SEISMIC VULNERABILITY ASSSESSMENT OF REINFORCED CONCRETE SCHOOL BUILDINGS IN TURKEY

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ABSTRACT

SEISMIC VULNERABILITY ASSESSMENT OF REINFORCED CONCRETE SCHOOL BUILDINGS IN TURKEY

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The walk-down and preliminary seismic vulnerability assessment procedures for low-to-mid-rise RC school buildings are examined by detailed analysis. The correlation of significant damage-inducing parameters and the performance is studied by using results obtained from detailed analysis. The databases of the Ministry of Environment and Urban Planning and the Governorship of İstanbul are used for this study. 321 RC School buildings that were assessed using the 2007 Turkish Earthquake Code were employed. The parameters used for walk-down and preliminary analysis methods are analyzed statistically for the databases used. A representative database of buildings consisting of 36 buildings from different heights was formed. All 36 RC school buildings are also analyzed by 2013 and 2019 versions of Urban Renewal Law and 2007 and the new (2018) version of the Turkish EQ Code. Fragility analysis for 3, 4, and 5-story RC school buildings are studied, and Fragility Curves are formed using both bilinear and stiffness and strength degradation (Modified Clough) material models. Obtained F.C.s are validated by the observed performance data of RC school buildings after 2003 Bingöl EQ F.C.s are formed for repeated EQs. Therefore, the study provides a wide and reliable database for seismic vulnerability assessment of RC school buildings in Turkey.

Keywords: Earthquake, Performance, Vulnerability, Fragility, School

ÖZ

TÜRKİYE'DEKİ BETONARME OKUL BİNALARININ DEPREM PERFORMANLARININ BELİRLENMESİ

Mazılıgüney, Levent
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Tez çalışması kapsamında, alçak ve orta yükseklikteki betonarme okul binaları için sokak taraması ve ikinci kademe deprem davranışı belirleme yöntemleri detaylı analizler kullanılarak incelenmiştir. Hasara neden olan belirgin etkenlerle hasar arasındaki ilişki detaylı analizlerden elde edilen sonuçlar kullanılarak araştırılmıştır. Çalışma kapsamında, Çevre ve Şehircilik Bakanlığı ve İstanbul Valiliği'nin verilerinden faydalanılmıştır. 2007 Türk Deprem Yönetmeliğine göre analiz edilen 321 betonarme okul binası kullanılmıştır. Sokak taraması ve ön değerlendirme yöntemlerinde kullanılan parametreler, incelenen binalar için istatistiksel olarak analiz edilmiştir. Farklı yüksekliklerde 36 binadan oluşan temsili bir bina seti oluşturulmuştur. 36 okul binasının tamamı Riskli Yapı Tespit Esasları 2012 ve 2019 versiyonlarına göre ve 2007 ve (2018) Türkiye Bina Deprem Yönetmeliği'ne göre analiz edilmiştir. 3, 4 ve 5 katlı betonarme okul binaları için hasar görebilirlik analizleri çalışılmış ve kırılganlık eğrileri iki doğrulu ve dayanım ve rijitlik azaltımlı (Modifiye Clough) malzeme modelleri kullanılarak oluşturulmuştur. Elde edilen kırılganlık eğrileri 2003 Bingöl depremi sonrası betonarme okul binalarından elde edilen performans verileri ile karşılaştırılmıştır. Tekrar eden depremler için kırılganlık eğrileri oluşturulmuştur. Sonuç olarak, bu çalışma Türkiye'deki betonarme okul binalarının deprem güvenliği açısından geniş ve güvenilir bir veri tabanı sağlamaktadır.

Anahtar Kelimeler: Deprem, Performans, Hassasiyet, Kırılganlık, Okul

To my daughters, Nazlı Hilâl, Azrâ, and Bernâ

They are the brightest lights of my sky, even in the darkest nights of life.

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Finally, I wish to dedicate this study to my family. Their enduring love, unconditional support, and encouragement have been the real inspiration that I always felt for this study, and every good thing I do throughout my life.

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LIST OF ABBREVIATIONS

ABBREVIATIONS

ADRS	Acceleration displacement response spectra
AFAD	Disaster and Emergency Management Presidency of Turkey
ASCE	American Society of Civil Engineers
ASCE/SEI	Structural Engineering Institute of American Society of Civil Engineers
ATC	Applied Technology Council
BCPI	Basic capacity index
BKS	Building use category
BSS	Basic structural score
BYS	Building height category
C	Collapse
CD	Controlled Damage
CO	Continued Operation
CP	Collapse prevention
DCM	Displacement Coefficient Method
DTS	Seismic design category
EI	Energy index
EPA	Effective peak acceleration
EQ	Earthquake
EQs	Earthquakes
ESDOF	Equivalent single degree of freedom
FBD	Force based design
F.C.	Fragility curve
F.C.s	Fragility curves
FEMA	Federal Emergency Management Agency
GM	Ground motion

HAZUS	Hazard United States
IM	Intensity measure
IO	Immediate occupancy
ISMEP	İstanbul Seismic Risk Mitigation and Emergency Preparedness Projects
IPCU	İstanbul Project Coordination Unit
LD	Limited Damage
LS	Life Safety
MDOF	Multi degree of freedom
METU	Middle East Technical University
PBD	Performance-Based assessment and re-Design
PGA	Peak ground acceleration
PGV	Peak ground velocity
PS	Performance score
PSA	Peak spectral acceleration
RC	Reinforced concrete
S	Satisfactory
SVA	Seismic vulnerability assessment
SDOF	Single degree of freedom
TBEC	Turkish Building Earthquake Code
TEC	Turkish Earthquake Code
TS	Turkish Standards
TSS	Total structural score
U	Unsatisfactory
URL	Urban Renewal Law

LIST OF SYMBOLS

SYMBOLS

A_c	Total cross-section area of the concrete columns
A_m	Total cross-section area of the masonry walls
A_s	Total cross-section area of the shear walls
C_0	Coefficient that converts the SDOF spectral displacement to MDOF (elastic) roof displacement
C_1	Coefficient for modifying the elastic displacement to the corresponding inelastic displacement
C_2	Coefficient that incorporates the effects of pinched hysteretic shape, stiffness degradation and strength deterioration
C_3	Coefficient that reflects the increase in the displacements due to dynamic P-Δ effects
c_A	A positive value used as the calibration exponent in the hysteretic energy deterioration model, used for accelerated stiffness loss deterioration
c_D	A positive value used as the calibration exponent in the hysteretic energy deterioration model, used for capping deterioration
C_s	Design strength coefficient (fraction of building weight)
c_S	A positive value used as the calibration exponent in the hysteretic energy deterioration model, used for strength deterioration
c_K	A positive value used as the calibration exponent in the hysteretic energy deterioration model, used for stiffness deterioration
C_m	effective mass factor
D	Damage (response)
DLS_i	Damage Limit State i

F_y	Yield force
F_y^+	A positive value for the yield strength in positive direction
F_y^-	A negative value for the yield strength in negative direction
g	Gravitational acceleration
IM_j	Ground motion level with an intensity measure of j
K_i	Elastic initial stiffness
K_e	Elastic effective stiffness
M_s	Surface magnitude
K^*	Effective stiffness of Equivalent SDOF system
M^*	Effective mass of Equivalent SDOF system
PF	Participation factor
PF_1	Modal participation factor for the first fundamental mode
R	Structural behavior factor
R	Residual stress ratio as a fraction of yield strength (for OPENSEES software)
S_1	Spectral acceleration at 1.0 second
S_a	Spectral acceleration
Sa_y	Yield spectral acceleration
Sa_u	Ultimate spectral acceleration
S_d	Spectral displacement
Sd_y	Yield spectral displacement
Sd_u	Ultimate spectral displacement
S_S	Spectral acceleration at 0.2 second
tag	Integer identifier used to tag the material model
T_e	Effective building period
T_i	Elastic initial period (period of the first mode)
V	Base shear force
V_y	Base shear force at yield
W	Weight of the structure
α	Modal weight factor

α_1	Modal mass coefficient for the first fundamental mode
α_{cap}	A negative value for the cap slope ratio as a fraction of stiffness
α_h	Isotropic hardening ratio as a fraction of elastic stiffness
Δ_d	Demand displacement
Δ_{roof}	Roof displacement
Δt_{eff}	Effective duration of eartquake
γ	Yield over-strength ratio
λ	Ultimate over-strength ratio
λ_A	A positive ratio for calculating the ultimate energy in the hysteretic energy deterioration model, used for accelerates stiffness loss deterioration
λ_D	A positive ratio for calculating the ultimate energy in the hysteretic energy deterioration model, used for capping deterioration
λ_K	A positive ratio for calculating the ultimate energy in the hysteretic energy deterioration model, used for stiffness deterioration
λ_S	A positive ratio for calculating the ultimate energy in the hysteretic energy deterioration model, used for strength deterioration
μ	Ductility ratio
$\phi_{roof,1}$	Amplitude of the first mode at the level of roof
δ_{cap}^{+}	Positive value for the cap displacement on positive side
δ_{cap}^{-}	Negative value for the cap displacement on negative side
δ_i	Average story drift for a given story
η_{bi}	Torsional irregularity factor
η_{ci}	Ratio of the effective shear area of any story to the effective shear area of the story immediately above
Γ_1	First mode participation factor

CHAPTER 1

INTRODUCTION

1.1 Background and Research Significance

Turkey is an earthquake (EQ) prone country located in one of the world's most seismically active regions. Recent EQs caused severe damage to buildings, tremendous amounts of economic losses, so many injuries, and, unfortunately, tens of thousands of human life losses. In EQ-prone countries like Turkey, the building stock's seismic performance needs to be investigated for mitigation efforts and the community's resilience.

Recent EQs in Turkey have shown that reinforced concrete (RC) buildings' structural performance plays a crucial role in terms of EQ losses. The vulnerability of RC construction to EQ ground motion was clearly apparent during the 1999 İzmit EQ and its aftermath, where the property losses reached 20 billion US$ and human life losses exceeded 18,000 (USGS, 2000). RC moment-resisting frame structures represent approximately 75% of Turkey's building stock (Vona, 2014).

RC school buildings have been observed among the most vulnerable and severely damaged buildings after August 17, 1999 Marmara (Mw=7,4), November 12, 1999 Düzce (Mw=7,2), May 1, 2003 Bingöl (Mw=6,4) and October 23, 2011 Van (Mw=7,2) EQs.

Özmen (2000) and the Ministry of Education determined the institutional and provincial distribution of school facilities damaged in the 1999 EQs, and it is summarized in Table 1-1 and

Table 1-2 below.

Table 1-1 The institutional distribution of damaged educational institutions after 1999 EQs (Özmen, 2000)

Institution	**Buildings destroyed or damaged beyond repair**	**Repair required**			
		Heavy	**Moderate**	**Minor**	**Total**
Primary schools	63	33	223	831	1087
Private education	-	-	-	2	2
Secondary education	18	8	46	120	174
Female technical education	10	7	25	78	110
Trade and tourism education	1	3	6	25	34
Apprenticeship and non-formal education	4	4	4	25	33
Teachers' guesthouse	3	1	3	11	15
Public housing	1	-	2	5	7
Other	2	4	8	29	41
Total	102	60	317	1126	1503

Table 1-2 The provincial distribution of damaged educational institutions in the 1999 EQs (Özmen, 2000)

Location	**Buildings destroyed or damaged beyond repair**	**Repair required**			
		Heavy	**Moderate**	**Minor**	**Total**
Bolu	9	39	48	44	185
Bursa	11	-	10	85	95
Eskişehir	1	-	3	29	32
İstanbul	28	-	99	659	758
Kocaeli	19	16	50	144	210
Sakarya	31	-	86	91	177
Yalova	3	5	21	20	46
Total	102	60	317	1126	1503

The collapse of the Çeltiksuyu Primary School Dormitory building in Bingöl EQ, which killed 84 students and a teacher, is a significant tragic evidence of the school buildings' seismic vulnerability in Turkey. Pictures that can be found from the internet and taken just after the collapse of the dormitory building can be seen in Figure 1-1.

Figure 1-1. Photographs taken from the dormitory building of Çeltiksuyu Basic Education School collapsed during the 2003 Bingöl EQ

The situation for school buildings' seismic vulnerability is not much different in many countries globally, especially in developing countries. As noted in OECD (2004), "schools built worldwide routinely collapse in EQs due to avoidable errors in design and construction, because existing technology is not applied, and existing laws and regulations are not sufficiently enforced". Some of the EQs that reveal school buildings' seismic vulnerability is listed in Table 1-3 (Wisner, 2006).

Table 1-3 EQs and damages to school facilities (Wisner, 2006)

Year	EQ-Location	Damage
1999	Chi-Chi-Taiwan	Destroyed 43 Taiwan schools in the Nantou and Taichung area, and a total of 700 schools nationwide were damaged.
2001	Gujarat-India	Caused severe damage to 11,600 schools.
2005	Kashmir-India	Resulted in the collapse of 8000 schools in the North-West part of the country and 1300 schools in Pakistan-administered Kashmir. UNICEF stated that in this EQ, at least 17,000 school children died because children were attending morning classes.
2008	Sichuan-China	Caused the death of 19,000 students and the destruction of about 7000 schools.

The characteristics that generate seismically vulnerable school buildings can be better understood by the school collapses after EQs. But unfortunately, efforts to collect and make available detailed data on EQ damage to school buildings have fallen far short of what is needed (Rodgers, 2012). Focused attention on both pre-EQ mitigation efforts and post-EQ reconnaissance reports for school buildings is a necessity.

Approximately 1.2 billion students are enrolled in primary and secondary schools; of these, 875 million school children live in high seismic risk zones worldwide (Hancılar et al., 2014). In Turkey, about 30 % of the population are children of school-age. During daytime hours, these children are mostly within the school buildings, which are usually vulnerable to EQs.

Some of the statistical information of education in Turkey (schools, teachers, and students) for the education year of 2018-2019 is given below in Table 1-4 (TUIK,

2019). There are also teachers and students (about 1.6 million) in the pre-school classrooms, which are not listed in the table. Including the administrative staff, there is approximately 20 million population within the school facilities during the daytime. There are projections for the year 2023, and it is accepted that 21.2 % of the population will be under 14 years old and 15.1 % of the population between the ages of 15 and 24 by the year 2023 (TUIK, 2016), which means Turkey will have more students and need more schools and teachers by the year 2023.

Table 1-4 The statistical information on education in Turkey (2018-2019) (TUIK, 2019)

2018-2019 Education Year	**Number of schools**	**Number of teachers**	**Number of students**
Primary School	24,749	300,732	5,267,378
Junior High School	18,395	354,198	5,627,075
Secondary School	12,506	371,234	5,649,594
Total	55,650	1,026,164	16,544,047

Schools play a vital role in the community's social life since they have an essential role in the educational process. When the schools are closed, education is delayed, and loss of social and cultural life comes after that (Ersoy and Kolçak, 2015). School buildings are houses for children during the daytime and serve as emergency shelters after any kind of disaster. EQ vulnerability assessment of school buildings is a prior step to avoid losses and increase the emergency preparedness of society.

The calculated duration for an expected major EQ (Mw ≥ 7) in the Sea of Marmara part of the North Anatolian Fault was probably a maximum of 30 years with the probability of 20%-65% (Barka, 2000). Two third of the determined duration has already been consumed, and time is passing rapidly. According to a prediction done by the Japan International Cooperation Agency (JICA), an EQ of magnitude ranging

between 6.9 and 7.7 will occur around İstanbul in the near future (Griffiths et al. 2007).

Decision-makers should urgently exhibit a proactive approach for EQ preparedness of school buildings. Since children are emotionally and physically delicate, they are more vulnerable than adults in EQs or any kind of emergency events. Protecting and defending children urgently during emergency events is the priority not only for parents but also for governments and regulations. Building a safe and better future for our children is a priority of the whole society. (Hancilar et al. 2014 & Ersoy and Kolçak, 2015)

Retrofitting/strengthening of schools or designing them in accordance with the requirements of the EQ-resistant design codes is considered as a primary objective in EQ risk reduction strategies due to the high importance of those facilities for the entire community (Hancılar et al., 2014). Nevertheless, due to economic constraints, a tiny fraction of the existing school building stock has been upgraded in the frame of pre-EQ strengthening programs worldwide (Chrysostomou et al., 2015).

Components for developing strategies against the EQ risk are EQ hazard estimation and seismic vulnerability assessment (SVA). Determination of the vulnerability of existing engineering structures requires the assessment of seismic performances of the building stock when subjected to a variety of potential EQs (Özün, 2007). The expected damage is generally considered as a measure of seismic vulnerability, and the associated loss caused by EQs can be estimated in this way.

SVA of school buildings located within EQ prone regions and determination of their performance levels under seismic actions play an essential role in the safety of children, teachers, and the education system overall. Seismic performance assessment procedures can be divided into three main categories in the literature. These are walk-down (street survey), preliminary evaluation procedures, and detailed assessment procedures.

Walk-down or street survey is the quickest and simplest method. Superficial data (usually composed of deficiencies or irregularities) collected from a quick inspection of the building is sufficient. Rapid evaluation techniques serve to identify or rank highly vulnerable buildings by providing a crude index used to rank a group of buildings to determine their priority for further evaluation. These highly vulnerable buildings are investigated in detail thereafter, if necessary. (Yakut, 2004 & Kalem, 2010)

When a more reliable and detailed assessment than the walk-down survey is needed, preliminary assessment techniques are employed. In addition to the data collected for the walk-down assessment procedures, the size and orientation of the structural components, material properties, and layout are needed to perform practical and straightforward functions (Yakut, 2004 & Erduran, 2005).

There are various methods to evaluate the seismic performance of existing buildings in the world. These methods are established so that they only have an application in different types of RC buildings constructed in a country. Therefore, the direct application of them in other countries is not possible (Mohsen, 2012).

Unfortunately, walk-down/street survey or preliminary assessment methods are generally developed for residential buildings in Turkey. There is not enough data to test these methods' accuracy for public buildings, including school facilities. In this study, the accuracy of these methods will be tested based on TEC-2007 assessment results.

The third category among assessment procedures that involve the in-depth evaluation of the buildings is the detailed assessment procedures. Linear or nonlinear analyzes are needed to determine the response quantities in these procedures. In order to perform the analysis, the geometrical properties of the components, mechanical properties of the materials, and detailing of the components are needed in addition to the available data collected for preliminary assessment methods. (Yakut, 2004 & Erduran, 2005)

Detailed analyzes are performed, and assessment results are compared by the results of TEC-2007 within the study. Detailed analyzes are also performed by TBEC-2018, and the results are compared by the results of TEC-2007.

Capacity curves are developed through parameters obtained with nonlinear analysis. The reliability and accuracy of the capacity curves have a significant role in seismic loss estimation studies. These curves directly affect building vulnerability and, Consequently, the EQ losses. They are recommended to be used in loss estimation, risk assessment, and quick evaluation studies for RC frame buildings. (Yakut, 2008)

Although comprehensive research is devoted to RC residential buildings' seismic vulnerabilities in Turkey, a similar endeavor has not been given to RC school buildings. Therefore, this study focuses on determining capacity curve parameters and the validity of existing assessment procedures for Turkey's school buildings.

SVA of school buildings can be performed based on fragility curves (F.C.s) after determining the seismic ground motion level that they can be exposed during their lifetime. F.C.s supply useful information about the relation between the ground motion intensity and the probability of exceeding a specific damage state.

Fragility based assessment is generally used to evaluate the seismic performance of building stock. For a successful performance assessment, the country-specific characteristics should necessarily be taken into account while generating the F.C.s for the related building structures. Construction practices may differ broadly among countries, and these differences directly influence the F.C.s of the buildings under consideration (Özün, 2007).

Structures in seismically active regions can be subjected to repeated earthquakes during their lifetimes before strengthening or by aftershocks. Damage accumulation in their structural systems could be observed during these events. School buildings encountering such occasions without being retrofitted after the first event might lead to catastrophic cases, including increased fatality rates. (Mazılıgüney et al., 2017).

