 2002 Afyon and 2003 Bingol earthquakes are analyzed for the verification of the proposed statistical models.

### 4.2.1. Life Safety Performance Level

In order to check the validity of the model proposed for the Life Safety Performance Level, the unstandardized discriminant score is calculated by using the unstandardized discriminant function given by Equation 3.1. The classification matrices obtained based on the cutoff value of 0.244 for the Erzincan, Afyon and Bingol damage databases are shown in Tables 4.1, 4.2 and 4.3, respectively.

As it is observed from these tables, the overall correct classification rates are found to be 100.0%, 83.3% and 78.6% for the Erzincan, Afyon and Bingol damage databases, respectively.

**Table 4.1.** Classification Results for the Erzincan Damage Database for LSPL

| | | | Predicted Group Membership | | Total |
| --- | --- | --- | --- | --- | --- |
| | | | 0 | 1 | |
| Original Group Membership | Count | 0 | 41 | 0 | 41 |
| | | 1 | 0 | 2 | 2 |
| | % | 0 | 100.0 | 0.0 | 100.0 |
| | | 1 | 0.0 | 100.0 | 100.0 |
| 100.0% of original grouped cases correctly classified | | | | | |

**Table 4.2.** Classification Results for the Afyon Damage Database for LSPL

| | | | Predicted Group Membership | | Total |
|---|---|---|---|---|---|
| | | | 0 | 1 | |
| Original Group Membership | Count | 0 | 8 | 0 | 8 |
| | | 1 | 3 | 7 | 10 |
| | % | 0 | 100.0 | 0.0 | 100.0 |
| | | 1 | 30.0 | 70.0 | 100.0 |
| 83.3% of original grouped cases correctly classified | | | | | |

**Table 4.3.** Classification Results for the Bingol Damage Database for LSPL

| | | | Predicted Group Membership | | Total |
|---|---|---|---|---|---|
| | | | 0 | 1 | |
| Original Group Membership | Count | 0 | 19 | 3 | 22 |
| | | 1 | 3 | 3 | 6 |
| | % | 0 | 86.4 | 13.6 | 100.0 |
| | | 1 | 50.0 | 50.0 | 100.0 |
| 78.6% of original grouped cases correctly classified | | | | | |

### 4.2.2. Immediate Occupancy Performance Level

The model proposed for the Immediate Occupancy Performance Level was applied to the Erzincan, Afyon and Bingol damage databases. The unstandardized discriminant score is calculated by using Equation 3.3 and the classification is performed by considering the cutoff value of -0.062.

The classification matrices obtained for the Erzincan, Afyon and Bingol damage databases are shown in Tables 4.4, 4.5 and 4.6, respectively. 74.4%, 50.0% and 75.0% overall correct classification rates are achieved for the Erzincan, Afyon and Bingol damage databases, respectively.

**Table 4.4.** Classification Results for the Erzincan Damage Database for IOPL

| | | | Predicted Group Membership | | Total |
|---|---|---|---|---|---|
| | | | 0 | 1 | |
| Original Group Membership | Count | 0 | 27 | 1 | 28 |
| | | 1 | 10 | 5 | 15 |
| | % | 0 | 96.4 | 3.6 | 100.0 |
| | | 1 | 66.7 | 33.3 | 100.0 |
| 74.4% of original grouped cases correctly classified | | | | | |

**Table 4.5.** Classification Results for the Afyon Damage Database for IOPL

| | | | Predicted Group Membership | | Total |
|---|---|---|---|---|---|
| | | | 0 | 1 | |
| Original Group Membership | Count | 0 | 4 | 0 | 4 |
| | | 1 | 9 | 5 | 14 |
| | % | 0 | 100.0 | 0.0 | 100.0 |
| | | 1 | 64.3 | 35.7 | 100.0 |
| 50.0% of original grouped cases correctly classified | | | | | |

**Table 4.6.** Classification Results for the Bingol Damage Database for IOPL

<table>
<tr><td colspan="3" rowspan="2"></td><td colspan="2">Predicted Group Membership</td><td rowspan="2">Total</td></tr>
<tr><td>0</td><td>1</td></tr>
<tr><td rowspan="4">Original Group Membership</td><td rowspan="2">Count</td><td>0</td><td>13</td><td>2</td><td>15</td></tr>
<tr><td>1</td><td>5</td><td>8</td><td>13</td></tr>
<tr><td rowspan="2">%</td><td>0</td><td>86.7</td><td>13.3</td><td>100.0</td></tr>
<tr><td>1</td><td>38.5</td><td>61.5</td><td>100.0</td></tr>
<tr><td colspan="6">75.0% of original grouped cases correctly classified</td></tr>
</table>

### 4.2.3. Optimal Classification for the Two Damage State Grouping

The optimal classification methodology described in Section 3.4.2 is applied to the Erzincan, Afyon and Bingol damage databases. The classification results of the optimal method for these databases are presented in Table 4.7, Table 4.8 and Table 4.9, respectively. For the Erzincan damage database (Table 4.7), the correct classification rate for severely damaged and collapsed buildings is the same as the results obtained for LSPL (100.0 %). Moreover, the correct classification rate for none and light damage is increased from 66.7 % to 96.4 %. The structures left for further detailed evaluation is only (1+0+3)/43×100 = 9.3 %. Moreover, correct classification rates for the Afyon and Bingol damage databases were improved with the use of the optimal classification methodology.

The individual and overall correct classification rates for the Erzincan, Afyon and Bingol damage databases for two damage state groupings and the optimal classification method show that the discriminant analysis technique provides a rational approach for seismic vulnerability assessment and the selected parameters are “good” indicators of seismic damage to reinforced concrete buildings in Turkey.

**Table 4.7.** Classification Results for the Erzincan Damage Database According to Optimal Classification Method

<table>
<tr><td colspan="3" rowspan="2"></td><td colspan="3">Predicted Group Membership</td><td rowspan="2">Total</td></tr>
<tr><td>0</td><td>1</td><td>2</td></tr>
<tr><td rowspan="6">Original Group Membership</td><td rowspan="3">Count</td><td>None or Light</td><td>27</td><td>0</td><td>1</td><td>28</td></tr>
<tr><td>Severe or Collapsed</td><td>0</td><td>2</td><td>0</td><td>2</td></tr>
<tr><td>Moderate</td><td>10</td><td>0</td><td>3</td><td>13</td></tr>
<tr><td rowspan="3">%</td><td>SAFE</td><td>96.4</td><td>0.0</td><td></td><td>100.0</td></tr>
<tr><td>UNSAFE</td><td>0.0</td><td>100.0</td><td></td><td>100.0</td></tr>
<tr><td>Requires Further Evaluation</td><td></td><td></td><td>9.3</td><td>100.0</td></tr>
</table>

**Table 4.8.** Classification Results for the Afyon Damage Database According to Optimal Classification Method

<table>
<tr><td colspan="3" rowspan="2"></td><td colspan="3">Predicted Group Membership</td><td rowspan="2">Total</td></tr>
<tr><td>0</td><td>1</td><td>2</td></tr>
<tr><td rowspan="6">Original Group Membership</td><td rowspan="3">Count</td><td>None or Light</td><td>3</td><td>0</td><td>1</td><td>4</td></tr>
<tr><td>Severe or Collapsed</td><td>1</td><td>8</td><td>1</td><td>10</td></tr>
<tr><td>Moderate</td><td>2</td><td>0</td><td>2</td><td>4</td></tr>
<tr><td rowspan="3">%</td><td>SAFE</td><td>75.0</td><td>0.0</td><td></td><td>100.0</td></tr>
<tr><td>UNSAFE</td><td>10.0</td><td>80.0</td><td></td><td>100.0</td></tr>
<tr><td>Requires Further Evaluation</td><td></td><td></td><td>22.2</td><td>100.0</td></tr>
</table>

**Table 4.9.** Classification Results for the Bingol Damage Database According to Optimal Classification Method

| | | | Predicted Group Membership | | | Total |
|---|---|---|---|---|---|---|
| | | | 0 | 1 | 2 | |
| **Original Group Membership** | Count | None or Light | 13 | 2 | 0 | 15 |
| | | Severe or Collapsed | 2 | 4 | 0 | 6 |
| | | Moderate | 2 | 1 | 4 | 7 |
| | % | SAFE | 86.7 | 13.3 | | 100.0 |
| | | UNSAFE | 0.33 | 66.7 | | 100.0 |
| | | Requires Further Evaluation | | | 14.2 | 100.0 |

## 4.3. APPLICATION OF OTHER METHODOLOGIES

In this section, two different methodologies previously proposed by Hassan and Sozen [17] and Ersoy and Tankut [13] are applied on the Duzce, Afyon and Bingol damage databases.

### 4.3.1. Method Proposed by Hassan and Sozen [17]

Hassan and Sozen [17] proposed a method for ranking low-rise monolithic reinforced concrete structures according to their seismic vulnerability. The proposed ranking process requires only the total floor area of the structure and cross-sectional areas of columns, shear walls and masonry walls. The method requires the calculation of two indices, namely: wall index (WI) and column index (CI). These indices are defined as follows:

$$WI = \frac{A_{wt}}{A_{ft}} \times 100 \tag{4.1}$$

$$CI = \frac{A_{ce}}{A_{ft}} \times 100 \tag{4.2}$$

in which;

$$A_{wt} = A_{cw} + \frac{A_{mw}}{10} \tag{4.3}$$

$$A_{ce} = \frac{A_{col}}{2} \tag{4.4}$$

where;

$A_{ft}$: total floor area above base level of the structure.

$A_{mw}$: total cross-sectional area of non-reinforced masonry walls in one horizontal direction at base level of the structure.

$A_{cw}$: total cross-sectional area of reinforced concrete walls in one horizontal direction at base level of the structure.

$A_{ce}$: effective cross-sectional area of columns above base level of the structure.

$A_{col}$: total cross-sectional area of columns above base level of the structure.

The wall and column indices are calculated for both directions of a structure. Then, the indices corresponding to the weaker direction are plotted such that x and y axes will represent column and wall indices, respectively. This plot indicates their relative seismic vulnerability with respect to each other in a given region. Priority index is defined to identify the buildings that require immediate rehabilitation. The priority index is calculated using the following relationship:

$$PI = WI + CI \tag{4.5}$$

Structures with the lowest priority index are considered to be candidates for severe damage in case of a strong ground motion. If decisions are to be made for immediate renewal or strengthening of structures in a given region, priority index could be used as an indicator. Since the method is designed to rank buildings in the same region, no additional factors are needed to reflect the relative seismic risk of those buildings.

The methodology proposed by Hassan and Sozen was applied to the Duzce damage database and the results are shown in Figure 4.1. In this figure, two boundary lines are shown. In the proposed method, in case the column and wall indices define a point within a triangle formed by Boundary 1 and the two axes of the graph, that particular building is considered to be more vulnerable than a building for which the indices intersect at a position, say, outside Boundary 2. However, there is no absolute basis for the determination of the boundary lines. For the application of proposed method to the Duzce damage database, the boundary lines suggested by authors for the 1992 Erzincan earthquake were used. Although these boundary lines were suitable for the Erzincan damage database, they seem to be unconservative for the Duzce damage database. A significant number of buildings that suffered severe damage intersect the region outside Boundary 2. Similarly, considerable number of buildings that experienced no or light damage fall in a triangle formed by Boundary 1 and the two axes of the graph. Moreover, almost 50 percent of the buildings intersect the region formed by these two boundary lines. Hence, it can be concluded that the proposed ranking procedure does not reflect the observed damage in Duzce earthquake, satisfactorily. Moreover, it can be concluded from Figures 4.2 and 4.3 that the method fails in ranking the observed damage for the Afyon and Bingol damage databases.

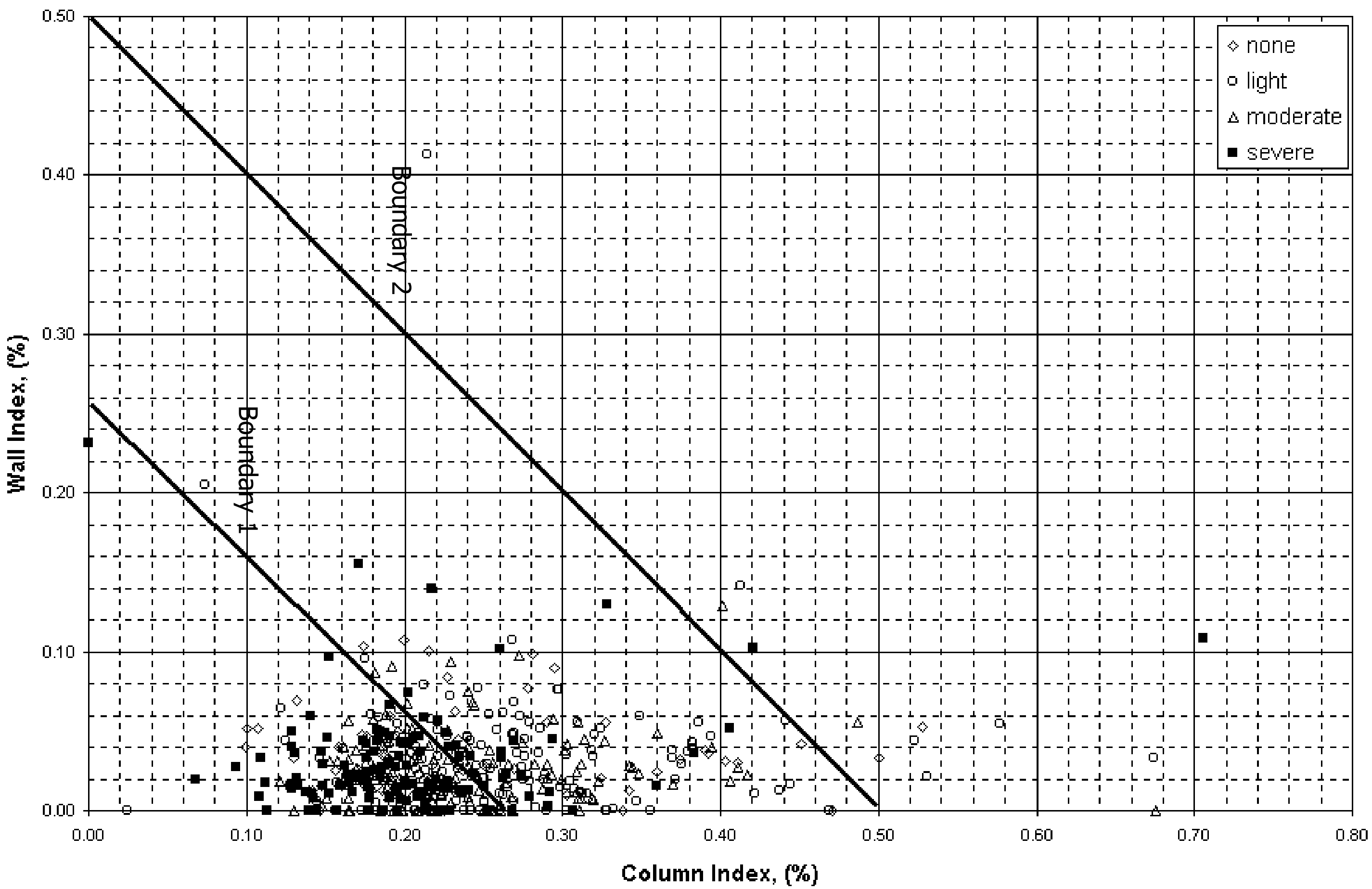



**Figure 4.1.** Results Obtained from the Application of Hassan and Sozen's Method [17] to the Duzce Damage Database

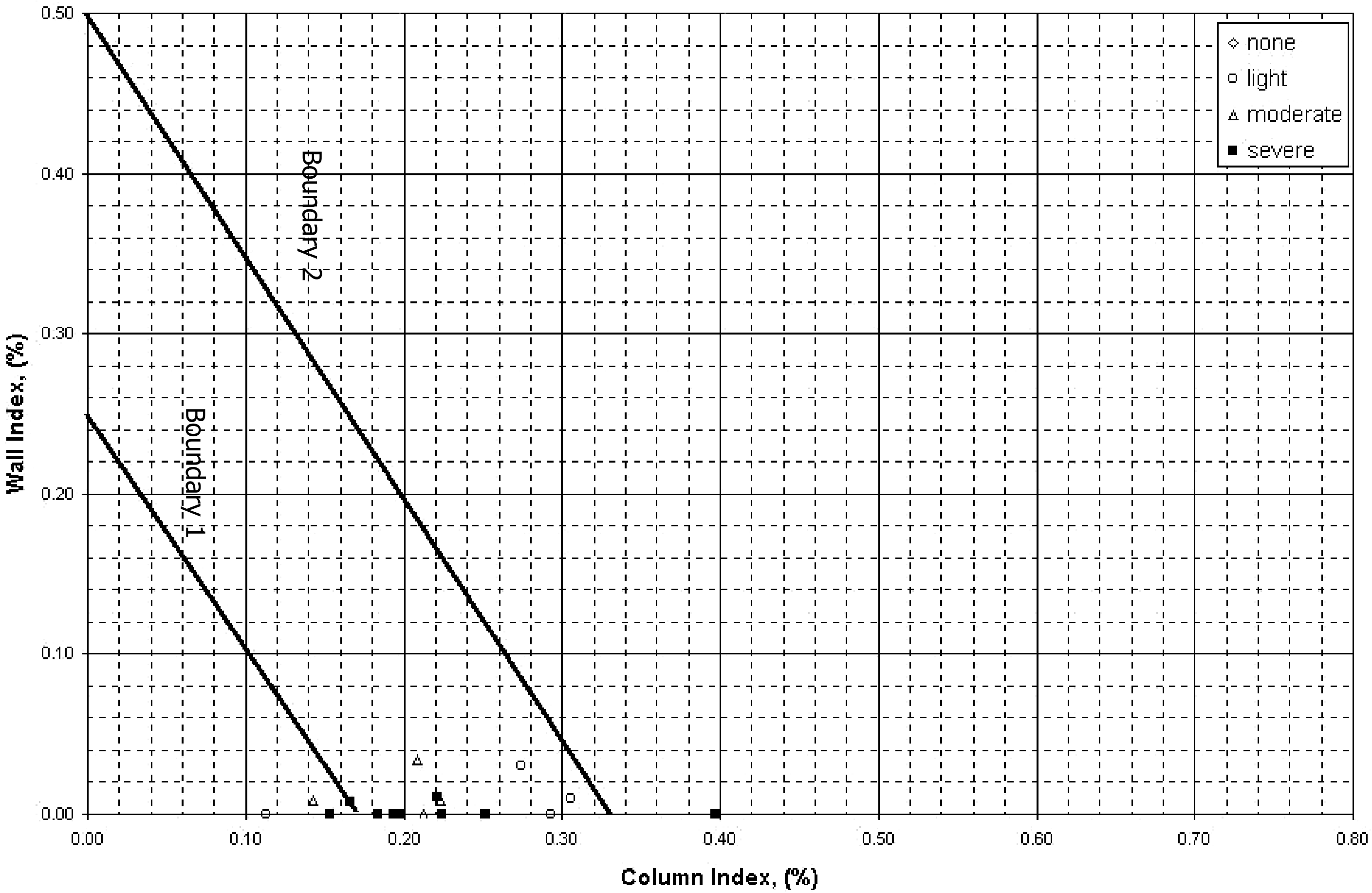



**Figure 4.2.** Results Obtained from the Application of Hassan and Sozen's Method [17] to the Afyon Damage Database

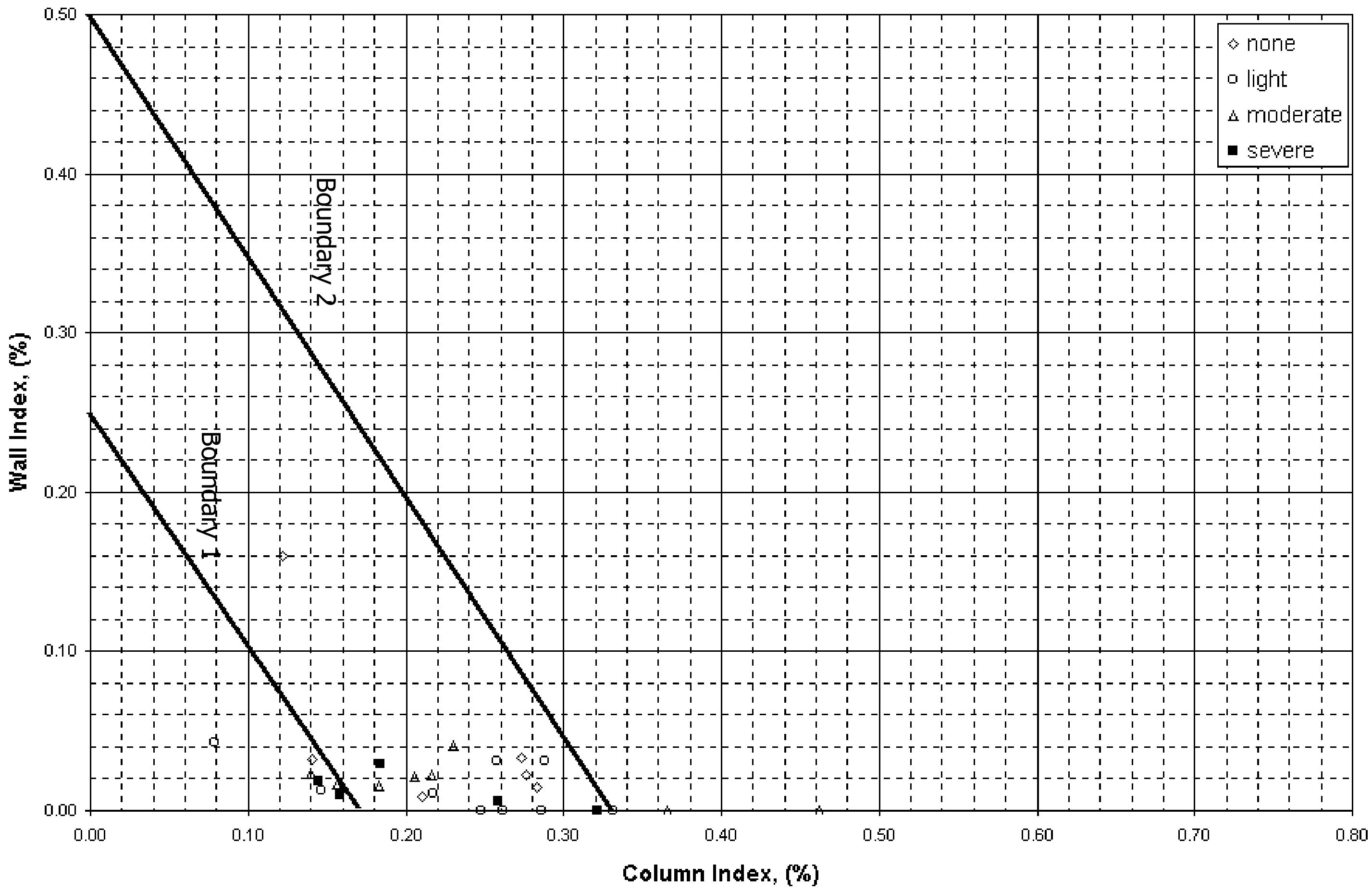



**Figure 4.3.** Results Obtained from the Application of Hassan and Sozen's Method [17] to the Bingol Damage Database

### 4.3.2. Method Proposed by Ersoy and Tankut [13]

Ersoy and Tankut [13] proposed a methodology for the seismic design of low-rise residential reinforced concrete buildings with less than seven stories. Authors stated that detailed structural analysis of buildings is not needed if the structure fulfills the minimum design requirements for the reinforcement ratios and the dimensions of members given in the "Specifications for Structures to be Built in Disaster Areas" [22] and the column and wall areas given in Equations 4.6 and 4.7 are satisfied.

$$(k \cdot \sum A_c + \sum A_w) \geq 0.003 \cdot \sum A_p \tag{4.6}$$

$$\sum A_w \geq 0.002 \sum A_p \geq 0.01 A_{pb} \tag{4.7}$$

where;

$\Sigma A_c$: total cross-sectional area of columns at the base level of the structure

$\Sigma A_w$: total cross-sectional area of shear walls in one horizontal direction at the base level of structure

$\Sigma A_p$: total floor area above base level of the structure

$A_{pb}$: floor area at the base level of the structure

k: 1/2 for square and circular columns, 1/3 for rectangular columns in their shorter directions, 2/3 for rectangular columns in their longer directions.