F.C.s for RC School Buildings due to repeated EQs are also derived, which are thought to be useful for rapid decisions for using structures after EQs.

"Urban Renewal Law" of Turkey, which has been in action since May 2012 for risk mitigation, focuses on reducing the expected seismic risk due to the vulnerability of existing buildings. For this purpose, new provisions are set forth to investigate and classify seismically vulnerable residential buildings as quickly as possible. (Binici et al., 2015) Although the provisions are designated for residential buildings mainly, 36 RC school buildings were also analyzed accordingly, and the results are compared by TEC-2007. Provisions for the seismic risk evaluation of existing RC buildings in Turkey under the Urban Renewal Law was revised in February 2019. 36 RC school buildings were also analyzed accordingly, and the results were compared by the previous ones and TEC-2007 and TBEC-2018.

Turkish EQ Code was substantially revised in 2018, and the new code is in force since January 2019. 36 RC school buildings were analyzed according to the new EQ code, and the results were compared by the previous code and by the provisions of Urban Renewal Law.

Time is running out rapidly for the expected EQ, which is threatening schools, children, teachers, and the overall society. The future of our children shall not be sacrificed to EQs.

1.2 Literature Survey

Fast urban growth after the 1970s, substantiating uncontrolled development of the physical environment, is the primary source of existing EQ vulnerability risks. RC buildings constitute the majority of the building stock, especially in developing countries. Due to their poor performance under major EQs occurring in the last decades, significant research has been initiated to assess the vulnerability of the existing building stock and provide means of improving the vulnerable buildings' expected seismic performance (Yakut and Erduran, 2005).

Vulnerability parameters and structural irregularities of RC buildings used by quick performance assessment procedures can be listed as follows;

- Number of stories,
- Apparent building quality,
- Soft-story,
- Weak story,
- Heavy overhangs,
- Short columns,
- Pounding between adjacent buildings,
- Torsional irregularity,
- Plan irregularity,
- Vertical irregularity,
- Redundancy,
- Local soil conditions,
- Topographic effects.

In the seismic performance of RC structures, one of the most critical parameters determining the building's survival is its energy dissipation capacity (Erduran and Yakut, 2004-2). Parameters used for quick performance assessment procedures also deal with the building's energy dissipation capacity.

School buildings all over the world have an essential role in the educational process. Damages to school buildings and losses of lives of teachers and children deeply affect society. Additionally, school buildings may be used as emergency shelters after EQs, contributing to the community's resilience. With these aspects, school

buildings' seismic vulnerability had taken attention among the EQ engineers, state authorities, and non-profit organizations.

There have been only a few studies relating to the seismic vulnerability of school buildings in Turkey, and unfortunately, they were all having limited content and scope. Some of the considerable studies from Turkey and worldwide are summarized below shortly.

OECD performed a comprehensive project under the Programme on Educational Building (PEB) in order to keep schools around the world safe in EQs. OECD work on school safety and security began in February 2002 with an experts' meeting in Washington, D.C., USA, and plenty of valuable resources are published within the project. The OECD PEB financed projects on the safety of school buildings within the member states. The OECD published a comprehensive report in 2004, and a chapter named "Obstacles to improving seismic safety of school buildings in Turkey" was prepared by Prof. Dr. Polat GÜLKAN. Obstacles to good quality construction in Turkey and observations from a site survey of school buildings after the EQ in Bingöl are described briefly within the mentioned chapter of Gülkan (2004). Figure showing the obstacles to good quality construction in Turkey (OECD, 2004) is copied below because of its importance.

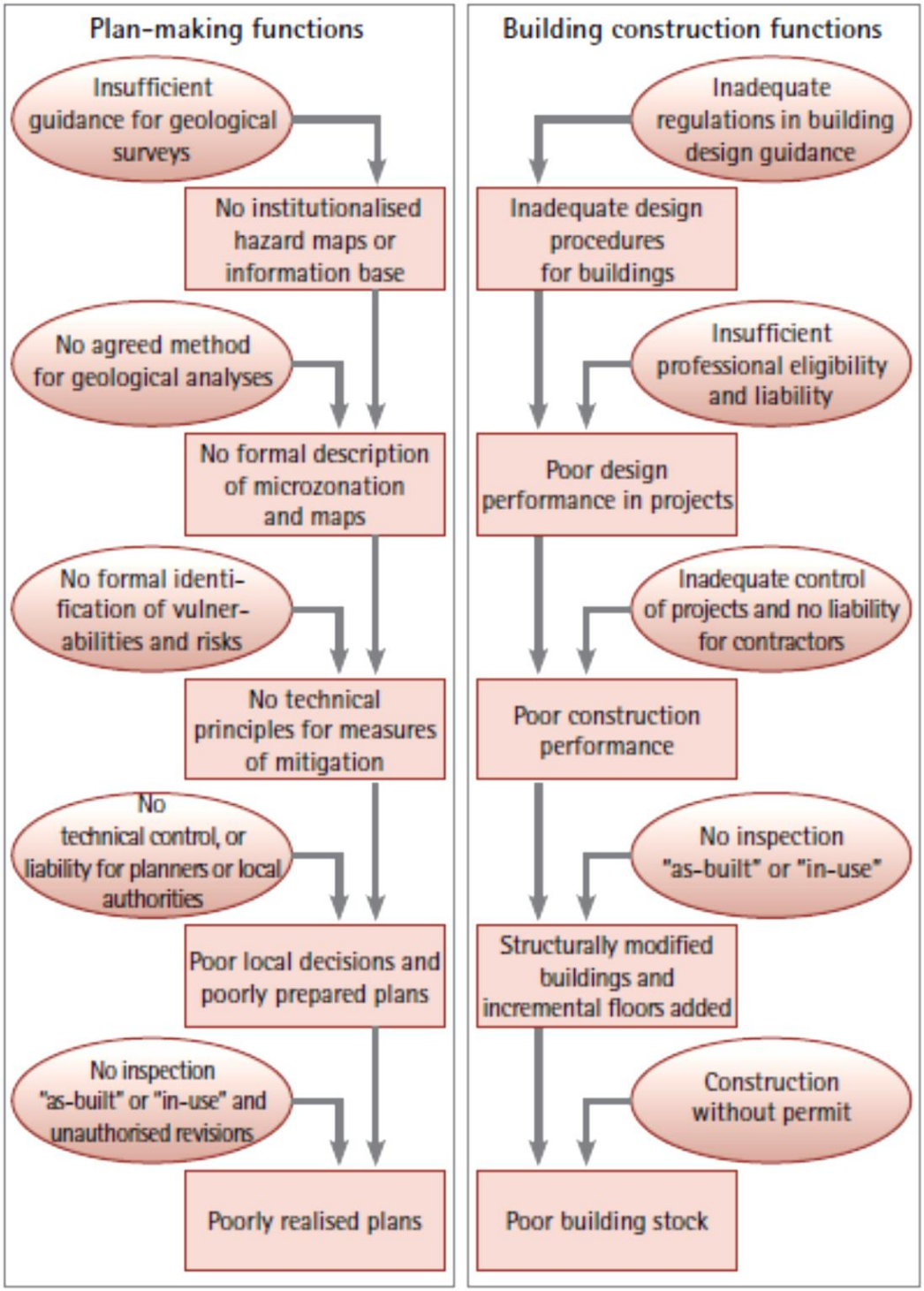

Figure 1-2. Obstacles to good quality construction in Turkey (Gülkan, 2004)

Gülkan and Utkutuğ (2003) discussed the Field Law of California and examined the damage to school buildings in Bingöl after the 2003 EQ Gülkan and Utkutuğ (2003)

concluded that the shear walls in both orthogonal directions are the most critical requirement for the EQ safety of RC school buildings. This requirement should be obligatory through a code similar to Field Law.

Dilmaç et al. (2018) studied the parameters influencing the seismic performance of existing RC buildings in Turkey and concluded the compressive strength of concrete and the transverse reinforcement are the two crucial parameters for EQ safety.

Until recently, the most extensive efforts in implementing school strengthening programs were made in Japan; some interesting examples of such applications are given by the Japan Ministry of Education (2006) publication.

Moreover, recent efforts towards setting up large-scale strengthening (also referred to as retrofit) programs of school buildings, such as that in British Columbia (Ventura et al., 2012) are useful in that they introduce concepts like performance-based assessment and compilation of web-based databases of results of advanced analysis of such buildings, but, to the authors' best knowledge, have not culminated into the actual implementation of strengthening schemes to even a limited number of schools.

Rodgers (2012) studied the physical characteristics of schools that cause vulnerability to damage and collapse, using data from EQ damage reports and SVAs of school buildings. Rodgers (2012) concluded that building configuration, type, materials and location; construction and inspection practices; and maintenance and modifications all contribute to building vulnerability. Scarce resources, inadequate seismic codes, unskilled building professionals, and lack of awareness of EQ risk and risk reduction measures are the underlying drivers for the vulnerability of schools as well as other buildings, according to Rodgers (2012).

Rodgers (2012) tabulated the prevalence of characteristics that create vulnerability by examining 32 EQs and 31 different vulnerability assessment reports worldwide, including studies from Turkey. The mentioned characteristics are tabulated concerning different categories below, as Table 1-5.

Rodgers (2012) also tabulated the underlying drivers of school vulnerability and "no construction site inspections" and "engineer of record is paid by the developer, no independent inspection, no liability" are two impressive underlying drivers from Turkey which were previously mentioned by Gülkan (2004).

Table 1-5 Vulnerability creating characteristics for school buildings (Rodgers, 2012)

Category	**Vulnerability creating characteristics**
Configuration	Large rooms - no cross walls
	Plan irregularity due to one-bay wide
	Plan irregularity general
	Captive (short) columns due to partial height infill walls
	Torsion due to windows on one side
	Torsion, general
	Weakness due to windows - masonry
	Soft or weak story
	Vertical irregularity, general
	Masonry gable walls
	Heavy roofs
Structural system type and construction materials	Vulnerable traditional construction
	Vulnerable non-engineered non-ductile RC frame
	Vulnerable non-engineered brick or block masonry
	Vulnerable non-engineered poorly confined masonry
	Vulnerable engineered non-ductile RC frame
	Vulnerable engineered brick or block masonry
	Safer traditional building types abandoned
	Standard types/plans have major seismic deficiencies
	Lack of seismic design understanding by engineers
	Local materials generate weak or brittle buildings
	Poor quality engineered materials general

Table 1-5 Continued

Category	**Vulnerability creating characteristics**
Location	Vulnerable sites / poor soil conditions
	Liquefaction
	Sloping site/landslides
	Cultural practices for site selection
Construction practices	Unskilled / low-skilled local labor
	Builders not aware of EQ-resistant practices
	Public contracting low bid rules
	Reducing quality to save money or time
	Poor construction quality, general
Construction inspection	Lack of inspection
	Corruption of inspection mechanisms
Maintenance	Deferred / not done, general
	No provision by builder or operator
Modifications	SubsEQuent structural modifications
	Ineffective retrofits
Falling hazards	Facade and exterior
	Interior / contents
Exit pathways	Inadequate doors, windows, halls/corridors, or stairs

Tan and Abdul Rezak (2011) studied the assessment of risk to school buildings resulting from distant EQs and concluded that the school buildings are safe to enter after an EQ of magnitude up to a PGA of 0,2g. But concrete compressive strength and yield strength of steel were accepted to be 25 MPA and 460 MPa, respectively, so the conclusions cannot be realistic for Turkish school buildings and school buildings of countries having similar construction practices with Turkey.

Sobaih and Nazif (2012) proposed a methodology for seismic risk evaluation of existing RC school buildings in Egypt to determine the priority to take action for mitigation. It is particularly mentioned that the school buildings need a rapid method to evaluate their seismic risk level and provide a basis for the next steps of the necessary mitigation process. The proposed methodology evaluates the school buildings' seismic risk levels by inspecting them, concentrating on the irregularities (soft-story, plan irregularity, short column, existence of adjacent building, i.e., pounding effect, etc.) especially to arrange a Priority List in ascending order of their seismic risk levels. The proposed method is more or less similar to the walk-down, and preliminary assessment methods discussed previously.

İnel et al. (2008) studied the seismic performance evaluation of school buildings in Turkey constructed per pre-modern seismic code (TEC-1975) with the six selected template designs to represent a significant percentage of school buildings in medium-size cities located in high seismic regions of Turkey. In the related study, capacity curves of investigated structures were determined by pushover analysis in two principal directions, Equivalent SDOF systems represented the inelastic dynamic characteristics of buildings, and their seismic displacement demands were calculated under selected ground motions. 10, 13, and 16 MPa concrete compressive strengths and two (15 and 25 cm) transverse reinforcement spacing were used to consider the uncertainties of material properties. Member sizes and reinforcements in the template design were used to model the selected buildings for nonlinear analysis. Twenty ground motion records from 7 different EQs in Turkey based on PGA were used for nonlinear time history analysis. Some of the remarkable conclusions of İnel et al. (2008) are taken below because of their importance;

i. Shear walls dominate the building response in buildings with shear wall area of at least 0,25 % of total building area,

ii. Concrete quality and detailing has a significant role in both displacement and lateral strength capacity of buildings,

iii. Column shear failures are a common problem for poor concrete and low amount of transverse reinforcement, resulting in brittle failure for existing school buildings,

iv. Existing school buildings are far from satisfying the objectives of the TEC 2007 during a possible EQ,

v. The primary deficiency of existing school buildings is high displacement demands due to their low load capacities.

İnel et al. (2008) suggested the addition of shear walls as the most practical and economical solution under the observed circumstances.

In his study, Kalem (2010) focused on determining capacity curve parameters and the validity of existing assessment procedures for Turkey's school buildings. Kalem (2010) documented valuable information about SVA of RC school buildings in Turkey, so his study became a general reference for researchers of this subject. Rodgers (2012) also referred to the study of Kalem (2010) for the vulnerability characteristics of RC school buildings in Turkey.

Bilgin (2013) studied the fragility based assessment of public buildings (administrative centers, health clinics, hospitals, schools, etc.) with representative template designs in Turkey, indicating that although the used projects of public buildings reveal minor differences from province to province, they are similar architecturally. A set of template designs consisting of school and hospital buildings were analyzed similarly with İnel et al. (2008). Some of the remarkable conclusions of Bilgin (2013) are taken below because of their importance.

i. The probability of exceedance for LS and CP levels of all buildings are close to each other.

ii. Examination of F.C.s reveals that the collapse probabilities of existing buildings may change approximately from 30% to 60% between the PGV values, which fall in the range of 40–70 cm/s.

iii. It can be said that the number of stories has a remarkable effect on the probability of exceeding moderate and severe damage limit states.

Hancılar et al. (2014) studied the EQ vulnerability of school buildings by probabilistic structural fragility analysis of four-story (typical project cited as 10403; there are 55 schools with the same type design in İstanbul) standardized/template designs. It is mentioned in the related study that methods to account for uncertainties are desirable because it is very often that school buildings may differ from the original blueprints. Uncertainties concerning material properties, geometrical characteristics, etc. are taken into account in different analysis cases wherein the parameter under consideration is randomly changing in each Monte Carlo simulation. The accepted concrete compressive strength and steel yield strength were 14,8 Mpa and 220 MPa, respectively. Fragility functions are produced in terms of PGA and PGV. Hancılar et al. (2014) mentioned that their study highlights the importance of uncertainties in school buildings' seismic performance assessment. The uncertainties cause a significant variation in the resulting fragility functions.

Turkey is an EQ-prone country, and especially İstanbul, Turkey's most crowded city, has a high probability of a severe EQ in the coming years. With the finance of the World Bank (at the first stages), the Governorship of İstanbul has started a project named ISMEP to assess the public buildings (schools, hospitals, administrative buildings, etc.) The Governorship of İstanbul has established an administrative unit named IPCU to manage the project.

The project's primary goals are to improve preparedness for a potential EQ and retrofit or reconstruction of priority public buildings in İstanbul. ISMEP project consists of three components:

- Component A: Enhancing Emergency Preparedness
- Component B: Seismic Risk Mitigation for Priority Public Buildings
- Component C: Enforcement of Building Codes (Elgin, 2007)

Within this aspect, a large number of school buildings have been selected for detailed assessment based on the national EQ code (TEC-2007) to determine their seismic performance (Yakut et al., 2008).

The project is financed by a World Bank (WB) loan at the beginning and is implemented through the İstanbul Special Provincial Administration (ISPA). The ISMEP project started on February 1 1, 2006, and was expected to be completed by the end of 2010, but extended afterward. The İstanbul Project Coordination Unit (IPCU), established under ISPA, is responsible for implementing the ISMEP. (IPCU, 2008)

ISMEP has reached a budget of 78 billion euros with the contribution of the European Investment Bank (EIB), Council of Europe Development Bank (CEB), and Islamic Development Bank (IDB). By January 2020, more than 900 school buildings are assessed, 247 are re-built, and 627 are strengthened.

Inadequate structural component detailing, low concrete compressive strength, soft or weak story created by a non-typical story height, and diaphragm deficiencies are the generally mentioned common seismic deficiencies of school buildings within the assessment reports (IPCU, 2008).

Ersoy and Koçak (2015) studied disasters and EQ preparedness of children and schools in İstanbul in detail by noting the disaster risks in İstanbul, emphasizing the losses from previous EQs, and summarizing, and evaluating the work done. The students and the conditions of the schools were evaluated. In this evaluation,

population, building stocks, geological threats and risks, assessment of educational buildings after 1999 Gölcük and Düzce EQs, and preparations against disasters for children were covered. Ersoy and Koçak mentioned the losses of children and school teachers and school damages in the last six EQs in Turkey, and damages to schools are copied below in Table 1-6 because of its importance.

Ersoy and Koçak (2015) concluded that; seismic risk mitigations of school buildings in İstanbul are not enough for İstanbul and far less than needed.

Ulutaş et al. (2019) concluded that school buildings could hardly satisfy the performance requirements without at least 2 separate shear walls in both orthogonal directions. Ulutaş et al. (2019-2) studied the shear wall ratio in school buildings and concluded that 1.12 %, 1.51 %, 1.79 %, and 2 % of shear wall wrt. floor area is needed for 2, 3, 4, and 5 story school buildings, respectively, for EQ safe school buildings.

Table 1-6 The school damages occurred in the last damaging EQs in Turkey. (Ersoy and Kolçak, 2015)

Location / Magnitude / Date	**Losses and Damage (Consequences)**
Van / Mw= 7.2 / October 23 2011	Many of the 500 schools were affected slightly or moderately. Two schools were totally destroyed.
Bingöl / Mw = 6.4 / May 1 2003	4 school buildings collapsed. The damage was moderate in 12 and slight in 10 schools. Education was negatively affected in 90% of all schools. (84 students and 1 teacher died.)
Düzce / Mw = 7.2 November 12 1999	Majority of buildings retrofitted after Kocaeli EQ destroyed completely or damaged heavily beyond repair.
Kocaeli / Mw = 7.4 / August 17 1999	43 schools were destroyed. 381 schools were damaged. 22 basic education and 21 secondary schools were damaged beyond repair. The 271 schools were damaged slightly or moderately. Unsafe schools closed for 4 months. Some Schools in İstanbul were also damaged slightly or moderately despite the long distance to the epicenter area. 35 unsafe schools were demolished. 131 schools were closed temporarily.
Erzincan / Mw = 6.8 March 12 1992	43 schools damaged (9 heavily and 34 moderately), 10 dormitory and public housing buildings damaged. (23 student girls died.)

1.3 Seismic Performance and Properties of RC Buildings in Turkey

Non-ductile reinforced-concrete moment frame buildings have been prevalent and are the primary construction type used in Turkey since the early 1900s till 1975. In 1975, with the introduction of a seismic code in Turkey (TEC-1975), RC frame structures were improved. Still, unfortunately, most of the practicing engineers were unfamiliar with the requirements of the seismic provisions over the following decades. The quality of construction was poor in most cases. Consequently, most of the moment-resisting RC frame buildings constructed according to TEC-1975 are also accepted as non-ductile, lacking the detailing necessary to prevent brittle failures and collapse. These buildings still constitute large amounts of occupancy and are vulnerable to future EQs. (IPCU, 2008)

Field surveys for determining the damage levels of buildings after EQs helped determine the seismic performance and general properties of structures in Turkey. After strong EQs, damage levels are determined, and buildings with none/light damage were allowed to be in use, but retrofitting was required generally for moderately damaged buildings. Severely damaged buildings are demolished after EQs, except for the buildings having particular historical or strategic importance. General properties and seismic vulnerability inducing parameters of Turkish RC construction practice are described shortly in this section.