Authors have examined the validity of the proposed method using the Erzincan damage database. The ratio of available column and shear wall areas of the buildings to the required area is denoted by R and is computed by Equation 4.8. The ratios were calculated in both directions for each structure by using Equations 4.6, 4.7 and 4.8. Smaller "R" values of structures were plotted against the number of stories. If the ratio is greater than one, no severe damage is expected in case of a strong ground motion.

$$R = \frac{(k \cdot \sum A_c + \sum A_w)}{0.003 \cdot \sum A_p} \tag{4.8}$$

The methodology proposed by Ersoy and Tankut was applied to the Duzce damage database. "R" values of each structure were calculated in both horizontal directions and the smaller of these ratios were plotted versus number of stories in Figure 4.4. As seen in this figure, seven buildings that suffered severe damage were not expected to experience severe damage during a strong ground motion. This figure also implies that R values of almost 85% of the building stock in Duzce damage database are less than one and the percents of buildings that suffered severe damage are almost same in both regions of the limit value of R (less than one and more than one). Therefore, it can be concluded that no evaluation is possible by considering the wall and column area ratios, only. The number of misclassifications indicates that the proposed method does not reflect the observed damage in Duzce inventory, satisfactorily, although it was found to be in good correlation with respect to the damage records associated with the Erzincan damage database.

Moreover, it can be concluded from Figures 4.5 and 4.6 that there is no possible discrimination by using the proposed boundary line for the Afyon and Bingol damage databases.

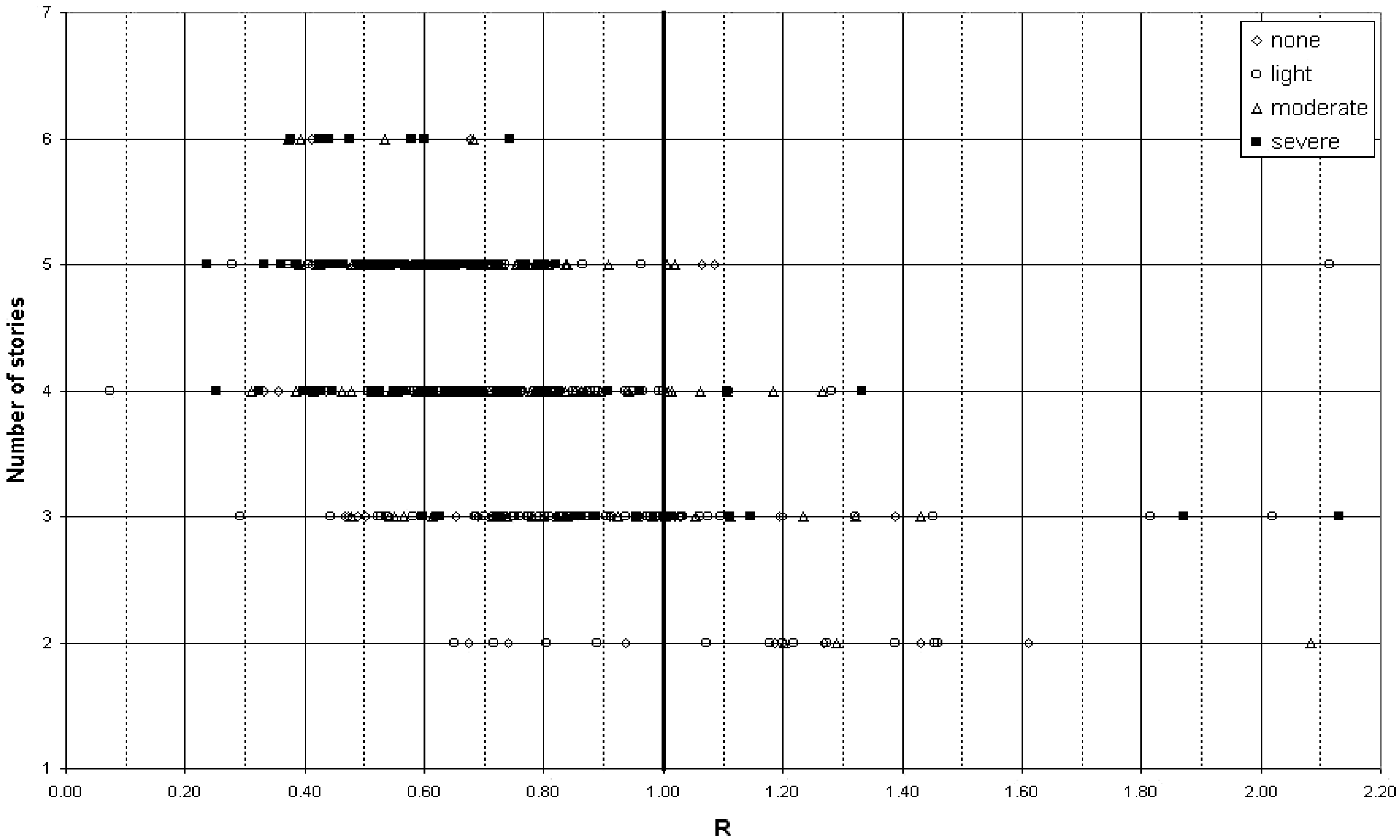



**Figure 4.4.** Results Obtained from the Application of Ersoy and Tankut's Method [13] to the Duzce Damage Database

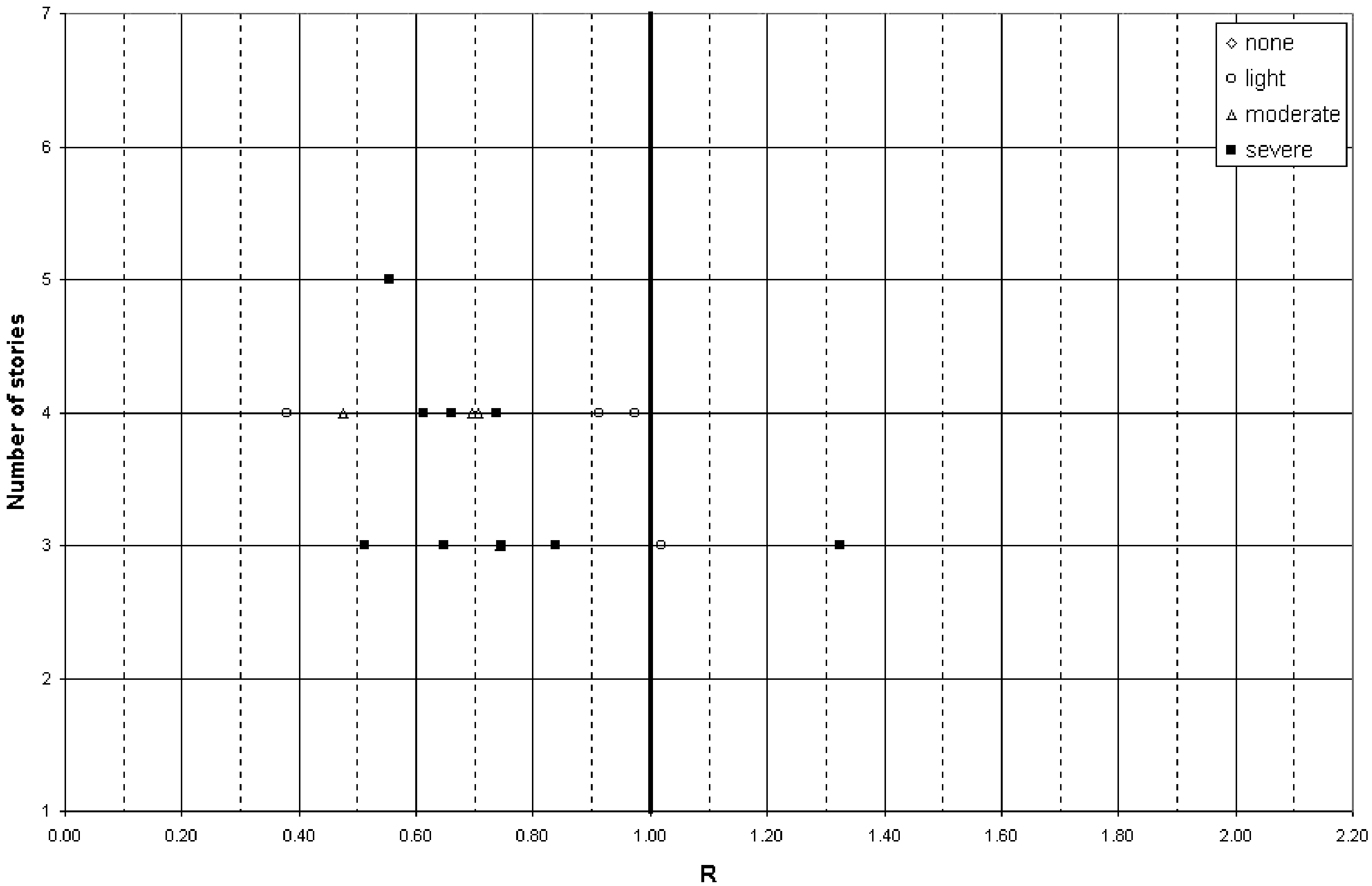



**Figure 4.5.** Results Obtained from the Application of Ersoy and Tankut's Method [13] to the Afyon Damage Database

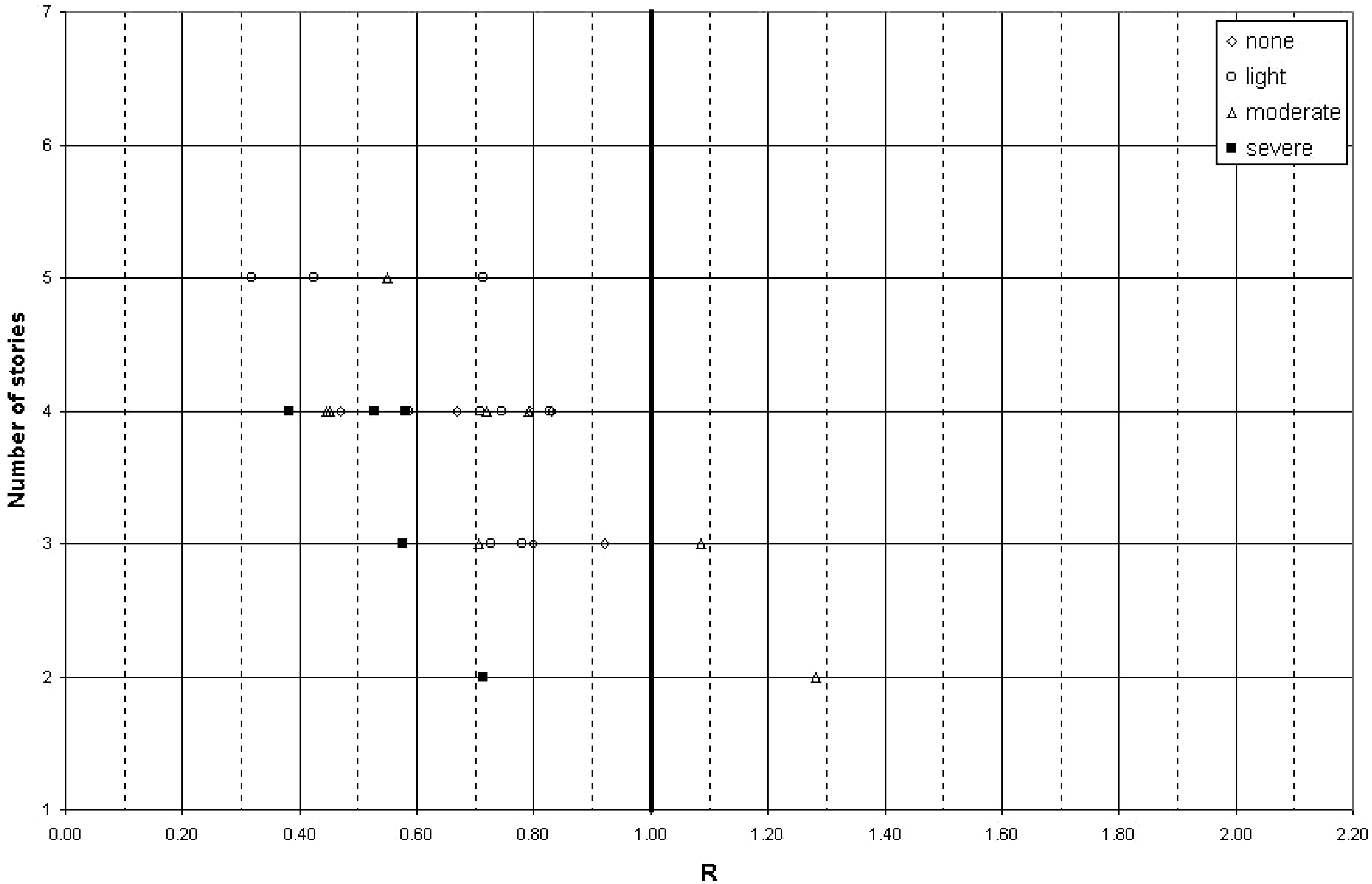



**Figure 4.6.** Results Obtained from the Application of Ersoy and Tankut's Method [13] to the Bingol Damage Database

# CHAPTER 5: INCLUSION OF SITE CHARATERISTICS

## 5.1. GENERAL

This chapter focuses on modifying the vulnerability assessment procedure, which was developed based on the structural characteristics of buildings in the city of Duzce with the purpose of including the site characteristics. The procedure, which is described in detail in the previous chapter, relies on the damage cutoff values developed using a statistical approach based on the damage data compiled from Duzce in the wake of 1999 earthquakes. Some selected building attributes are entered into a relation obtained from the discriminant analysis to compute a damage score. This damage score is then compared with a cutoff value, which identifies the buildings as "safe", "unsafe" or "requires further evaluation". The cutoff values recommended are considered to be valid for damaging earthquakes and the regions that have similar "distance to source" and site conditions to that of 1999 Duzce earthquake. To apply this procedure to the sites, which have different "distance to source" and soil properties than Duzce, modifications to the cutoff values have been computed.