Mazılıgüney et al. (2008) analyzed concrete compressive strengths of 4,647 core specimens taken from 693 buildings, mostly in İstanbul by grouping the buildings as public, residential, and military. Concrete compressive strengths of existing buildings in Turkey, which mostly need retrofit for EQ resistance, range from 5 to 16 MPa with an overall average of 10,64 MPa. Public buildings have an average concrete compressive strength of 5.86 MPa, while residential and military structures have average concrete compressive strengths of 8.96 MPa and 14.80 MPa.

The concrete compressive strengths of the buildings in Turkey are far less than the requirements of TEC-2007, which allows at least 20 MPa of concrete compressive

strength. Mazılıgüney et al. (2008) also concluded that there is a small tendency to increase the concrete compressive strength of structures by years until the 1970s. Still, after the mid-1970s, the trend changes. The concrete compressive strength of structures decreased by years until the time (ends of the 1990s) when ready mixed concrete became obligatory and afterward a common construction practice.

Yakut and Erduran (2005) mentioned that the buildings with favorable material properties would probably not suffer heavy damage or collapse, while the ones with poor material properties are highly vulnerable to devastating EQs. Material properties and particularly concrete compressive strengths of the buildings are strong indications of the construction quality and the degree of conformance to EQ codes.

Sucuoğlu and Yazgan (2003) mentioned a close relationship between the apparent quality and the experienced damage during Turkey's recent EQs. Buildings with poor apparent quality can be expected to possess weak material strengths and inadequate detailing so that they will be more vulnerable to EQs. There exists a strong correlation between the apparent quality and the concrete compressive strengths of buildings.

After the two EQs in 1999, Sucuoğlu and Yazgan (2003) analyzed the damage distribution for 9685 buildings in Düzce concerning the number of stories. They concluded that damage grades shift linearly with the number of stories. As the number of stories increases, the ratio of undamaged and lightly damaged buildings decreases steadily, whereas moderately and severely damaged buildings increase almost linearly. Many researchers mentioned a strong correlation between the number of stories and RC buildings' damage levels in Turkey. The number of stories is perhaps the most dominant parameter in the seismic vulnerability of typical RC buildings in Turkey (Sucuoğlu and Yazgan, 2003).

For buildings conforming to EQ codes are not accepted to have such a correlation between the number of stories and the damage level. Turkey's unaccepted situation is an indication of the conformance of the buildings to Turkey's EQ codes.

Consequently, the number of stories is a primary indicator for determining the sub-classess of RC school buildings in Turkey for this study.

A soft-story (B2 type irregularity) occurs when the lateral rigidity of the horizontal load resisting members at any level is such that the average lateral drift under the design load at that level is at least 50% larger than the same quantity calculated for the next story (TEC-2007). In TEC-2007, stiffness irregularity factor, η_{ki}, is defined as the ratio of the average story drift at any story to the average story drift at the story immediately above. The same definition for the B2 type irregularity is included in TBEC-2018.

$$\eta_{ki} = \frac{\delta_i}{\delta_{i+1}} \tag{1-1}$$

Stiffness irregularity or weak story (B1 type irregularity) exits when η_{ki} exceeds 2.0. Masonry non-structural elements are not typically modeled and can have an impact on the soft-story determination.

For reinforced concrete buildings TEC-2007, bases the effective strength of a floor is on its effective shear area. The effective shear area for each floor is computed from:

$$A_e=A_s+A_c+0.15A_m \tag{1-2}$$

Where As, Ac, and Am are the total cross-section area of the shear walls, concrete columns, and masonry walls, respectively, in the lateral direction under consideration. This approach accounts for shear (area) contribution but not flexural (moment of inertia) effect and appears inconsistent. The same definition for the B1 type irregularity is included in TBEC-2018.

In TEC-2007, the test for the presence of a weak story is the value of the coefficient, η_{ci}.

$$\eta ci = \frac{\Sigma Ae_i}{\Sigma Ae_{i+1}} \tag{1-3}$$

Depending on the value of ηci, several conditions exist, which are summarized in Table 1-7.

Table 1-7 Conditions depending on η_{ci} values (TEC 2007)

Case	**Action**
$0{,}8 \leq \eta_{ci}$	No weak story modification is needed.
$0{,}6 \leq \eta_{ci} \leq 0{,}8$	Increase the total base shear by the factor $1/1{,}25\ \eta_{ci}$
$\eta_{ci} \leq 0{,}6$	Increase floor strength until η_{ci} reaches 0,6 or above.

In Turkish construction practice, there is commonly a mixed form of occupation, where small businesses (grocery shops, offices, etc.) are dispersed among the residential spaces under the same roof (Yakut et al., 2005). The commercial areas at the ground levels having level access from the streets are usually made as free of obstructions (like columns or masonry walls), mostly left open between the frame members with no masonry walls inside, for human circulation, and have taller story heights (Sucuoğlu and Yazgan, 2003). These negative factors lead to weak or soft-story at the ground level, where the demand for seismic resistance is the largest. Many buildings with soft or weak stories were observed to collapse due to a pancaked vulnerable story in the past strong EQs worldwide, and likewise in Turkey (Sucuoğlu and Yazgan, 2003).

Another frequent application in Turkish construction practice is heavy balconies or overhanging floors in multistory residential buildings. This construction type shifts

the mass center upwards, resulting in higher seismic lateral forces and overturning moments affecting the structures during EQs. Heavy overhangs may also cause discontinuity of frames. Buildings with heavy overhangs sustained more severe damage with respect to regular buildings during recent strong EQs in Turkey (Ay, 2006; Sucuoğlu and Yazgan, 2003).

Torsional (A1 type) irregularity is another common vulnerability inducing parameters in Turkish construction practice. In TEC-2007, torsional irregularity is defined as when the average lateral displacement under the design forces at a given story differs by more than 20% from the largest lateral situation in the same story. In TEC-2007 torsional irregularity is defined as irregularity type A1 and described by Torsional Irregularity Factor η_{bi}.

Torsional Irregularity Factor is defined for any of the two orthogonal EQ directions as the ratio of the maximum relative story drift at any story to the average relative story drift at the same story in the same direction. When the so-called factor is greater than 1.2, the related building is accepted to have type A1 irregularity, according to TEC-2007. The same definition for the A1 type irregularity is included in TBEC-2018.

$$\eta_{bi} = (\Delta_i)_{max} / (\Delta_i)_{avr} \tag{1-4}$$

Landowners generally force architects to design the buildings utilizing the whole area of the land permitted by the local authorities, and this may lead to torsional irregularity usually at the extreme ends of the floor slabs (Yakut et al., 2005).

Plan irregularity is defined as irregularity type A3 and called "projections in the plan" in TEC-2007. The buildings where dimensions of projections in both of the two perpendicular directions in plan exceed the total plan dimensions of that story of the building in the respective directions by more than 20% are accepted to have type A3 irregularity according to TEC2007. In addition to the increased likelihood of causing

torsional irregularity, this situation creates re-entrant corners where stresses become concentrated and may cause unexpected distress (Yakut et al., 2005). In Turkey, taller buildings seem more likely to have this type of irregularity. The same definition for the A3 type irregularity is included in TBEC-2018.

Vertical (B3 type) irregularity is defined as irregularity type B3 and called "discontinuity of vertical structural elements" in TEC-2007. The buildings where vertical structural elements (columns or structural walls) are removed at some stories and supported by beams or gusseted columns underneath, or the structural walls of upper stories are supported by columns or beams underneath are accepted to have type B3 irregularity according to TEC-2007. In Turkey, taller buildings seem more likely to have vertical irregularity. The same definition for the B3 type irregularity is included in TBEC-2018.

Continuous and/or band windows, semi-infilled walls, semi-buried basements, and intermediate beams may result in the accumulation of shear forces in columns. Such a column is called a short column (Ay, 2006). Short columns are usually formed along the exterior side of the buildings so they can be identified from outside easily (Sucuoğlu and Yazgan, 2003). Unfortunately, in Turkish construction practice, short columns are frequent at the ground level, where seismic demand is the largest.

Especially in urban areas and along the main streets, buildings are constructed side by side without sufficient clearance between each other. These types of buildings pound each other during an EQ since they have different vibration periods. Different story heights resulting in uneven floor levels worsen the effect of pounding, and such buildings sustain heavier damages with respect to regular ones (Sucuoğlu and Yazgan, 2003).

Building systems need continuous frames or sufficient number of bays in order to distribute the lateral loads evenly to structural frame members. Buildings having insufficient redundancy sustained localized heavy damages during recent strong EQs in Turkey (Özcebe et al., 2003). Unfortunately, in Turkey, RC school buildings

generally have 3 or 4 continuous frames along the long direction or axes of the structure, which means they do not have sufficient redundancy in the long direction.

Topographic conditions may affect the seismic performance of buildings in Turkey, especially when the building is located on a slope steeper than 30° and have stepped foundations (Ay, 2006; Sucuoğlu and Yazgan, 2003). There are many RC buildings located on top of hills or steep slopes in Turkish construction practice.

Local site conditions and site amplification are other critical factors for RC buildings' seismic vulnerability in Turkey. There were many RC buildings sustained damages because of liquefaction during 1999 EQs.

Akkar et al. (2005) mentioned that low-and mid-rise ordinary RC buildings constitute the most vulnerable construction type in Turkey like other EQ prone and developing countries.

The capacity curves composed according to the design codes are used to determine seismic vulnerability in the HAZUS (Hazard United States) (1997) methodology. Still, these curves do not take into account local construction practices. Capacity curves are represented in the ADRS format. The parameters (such as fundamental period, yield over-strength ratio, post elastic stiffness, yield base shear coefficient, yield and ultimate drift ratios) used to determine the seismic response of buildings under a given hazard intensity can be determined from the idealized capacity curves. (Yakut, 2008)

A typical capacity curve recommended by HAZUS (1997) is illustrated in Figure 1-3. In this curve, building response to EQ is represented in three segments, and points A, B, and C show the code design point, actual elastic limit, and the ultimate point beyond which the structure fails, respectively.

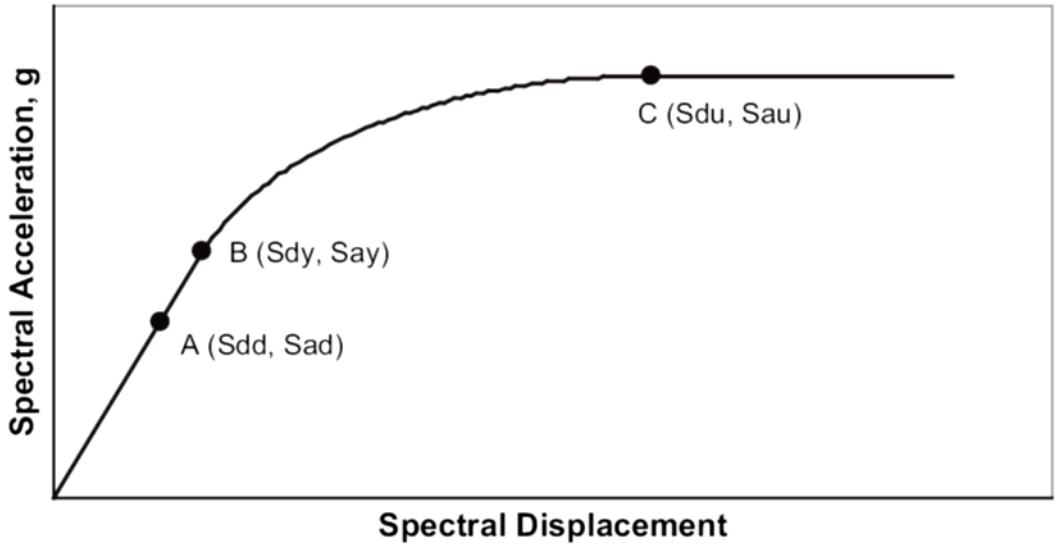

Figure 1-3. Typical Capacity Curve (HAZUS, 1997)

Where;

$$Sa_y = \frac{C_s \gamma}{\alpha_1} \tag{1-5}$$

$$Sd_y = \frac{Sa_y T_e^2 g}{4\pi^2} \tag{1-6}$$

$$Sa_u = \lambda Sa_y \tag{1-7}$$

$$Sd_u = \lambda \mu Sd_y \tag{1-8}$$

In construction practices of countries like Turkey, poor EQ code compliance is widespread, and the existing or as-built properties of structures may differ from the design drawings. To overcome this problem, as-built specific properties should be used to analyze buildings' seismic vulnerability analysis whenever the structural information is available. (Yakut, 2008)

In order to obtain the capacity-related properties of RC frame buildings in Turkey, Yakut (2008) performed a comprehensive study in which 33 sample buildings were selected to represent a typical subset of a comprehensive database consisting of nearly 500 buildings. All buildings were located in either Zone 1 or Zone 2 EQ region according to TEC-1997. The statistical properties of all capacity parameters needed to describe the capacity curve are given in Table 1-8 for each number of stories (Yakut, 2008).

Table 1-8 Statistics of Capacity Curve Parameters (Yakut, 2008)

Parameter		Number of Stories			
		2	3	4	5
Sd_y (cm)	mean	0.75	1.06	1.67	1.52
	st. dev.	0.25	0.29	0.71	0.35
Sa_y (g)	mean	0.25	0.18	0.16	0.12
	st. dev.	0.09	0.07	0.04	0.04
Sd_u (cm)	mean	7.30	10.65	12.84	14.06
	st. dev.	1.50	2.65	3.61	5.40
Sa_u (g)	mean	0.28	0.21	0.18	0.14
	st. dev.	0.10	0.08	0.05	0.04
C_s	mean	0.10	0.10	0.10	0.10
	st. dev.	0.00	0.00	0.00	0.00
T_e	mean	0.27	0.34	0.47	0.55
	st. dev.	0.05	0.11	0.11	0.14
PF	mean	1.17	1.24	1.28	1.29
	st. dev.	0.02	0.04	0.02	0.01
α	mean	0.94	0.88	0.83	0.83
	st. dev.	0.02	0.06	0.05	0.03
γ	mean	2.31	1.58	1.30	1.02
	st. dev.	0.85	0.52	0.39	0.32
λ	mean	1.13	1.15	1.14	1.16
	st. dev.	0.04	0.12	0.03	0.06
μ	mean	9.23	9.26	7.67	7.86
	st. dev.	3.03	3.12	3.01	2.22

The statistical properties of all capacity parameters needed to describe RC school buildings' capacity curves in Turkey with respect to the number of stories will be compared with this table in the following chapters.

Yakut (2004) mentioned that the evaluations made after recent strong EQs in Turkey indicated that RC buildings having over-strength ratios of less than 1.65 would perform poorly against a strong EQ Selected capacity curves for 2, 3, 4, and 5 story RC buildings in Turkey are illustrated in Figure 1-4 (Akkar et al., 2005).

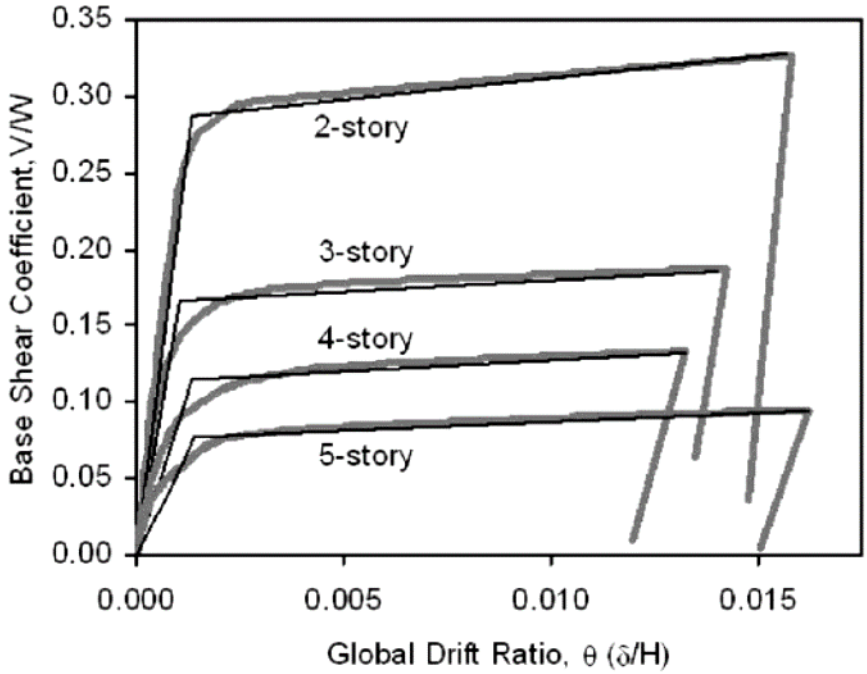

Figure 1-4. Selected capacity curves for 2, 3, 4, and 5 story buildings in Turkey (Akkar et al., 2005).

1.4 Object and Scope

SVA of existing RC residential buildings has been focused by plenty of recent studies in Turkey. However, SVA of facilities where essential public functions are carried out, such as schools, has not been studied comprehensively. The public buildings' vulnerability drew attention since they experienced considerable damage compared to residential buildings after recent EQs in Turkey (Bilgin, 2013). The seismic safety of the public buildings, especially school buildings, is essential for the community's resilience after EQs. National attention has focused on the problem of the high seismic vulnerability of schools for the sake of children's safety.

EQ hazard identification is out of the scope of this study. Identifying high vulnerability buildings is of significant importance for both reliable loss estimation and setting priority criteria for mitigation and strengthening of structures (Erduran and Yakut, 2004). The study's primary concern and scope is the structural vulnerability assessment of RC school buildings in Turkey.

The primary aims of this study are listed below:

- To determine comprehensive statistical information for the RC school buildings in Turkey,
- To determine the capacity-related properties of school buildings located in Turkey,
- To investigate the applicability, on school buildings, of existing walk-down and preliminary SVA procedures developed for residential buildings in Turkey,
- To analyze RC school buildings for assessing their performance using different approximate methods and compare the results with TEC-2007,
- To generate F.C.s for RC school buildings and validate with the observed data,
- To generate F.C.s for RC school buildings for repeated EQs,
- To analyze RC school buildings according to the provisions of Urban Renewal Law and discuss the applicability of provisions,
- To analyze the RC school buildings according to the new EQ code (TBEC-2018) and compare the results with the previous code.

The work done in order to achieve the aims of the study are shortly explained below.

The database of 321 RC school buildings examined through site surveys and assessed by TEC-2007 under the consultancy of professors of Middle East Technical

University (METU) is obtained from the IPCU, the administrative unit of ISMEP. All buildings' as-built properties were determined by site surveys, in-situ, and laboratory tests within the database. The material and structural properties, including the as-built structural drawings of the buildings, soil properties, site observations, and laboratory test results, were explained in detail with the comprehensive reports, and professors of METU confirmed all studies. These buildings formed a comprehensive statistical database reflecting importing properties of school buildings in Turkey.

The walk-down and preliminary SVA procedures (ATC-21/FEMA154, Sucuoğlu et al., Hassan and Sözen, Özcebe et al., and Yakut) are applied to RC school buildings in the database. Assessment results are compared with the linear elastic assessment results of TEC-2007. The parameters used for walk-down and preliminary assessment methods are analyzed statistically for the database. An alternative preliminary assessment method, a merging, and revision of ATC-21 and Yakut's procedures, is proposed for RC school buildings in Turkey consistent with the Turkish EQ Code.