The effects of distance to source and soil type are the two main parameters that should be taken into account, since the size of the earthquake considered is assumed to be similar to that of Duzce. The sites are classified according to the Turkish Seismic Design Code's [22] definitions based on the shear wave velocity. Various attenuation relations are used to account for the variation of the ground motion with distance and the soil type. Later, in this chapter, the modified cutoff values have been applied to Istanbul to get an estimate of what percentage of the buildings would perform unsatisfactorily if an expected earthquake of the same size as Duzce takes place in the vicinity of Istanbul.

## 5.2. PROCEDURE

The central point of the study presented in this chapter is to capture the relative variation of the ground motion intensity with the distance to source and the soil type. The spectral displacement ($S_d$) value was selected as the damage inducing ground motion parameter, as it is a widely used parameter for expressing the vulnerability of buildings.

A typical damage curve expressed in terms of the spectral displacement is shown in Figure 5.1 [10]. It is important to observe that the variation of damage with $S_d$ follows the form of an exponential function. This inference is used to link the change in $S_d$ to the change to be imposed on the cutoff values obtained for the Duzce damage database [7, 25]. In Figure 5.1, H denotes the height of the building.

The spectral displacement can be obtained from elastic site spectra computed using available attenuation relations. A number of relations, available in the literature, can be employed to relate inelastic spectral displacement ($S_{di}$) to the elastic one. Although the expressions seem quite different, their influence on the cutoff modifications is shown to be insignificant, especially in the range considered in this study as illustrated in Figure 5.2 [5, 9]. For this reason, equal displacement rule is considered to be adequate.

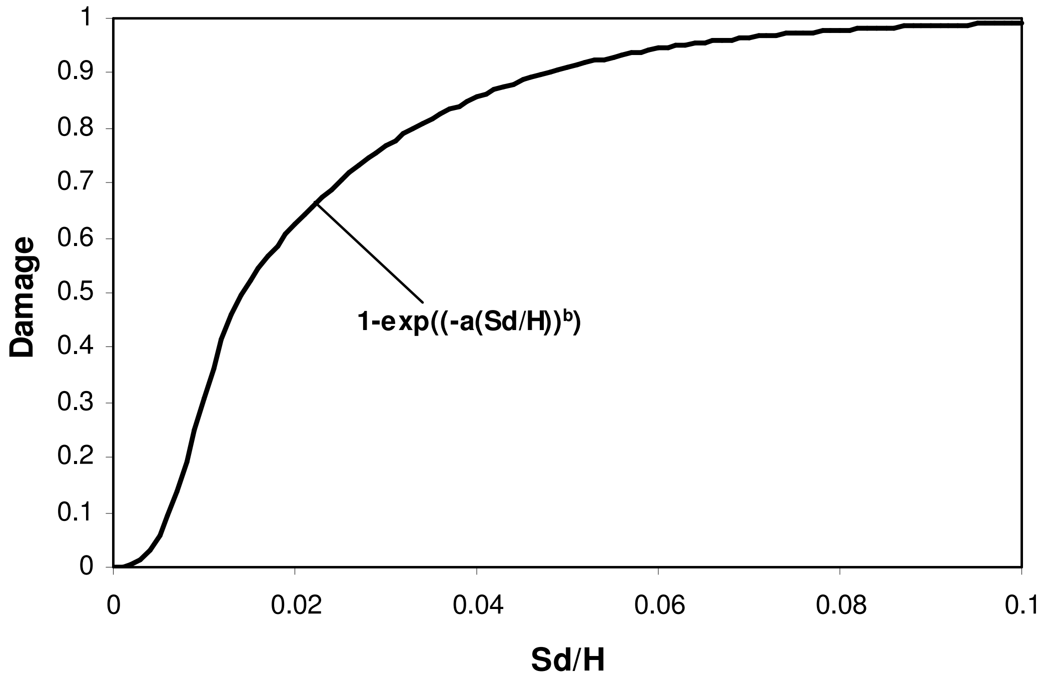



**Figure 5.1.** Typical Damage Curve

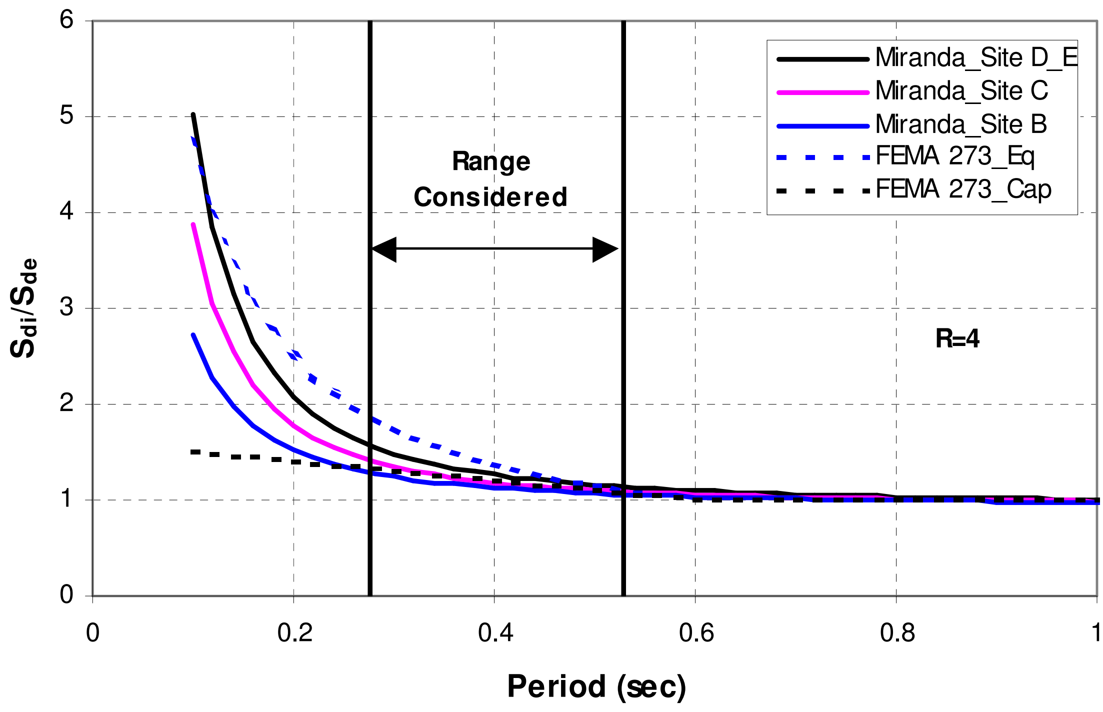



**Figure 5.2.** Comparison of $S_{di}/S_{de}$ Relations

The proposed procedure is developed on the basis of several assumptions, which are listed below:

- The earthquake magnitude in the region to which the method is applied is similar to the one that affected the reference site, i.e. Duzce.
- Attenuation relations are believed to represent the variation of the ground motion adequately.
- Construction practice does not show regional variations.
- Damage pattern observed in the reference site would be the same for other sites that have same distance to source and soil type.

The steps involved in this procedure can be outlined as follows;

- Step 1: Obtain site-specific response spectra using an appropriate attenuation model.
- Step 2: Calculate spectral displacement at the fundamental periods of interest.
- Step 3: Plot spectral displacement/n as a function of the fundamental period (or ns), ns representing number of stories considered in the Duzce study.
- Step 4: Convert spectral displacement to a damage index (cutoff value) by assuming an exponential relation.
- Step 5: Normalize all damage indexes at different sites and distances with the damage index obtained for the reference site, i.e. Duzce.
- Step 6: Modify Duzce cutoff values by multiplying them with the cutoff modification coefficients, i.e. normalized values calculated in Step 4.

### 5.2.1. Site Classification

Two major parameters used for site classification are the "distance to source ($d_s$)" and the "soil type (ST)". The sites were characterized by a pair of $d_s$ and ST bins. Five $d_s$ bins were selected in view of the variation in the response spectra with the distance. ST bins were determined based on the shear wave velocity (Vs) of the soil types employed by the Turkish Seismic Design Code [22]. Twenty different site classes were obtained from the combination of $d_s$ and ST bins, which are illustrated in Table 5.1. Note that type C2 represents the reference site (Duzce). This way, any region with a certain $d_s$ and ST is assigned a site class according to Table 5.1, excluding the sites located farther than 50 km from the source. The number of sites can easily be increased by incorporating other distance ranges and soil types (Vs>1000 m/s).

**Table 5.1.** Site Classification

| Soil Type | Shear Wave Velocity (m/s) | Distance to Source (km) | | | | |
|---|---|---|---|---|---|---|
| | | **0-4** | **5-8** | **9-15** | **16-25** | **26-50** |
| **A** | **701-1000** | A1 | A2 | A3 | A4 | A5 |
| **B** | **401-700** | B1 | B2 | B3 | B4 | B5 |
| **C** | **201-400** | C1 | C2 | C3 | C4 | C5 |
| **D** | **<200** | D1 | D2 | D3 | D4 | D5 |

### 5.2.2. Attenuation Models

Three attenuation relationships that are suitable for the source mechanism of the North Anatolian Fault were considered. The models developed by Boore et al. [8], Gulkan and Kalkan [15], and Abrahamson and Silva [1] were used to generate site-specific response spectra for all twenty sites included in Table 5.1. The relationships proposed by Boore et al., and Gulkan and Kalkan are the most convenient ones because they use the shear wave velocity directly to account for the soil type. For the attenuation relationship of Abrahamson and Silva, however, NEHRP amplification functions were applied on the rock motion to obtain response spectra. Since the uncertainty in attenuation models can be substantial, using different attenuation models is believed to give a better representation of the actual condition. Among the ones selected, Gulkan and Kalkan's model has been developed based on the local data recorded in Turkey. These models are compared at different distances as shown in Figure 5.3. Although at short distances Gulkan and Kalkan's model suggest lower estimates as compared to others, at far distances the situation is the other way around.