The majority (80 %) of the database are 3, 4, and 5 story RC school buildings, so each story group is considered as a sub-class of the database. 12 RC school buildings are selected for each sub-class. Thus a total of 36 RC school buildings are selected, and 3-D structural models of all buildings are generated using SAP2000 software. The capacity curves of the buildings in both orthogonal directions are obtained by pushover analysis. Bilinear idealization (Acceleration Displacement Response Spectra-ADRS) of the capacity or the pushover curves are obtained. Equivalent single degree of freedom (ESDOF) models are generated for both orthogonal directions of the selected buildings, which makes 72 Equivalent models in total.

The selected buildings are analyzed and assessed by the Displacement Coefficient Method (DCM), ASCE-41 detailed assessment method, and the method proposed by Urban Renewal Law of Turkey. All assessment results are compared with the linear elastic assessment results of TEC-2007.

The selected buildings are also analyzed according to the new version of the provisions under Urban Renewal Law and TBEC-2018. The results were compared by the previous ones and linear elastic assessment results of TEC-2007

Fragility analysis for 3, 4, and 5-story RC school buildings are carried out using Equivalent SDOF models, and F.C.s are developed using both bilinearand, stiffness and strength degradation (Modified Clough) material models. The outcomes and the limitations of the other studies explained in the literature survey on F.C. generation are considered.

Obtained F.C.s are validated by the observed performance data of RC school buildings after 2003 Bingöl EQ

F.C.s for RC School Buildings due to repeated EQs are also derived, which are thought to be useful for rapid decisions for using structures after EQs.

1.5 Organization of the Dissertation

The thesis study is composed of 8 chapters. The first chapter is an introduction section that gives a general overview of the background and research significance, literature survey, and object and scope. The previous studies on school buildings' seismic vulnerability are discussed in the first chapter's literature survey section.

The second chapter is devoted to describing the database composed of 321 RC school buildings used in the study. Seismic performance and general properties of RC buildings in Turkey are mentioned in the first section of the second chapter. The selection procedure of the buildings for 3, 4, and 5 story sub-classes is also described in this chapter.

The assessments of RC school buildings in the database by existing walk-down and preliminary methods and the proposed preliminary assessment method are discussed in the third chapter. The comparisons of assessment results with TEC-2007 are also

mentioned, and the results are discussed. The proposed (Modified ATC-21) Preliminary Assessment Method is also described in the third chapter.

In the fourth chapter, the work done for 3-D modeling, pushover analysis, bilinear idealization of capacity curves, ESDOF model formation, determining performance limit states for each building, and assessment by target displacements of DCM, ASCE-41, and the discussion of the assessment results are described. Detailed assessment results are also compared with TEC-2007 results within the chapter.

In the fifth chapter, the new EQ code of Turkey, TBEC-2018, is examined for RC school buildings, and all 36 RC school buildings are analyzed accordingly. The assessment results are compared by the results of other methods and the results of TEC-2007.

In the sixth chapter, the assessment of 36 RC school buildings under the provisions of the Urban Renewal Law is carried out. Both 2013 and 2019 versions of the provisions are used for assessment, and the results are compared for the two versions, TEC-2007 and TBEC-2018.

The seventh chapter is devoted to the generation of the F.C.s for 3, 4, and 5 story RC school buildings using both bilinear and stiffness and strength degradation (Modified Clough) material models in Turkey, the validation of the F.C.s, and the assessment of related buildings in the database are described.

Finally, the last chapter is devoted to a summary and conclusions of the study and information obtained as well as recommendations for future studies.

CHAPTER 2

BUILDING DATABASE

2.1 Description of the Buildings Database in the Study

The majority of the school buildings worldwide are generally two classrooms wide, with classrooms placed next to each other with a corridor between them (Rodgers, 2012). With this aspect, there are generally three of four continuous RC frames along the long sides of the school buildings, which generates a low redundancy problem for the long sides/directions.

There are generally exterior windows on the exterior side walls to sustain the need for natural lighting and ventilation. Sometimes, these windows are constructed from column to column, which may lead to short column irregularity on the exterior sides.

In order to provide unobstructed sight between the teacher and the students, there are no columns or any kind of interior supports within the classrooms, and the bay widths are generally larger than residential buildings. The school buildings' story heights are usually more than residential buildings to provide enough ventilation for children. Larger bay widths and taller story heights are the main differences and the vulnerability deriving parameters of schools when compared with residential buildings.

The above mentioned architectural configurations may support schools' functions but, unfortunately, increase the seismic vulnerability. Rodgers (2012) noted that other characteristics such as weak or soft stories and heavy roofs are not unique to schools, but it is not totally valid for the Turkish construction practice.

Public buildings like schools and hospitals in Turkey are generally designed by templates developed by the Ministry of Environment and Urban Planning (the previous name was Ministry of Public Works and Settlement) (General Directorate of Construction Affairs in general) according to the former EQ-codes for practical reasons. Still, unfortunately, there are many deficiencies from templates during construction (Bilgin, 2013). Although the used projects reveal minor differences from province to province, they are similar architecturally. Unfortunately, throughout the thesis study, it is observed that this is not the situation for most of the school buildings in İstanbul as well as in the other provinces of Turkey.

There are many typical types of school buildings in İstanbul. IPCU defined 66 different types of school buildings in ISMEP (2008) guidelines, and the number increased in the following years. Detailed information about typical school buildings in İstanbul has been covered in the ISMEP (2008) guideline. It is also mentioned in the guideline that many school buildings have characteristics incorporating several of the listed types.

Using only template designs for determining the seismic vulnerabilities of school buildings will lead to inconsistent results. Considering the uncertainties in material properties and detailing would not be enough to reach adequate validity of assessment analysis. In order to overcome the nonconformance problem, the as-built properties of the RC school buildings are used in this study. Using the as-built properties obtained by site surveys and laboratory tests results for materials reduces the effects of uncertainties.

RC school buildings in Turkey usually consist of RC framing, only a few of them having shear walls. They typically have masonry infill walls, and floor slabs are generally of cast-in-place concrete. Lateral forces are usually resisted by cast-in-place moment frames that develop stiffness through column-beam connections.

A database of 321 RC school buildings located in İstanbul that were assessed in detail under the World Bank program ISMEP and with the consultancy of professors of METU has been compiled for this study. The determined properties of 321 RC

school buildings considered in this study are given in Appendix-A. The general properties of 321 RC school buildings and the sub-classes of 3, 4, and 5 story buildings selected for further analyzes are described below. Each 3, 4, and 5 story subclasses consists of 12 RC school buildings representing the database's general properties, and templates are also taken into account in the selection process.

The general parameters of the school buildings in the database are listed in Table 2-1.

Table 2-1 General parameters of school buildings in the database.

Parameter	**Quantity**
Construction Year	1951-2005
Average Plan Area	475 m^2
Average Story Height	3.30 m
Average Distance to Fault	18.76 km
Average Concrete Compressive Strength	10.22 MPa
Average Steel Tensile Strength	253 MPa
Average Period (T_x/T_y)	0.66/0.62 sec

The distribution of the construction years of RC school buildings in the database is shown below in Figure 2-1.

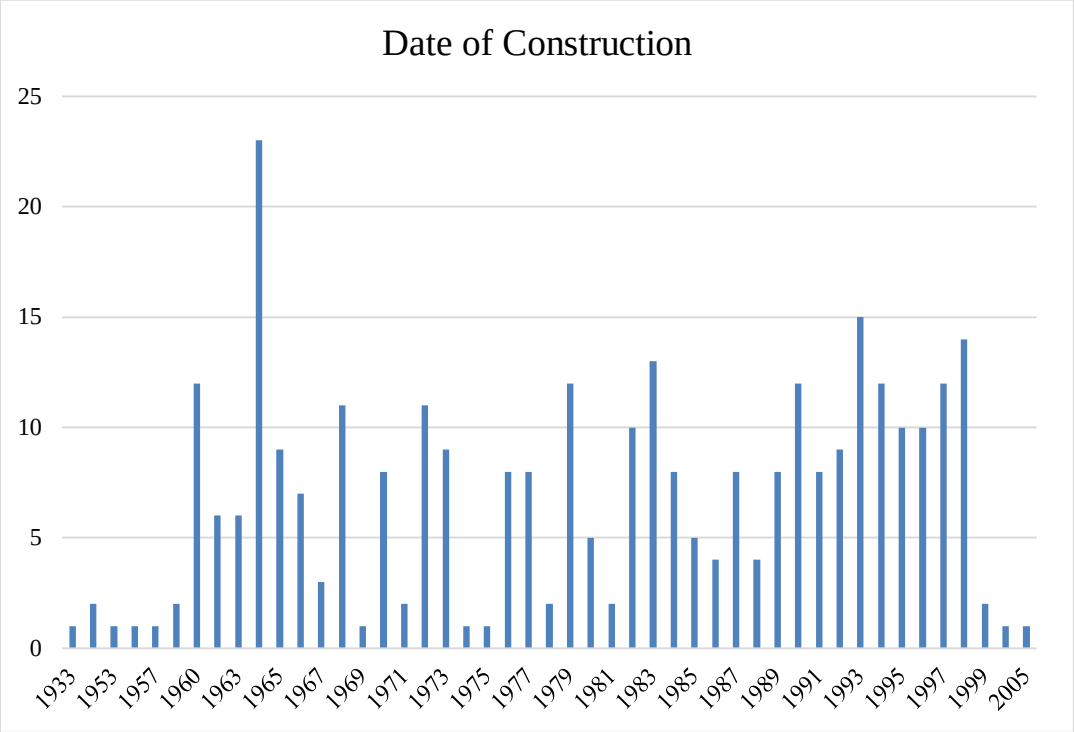

Figure 2-1. The distribution of date of construction of RC school buildings

Concrete compressive strengths of existing buildings in Turkey, which need retrofit for EQ resistance, range from 5 to 16 MPa and public buildings have an average concrete compressive strength of 5.86 MPa (Mazılıgüney et al., 2008). School buildings have an average concrete compressive strength of 10.22 MPa, consistent with Turkey's building stock but slightly better than other public buildings. The minimum permitted concrete strength in the regions of high seismicity is specified as 20 MPa in TEC-2007. The distribution of concrete compressive strengths of the buildings in the database is shown in Figure 2-2 below.

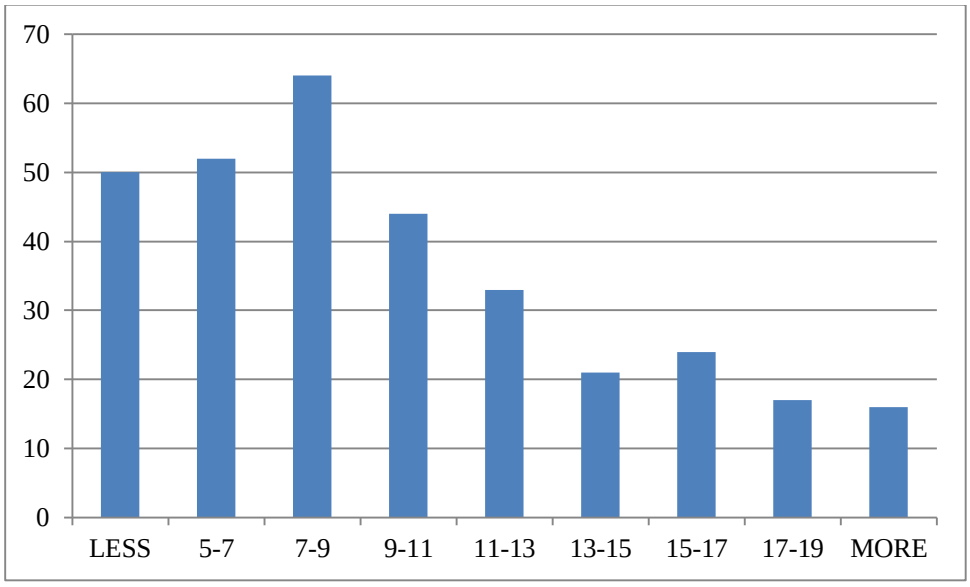

Figure 2-2. The distribution of concrete compressive strengths (MPa)

The periods of 3, 4, and 5 story RC school buildings are listed in Table 2-2. It is observed that the periods in X and Y directions are close to each other. The X-directions of RC school buildings are longer, but there are more shear walls in Y directions, so the average periods in both directions are close to each other.

Table 2-2 Average periods of buildings with respect to story numbers.

Story Number	**T_x (s)**	**T_y (s)**
3	0.52	0.50
4	0.77	0.71
5	0.84	0.76

The average steel tensile strength of the steel used for the database is 253 MPa since STIIIa is used only in 53 of the RC school buildings, and STI is used for the remaining 268 buildings.

These RC school buildings' distance to the nearest fault (mostly the Marmara Sea segment of North Anatolian Fault) was considered the primary seismic source and ranges from 11 to 44 km. 111, 206, and 4 of the school buildings in the database are

located in the 1st, 2nd, and 3rd EQ zones according to the 2007 code. Only four buildings were constructed after TEC-1997, so these four buildings are not included in the subclasses.

The number of basement stories, the number of stories above ground, and unrestrained stories are given in Table 2-3. There are 118 (36.8 %) RC school buildings that have 1 basement story. None of the buildings in the database has 2 or more basement stories. Stories above ground represent the story number of the buildings except for the basement story. If the basement story is above the ground for at least half of its height, it is accepted as a story above the ground. If the basement story is surrounded by shear walls, it is accepted as restrained. The number of unrestrained stories is taken into account for determining the subclasses.

Table 2-3 Number of stories of the school buildings in the database

Number of Stories	**Basement Stories**	**Stories Above Ground**	**Unrestrained Stories**
1	118	21	13
2	-	47	39
3	-	118	96
4	-	113	130
5	-	19	30
6	-	3	12
7	-	-	1

3, 4, and 5 story buildings comprise (256/321=0,7975) nearly 80 % of the RC school buildings stock. 1 and 2 story ones are less vulnerable, while structures having more stories than 5 are more vulnerable with respect to 3, 4, and 5 story buildings according to assessment results of TEC-2007 analysis that will be given in the next section. It was also mentioned in the previous chapter that there is a strong correlation between the number of stories and the damage level for RC buildings in

Turkey. Consequently, 3, 4, and 5 story building sub-classess are determined for further analysis in this study.

The number of continuous frames in the long (X) and short (Y) directions of the RC school buildings in the database is an important parameter. The distribution of the number of continuous frames is given in Figure 2-3 and Figure 2-4 below for long and short directions.

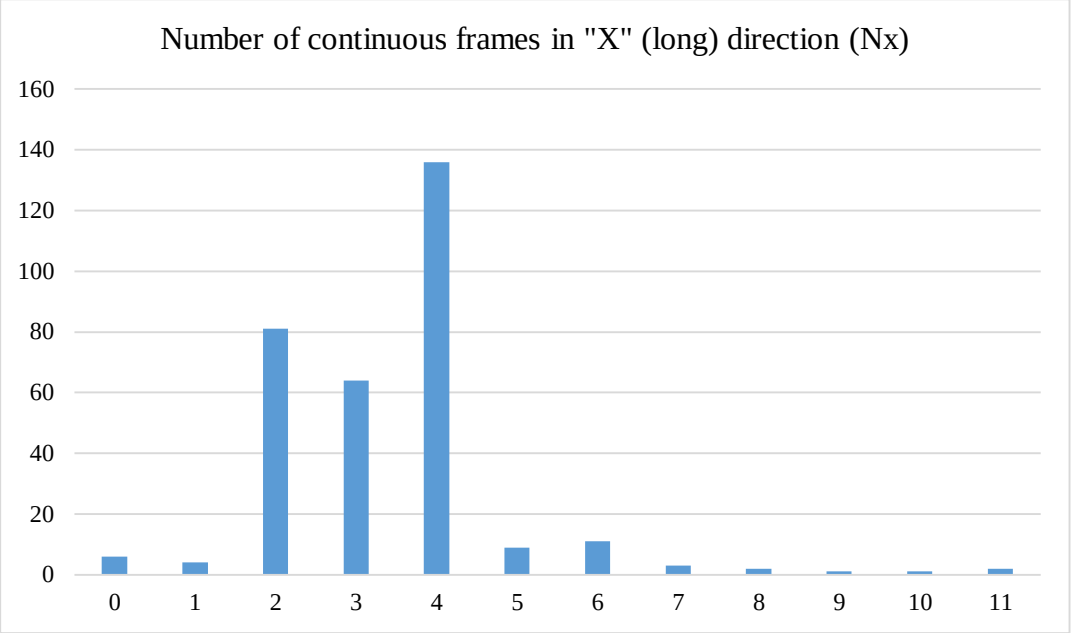

Figure 2-3. The distribution of continuous frames in the long direction of RC school buildings (Nx)

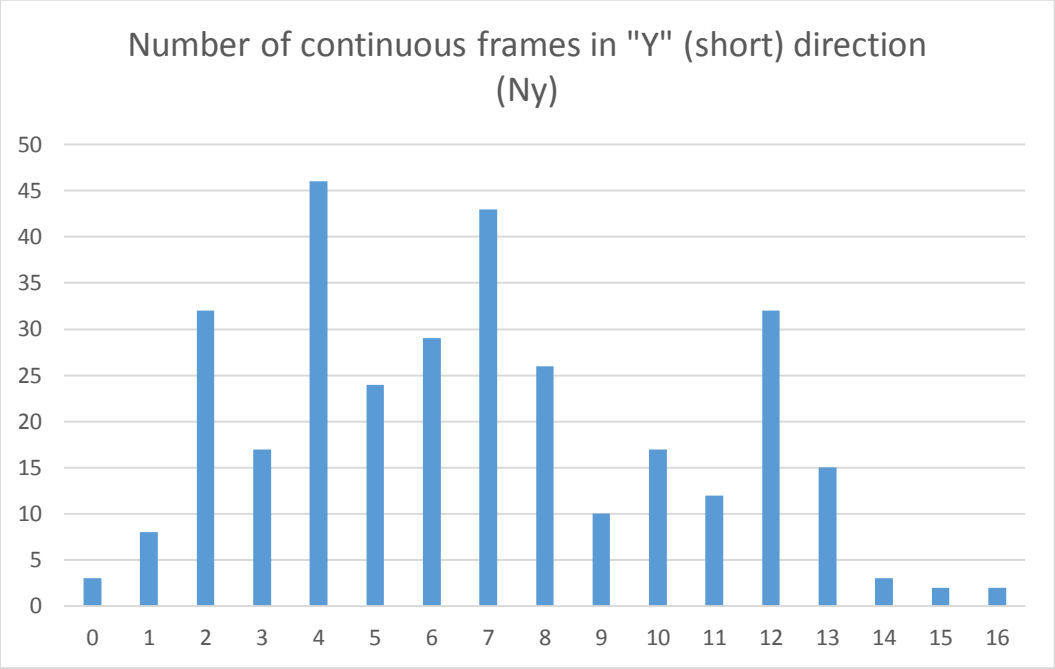

Figure 2-4. The distribution of continuous frames in the short direction of RC school buildings (Ny)

Only 2 of the school buildings have an apparent building quality of “good”, while 114 of them have “average” and 205 of them have “poor” apparent building quality. Factory brick was used in 319 of the school buildings, and local brick was used in only 2 of them, which are not selected for subclasses. 27 of the buildings are located on soil class Z1, while 261 are located on site Z2, and 33 are located on site Z3.

The irregularities observed on the school buildings are listed in Table 2-4. As can be seen from Table 2-4 pounding effect, topographic effects, and the short column are the most frequently observed features for school buildings.

Table 2-4 Number of school buildings having irregularities

Irregularity Definition	**Number of School Buildings Observed**
Topographic Effects	119
Soft-story	43
Weak Story	7
Vertical Irregularity	6
Plan Irregularity	9
Short Column	101
Heavy Overhangs	43
Torsion	7
Pounding Effect	186

43 of RC school buildings in the database have soft-story irregularities, and 7 have a weak story. Soft-story and weak story irregularities in RC school buildings are due to higher ground stories and large entrance halls.