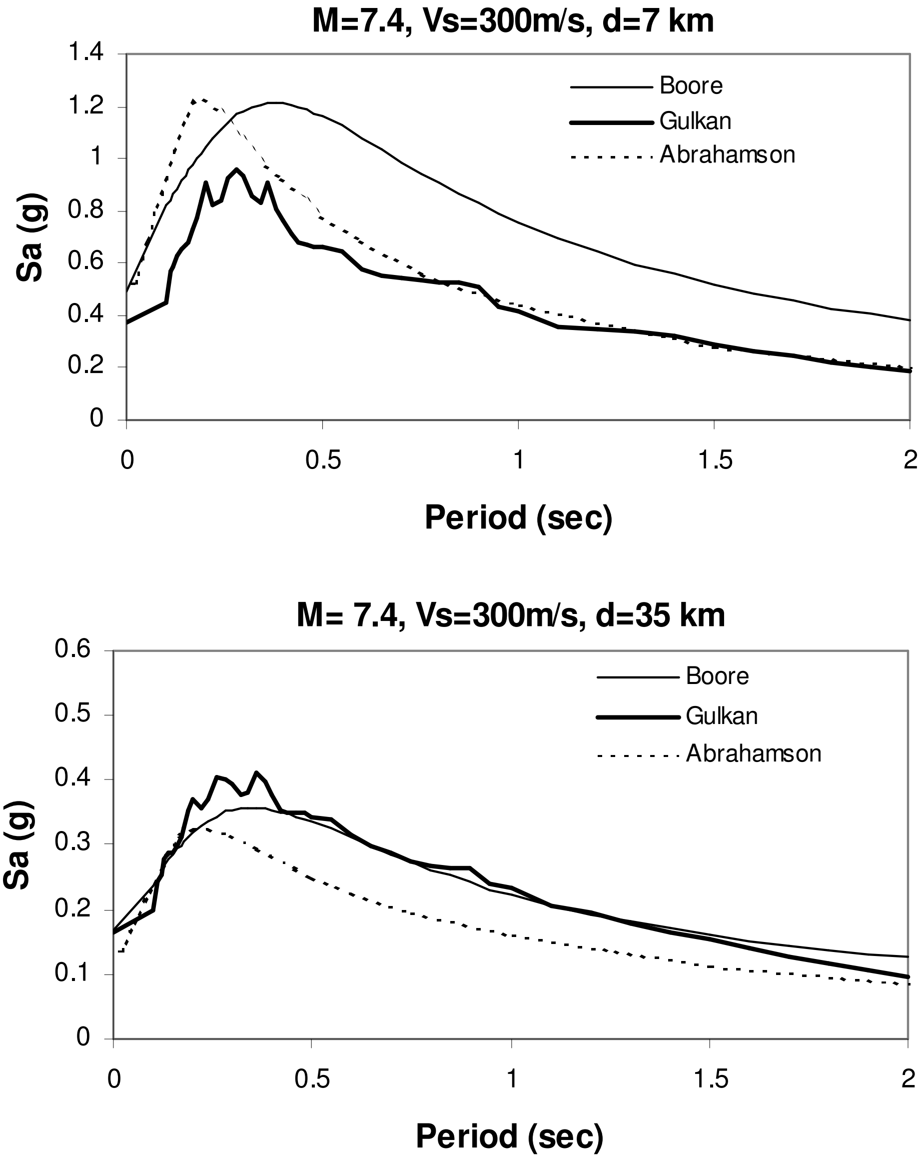



**Figure 5.3.** Comparisons of the Attenuation Models for d = 7 km and d = 35 km

### 5.2.3. Number of Stories and Period Relationship

Since the reference cutoff values were obtained as a function of the building height (number of stories), modification factors were also intended for the discrete height levels included in the database. Hence, a relationship between number of stories and the fundamental period was established based on the Turkish Seismic Code formulae. The mean values of the period and the number of stories obtained for the buildings contained in the Duzce seismic damage database are given in Table 5.2. Although the variation and dispersion of the period with number of stories is large for the buildings in the database, this would not significantly affect the modification factors as will be shown later.

**Table 5.2.** Period versus Number of Stories for Duzce Damage Database

| Number of Stories | Period (sec) |
|---|---|
| 2 | 0.275 |
| 3 | 0.355 |
| 4 | 0.433 |
| 5 | 0.504 |
| 6 | 0.529 |

### 5.2.4. Calculation of Spectral Displacement

A series of site-specific response spectra computed for a magnitude 7.4 earthquake and a shear wave velocity of 350 m/s are shown in Figure 5.4. The variations in the spectral ordinates were considered insignificant within the distance bins that were selected. Spectral displacement values were obtained from the calculated spectral accelerations ($S_a$) at all periods given in Table 5.2 for each of the twenty site classes.

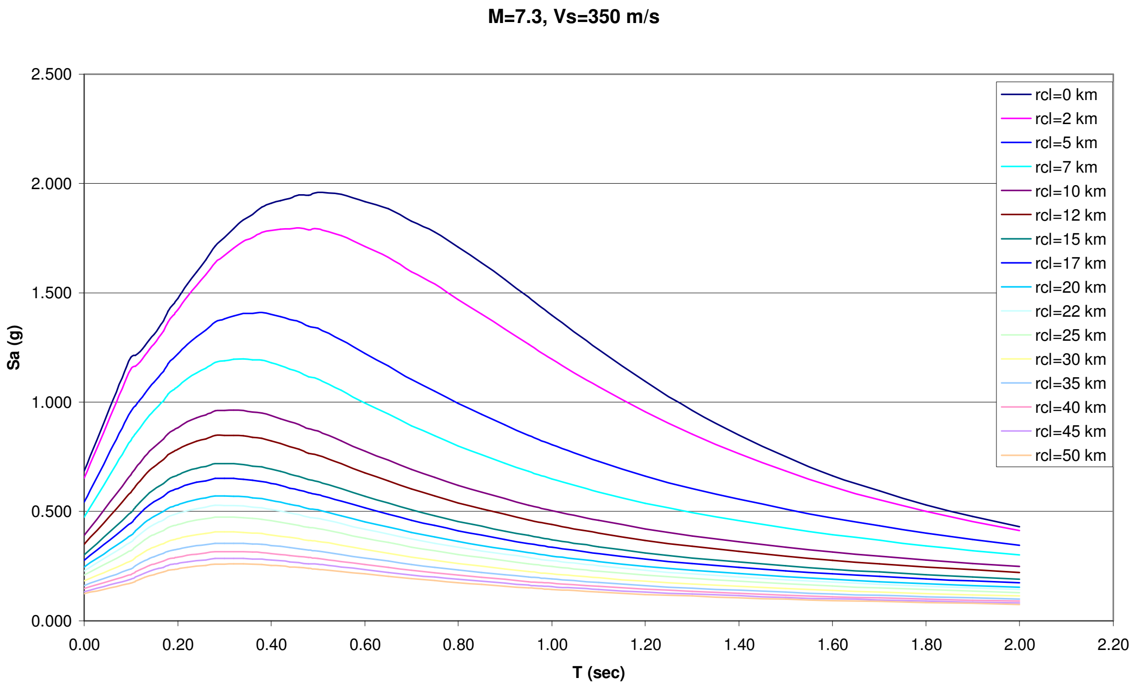
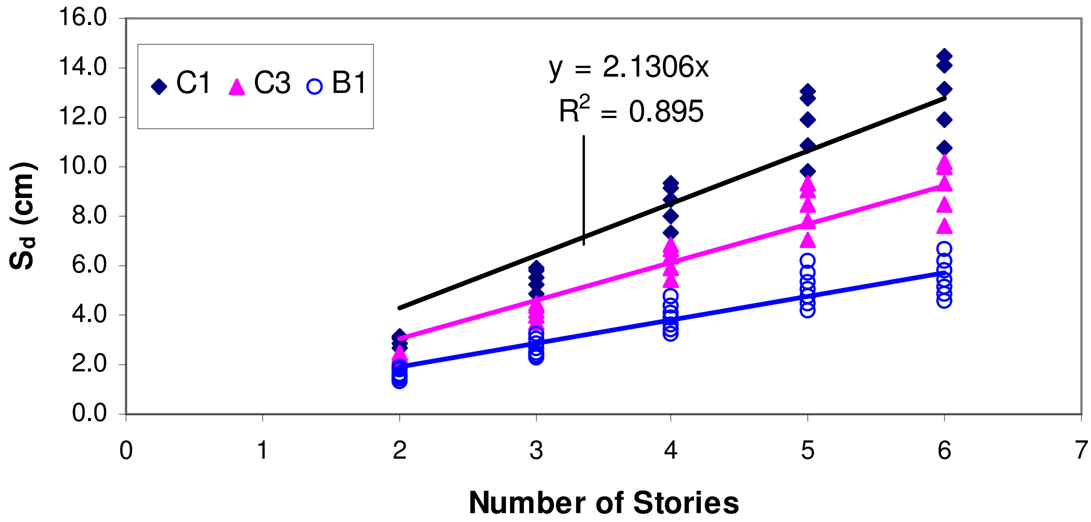



**Figure 5.4.** Acceleration Response Spectra

The spectral displacement normalized with number of stories (corresponding to the building period) is plotted against the number of stories as shown in Figure 5.5.



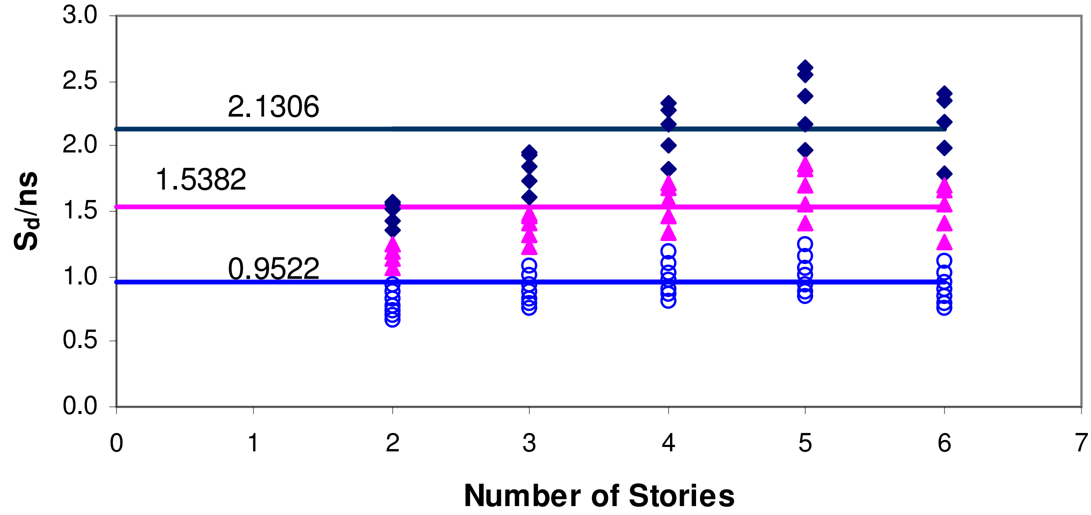




**Figure 5.5.** Normalized $S_d$ versus Number of Stories (Linear Representation)

This normalization was done to obtain a similar term that would mimic the average drift. The change of $S_d$ with the site class is also evident from these plots. When a linear regression is used to represent data a constant line develops, this is the simplest and the most convenient choice because it leaves out the number of stories. The trend of data implies a nonlinear behavior, so power function was used as an alternative to represent the data as displayed in Figure 5.6.

The modification coefficients were developed for both cases. The influence of the attenuation functions on the calculated response for site C3 is shown in Figure 5.7. Abrahamson and Silva yields similar results to that of Boore et al., Gulkan and Kalkan, however, provides lower estimates of $S_d$ at all periods.

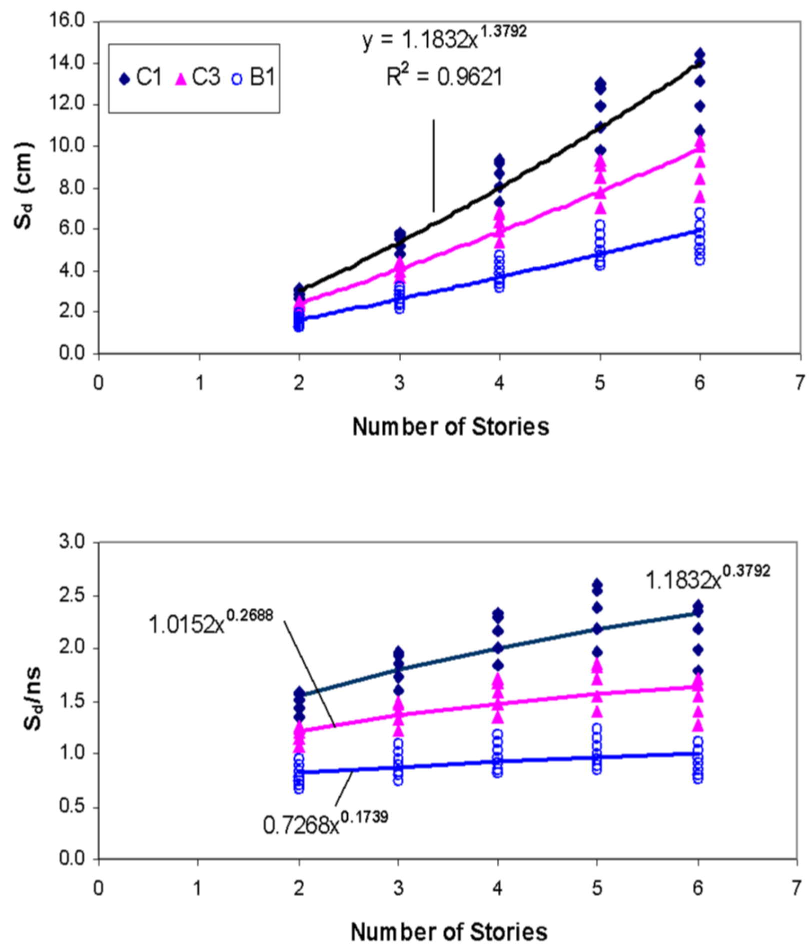



**Figure 5.6.** Normalized $S_d$ versus Number of Stories (Non-linear Representation)

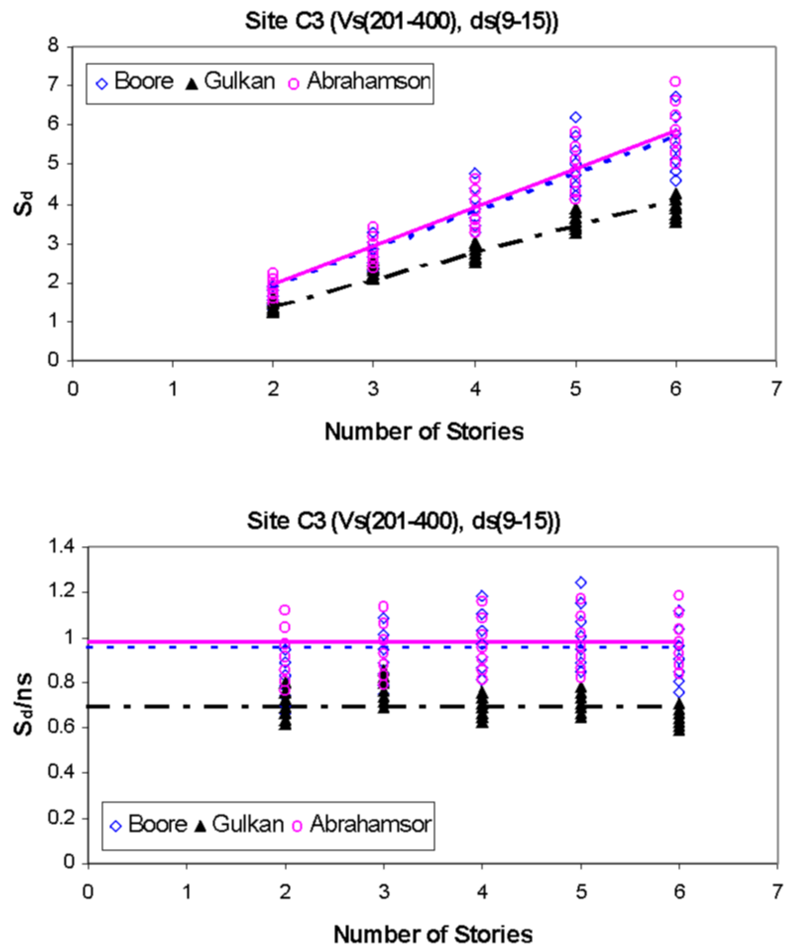



**Figure 5.7.** Influence of Attenuation Relationship

### 5.2.5. Calculation of Modification Factors

Once $S_d$ values for all sites are computed, they are translated into damage terms. In the vulnerability assessment procedure developed for Duzce, there is a reverse relationship between the cutoff value and the damage score of the evaluated building. In other words, as the cutoff value is raised the number of "unsafe" buildings decreases. In view of this relation, the change of the cutoff value (CV) with the normalized spectral displacement was assumed to follow a similar trend observed between damage and $S_d/H$ (Figure 5.1). Thus, the following function is assumed to reflect the relation between the CV and the normalized spectral displacement ($S_d$/ns);

$$CV = f\left[\frac{1}{1-e^{-S_d/ns}}\right] \tag{5.1}$$

Since the objective is to obtain cutoff modification coefficients (CMC) to be applied on the reference cutoff values ($CV_r$), the variable of the function in Equation 5.1 can be used to get CMC values. The calculated CMC values are presented in Tables 5.3, 5.4 and 5.5 for the three attenuation models employed. These results verify the observation that non-linear and linear formulations of the spectral displacement versus number of story relation provide similar values. The CMC can take values between 0.78-3.90, 0.80-2.14, and 0.83-3.03 for the attenuation relationships developed by Boore et al., Gulkan and Kalkan, and Abrahamson and Silva, respectively. The CMC value for the reference site class C2 is 1.0 because of the normalization with respect to this site. Obviously, at better site conditions and farther distances cutoff values should be larger. These CMC values were multiplied with the respective reference cutoff values to obtain the cutoff values for other site classes. Modified cutoff values are computed merely from Equation 5.2, which can handle negative as well as positive values of reference cutoff values. As evidenced in these tables, among all of the attenuation models, the one by Gulkan and Kalkan led to narrower range of modification values, meaning that performance differences of the buildings between the sites would be less.

**Table 5.3.** Cutoff Modification Coefficients (CMC) for the Attenuation Relationship of Boore et. al. [8]

| | | LINEAR | | | | | NON-LINEAR | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Distance (km) | | | | | | | | | |
| **ns** | **Vs (m/s)** | **0-4** | **5-8** | **9-15** | **16-25** | **26+** | **0-4** | **5-8** | **9-15** | **16-25** | **26+** |
| **2-3** | **0-200** | 0.778 | 0.824 | 0.928 | 1.128 | 1.538 | 0.764 | 0.826 | 0.959 | 1.207 | 1.726 |
| | **201-400** | 0.864 | 1.000 | 1.240 | 1.642 | 2.414 | 0.875 | 1.000 | 1.239 | 1.654 | 2.496 |
| | **401-700** | 0.970 | 1.180 | 1.530 | 2.099 | 3.177 | 0.978 | 1.150 | 1.468 | 2.010 | 3.101 |
| | **701+** | 1.082 | 1.360 | 1.810 | 2.534 | 3.900 | 1.075 | 1.288 | 1.675 | 2.329 | 3.640 |
| **4-6** | **0-200** | 0.778 | 0.824 | 0.928 | 1.128 | 1.538 | 0.781 | 0.825 | 0.928 | 1.125 | 1.535 |
| | **201-400** | 0.864 | 1.000 | 1.240 | 1.642 | 2.414 | 0.865 | 1.000 | 1.242 | 1.642 | 2.424 |
| | **401-700** | 0.970 | 1.180 | 1.530 | 2.099 | 3.177 | 0.970 | 1.182 | 1.537 | 2.106 | 3.204 |
| | **701+** | 1.082 | 1.360 | 1.810 | 2.534 | 3.900 | 1.082 | 1.364 | 1.824 | 2.552 | 3.948 |

$$CV = CV_r + ABS(CV_r)*(CM-1) \tag{5.2}$$

**Table 5.4.** Cutoff Modification Coefficients (CMC) for the Attenuation Relationship of Gulkan and Kalkan [15]

| | | LINEAR | | | | | NON-LINEAR | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Distance (km) | | | | | | | | | |
| **ns** | **Vs (m/s)** | **0-4** | **5-8** | **9-15** | **16-25** | **26+** | **0-4** | **5-8** | **9-15** | **16-25** | **26+** |
| **2-3** | **0-200** | 0.791 | 0.840 | 0.931 | 1.083 | 1.359 | 0.748 | 0.815 | 0.926 | 1.099 | 1.413 |
| | **201-400** | 0.932 | 1.000 | 1.126 | 1.334 | 1.706 | 0.892 | 1.000 | 1.171 | 1.431 | 1.896 |
| | **401-700** | 1.032 | 1.113 | 1.263 | 1.508 | 1.946 | 1.006 | 1.142 | 1.355 | 1.678 | 2.252 |
| | **701+** | 1.115 | 1.207 | 1.376 | 1.652 | 2.144 | 1.106 | 1.265 | 1.514 | 1.891 | 2.558 |
| **4-6** | **0-200** | 0.791 | 0.840 | 0.931 | 1.083 | 1.359 | 0.799 | 0.843 | 0.932 | 1.081 | 1.357 |
| | **201-400** | 0.932 | 1.000 | 1.126 | 1.334 | 1.706 | 0.939 | 1.000 | 1.121 | 1.324 | 1.695 |
| | **401-700** | 1.032 | 1.113 | 1.263 | 1.508 | 1.946 | 1.037 | 1.110 | 1.253 | 1.492 | 1.927 |
| | **701+** | 1.115 | 1.207 | 1.376 | 1.652 | 2.144 | 1.120 | 1.201 | 1.363 | 1.630 | 2.118 |

**Table 5.5.** Cutoff Modification Coefficients (CMC) for the Attenuation Relationship of Abrahamson and Silva [1]

| | | LINEAR | | | | | NON-LINEAR | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Distance (km) | | | | | | | | | |
| **ns** | **Vs (m/s)** | 0-4 | 5-8 | 9-15 | 16-25 | 26+ | 0-4 | 5-8 | 9-15 | 16-25 | 26+ |
| **2-3** | **0-200** | 0.826 | 0.917 | 1.084 | 1.362 | 1.887 | 0.850 | 0.967 | 1.185 | 1.554 | 2.288 |
| | **201-400** | 0.873 | 1.000 | 1.219 | 1.575 | 2.236 | 0.870 | 1.000 | 1.240 | 1.642 | 2.438 |
| | **401-700** | 0.919 | 1.077 | 1.341 | 1.765 | 2.542 | 0.903 | 1.055 | 1.329 | 1.783 | 2.676 |
| | **701+** | 0.999 | 1.205 | 1.539 | 2.065 | 3.032 | 0.947 | 1.125 | 1.439 | 1.957 | 2.970 |
| **4-6** | **0-200** | 0.826 | 0.917 | 1.084 | 1.362 | 1.887 | 0.825 | 0.917 | 1.085 | 1.362 | 1.894 |
| | **201-400** | 0.873 | 1.000 | 1.219 | 1.575 | 2.236 | 0.872 | 1.000 | 1.221 | 1.574 | 2.241 |
| | **401-700** | 0.919 | 1.077 | 1.341 | 1.765 | 2.542 | 0.919 | 1.078 | 1.344 | 1.763 | 2.550 |
| | **701+** | 0.999 | 1.205 | 1.539 | 2.065 | 3.032 | 1.001 | 1.208 | 1.545 | 2.069 | 3.046 |

## 5.3. AN APPLICATION OF THE PROPOSED PROCEDURE

As alluded to before, Istanbul is on the verge of being struck by a devastating earthquake, similar to the one that hit Duzce. Assuming that the construction practices in Duzce and in Istanbul are similar, the procedure would provide reasonable results when applied to Istanbul. To see the extent and relativity of the expected damage or the layout of the risk within Istanbul an exercise was undertaken, in which, all buildings in Duzce database were assumed to portray buildings all over Istanbul. In other words, a uniform exposure that is identical to the compiled database for Duzce is assigned to all districts of Istanbul. The earthquake scenario "Model A" and shear wave velocity estimates of JICA study [19] were employed to model the fault and to classify the sites. The modified cutoff values were applied and all buildings were identified as "safe", "unsafe" or "intermediate" in all districts of Istanbul. It should be pointed out that “safe” buildings represent the structures that would experience none or light damage states, “unsafe” buildings include those that are expected to suffer severe damage or would collapse, and “intermediate” buildings might encompass buildings with all degrees of damage that can not be clearly identified.

Figures 5.8 to 5.10 display results obtained by using the attenuation relationship of Boore et al. [8]. In these figures, results are presented in the form of the ratio of the classified buildings to the total number of buildings. The visual plots indicate some spotty areas, which reflect the local soil profile. The effect of distance to source is clearly observed. The range of safe buildings varies from 38% to 60% depending on the site class. Unsafe buildings constitute 1-40 % and buildings identified as intermediate, which represent buildings that could not be clearly classified as safe or unsafe, have a share of 21-39%. Of the indeterminate buildings, around 50% were moderately damaged, 38% had light or no damage and 10% were severely damaged in Duzce.

The JICA estimates of the heavily damaged building percentages are shown in Figure 5.11. These results were obtained based on the actual exposures extracted from the data released by the State Statistics Institute of Turkey; the apparent discrepancy is due to this fact.