The long sides and the short sides are always named as X and Y directions, respectively, in this study. The density of shear walls in a given principal direction is believed to have a prominent role in the buildings' seismic performance (Gülkan and Sözen, 1999; Yakut, 2004), but 170 of the buildings in the database have no shear walls in either direction.

Detailed assessment of these buildings according to TEC-2007 revealed that 258 (80.37 %) of the school buildings in the database do not satisfy the IO performance state for an EQ having a probability of exceedance of 10 % in 50 years and 219 (68,22 %) of them do not satisfy LS for an EQ having a probability of occurrence of 2 % in 50 years.

It is examined if there is a correlation between concrete compressive strength and EQ performance for the RC school buildings in the database. The concrete

compressive strength distribution of RC school buildings, which do not satisfy the IO performance state for the (design) EQ having a probability of 10 % in 50 years, is shown in Figure 2.5. Similarly, the concrete compressive strengths of RC school buildings, which do not satisfy the LS performance state for the EQ having a probability of exceedance of 2 % in 50 years, is shown in Figure 2.6. The concrete compressive strength distribution of the satisfactory buildings are shown in Figure 2.7 and Figure 2.8. As can be seen from the corresponding figures, there is no correlation between the concrete compressive strengths of RC school buildings and the performance states of them.

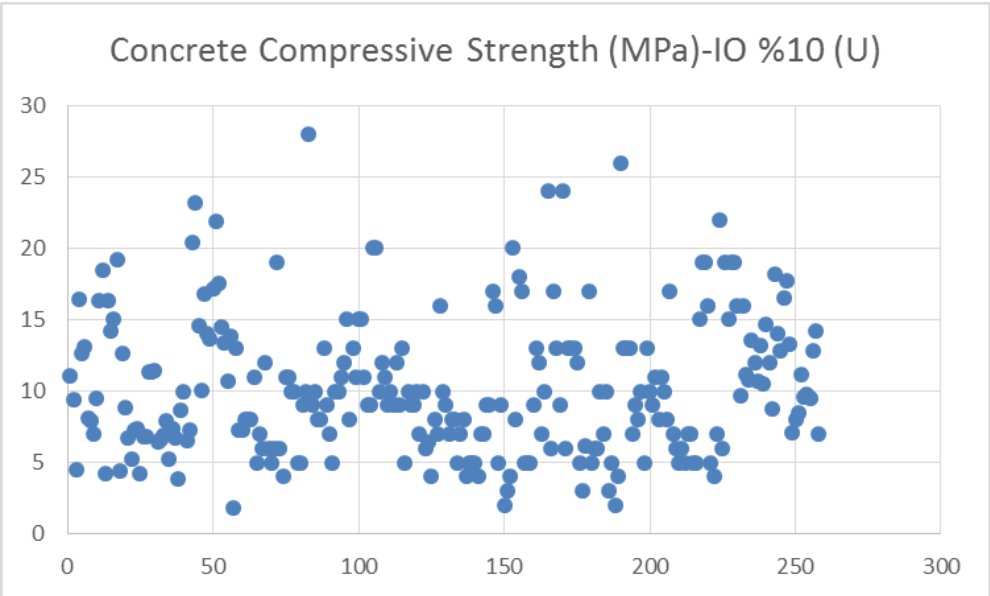

Figure 2.5. Concrete compressive strengths of RC school buildings which do not satisfy IO performance state for the design EQ (%10 in 50 years)

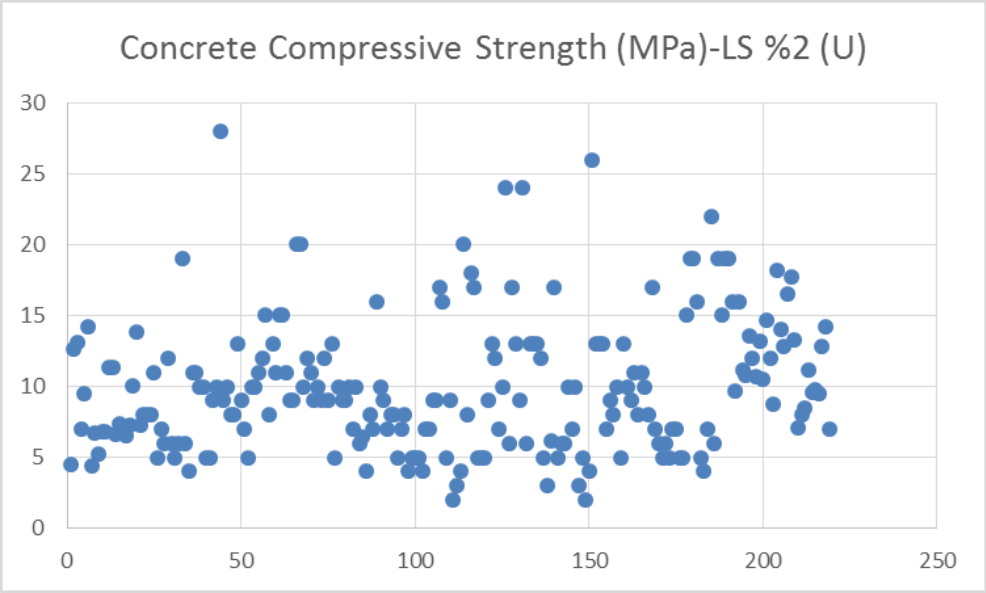

Figure 2.6. Concrete compressive strengths of RC school buildings which do not satisfy IO performance state for the maximum expected EQ (%2 in 50 years)

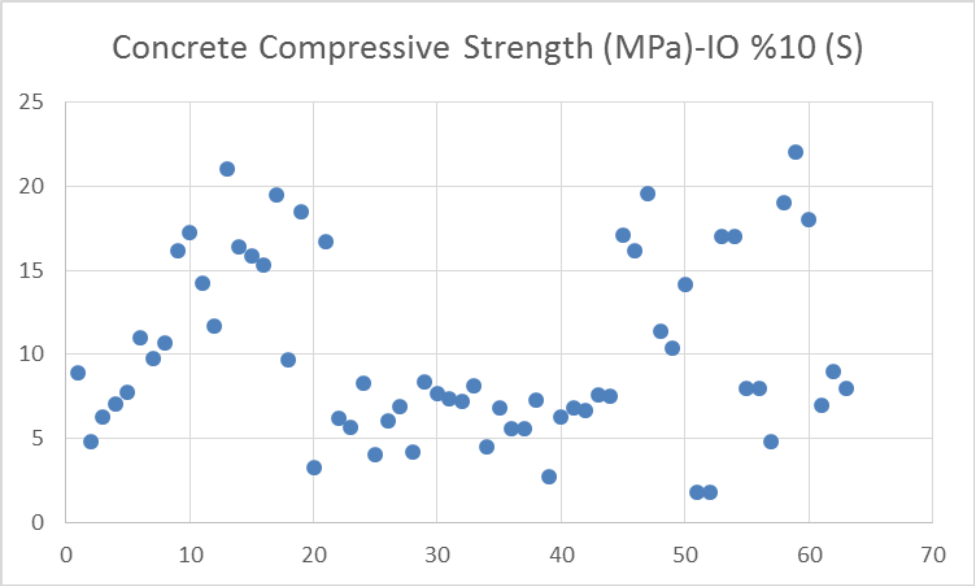

Figure 2.7. Concrete compressive strengths of RC school buildings which satisfy IO performance state for the design EQ (%10 in 50 years)

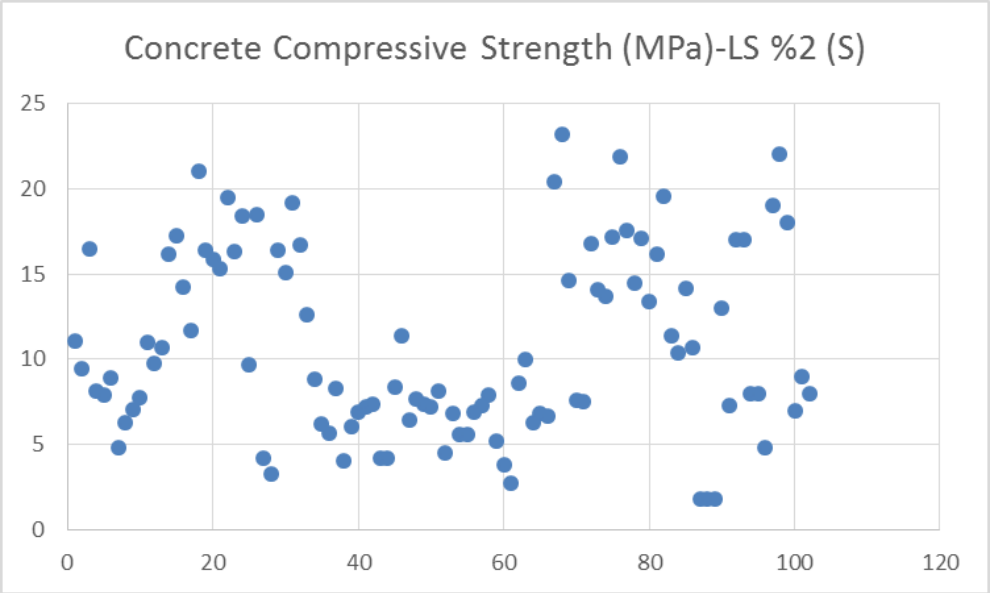

Figure 2.8. Concrete compressive strengths of RC school buildings which satisfy LS performance state for the maximum expected EQ (%2 in 50 years)

RC school buildings in the database are examined for the correlation between the number of stories and performance states. It is observed that all RC school buildings in the database having more than 5 stories cannot satisfy the IO or LS performance states for the design EQ and the maximum expected EQ, respectively. RC school buildings having 1 or 2 stories are less vulnerable to EQs. The situation for 3, 4, and 5-story RC school buildings are close to each other. The percentage of unsatisfactory RC school buildings having 3, 4, and 5 stories in the database are shown in Figure 2.9. The blue bars correspond to the percentages of RC school buildings that do not satisfy the IO performance state for the design (%10 in 50 years) EQ, while the red ones for the maximum expected (%2 in 50 years) EQ

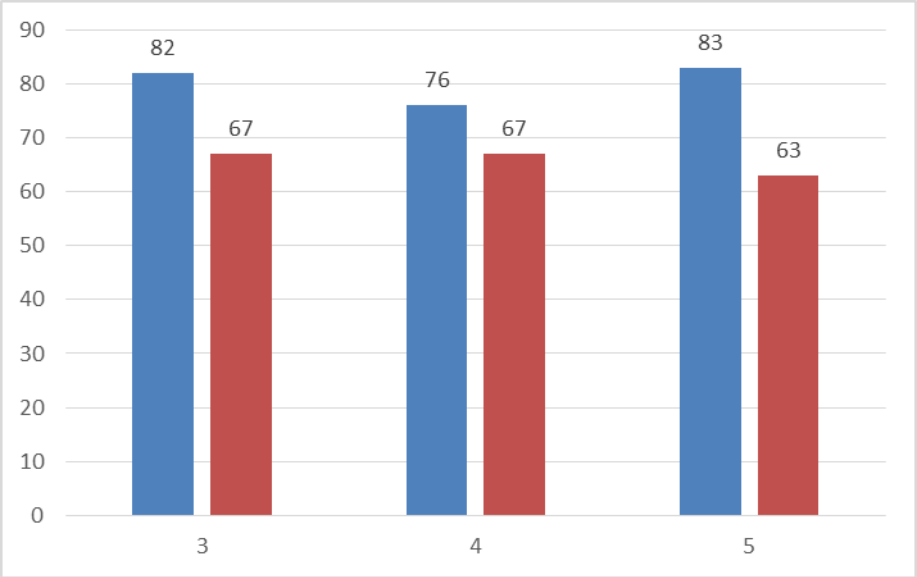

Figure 2.9. Percentages of unsatisfactory buildings for design and maximum expected EQs

It is well known that the number of stories is one of the primary governing factors for the EQ vulnerability of RC building stock in Turkey. The percentages of unsatisfactory RC school buildings of 3, 4, and 5 stories are close since there are other contributing factors, especially structural irregularities.

None of the contributing factors governs the performance states of the RC school buildings, but statistical analysis may give an insight into the weightiness of the contributing factors. Logistic regression analyses are performed by using Minitab-18 software, and the deviance table is constructed to determine the p-values of the selected contributing factors, which are F_{ck}, N_x, N_y, $\%A_{swx}$, and $\%A_{swy}$. There is no statistically meaningful correlation, but the p-values give an insight since the p-values get smaller when the contributing factors get more effective. The screenshots of the deviance tables taken from Minitab-18 software is given in **Error! Reference ource not found.** and Figure 2.11, which shows the most influential factor for EQ safety of RC school buildings is the shear wall ratio in the short direction.

Source	DF	Adj Dev	Adj Mean	Chi-Square	P-Value
Regression	5	54,517	10,9034	54,52	0,000
Fck	1	0,008	0,0077	0,01	0,930
%Aswx	1	0,205	0,2047	0,20	0,651
%Aswy	1	21,187	21,1866	21,19	0,000
Ny	1	3,726	3,7259	3,73	0,054
Nx	1	0,189	0,1890	0,19	0,664

Figure 2.10. Deviance table (screenshot) of logistic regression analysis for design EQ

Source	DF	Adj Dev	Adj Mean	Chi-Square	P-Value
Regression	5	44,478	8,8956	44,48	0,000
Fck	1	1,981	1,9807	1,98	0,159
%Aswx	1	0,028	0,0276	0,03	0,868
%Aswy	1	18,258	18,2579	18,26	0,000
Ny	1	7,815	7,8152	7,82	0,005
Nx	1	0,242	0,2421	0,24	0,623

Figure 2.11. Deviance table (screenshot) of logistic regression analysis for design EQ

Appendix-A provides detailed information about the database and the selected buildings.

2.2 Properties of Selected 36 RC School Buildings

The assessment reports of the 321 RC school buildings are taken from the IPCU and determined as-built properties are provided in Appendix A.

All determined parameters of the selected RC school buildings are tabulated in Appendix A. The selected RC school buildings are labeled with red ID numbers, and

all parameters are written with bold characters in Appendix A. Likewise, the selected 36 RC school buildings are extracted and tabulated separately in Appendix B.

CHAPTER 3

WALK-DOWN AND PRELIMINARY ASSESSMENT

3.1 Introduction to Walk-down and Preliminary Assessment

After severe EQs, extensive losses of human lives and economic property may be faced in Turkey's urban areas or elsewhere, like in the past. For seismic risk reduction in urban areas, it is a high priority to determine the addresses of seismically vulnerable buildings within the existing building stock. Once the seismically vulnerable structures are appropriately identified, the urban seismic risk may be reduced either by retrofitting such buildings individually or applying urban renewal procedures in areas where their density is high (Sucuoğlu and Yazgan, 2003).

As a significant example, the built environment of İstanbul alone consists of about a million buildings. It is anticipated that some 50,000 to 70,000 buildings in İstanbul are expected to experience severe damage or collapse if the probable scenario EQ occurs (JICA, 2002). In order to perform effective seismic reduction for İstanbul, determining the vulnerable buildings, and concentrating the efforts on them should have a high priority.

An experienced professional's judgment is accepted to be the best instrument to identify a vulnerable building, but determining all vulnerable structures by experienced professionals is very expensive for cities like İstanbul and all other EQ prone urban regions. The building inventory should be filtered initially, and the buildings need further examination determined by objective criteria. These criteria and the required calculation level need to be at a very low level of sophistication (Hassan and Sözen, 1997).

As described in the first chapter, SVA procedures can be divided into three main categories depending on their complexity in the literature. These are walk-down

(street survey), preliminary evaluation procedures, and detailed assessment procedures.

Walk-down or street survey is the quickest and most straightforward (the easiest) method. Superficial data (usually composed of deficiencies or irregularities) collected from a quick inspection of the building is sufficient to use these assessment methods. Rapid evaluation techniques serve to identify or rank highly vulnerable buildings by providing a crude index used to rank a group of buildings to determine their priority for further evaluation. No analysis is needed for the first level assessment of building inventory. Walk-down assessment methods determine the priority levels of buildings requiring more detailed studies. These vulnerable buildings determined in the first level are investigated in detail thereafter, if necessary. (Yakut, 2004, and 2004-2 & Kalem, 2010)

Sucuoğlu and Yazgan (2003) mentioned that a street survey procedure must be based on simple structural and geotechnical parameters that can be observed quickly by a sidewalk. The time required for an observer to collect one building's data by the sidewalk is expected not to exceed 10 minutes (Hassan and Sözen, 1997).

The parameters examined by street survey methods are determined by expert opinions or statistical data. After EQs, experts examine the performances of buildings on-site and correlate the performances with structural parameters. Then the importance of the parameters is determined through statistical analysis.

The typical (vulnerability) parameters used by walk-down assessment methods are the number of stories, vertical and plan irregularities, location of the building (including soil properties), age of the building, its structural system, apparent building (including material and workmanship) quality, existence of soft and weak story, overhangs, short columns, etc.

Generally, in walk-down assessment procedures, an initial score is first given to the building examined depending on structural parameters, soil conditions, or seismic risk zones. Then the structure is examined to determine the existence of vulnerability

parameters described above. Each parameter generally reflects a negative effect on the buildings' seismic vulnerability, so the initial score is reduced by penalties appropriately for each parameter observed. By this procedure, a performance score is obtained for each building individually. A cut-off score is determined or calculated, and the buildings with performance scores lower than the cut-off value are accepted to be seismically vulnerable than others. The performance scores of each building also reflect the building's priority for the next level of assessments. A general equation for calculating the seismic PS is given below (Sucuoğlu and Yazgan, 2003):

$$PS=(\text{Initial Score})-\sum(\text{Vulnerability parameter})x(\text{Penalty Score}) \quad (3\text{-}1)$$

FEMA-154 or ATC-21, and Sucuoğlu and Yazgan (2003), and Sucuoğlu et al. (2007) are three examples of walk-down assessment procedures. ATC-21/FEMA-154 (1998) method has been developed to determine the vulnerable buildings that would show collapse or collapse prevention performance states under the design EQ Sucuoğlu and Yazgan (2003) and Sucuoğlu et al. (2007) walk-down assessment procedures are based on the Düzce (1999) database consisting of 9685 RC residential buildings. They aim to determine the buildings to be collapsed during the design EQ ATC-21, Sucuoğlu and Yazgan (2003), and Sucuoğlu et al. (2007) methods are used in this study and the assessment results are evaluated with respect to TEC-2007 (linear elastic) assessment results.

When a more reliable and detailed assessment than the walk-down survey is needed, preliminary assessment techniques are employed. In addition to the data collected for the walk-down assessment procedures, the size and orientation of the structural components, material properties, and layout are needed to perform practical and straightforward procedures. In this level, a simplified analysis of the buildings under investigation can be performed; entry to the buildings and examining the drawings may be necessary depending on the method used. The building's expected performance state is usually predicted by comparing the index derived by preliminary assessment methods against anticipated demand. The success of these

techniques depends on the availability and quality of data. (Yakut, 2004 and 2004-2; Erduran, 2005)

Hassan and Sözen (1997), Özcebe et al. (2003), and Yakut's (2004) methods are three examples of preliminary assessment methods, and they are used in this study. These methods generally depend on the dimensions of the lateral (EQ) load-carrying/resisting structural elements.

Hassan and Sözen (1997) method is based on the Erzincan (1992) database consisting of 46 institutional buildings. Özcebe et al. (2003) method is based on the Düzce (1999) database composed of 484 RC residential buildings. Yakut's (2004) method depends on both the material properties and strength of the structures. Yakut's (2004) method aims to distinguish buildings that would have heavy damage during the design EQ

It is possible to survey large building stocks by employing the walk-down and preliminary assessment methods within a reasonable time and reasonable cost. The primary point is that these practical methods may provide valuable information about a building stock's seismic vulnerability. They are not tailored to estimate a single building's expected seismic performance (Binici et al., 2015).

The in-depth evaluation of the buildings by detailed and sophisticated structural analysis is the third level of SVA, which will be discussed in the next chapters. The preliminary assessment is the most widely used one when a quick and reliable evaluation is needed among three assessment levels.