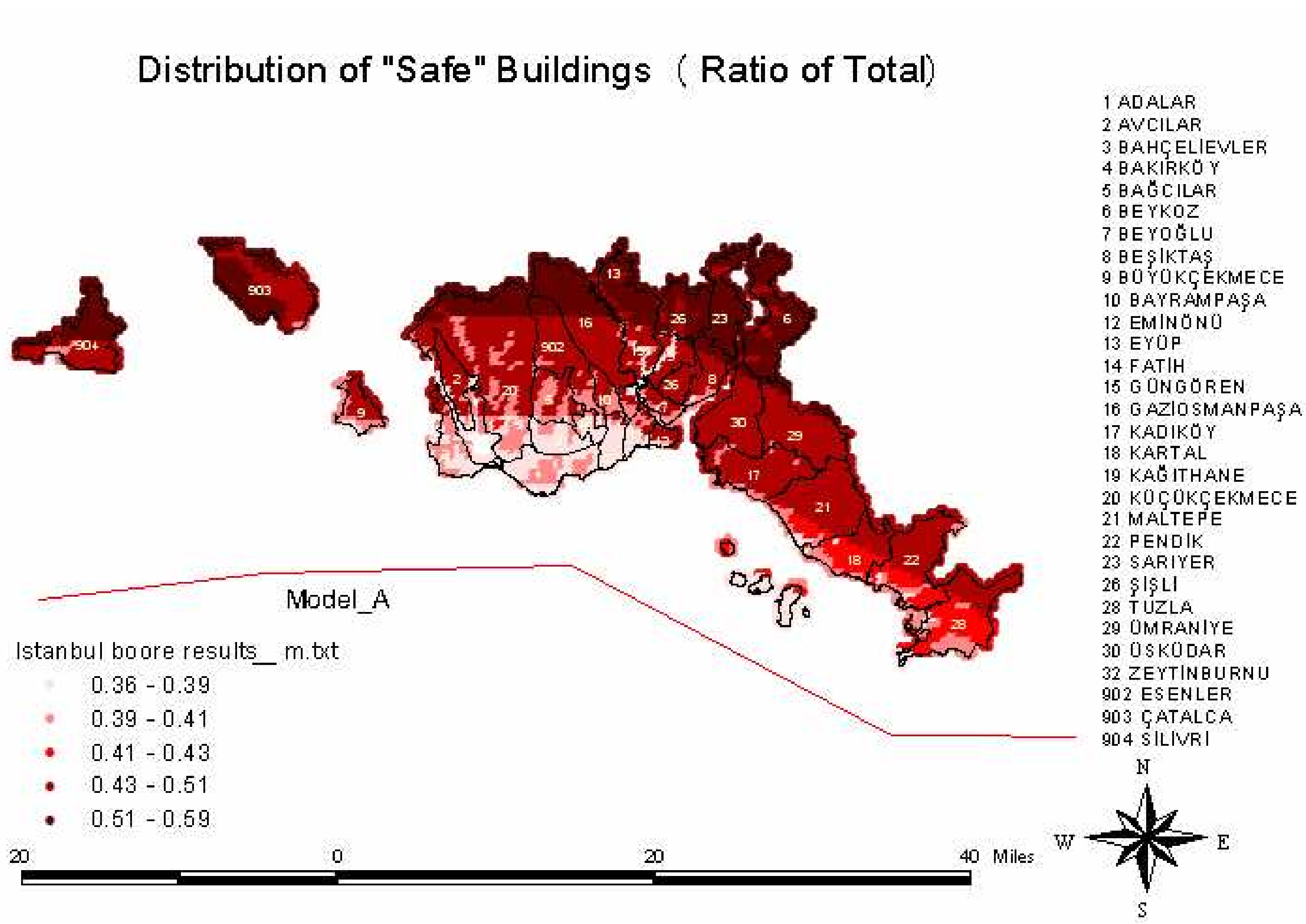



**Figure 5.8.** Identification of Safe Buildings Based on Boore et al.'s [8] Attenuation Relationship

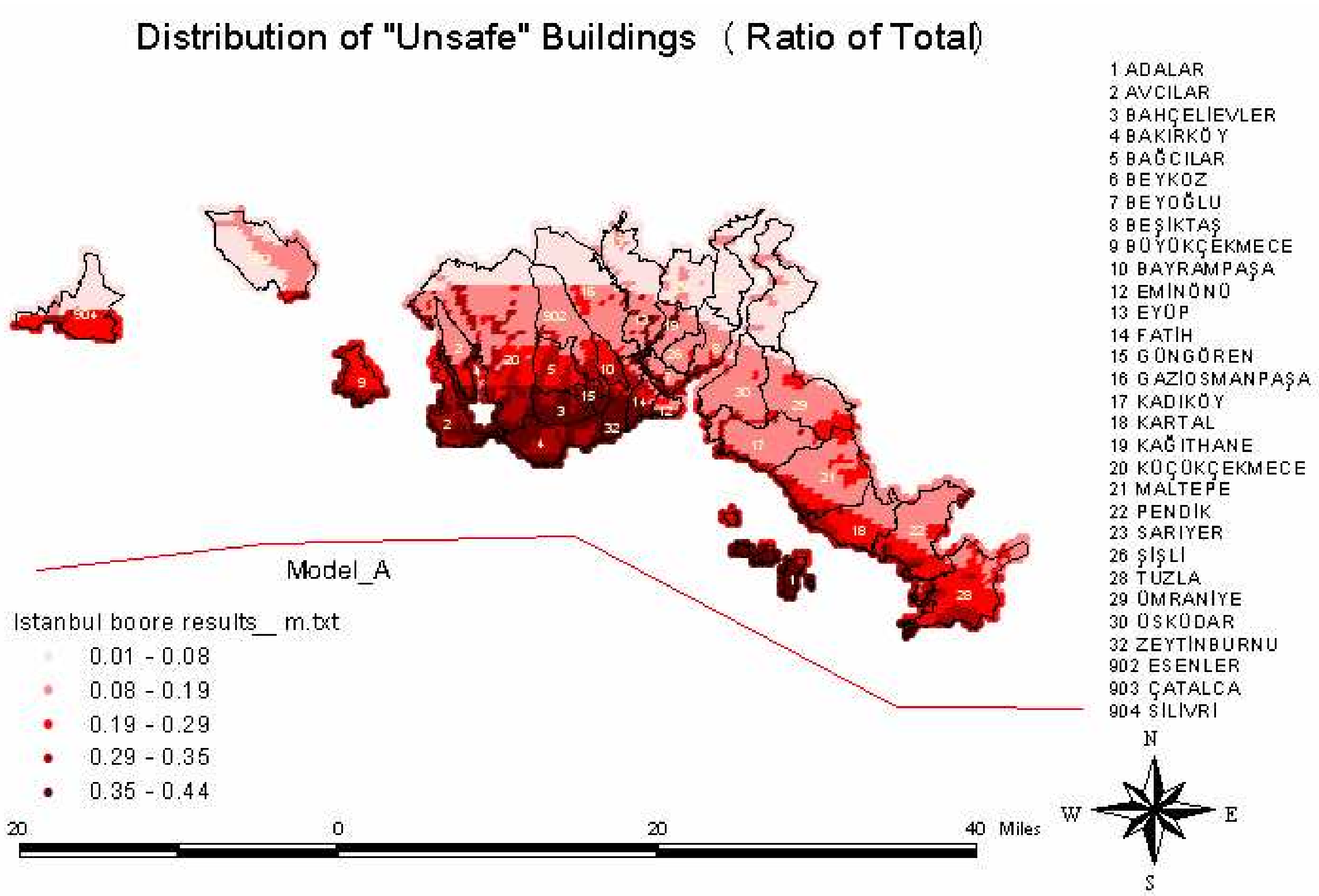



**Figure 5.9.** Identification of Unsafe Buildings Based on Boore et al.'s [8] Attenuation Relationship

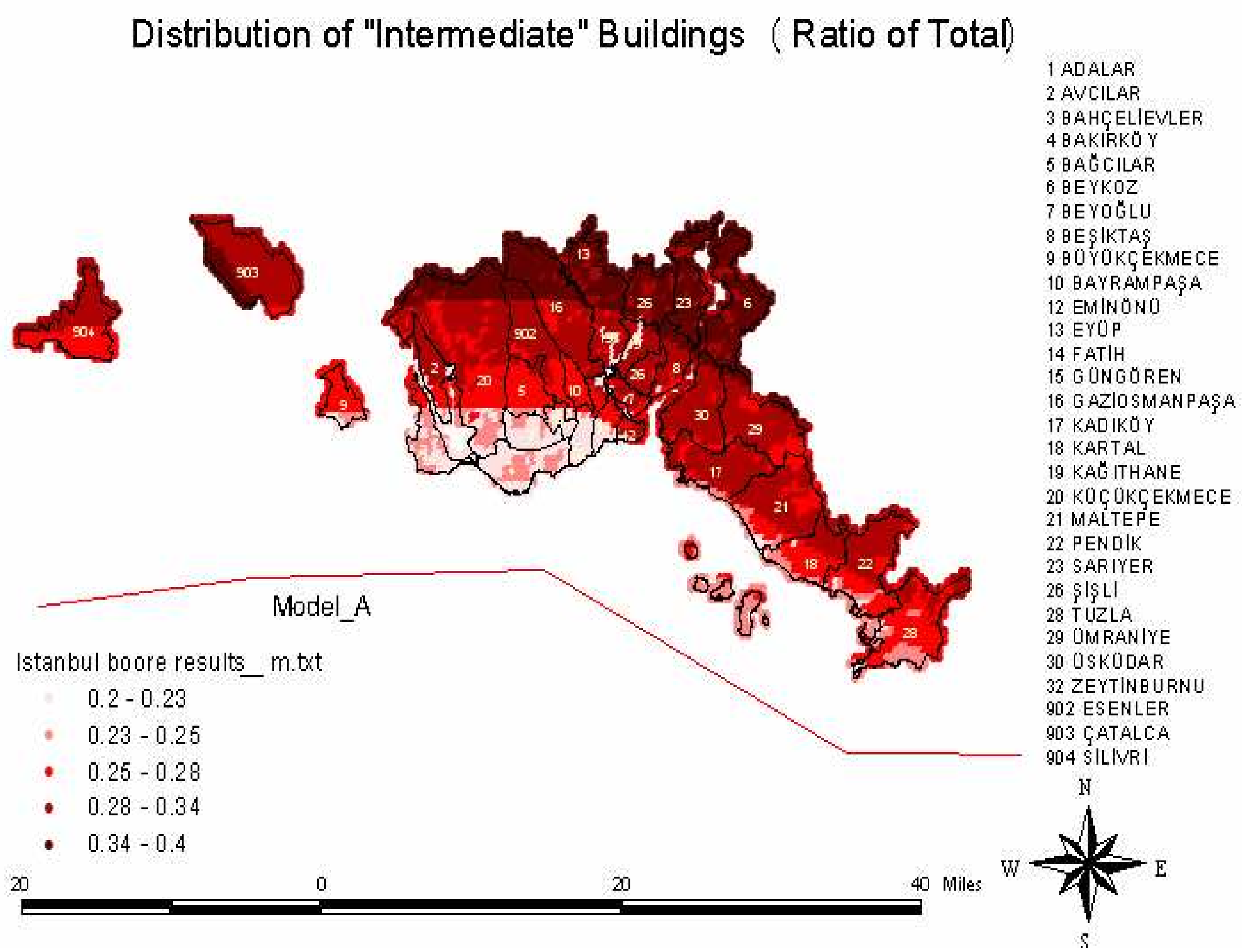



**Figure 5.10.** Identification of Intermediate Buildings Based on Boore et al.'s [8] Attenuation Relationship

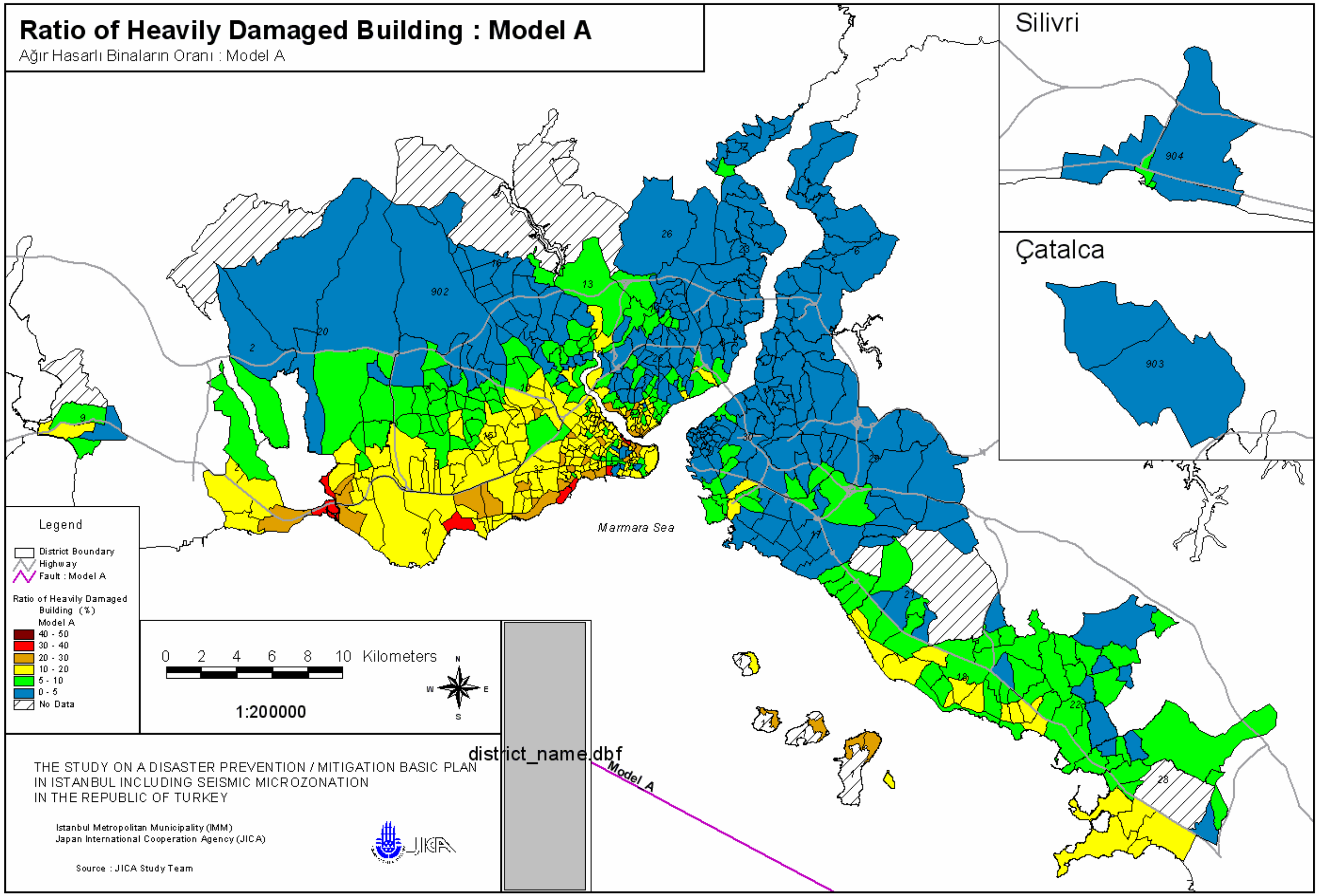



**Figure 5.11.** JICA Estimates of Heavily Damaged Buildings (from JICA, 2002)

# CHAPTER 6: SUMMARY AND CONCLUSIONS

## 6.1. SUMMARY

In this study, a new statistical model was developed for the preliminary seismic vulnerability assessment of low- to mid-rise existing reinforced concrete structures by using a multivariate statistical procedure, called discriminant analysis. The database used in the discriminant analysis was compiled by reconnaissance surveys conducted in Duzce after 17 August 1999 Kocaeli and 12 November 1999 Duzce earthquakes.

Considering the characteristics of the damaged reinforced concrete structures, the following parameters were selected as the basic estimation parameters to be used in the proposed models: "number of stories" (*ns*), "minimum normalized lateral stiffness index" (*mnlstf*), "minimum normalized lateral strength index" (*mnlstr*), "normalized redundancy score" (*nrs*), "soft story index" (*ssi*) and "overhang ratio" (*or*). The discriminant analyses were conducted in two different stages. In the first stage, two damage state groupings were utilized by considering LSPL and IOPL. Later, optimal classification methodology was developed for the two-damage state grouping for the purpose of improving the correct classification rates. All statistical analyses were carried out by using the SPSS software.

The validity of the proposed statistical models was verified by applying the models to the Erzincan, Afyon and Bingol damage databases. Moreover, applications of the previous vulnerability assessment methodologies proposed by Hassan and Sozen [17] and Ersoy and Tankut [13] are utilized in order to compare the effectiveness of the proposed model in this study with respect to damage estimation.

It has been shown that vulnerability assessment procedures based on observed damage from a particular region can be extrapolated to other sites having similar construction practices and building stock. The variation of ground motion parameters that have known relationship to the damage of buildings are captured using attenuation models that reflect the properties of the sites, i.e. the distance to source and soil type. When the assumptions made are considered to be convincing, which is the case for Istanbul, high-risk areas and vulnerable regions can be identified in a reliable way. This would help determine the rank of regional vulnerability and the mitigation priorities, especially for the mega city of Istanbul for which a large earthquake is due.

## 6.2. CONCLUSIONS

The following conclusions can be stated based on the results of this study:

- Among the six damage-inducing parameters considered, the number of stories is found to be the most effective discriminating parameter in all damage state classifications.
- The discriminant analyses are carried out in two stages. In the case of two damage state groupings, correct classification rates of 69.0% in LSPL and 72.5% in IOPL are achieved.
- Optimal classification procedure improves the efficiency and accuracy of the proposed model in the case of two damage state groupings. The correct classification rate in determining the severely damaged and collapsed structures is increased to 80.3 %, whereas total misclassification rate that may

lead to loss in human lives is reduced to 2.7 %, in the case of Duzce damage database.

- The proposed classification models are checked by using the Erzincan, Afyon and Bingol damage databases. The high correct classification rates demonstrate the predictive ability of the proposed methodology. The individual correct classification rates as well as the overall classification rates for all damage databases in all models indicate that the discriminant analysis technique is a rational approach for the seismic vulnerability assessment of existing reinforced concrete buildings.
- Applications of the previous vulnerability assessment methodologies proposed by Hassan and Sozen [17] and Ersoy and Tankut [13] were utilized in order to compare the effectiveness of the models proposed in this study with respect to those proposed by above research works. The results indicate that the parameters considered in this study are more efficient in identifying the damage states.
- The technique presented in Chapter 4 is a reasonable theoretical approach that uses available tools to predict the spatial variation of ground motion. Further improvements to the procedure can be made, especially in the intermediate steps, but the end results, which are the modification coefficients, would not be influenced considerably. Besides, the assumptions and approximations already introduced are far beyond the accuracy that would be gained this way.
- The seismic vulnerability assessment methodology for existing low-rise to mid-rise reinforced concrete buildings developed in this study is still open to improvements and revisions based on new additional data and the inclusion of more estimation parameters.

## 6.3. FUTURE RECOMMENDATIONS

In view of the results and conclusions of this study, the following points should be taken into consideration in future studies:

- The earthquake data, collected immediately in the aftermath of an earthquake, should be as complete and random as possible in order to avoid any bias towards a particular damage state.
- The subjectivity in the assignment of the damage states of structures should be minimized by developing a common damage assessment technique.
- Additional damage-inducing parameters like excitation and soil parameters might be developed as input to the discriminant analysis and their significance should be investigated.

# REFERENCES


[1] Abrahamson, N.A. and Silva, W.J, "Empirical Response Spectral Attenuation Relations for Shallow Crustal Earthquakes", Seismological Research Letters, Volume 68, Number 1, January/February, pp.94-127, 1997.

[2] Altin, S., "Strengthening of Reinforced Concrete Frames with Reinforced Concrete Infills", Ph.D. Thesis, Middle East Technical University, 1990.

[3] Applied Technology Council, "Rapid Visual Screening of Buildings for Potential Hazards" A Handbook, (ATC-21), Federal Management Agency Report FEMA-154, California, 1988.

[4] Applied Technology Council, "A Handbook for Seismic Evaluation of Existing Building" (ATC-22), Federal Management Agency Report FEMA-178, California, 1989.

[5] Applied Technology Council, "NEHRP Guidelines for the Seismic Rehabilitation of Buildings" (ATC 33), Federal Management Agency Report FEMA-273, Washington, 1997.

[6] Askan, A., "Stochastic Methods for the Estimation of Potential Seismic Damage", M.Sc. Thesis, Middle East Technical University, Ankara, 2002.

[7] Aydogan, V., "Seismic Vulnerability Assessment of Existing Reinforced Concrete Buildings in Turkey", M.Sc. Thesis, Middle East Technical University, Ankara, 2003.

[8] Boore, D.M, Joyner, W.B. and Fumal, T.E., "Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work", Seismological Research Letters, Volume 68, Number 1, January/February, pp.128-153, 1997.

[9] Comartin, C. D., "A Progress Report on ATC-55: Evaluation and Improvement of Inelastic Seismic Analysis Procedures", Fall, 2002.

[10] Erduran, E. and Yakut A., "Drift Based Damage Functions for Reinforced Concrete Columns", Submitted to Computers & Structures for possible publication, in Review, 2003

[11] Ersoy, U., "Basic Principles for the Design of Seismic Resistant Reinforced Concrete Structures", Turkish Jordanian Symposium on Earthquake, Amman, 1987.

[12] Ersoy, U., "Seismic Resistant Reinforced Concrete Structures-Design Principles", Journal of Islamic Academy of Sciences, pp.20-26, 1988.

[13] Ersoy, U. and Tankut, T., " A Proposal for the Seismic Design of Low-rise Buildings, " Turkiye Muhendislik Haberleri (Turkish Engineering News), No.386, pp. 40-43, 1996 (in Turkish).

[14] Ersoy, U., "Influence of Architecture and Structural System of Buildings on the Seismic Capacity", Deprem Guvenli Konut Sempozyumu (Earthquake-safe Building Symposium), Ankara, Boyut Matbaacilik, 1999. (in Turkish).

[15] Gulkan and Kalkan, "Attenuation Modeling of Recent Earthquakes in Turkey", Journal of Seismology, 6:397-409. Kluwer Academic Publishers, 2002.

[16] Gulkan, P. and Sozen, M.A., “Procedure for Determining Seismic Vulnerability of Building Structures”, ACI Structural Journal, Vol. 96, No.3, pp.336-342, 1999.

[17] Hassan, A.F. and Sozen, M.A., “Seismic Vulnerability Assessment of Low-Rise Buildings in Regions with Infrequent Earthquakes”, ACI Structural Journal, Vol.94, No.1, pp.31-39, 1997.

[18] Japan Building Disaster Prevention Association, “Standard and Commentary for Evaluation of Seismic Capacity of Existing Reinforced Concrete Buildings”, Tokyo, 2001.

[19] Japan International Cooperation Agency (JICA) and Istanbul Metropolitan Municipality (IBB), “The Study on A Disaster Prevention / Mitigation Basic Plan in Istanbul including Seismic Microzonation in the Republic of Turkey”, Final Report, December, 2002.

[20] Klecka, W.R., “Discriminant Analysis”, Sage University Papers, No:07-019, 1980.

[21] Marjani, F., “Behavior of Brick Infilled Reinforced Concrete Frames Under Reversed Cyclic Loading”, Ph.D. Thesis, Middle East Technical University, 1990.

[22] Ministry of Public Works and Settlement, General Directorate of Disaster Affairs, “ Specifications for Structures to be Built in Disaster Areas”, Ankara, 1998.

[23] Ohkuba, M., “Current Japanese System on Capacity and Retrofit Techniques for Existing Reinforced Concrete Buildings and Post-Earthquake Damage Inspection and Restoration Techniques”, University of California, San Diego, Report No. SSRP-91/02, 1991.

[24] Otani, S., “Seismic Vulnerability Evaluation Methods in Japan,” Seismic Vulnerability Assessment and Rehabilitation Strategies for Turkey, Scientific and Technical Research Council of Turkey-World Bank Workshop, Ankara, May 2000.

[25] Ozcebe, G., Yucemen, M.S, Aydogan V. and Yakut A., “Preliminary Seismic Vulnerability Assessment of Existing Reinforced Concrete Buildings in Turkey - Part I: Statistical Model Based on Structural Characteristics”, NATO Workshop, Izmir May 2003.

[26] Pay, C., “A New Methodology for the Seismic Vulnerability Assessment of Existing Buildings in Turkey”, M.Sc. Thesis, Middle East Technical University, Ankara, 2001.

[27] Sharma, S., “Applied Multivariate Techniques”, John Wiley & Sons, 1996.

[28] SPSS Inc., SPSS Base 11.0 User’s Guide, Chicago, Illinois, 2001.

[29] Yakut, A., Aydogan, V., Ozcebe, G. and Yucemen, M.S., “Preliminary Seismic Vulnerability Assessment of Existing Reinforced Concrete Buildings in Turkey - Part II: Inclusion of Site Characteristics” NATO Workshop, Izmir May 2003.