3.2 Assessment with Existing Methods

The objective of walk-down and preliminary analysis is to perform a quick and straightforward evaluation to decide about the expected seismic performance of the buildings for which detailed evaluation procedures could not be applied due to lack of data and enough time.

Turkish Earthquake Code (TEC-2007 and TBEC-2018 after January 2019) is Turkey's legal technical seismic design code. TEC-1997 was updated, with a new version issued in March 2007, to include a section in Chapter 7 on seismic assessment and retrofit of existing buildings. TEC-2007 has been legally in effect since March 2007 till 2019 when TBEC-2018 came to be in effect. It is used for the design of new buildings and the assessment and retrofit of existing buildings.

Although comprehensive research devoted to reinforced concrete residential buildings yielded capacity curve parameters and SVA procedures for residential buildings in Turkey, a similar endeavor has not been given to school buildings. Therefore, this section of the study focuses on determining the consistency of existing assessment procedures with TEC-2007 for school buildings in Turkey. Analytical and statistical analyzes are performed to investigate the applicability, on school buildings, of existing SVA procedures developed for residential buildings in Turkey.

The performance of the buildings is mainly grouped into three as IO, LS, and CP by TEC-2007. In addition, the physical damage states of the buildings were also identified based on the performance levels. There are mainly four damage levels that are negligible, light, moderate, and heavy. The negligible and light damage states correspond to the immediate occupancy performance level. The moderate damage state corresponds to the life safety performance level, and the heavy damage level corresponds to the collapse prevention.

For school buildings, TEC-2007 requires two different performance states for two different earthquake load levels. School buildings should satisfy IO performance state for an earthquake loading having a probability of exceedance of 10 % in 50 years lifetime and LS for an earthquake ground motion having a probability of exceedance of 2 % in 50 years lifetime.

The walk-down and preliminary assessment procedures used in this study will not be discussed in detail. For detailed information about the assessment procedures used, researchers may examine the given references for each method.

Testing the predictive success of walk-down and preliminary analysis of structures in Turkey can best be accomplished by observed damage after EQs in a collection of buildings with dimensional and material properties on the basis of random choices made during the design and construction stages. In this way, whether a rational explanation of the empirical evidence is contained in theory can be answered with better confidence (Yakut et al., 2005).

In this study, the predictive success of the walk-down and preliminary SVA methods will be checked by comparing the assessment results of each technique with the assessment results of TEC-2007. The proposed check in this study neither checks the validity of the methods nor describes the one as better than others, but reflects the consistency of the methods with TEC-2007.

As mentioned earlier, the buildings employed were assessed using the current seismic design code in force in Turkey within the ISMEP project under the consultancy of the academic staff of METU. In order to evaluate the efficiency and consistency of existing walk-down and preliminary seismic performance assessment procedures developed by ATC-21/FEMA-154 (FEMA, 1988), Sucuoğlu and Yazgan (2003), Sucuoğlu et al. (2007), Yakut (2004), Hassan & Sözen (1997) and Özcebe et al. (2003), these procedures were used to determine the expected seismic performance of all 321 school buildings in the database. The assessments based on TEC-2007 have been listed, and according to these results, the consistency classification rates of walk-down and preliminary seismic assessment procedures were determined.

A similar tabulation is used to evaluate each performance method, as can be seen in the tables below. The assessment found by TEC-2007 analysis is given in the first column, and the assessment based on the used procedure is provided in the next column. The desired performance state for the related probability of occurrence of EQ is also mentioned shortly. “IO 10 %” stands for the immediate occupancy performance state for the EQ having a probability of occurrence of 10 % in 50 years, and “LS 2 %” stands for life safety performance state for the EQ having a probability

of exceedance of 2 % in 50 years lifetime. Evaluations for the code-required EQ intensities are shown by individual tables. The letter “U” shows unsatisfactory, “S” corresponds to satisfactory, and “M” stands for moderate. The classification rates and the correlations are given in percentages in the third column. Also, the relative percentages are given for U and S individually in the last column. As an example, the relative percentage of U-U stands for the percentage of correctly determined unsatisfactory school buildings. Since one of the critical parameters for the consistency of the related method with TEC-2007 is the relative ratio (%) of the U-U case, it is written in bold characters. The consistency ratio (i.e., the sum of consistency ratios of U-U and S-S) is another important parameter and is also written in bold characters.

Evaluation of ATC-21/FEMA-154 (FEMA, 1988) procedure with TEC-2007 for the IO performance state can be seen in Table 3-1. The unsatisfactory buildings were assessed by a 78.68 % relative consistency ratio, and the method is evaluated to be 75.39 % consistent with TEC-2007.

Table 3-1 Evaluation of ATC-21 (FEMA, 1988) procedure with TEC-2007 for IO 10 % case

TEC IO 10 %	**ATC-21(FEMA154)**	**Ratio (%)**	**Relative Ratio (%)**
U	U	63.24	**78.68**
U	S	17.13	21.32
S	U	7.48	38.10
S	S	12.15	61.90
	Consistency (%)	**75.39**	

Evaluation of ATC-21 (FEMA-154) procedure with TEC-2007 for the LS performance state is summarized in Table 3-2. The unsatisfactory buildings may be assessed by a 77.63 % relative consistency ratio, and the method is evaluated to be 66.98 % consistent with TEC-2007.

Table 3-2 Evaluation of ATC-21(FEMA, 1988) procedure with TEC-2007 for LS 2 % case

TEC IO 2 %	ATC-21(FEMA154)	Ratio (%)	Relative Ratio (%)
U	U	52.96	**77.63**
U	S	15.26	22.37
S	U	17.76	55.88
S	S	14.02	44.12
	Consistency (%)	**66.98**	

Evaluation of Sucuoğlu and Yazgan (2003) procedure with TEC-2007 for the IO performance state is given in Table 3-3. The unsatisfactory buildings may be assessed by a 7.75 % relative consistency ratio, and the method is evaluated to be 24.3 % consistent with TEC-2007. The cut-off value is assumed as 50 for both cases.

Table 3-3 Evaluation of Sucuoğluand Yazgan (2003) procedure with TEC-2007 for IO 10 % case

TEC IO 10 %	Sucuoğlu et al.	Ratio (%)	Relative Ratio (%)
U	U	6.23	**7.75**
U	S	74.14	92.25
S	U	1.56	7.94
S	S	18.07	92.06
	Consistency (%)	**24.30**	

Evaluation of Sucuoğlu and Yazgan (2003) procedure with TEC-2007 for the LS performance state can be seen in Table 3-4. The unsatisfactory buildings may be assessed by a 7.31 % relative consistency ratio, and the method is evaluated to be 33.95 % consistent with TEC-2007.

Table 3-4 Evaluation of Sucuoğlu and Yazgan (2003) procedure with TEC-2007 for LS 2 % case

TEC LS 2 %	Sucuoğlu et all.	Ratio (%)	Relative Ratio (%)
U	U	4.98	7.31
U	S	63.24	92.69
S	U	2.80	8.82
S	S	28.97	91.18
	Consistency (%)	**33.95**	

All school buildings are assessed to be unsatisfactory with the Sucuoğlu et al. (2007) method, i.e., none of the school buildings would behave satisfactorily according to this method. Evaluation of Sucuoğlu et al. (2007) procedure with TEC-2007 for the IO performance state can be seen in Table 3-5. The relative ratios seem to be high for the unsatisfactory case, but it does not have a logical meaning since the method evaluates all buildings as unsatisfactory. The process determines the unsatisfactory buildings by 100 % consistency with TEC-2007 but also has a 100 % inconsistency with TEC-2007 for the satisfactory buildings.

Table 3-5 Evaluation of Sucuoğlu et al. (2007) procedure with TEC-2007 for IO 10 % case

TEC IO 10 %	Sucuoğlu et al.	Ratio (%)	Relative Ratio (%)
U	U	80,37	**100**
U	S	0	0
S	U	19,63	100
S	S	0	0

Evaluation of Sucuoğlu et al. (2007) procedure with TEC-2007 for the LS performance state can be seen in Table 3-6. The relative ratios seem to be high for

the unsatisfactory case, but it does not have a logical meaning since the method evaluates all buildings as unsatisfactory as described above.

Table 3-6 Evaluation of Sucuoğlu et al. (2007) procedure with TEC-2007 for LS 2 % case

TEC LS 2 %	Sucuoğlu et al.	Ratio (%)	Relative Ratio (%)
U	U	68,22	**100**
U	S	0	0
S	U	31,78	100
S	S	0	0

Evaluation of Hassan and Sözen (1997) procedure with TEC-2007 for the IO performance state can be seen in Table 3-7. The unsatisfactory buildings may be assessed by 51 % relative consistency ratio, and the method is evaluated to be 68.69 % consistent with TEC-2007. The total consistency ratio of this method is calculated by summing the percentages of U-U, S-S ratios, and half of the U-M and S-M ratios.

Table 3-7 Evaluation of Hassan and Sözen procedure with TEC-2007 for IO 10 % case

TEC IO 10 %	Hassan-Sözen	Ratio (%)	Relative Ratio (%)
U	U	40.81	**50.78**
U	S	7.48	9.30
U	M	32.09	39.92
S	U	1.87	9.52
S	S	5.92	30.16
S	M	11.84	60.32
	Consistency (%)	**68.69**	

Evaluation of Hassan and Sözen procedure with TEC-2007 for the LS performance state is given in Table 3-8. The unsatisfactory buildings may be assessed by a 52 % relative consistency ratio, and the method is evaluated to be 64.65 % consistent with TEC-2007. The total consistency ratio is calculated as described in the above paragraph.

Table 3-8 Evaluation of Hassan and Sözen procedure with TEC-2007 for LS 2 % case

TEC LS 2 %	Hassan-Sözen	Ratio (%)	Relative Ratio (%)
U	U	35.20	**51.60**
U	S	6.85	10.05
U	M	26.17	38.36
S	U	7.48	23.53
S	S	6.54	20.59
S	M	17.76	55.88
	Consistency (%)	**64.65**	

Hassan and Sözen (1997) criticized their method themselves by the words below:

"Undoubtedly, the method leaves out more variables than it includes. It is insensitive to changes in material quality, story height, girder properties, framing in upper stories, and detail. It lacks proof by theory or experiment. It cannot make absolute judgments about structural safety. Nevertheless, it offers a pragmatic method for identifying the most vulnerable in a regional inventory of low-rise buildings with monolithic RC framing. Granted that an engineer's judgment is the most important criterion for determining seismic vulnerability, the proposed method conserves the most expensive ingredient: time of the experienced professional."

Gulkan and Sözen (1999) showed that the wall and column areas influence the drift on the ground floor. Hassan and Sözen (1997) method is practical and

straightforward to use. Still, the major drawback of this method is the underlying assumption that the quality of construction, quality of material, and as-built properties of the buildings examined are uniform. The method itself is somehow a force-based assessment procedure, but the effect of concrete strength is ignored (Yakut, 2004).

Evaluation of Özcebe et al. (2003) procedure with TEC-2007 for the IO performance state can be seen in Table 3-9. The unsatisfactory buildings may be assessed by a 3.57 % relative consistency ratio, and the method is evaluated to be 30.64 % consistent with TEC-2007. The total consistency ratio is calculated as described in the above paragraphs.

Table 3-9 Evaluation of Özcebe et al. (2003) procedure with TEC-2007 for IO 10 % case

TEC IO 10 %	**Özcebe et al. 2003**	**Ratio (%)**	**Relative Ratio (%)**
U	U	2.86	**3.57**
U	S	60.32	75.40
U	M	16.83	21.03
S	U	0.00	0.00
S	S	18.73	93.65
S	M	1.27	6.35
	Consistency (%)	**30.64**	

Evaluation of Özcebe et al. (2003) procedure with TEC-2007 for the LS performance state can be seen in Table 3-10. The unsatisfactory buildings may be assessed by a 3.76 % relative consistency ratio, and the method is evaluated to be 39.20 % consistent with TEC-2007. The total consistency ratio is calculated as described in the above paragraphs.

Table 3-10 Evaluation of Özcebe et al. (2003) procedure with TEC-2007 for LS 2 % case

TEC LS 2 %	Özcebe et al. 2003	Ratio (%)	Relative Ratio (%)
U	U	2.54	**3.76**
U	S	51.43	76.06
U	M	13.65	20.19
S	U	0.32	0.98
S	S	27.62	85.29
S	M	4.44	13.73
	Consistency (%)	**39.20**	

Yakut's (2004) procedure has two different assessment versions. The only difference between them is including or excluding the masonry filler walls in the assessment procedure. Evaluation of Yakut (including walls) procedure with TEC-2007 for the IO performance can be seen in Table 3-11. The unsatisfactory buildings may be assessed by a 50.78 % relative consistency ratio, and the method is evaluated to be 46.39 % consistent with TEC-2007.

Table 3-11 Evaluation of Yakut (2004) procedure (including walls) with TEC-2007 for IO 10 % case

TEC IO 10 %	Yakut (including walls)	Ratio (%)	Relative Ratio (%)
U	U	40.75	**50.78**
U	S	39.50	49.22
S	U	14.11	71.43
S	S	5.64	28.57
	Consistency (%)	**46.39**	

Evaluation of Yakut (2004) (including walls) procedure with TEC-2007 for the LS performance state can be seen in Table 3-12. The unsatisfactory buildings may be

assessed by a 49.77 % relative consistency ratio, and the method is evaluated to be 44.83 % consistent with TEC-2007.

Table 3-12 Evaluation of Yakut (2004) procedure (including walls) with TEC-2007 for LS 2 % case

TEC LS 2 %	Yakut (including walls)	Ratio (%)	Relative Ratio (%)
U	U	33.86	**49.77**
U	S	34.17	50.23
S	U	21.00	65.69
S	S	10.97	34.31
	Consistency (%)	**44.83**	

Evaluation of Yakut (2004) (excluding walls) procedure with TEC-2007 for the IO performance state for the EQ having a probability of exceedance of 10 % in 50 years lifetime can be seen in Table 3-13. The unsatisfactory buildings may be assessed by a 50.00 % relative consistency ratio, and the method is evaluated to be 45.77 % consistent with TEC-2007.

Table 3-13 Evaluation of Yakut (2004) procedure (excluding walls) with TEC-2007 for IO 10 % case

TEC IO 10 %	Yakut (excluding walls)	Ratio (%)	Relative Ratio (%)
U	U	40.13	**50.00**
U	S	40.13	50.00
S	U	14.11	71.43
S	S	5.64	28.57
	Consistency (%)	**45.77**	

Evaluation of Yakut (2004) (excluding walls) procedure with TEC-2007 for the LS performance state for the EQ having a probability of exceedance of 2 % in 50 years

lifetime can be seen in Table 3-14. The unsatisfactory buildings may be assessed by a 48.85 % relative consistency ratio, and the method is evaluated to be 44.20 % consistent with TEC-2007.

Table 3-14 Evaluation of Yakut (2004) procedure (excluding walls) with TEC-2007 for LS 2 % case

TEC LS 2 %	Yakut (excluding walls)	Ratio (%)	Relative Ratio (%)
U	U	33.23	**48.85**
U	S	34.80	51.15
S	U	21.00	65.69
S	S	10.97	34.31
	Consistency (%)	**44.20**	

The primary objective for walk-down and preliminary SVA procedures is to assess the unsatisfactory buildings correctly. In order to compare the assessment procedures used in this study, the relative consistency percentages for unsatisfactory buildings are tabulated, and this comparison can be seen in Table 3-15. Sucuoğlu et al. (2007) method is not included in the table since all 321 school buildings are determined to be unsatisfactory by this method.

Table 3-15 Comparison of assessment procedures with TEC-2007 for school buildings

Assessment Procedure	Ratios for Relative Consistency of U-U case		Ratios of Total Consistency	
	TEC IO 10 %	TEC LS 2 %	TEC IO 10 %	TEC LS 2 %
ATC-21/FEMA-154 (1998)	78.68	77.63	75.39	66.98
Sucuoğlu and Yazgan (2003)	7.75	7.31	24.30	33.95
Hassan and Sözen (1997)	50.78	51.60	68.69	64.65
Özcebe et al. (2003)	3.57	3.76	30.64	39.20
Yakut (2004) (including walls)	50.78	49.77	46.39	44.83
Yakut (2004) (excluding walls)	50.00	48.85	45.77	44.20

ATC-21/FEMA-154 (1998) SVA method is evaluated to be approximately 80 percent relatively and 75 % overall consistent with TEC-2007 for RC school buildings in Turkey.

Hassan and Sözen (1997) and Yakut (2004) procedures are approximately 50 percent relative consistent with TEC-2007 for RC school buildings in Turkey. Since there are usually big openings on the masonry walls, including or excluding the masonry walls of RC school buildings, do not cause a significant change in Yakut's (2004) procedure.

Sucuoğlu and Yazgan (2003), Sucuoğlu et al. (2007), and Özcebe et al. (2003) procedures that rely mostly on visual properties of buildings such as irregularities are not consistent with TEC-2007 for RC school buildings in Turkey.

Muvafık and Özdemir (2018) studied the correlation of walk-down and preliminary assessment methods with 50 buildings that are totally collapsed or severely damaged in Van EQ, 23rd October 2011. Muvafık and Özdemir (2018) concluded that Yakut's preliminary assessment method is examined to be the most successful one, with 86% accuracy.

The differences in assessment results are due to different parameters taken into consideration in different assessment procedures. It should be noted that the performances of the RC school buildings are determined by analysis software, and the real performances may, of course, be different.

Although none of the procedures were found to adequately determine the performance in conformance with TEC-2007, the ATC-21/FEMA-154 (1998) method is the most suitable one to assess approximately the performance of existing RC school buildings consistent with TEC-2007. Since the Turkish EQ Code is in force and obligatory for all RC school buildings, the method which has the best consistency with TEC-2007 is advised to be preferred for rapid assessment purposes, which is ATC-21/FEMA-154 (1998).

3.3 Proposed Preliminary Assessment Method

As mentioned in the previous section, the ATC-21/FEMA-154 (1998) walk-down SVA method is determined to be 75.39 % overall consistent with TEC-2007 for RC school buildings in Turkey. As mentioned in Table 3-1, these ratios are for IO performance level and an EQ with a probability of occurrence of 10 % in 50 years; i.e., if the building satisfies IO performance, it is labeled as "Satisfactory" (S), and if not it is labeled as "Unsatisfactory"(U). The ratios, i.e., the evaluation of ATC-21 (FEMA 154) method for the EQ having a probability of exceedance of 2 % in 50

years, is given in Table 3-2. Comparison of assessment procedures with TEC-2007 for school buildings is shown in Table 3-15.

Assessment of vulnerable buildings in EQ prone regions has always become a challenging task for structural engineering. Since the ATC-21/FEMA-154 (1998) method has been developed to determine the vulnerable buildings that would show collapse or collapse prevention performance states under the design EQ, it is not expected to match one to one with TEC-2007 assessment results. The basic disadvantage for ATC-21 (FEM-154) (1998) procedure is the lack of contribution of material properties for the assessment of the buildings.

Considering the large number (more than 55 thousand) of RC school buildings in Turkey and inaduacy of inelastic analysis, it is clear that a simple and convenient practical assessment method would be useful for structural engineers. In order to develop a more convenient assessment method, material properties of structural components as well as size and orientation of them should be taken into account.

Yakut's (2004) preliminary assessment procedure for existing RC buildings is suitable for this purpose, and its overall consistency with TEC-2007 is 46 %. Only the continuous infill walls without openings are taken into account by Yakut's (2004) method, likewise other structural analysis procedures. The infill walls of RC school buildings generally have large openings. Because the number of continuous infill walls without openings is low, the BCPI values calculated by including or excluding the infill walls are quite close to each other. It can be seen in Table 3-11, Table 3-12, Table 3-13, and Table 3-14 that the difference between TEC-2007 consistencies of including or excluding infill walls are quite low. Since there is not much difference between the BCPI values of RC school buildings by including or excluding the masonry walls, BCPI values calculated without including masonry walls are preferred.

In order to develop a preliminary assessment method that is more consistent with TEC-2007, ATC-21 (FEMA-154) (1998) method is improved by modifying the penalties for structural irregularities and determining the basic structural score by

taking into account BCPI of Yakut (2004) preliminary assessment method. Combining the two approaches, size, orientation, and material properties of the structural components are taken into account with structural irregularities. The study's database consists of 321 RC school buildings used to find the best penalties for structural irregularities. The proposed preliminary assessment method is explained briefly below.

BCPI of RC school buildings is calculated as described in Yakut's (2004) preliminary assessment method. Since the amount of continuous walls is negligible for most RC school buildings, and there is not much difference between the TEC-2007 consistency results of Yakut's (2004) methods, BCPI is preferred to be calculated by excluding walls.

BSS is determined concerning BCPI in both orthogonal directions. If BCPI is greater than equal to 1.2, the BSS is 100, otherwise 90.

$BCPI \geq 1.2 \Rightarrow BSS = 100$

$BCPI < 1.2 \Rightarrow BSS = 90$

The penalties determined for the structural irregularities are subtracted from the BSS to find the Total Structural Score (TSS). The penalty scores of structural irregularities determined by many trials for the best consistency with TEC-2007 are listed below in Table 3-16.

Table 3-16 Penalties for structural irregularities

Structural Irregularity	Penalty Score
Average Condition	-10
Poor Condition	-20
Vertical Irregularity	-40
Torsion	-40
Soft-story	-40
Plan Irregularity	-40
Pounding Effect	-10
Short Column	-20
Weak Story	-40
Heavy Overhang	-10

TSS > 80 => “S” (Satisfactory)

TSS < 80 => “U” (Unsatisfactory)

If TSS = 80 and Total Penalty Score ≥ -10 => “S”

If the shear wall ratio (with respect to surface area) for all stories for both of the orthogonal directions is greater than 0.004 (0.4 %), the building is accepted to be “S”.

Evaluation of the proposed procedure with TEC-2007 for the LS performance state for the EQ having a probability of 2 % in 50 years lifetime can be seen in Table 3-17. The unsatisfactory buildings may be assessed by a 91.09 % relative consistency ratio, and the method is evaluated to be 81.00 % consistent with TEC-2007.

Table 3-17 Evaluation of proposed procedure with TEC-2007 for LS 2 % case

TEC IO 10 %	Proposed Method	Ratio (%)	Relative Ratio (%)
U	U	73.21	**91.09**
U	S	7.17	9.91
S	U	11.83	60.32
S	S	7.79	39.68
	Consistency (%)	**81.00**	

Evaluation of the proposed procedure with TEC-2007 for the IO performance state for the EQ having a probability of 10 % in 50 years lifetime can be seen in Table 3-18. The unsatisfactory buildings may be assessed by a 91.78 % relative consistency ratio, and the method is evaluated to be 71.96 % consistent with TEC-2007.

Table 3-18 Evaluation of proposed procedure with TEC-2007 for IO 10 % case

TEC IO 10 %	Proposed Method	Ratio (%)	Relative Ratio (%)
U	U	62.62	**91.78**
U	S	5.61	8.22
S	U	22.43	70.59
S	S	9.35	29.41
	Consistency (%)	**71.96**	

It is believed that the proposed preliminary assessment method would be useful for structural engineers for the assessment of a group of RC school buildings with respect to TEC-2007. Although TEC-2007 has been modified, and TBEC-2018 is in effect since January 2019, the proposed method may give insight to structural engineers about the seismic vulnerabilities of RC school buildings.

Since the consistency of the proposed method with TEC-2007 is more than 90 % for unsatisfactory buildings, it is advised to be used for preliminary assessment of RC school building groups.

CHAPTER 4

DETAILED ASSESSMENT

4.1 Introduction to Detailed Assessment Procedures

The third category among assessment procedures that involve the in-depth evaluation of the buildings is the detailed assessment procedures. Detailed assessment procedures employ linear or non-linear analysis of the related structures and require dimensions of structural components and the reinforcement details of the buildings. Displacement Coefficient Method (FEMA-440, 2005), TEC-2007, TBEC-2018, FEMA-356 (2000), and Urban Renewal Law (URL) (MEU, 2013 and 2019) are examples of detailed assessment procedures which are used in this study.

Dimensions of structural components, the reinforcement details, and the mechanical properties of materials of the related buildings may be obtained from structural application drawings or as-built features of the structure determined by site surveys, in-situ, and laboratory testing. There may be many deficiencies between structural drawings and as-built properties of buildings in developing countries like Turkey. It is strongly advised to use the as-built properties to overcome the inconsistent assessment results for countries like Turkey.

After obtaining the buildings' required data, a representative structural model is formed, and linear or non-linear analysis techniques are employed to determine the response of the structure for an anticipated EQ. The obtained response quantities are then compared with specific accepted values to arrive at a decision regarding the building's expected performance (Yakut, 2004-2).

Methods for EQ vulnerability assessment of structures are legally introduced by TEC-2007 (TBEC-2018 since January 2019) in Turkish practice. Chapter 7 of TEC-2007, named “Evaluation and Strengthening of Existing Buildings,” declares the

rules of calculation to be used to assess the existing buildings in EQ zones and the procedures to be followed for the decisions of strengthening them. Chapter 7 of TEC-2007 requires member forces to be calculated with unreduced EQ demands, i.e., the R-value is taken as "1" in the calculations for linear elastic procedures. The seismic demand on each structural member is calculated and compared with the related acceptance criteria for that member. The acceptance criteria depend on the state of stress and the ductility of the corresponding structural member. Demand to capacity ratios calculated, and the values defining the damage limit states are used for member assessment. Tables 7.2, 7.3, and 7.4 of TEC-2007 present the demand/capacity ratios (r) and the damage limit states for structural members of beams, columns, and shear walls, respectively. Table 7.5 of TEC-2007 states the demand/capacity ratios for strengthened filled walls, and relative story drifts with respect to defined damage limit states.

During retrofitting and rehabilitation studies conducted under the scope of ISMEP, it has been observed that TEC-2007 is likely more conservative in the assessment and retrofit requirements of existing buildings compared with other code applications. Therefore, engineering judgment and experience are key factors in implementing TEC-2007 for seismic assessment and retrofit (IPCU, 2008). The methods proposed in TEC-2007 are also criticized by Sucuoğlu (2006) in detail. Yakut et al. (2008) also mentioned that the code recommendations were found to be too conservative in the linear and nonlinear analysis.

TEC-2007 defines three limit states, i.e., four regions of performance states for structural members (columns, beams, and shear walls). These limit states are *Minimum Damage Limit* (MN), which also corresponds to IO, *Safety Limit* (GV), which also corresponds to LS, and *Collapsing Limit* (GÇ) corresponds to CP. The minimum damage limit defines the beginning of the behavior beyond elasticity. The safety limit defines the limit of the behavior beyond elasticity that the section can safely ensure the strength, and the collapsing limit defines the limit of the behavior before collapsing (Kalem, 2010). This classification is for ductile elements and does

not apply to structural members damaged in a brittle behavior. The schematic description of the damage limit states is shown in Figure 4-1 below.

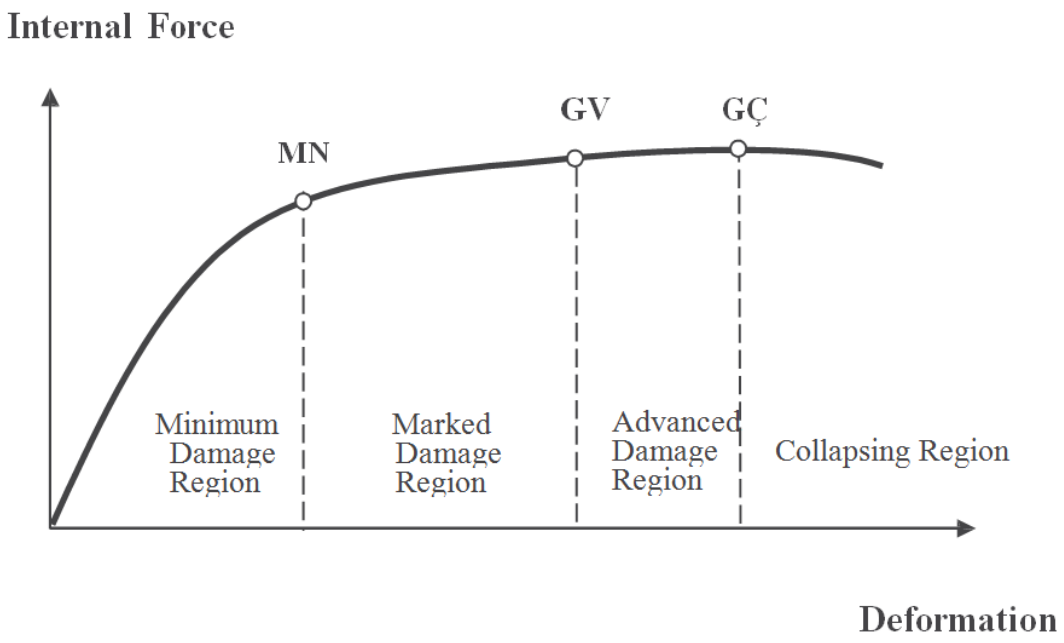

Figure 4-1. Schematic description of damage limit states (TEC-2007)

Beams, columns, and shear walls are primary structural elements for RC buildings. TEC-2007 requires all structural members to be ductile on their expected failure mode. It allows a certain percentage of members that do not satisfy the code's acceptable limits for each performance state. First, each member is checked by damage limits to determine its expected damage state. After all members are checked against damage limits, the global evaluation of the building is done. (Yakut et al., 2008)

Procedures to determine the seismic performance of the buildings and the targeted seismic performance of the buildings are also declared and described in TEC 2007. IO (Ready for Use), LS, and CP performance states are defined according to the corresponding damage states MN (IO), GV (LS), and GÇ (CP), respectively (Yakut et al., 2008). Buildings having deformations beyond CP are in a Collapse

performance state. The requirements declared for performance states of buildings in TEC-2007 are tabulated below in Table 4-1.

Table 4-1 Requirements of performance states given in TEC-2007

Performance State	**Requirements**
IO	1. No columns or shear walls are allowed beyond the IO level. 2. In any story, at most, 10 % of beams are allowed to be in LS state. 3. No beams are allowed beyond LS level.
LS	1. No columns or shear walls are allowed beyond the CP level. 2. In any story, at most, 30 % of beams are allowed to be in CP state. 3. The total contribution of the columns in the CP state to the shear force should not exceed 20 % in any story. It can be 40 % for the roof story only. 4. The shear carried by columns or shear walls yielded at both ends shall not exceed 30 % of the total story shear in any story.
CP	1. No columns or shear walls are allowed beyond the CP level, 2. In any story, at most, 20 % of beams are allowed to be in a Collapse state. 3. The shear carried by columns or shear walls yielded at both ends shall not exceed 30 % of the total story shear in any story.

TEC-2007 determines the performance states according to the type of the building and the probability of occurrence of the estimated EQ within 50 years. For school

buildings, two performance objectives, namely IO and LS, are required to be satisfied. For an EQ having a probability of 10 % of exceedance in 50 years, school buildings are required to satisfy the IO performance state. For an EQ having a probability of 2 % of exceedance in 50 years, school buildings are required to satisfy the LS performance state. For an EQ having a probability of occurrence of 2 % in 50 years, the spectrum corresponds to a probability of exceedance of 10 percent in 50 years is used by multiplying 1.5.

The database of RC school buildings used in this study consists of 321 buildings, which are assessed according to TEC-2007 by the contractors of IPCU and under the consultancy of METU professors. In the following sections, the RC school buildings chosen for the subclasses will be modeled and assessed by other methods and compared with the assessment results of TEC-2007.

An essential phenomenon about structural or EQ engineering that should be mentioned while dealing with detailed analysis/assessment methods is the "exactness." Sometimes, researchers of structural engineering define somehow an "exact" solution and compare the other so-called "approximate" solutions to that "exact" solution (Freeman et al., 2004). Dr. Dykes defined structural engineering as: *"Structural Engineering is the Art of molding materials we do not wholly understand into shapes we cannot precisely analyze, so as to withstand forces we cannot really assess, in such a way that the community at large has no reason to suspect the extent of our ignorance"* (Schmidt, 2009).

Performance-based EQ engineering recently focused on the simplified procedures that resulted in improved techniques for buildings with generally regular structural and geometrical properties. Gülkan et al. (2005) mentioned that *"the performance of performance-based methods has not been tested by nature, so the goodness of a given method is assessed against results calculated for another method or technique."* Generally, a nonlinear dynamic time history is deemed to be the baseline "exact" solution for structural engineering studies, but unfortunately, there is nothing to be called as "really or naturally exact" for structural engineers.

4.2 Modeling and Pushover Analysis of RC School Buildings

Recent developments in performance-based procedures provide valuable information for the SVA of existing structures (ATC-40, 1996; FEMA-356, 2000). Unfortunately, there are important shortcomings in these procedures in general. The first shortcoming is the requirement of nonlinear analysis capabilities that most structural engineers do not have in common.

Nonlinear structural analysis procedures are not as standard as the linear elastic analysis methods (Wight et al., 1999). Another significant disadvantage is the number of buildings that should be evaluated for seismic performance before the next EQ is extensive, and the procedures developed are time-consuming. Therefore, engineers need more straightforward methods for SVA of buildings. (Sucuoğlu and Günay, 2005)

Nonlinear Static Procedure based on pushover analysis has been introduced into EQ engineering by ATC-40 (1996) and FEMA 273 and 274 (1997) within the context of performance-based seismic evaluation and design (Aydınoğlu and Celep, 2005) and in use for more than two decades. Pushover analysis has become popular for seismic performance evaluation since it is relatively simple while considering the post-elastic behavior of the whole structure (Kalem, 2010).

Pushover analysis is a static and nonlinear procedure. The structure is pushed by increasing the magnitude of the lateral load step by step until the ultimate/failure deformation is reached. The lateral load is applied according to a predefined manner. The increments are small enough to determine plastification and the failure sequence of the structural members of the whole structure. The overall capacity of the structure is determined by considering the strength and deformation capacities of structural members (columns and beams). In fact, the standard elastic and geometric stiffness matrices for frame elements are progressively modified at each step of lateral load increment under constant gravity loads. By this step-by-step procedure, pushover analysis monitors the progressive stiffness degradation of a building while it is

loaded beyond the elastic behavior. (Hasan et al., 2002; Oğuz, 2005; Güneyisi, 2007; Kalem, 2010)

The capacity curve of the building is drawn by using the corresponding lateral load and deformation values obtained by pushover analysis. After that, the building's expected performance under an estimated EQ can be evaluated by matching the corresponding deformation demand for the related lateral load caused by that EQ

The major advantages of the pushover analysis method are its simplicity and its' potential to expose weak links in the structure; its' major disadvantage is the questionable validity of a predefined/fixed lateral load pattern determined by only one mode of vibration of the structure (Riddell et al., 2002). Therefore assessment/design procedures based on pushover analysis can be reliably used only for regular and low- to mid-rise buildings (Aydınoğlu and Celep, 2005). Considering this major drawback, pushover analysis based assessment procedures are reliably applicable for RC school buildings in Turkey.

A general sequence of steps is involved in performing the pushover analysis of the selected 36 RC school buildings in this study. Below, the steps of performed pushover analyzes are described shortly one by one. First of all, the 3-D models of the selected buildings are formed, and the gravity loads composed of dead and live loads are applied to the structural model.

Throughout the thesis study, it is observed that the as-built properties of school buildings may have deficiencies from blueprints of application projects or template designs. Using member sizes and amount of reinforcement somewhat different than application projects or template designs is not uncommon in Turkish construction practice. Using the blueprints of application or template designs for modeling and analysis purposes would not give proper solutions in Turkish construction practice.

In order to overcome the inconsistencies between the blueprints and as-built construction, as-built properties are used for modeling purposes for all of the selected RC school buildings. SAP2000 v.14.1 software is used for modeling and analysis

purposes. 3-D (three dimensional) models of all selected 36 RC school buildings were prepared using SAP2000 software.

All dimensions of the structural members are modeled according to site surveys. Columns and beams are modeled as line elements (flexural members). RC shear walls are modeled by using wide column analogy. Rigid diaphragms are employed for modeling the story floors. The reinforcement details of the structural members are also modeled according to the destructive and non-destructive in-situ test results. Infill walls are not modeled since there are no continuous infill walls, i.e., there are large windows or door openings on the walls, and the effect of the non-continuous infill walls for seismic vulnerability is negligible for RC school buildings.

As-built material properties determined by field investigations and experimental works were considered for 3-D modeling of the selected buildings. The concrete compressive strength of each RC school building was determined using both destructive (concrete core sampling and uniaxial compression laboratory testing) and non-destructive (Schmidt hammer testing) methods.

Dead loads were calculated and applied to the model according to the load analysis. Live loads were calculated and used according to TEC-2007, TS-498 (1987), and TS-500 (2000). The buildings were then analyzed to calculate elastic forces and displacements under gravity loads only at first. The pushover analysis phase starts after the analysis under gravity loads. It is observed that all RC school buildings are safe under gravity loads.

The masses of each story were applied/lumped to the mass centers of each floor, and all seismic forces were applied to the mass centers. The weights of the stories are in compliance with the story masses, and no eccentricity was applied. Viscous damping values are taken as 5 % for all selected buildings.

Cracked section rigidities were used for defining the stiffness of beams and columns. The cracked section stiffness of beams is defined as 40 % of uncracked section stiffness, and cracked section stiffness of columns and shear walls are defined as 65

% of uncracked section stiffness. The rigidity of the cracked sections is modeled by multiplying the moment of inertias by 0.40 for beams and by 0.65 for columns.

The material fibers furthest from the neutral axis starts initial yielding first. Under the effect of the increasing moment, plasticity propagates through the neutral axis and along the length of the member until a fully-developed plastic hinge is formed. The spread of the plasticity causes post-elastic degradation of flexural stiffness of a frame member, and the flexural stiffness of the member is exhausted when increasing moment, continues as plasticity spreads through the section depth and along the member length to form a fully-developed plastic hinge, at which point the flexural stiffness of the member section is exhausted (Hasan et al., 2002).

SAP2000 provides default and user-defined hinge properties options to model the nonlinear behavior of structural components. For beams, default M-M hinges, and for columns default, P-M_2-M_3 hinges are defined at the member ends for all selected 36 buildings. Built-in default hinge properties of SAP2000 software are typically based on ATC-40 (1996) and FEMA-356 (2000) criteria. Half of the cross-section depth is suggested as the plastic hinge length by ATC-40 (1996). The modeled default moment-rotation relationship can be seen below in Figure 4-2.

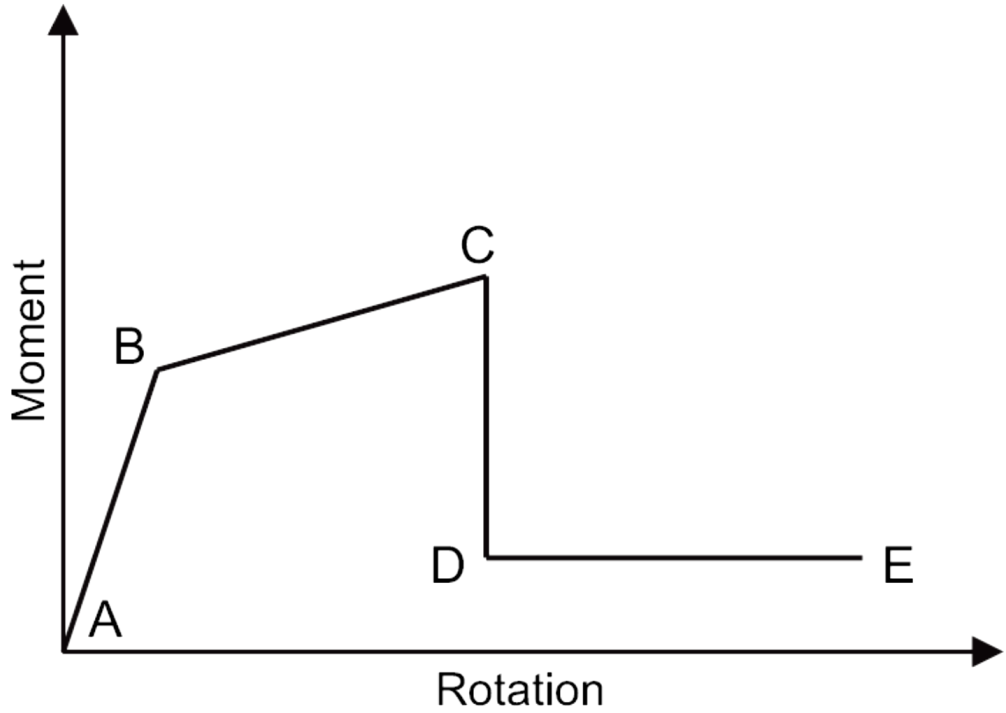

Figure 4-2. Modeled default moment-rotation relationship (Yakut, 2008)

Total equivalent seismic loads acting on the RC school buildings laterally (base shear forces) were also calculated in compliance with TEC-2007 in the EQ direction considered. The structural behavior factor, R, was taken as "1," as described by TEC-2007. The total base shear due to EQ was distributed to the stories as an inverted triangle in compliance with TEC-2007.

The possibility of shear failure should be controlled before pushover analysis with the full seismic loads applied. It is observed that there is no shear failure for the selected buildings. It is followed and checked by screening the hinge propagation, i.e., damage sequence of the analyzed RC school buildings using SAP-2000 software.

After that lateral seismic loads are extracted from the model, and the predefined patterns for the pushover analysis were applied. A unit force of 1 N is applied to the mass center of the first floor, and loads proportional to the triangular distribution calculated are applied to the mass centers of upper floors. Lateral loads proportional

to the product of mass and the height of the story are used at each story level under consideration. The obtained load pattern for each related building is the so-called predefined load pattern of the pushover analysis. A schematic description of pushover analysis and the inverted triangle load distribution is shown in Figure 4-3 (Ay, 2006).

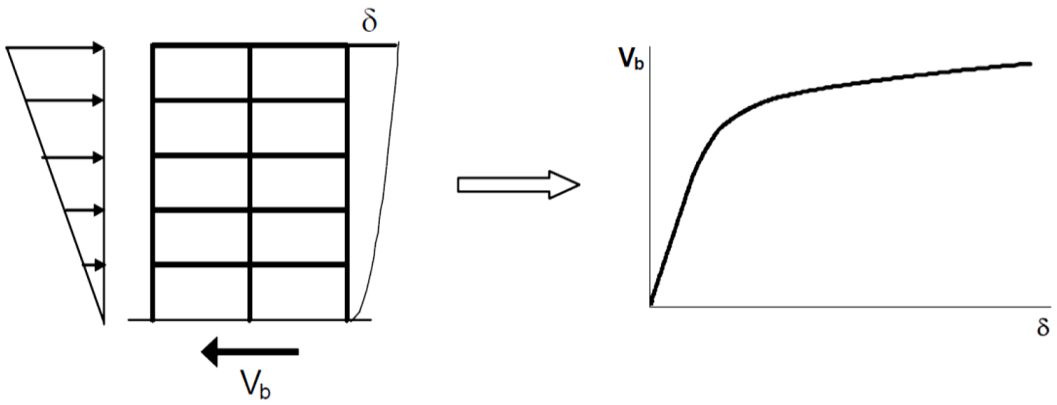

Figure 4-3. Schematic description of pushover analysis (Ay, 2006)

Deformations of the structures are increasingly considered more appropriate measures of seismic vulnerability or seismic performance than forces (Bommer et al., 2000). Specified deformation states at corresponding load levels are often accepted to measure building performance (Hasan et al., 2002). Therefore, the deformation-controlled pushover analysis procedure is preferred in general and chosen in this study.

Lateral loads are then increased step by step until a member or some members yield under the combined effects of gravity and lateral loads. In this study, the roof displacement is chosen to identify the damage limit states of the buildings since it is widely used. Base shear values and the corresponding roof displacements are recorded at each step. The structural model is modified to consider the reduced stiffness of the yielded members by the software. The lateral load increments and the corresponding roof displacements are recorded until the total collapse of the building. The building becomes unstable, i.e., a mechanism at the total collapse or the failure level. The corresponding displacement at the point where the structure

becomes a mechanism is called the ultimate displacement. The base shear force at the ultimate displacement point is called the ultimate base shear force.

The base shear values and the corresponding roof displacements are plotted to obtain the pushover curve of the entire structure. A representative obtained pushover curve of the buildings is shown in Figure 4-4 (Kalem, 2010).

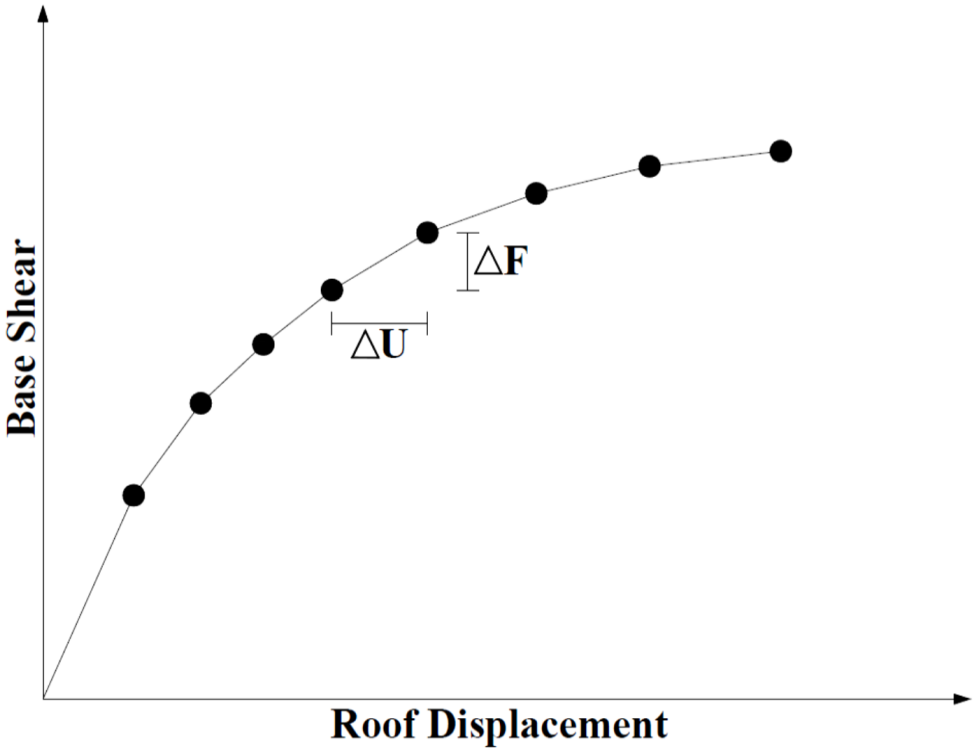

Figure 4-4. Obtained Pushover Curve of Structure (Kalem, 2010)

Push-over analysis and pushover curves provide valuable information for the SVA and rehabilitation of existing RC buildings. The increasing roof displacement that buildings experience is associated with EQs having increasing intensities of ground motion, and the displacement is directly associated with the building's performance level (Hasan et al., 2002). Pushover-based seismic evaluation methods might be considered a breakthrough in EQ engineering since engineers can directly calculate the nonlinear seismic demand and evaluate the response of the structure by using pushover curves (Fajfar, 2002).

3-D models of each selected 36 buildings were developed in SAP2000 and then subjected to pushover analysis together with the modal analysis in two principal/orthogonal directions to obtain the base shear versus roof displacement curves as described above. After that, these obtained capacity curves were bilinearized to determine the yield base shear force (V_y), yield roof drift ratio (δ_y), ultimate base shear force (V_u), and the ultimate drift ratio of (δ_u) of the selected buildings for both directions. Then modal properties that conform to the initial linear part of the bilinearized pushover curves were determined consistently.

4.3 Bilinearization and Obtaining Equivalent SDOF Parameters

Using 3-D models for seismic analysis is complicated and time-consuming. Seismic behaviors of Multi-Degree-of-Freedom (MDOF) systems can be approximated with certain accuracy by Equivalent Single Degree of Freedom (ESDOF) systems whose properties are computed by conducting the push-over analysis (Kuramoto et al., 2000, and Decanini et al., 2001). The primary requirement for certain accuracy of conversion is the governance of the first mode for EQ response. Other requirements for accuracy of determining EQ response using ESDOF conversion are; there should not be torsion, push-over curves should be suitable for bilinear conversion, and strength degradation should not be severe (Kuramoto et al., 2000, and Decanini et al., 2001). The selected RC school buildings in this study satisfy the listed requirements. RC school buildings are ideal for determining the EQ response by using SDOF models.

It should be mentioned that there is no standard method for bilinearization of pushover curves. In this study, pushover curves of each building approximated by a bilinear curve using bilinearization of pushover curves are engineering judgment and equal-area criterion as recommended in ATC-40 (1996) and FEMA 356 (FEMA, 2000). The procedure is also described by Ay (2006) in detail.

The areas under the actual pushover curve and the approximated bilinear curve should be the same, and this phenomenon is stated as the equal-area rule in ATC-40 (1996) and FEMA 356 (FEMA, 2000). There is no specific point on the pushover curve to describe the yielding of the building. The yield point on the pushover curves was defined as the point where the building starts to soften definitely. Variations in the initial stiffness of the bilinear curve would lead to considerable variations in V_y and δ_y. Therefore, the first linear part of the bilinear curve should coincide with the actual pushover curve at the 0.60 Vy point to overcome the variations.

Line segments of the idealized bilinear curves should be located using an iterative procedure which approximately balances the areas above and below the curve. The effective lateral stiffness, K_e, is taken as the secant stiffness calculated at a base shear force equal to 60% of the structure's effective yield strength. The post-elastic stiffness ratio, α, is determined by a line segment that starts from the V_y and δ_y points, passes through the actual curve, and ends at the calculated target displacement. (Özün, 2007)

Schematic descriptions of the bilinearization of pushover curves can be seen in Figure 4-5 below.

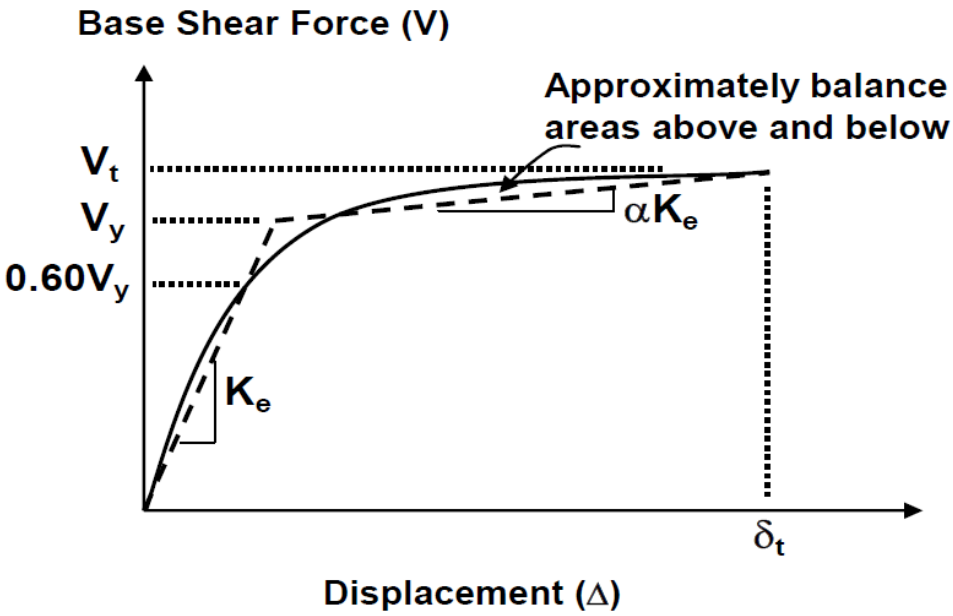

Figure 4-5. Idealized Bilinear Force-Displacement Curve (FEMA 356, 2000)

After bilinearization of all pushover curves is completed, all curves are converted to acceleration displacement response spectra (ADRS) format, i.e., the pushover curve of the MDOF model is plotted in the S_a vs. S_d domain of the equivalent SDOF system by using the corresponding modal properties of each RC school building. The following equations were used to determine the spectral acceleration and spectral displacement values.

$$S_a = \frac{V}{\alpha_1 W} \tag{4-1}$$

$$S_d = \frac{\Delta_{roof}}{PF_1 x \phi_{roof,1}} \tag{4-2}$$

According to the well-known equal displacement rule, spectral displacement of an inelastic SDOF system and that of the corresponding elastic system are assumed practically equal, provided that the effective-initial period of the SDOF system is longer than the *characteristic period* of the EQ For periods shorter than the characteristic period, elastic spectral displacement is appropriately amplified. (Aydınoğlu and Celep, 2005)

Instead of analytical models, each of the 36 buildings chosen from the database is represented by an equivalent Single-Degree-of-Freedom (SDOF) model with structural parameters described below. After converting all actual pushover curves to ADRS format, the parameters that define the SDOF system are obtained through the equations below.

$$M^* = \alpha_1 x M \tag{4-3}$$

$$K^* = \frac{4\Pi^2}{{T_e}^2} M^* \tag{4-4}$$

$$F_y = V_y = S_{a,y} W^* \tag{4-5}$$

Where;

The pushover curves obtained from nonlinear static analysis in two orthogonal directions using the software SAP2000 are used to obtain the SDOF structural parameters. Hence, there exist 72 (36x2) actual and bilinear pushover curves, and the corresponding SDOF parameters (with 5 % damping ratio) are obtained for each building and each orthogonal (X and Y) direction.

Then, the capacity-related properties of RC school buildings are determined with respect to story numbers and the EQ direction considered. As mentioned earlier, the longer directions of the buildings were accepted to be the "X" direction, and the orthogonal and shorter directions were taken to be the "Y" direction. EQ lateral loads are applied to these two orthogonal directions.

All structural parameters for the selected 36 buildings with respect to orthogonal directions are given in the tabular form in Appendix C. The equivalent SDOF system parameters are also provided for all selected buildings in Appendix D.

4.4 The Structural Parameters of School Buildings

In order to obtain the capacity-related properties of RC frame buildings in Turkey, Yakut (2008) performed a comprehensive study in which 33 sample buildings were selected to represent a typical subset of a comprehensive database consisting of nearly 500 buildings. The statistical properties of all capacity parameters needed to describe the capacity curve were given in Table 1-8 for each number of stories (Yakut, 2008). The capacity-related properties of RC school buildings determined in this study are listed below (Table 4-2, Table 4-3, and Table 4-4) with parameters of Yakut (2008) in the same tables with respect to the number of stories. The parameters and the deriving formulas were described in Section 2.1, but the parameters are listed below again. Remembering that in Yakut's (2008) study, building databases were mostly composed of residential buildings. By listing the capacity curve parameters on the same table, we have the opportunity to compare the RC school buildings with the RC residential buildings in Turkey.

$$Sa_y = \frac{C_s \gamma}{\alpha_1} \tag{4-6}$$

$$Sd_y = \frac{Sa_y T_e^2 g}{4\pi^2} \tag{4-7}$$

$$Sa_u = \lambda Sa_y \tag{4-8}$$

$$Sd_u = \lambda \mu Sd_y \tag{4-9}$$

The value of C_s is taken as 0.10 for all cases in order to compare with the results of Yakut (2008). The value of C_s only affects the γ, and these values did not affect previous studies.

Table 4-2 Statistics of Capacity Curve Parameters for 3 Story RC Buildings

Parameter		3 Story (X)	3 Story (Y)	Yakut (2008)
Sdy (cm)	Mean	2,51	2,12	1,06
	Std.Dev.	2,29	1,37	0,29
Say (g)	Mean	0,17	0,20	0,18
	Std.Dev.	0,08	0,15	0,07
Sdu (cm)	Mean	3,36	5,56	10,65
	Std.Dev.	2,79	5,22	2,65
Sau (g)	Mean	0,23	0,29	0,21
	Std.Dev.	0,12	0,25	0,08
Cs	Mean	0,10	0,10	0,10
	Std.Dev.	0,00	0,00	0,00
Te	Mean	0,80	0,67	0,34
	Std.Dev.	0,61	0,44	0,11
PF	Mean	1,31	1,26	1,24
	Std.Dev.	0,06	0,12	0,04
α	Mean	0,83	0,77	0,88
	Std.Dev.	0,07	0,08	0,06
γ	Mean	1,26	1,47	1,58
	Std.Dev.	0,63	1,23	0,52
λ	Mean	1,33	1,45	1,15
	Std.Dev.	0,14	0,32	0,12
μ	Mean	2,84	3,39	9,26
	Std.Dev.	0,66	1,06	3,12

Table 4-3 Statistics of Capacity Curve Parameters for 4 Story RC Buildings

Parameter		4 Story (X)	4 Story (Y)	Yakut (2008)
Sdy (cm)	Mean	3,24	3,28	1,67
	Std.Dev.	3,34	1,76	0,71
Say (g)	Mean	0,11	0,14	0,16
	Std.Dev.	0,08	0,12	0,04
Sdu (cm)	Mean	7,55	9,06	12,84
	Std.Dev.	4,09	4,23	3,61
Sau (g)	Mean	0,13	0,17	0,18
	Std.Dev.	0,10	0,16	0,05
Cs	Mean	0,10	0,10	0,10
	Std.Dev.	0,00	0,00	0,00
Te	Mean	1,21	1,16	0,47
	Std.Dev.	0,62	0,49	0,11
PF	Mean	1,31	1,22	1,28
	Std.Dev.	0,08	0,19	0,02
α	Mean	0,80	0,72	0,83
	Std.Dev.	0,07	0,10	0,05
γ	Mean	0,84	0,96	1,30
	Std.Dev.	0,59	0,93	0,39
λ	Mean	1,25	1,27	1,14
	Std.Dev.	0,19	0,11	0,03
μ	Mean	3,58	3,42	7,67
	Std.Dev.	1,81	1,79	3,01

Table 4-4 Statistics of Capacity Curve Parameters for 5 Story RC Buildings

Parameter		5 Story (X)	5 Story (Y)	Yakut (2008)
Sdy (cm)	Mean	4,62	4,18	1,52
	Std.Dev.	3,98	2,82	0,35
Say (g)	Mean	0,04	0,07	0,12
	Std.Dev.	0,02	0,07	0,04
Sdu (cm)	Mean	14,01	12,65	14,06
	Std.Dev.	11,41	6,54	5,40
Sau (g)	Mean	0,04	0,09	0,14
	Std.Dev.	0,03	0,09	0,04
Cs	Mean	0,10	0,10	0,10
	Std.Dev.	0,00	0,00	0,00
Te	Mean	2,40	1,85	0,55
	Std.Dev.	1,27	1,02	0,14
PF	Mean	1,46	1,40	1,29
	Std.Dev.	0,22	0,21	0,01
α	Mean	0,76	0,72	0,83
	Std.Dev.	0,06	0,08	0,03
γ	Mean	0,28	0,50	1,02
	Std.Dev.	0,20	0,43	0,32
λ	Mean	1,19	1,23	1,16
	Std.Dev.	0,16	0,11	0,06
μ	Mean	3,79	3,83	7,86
	Std.Dev.	1,67	2,43	2,22

For all 3, 4, and 5 story RC school buildings, yield spectral displacements for X and Y directions are greater than the yield spectral displacements of RC residential buildings calculated by Yakut (2008). But the situation is vice versa for ultimate spectral displacements. It also means that, in Turkey, the ductilities of 3, 4, and 5

story RC school buildings are smaller than the ductilities of residential buildings in general. The yield and ultimate spectral accelerations of 4 and 5 story RC school buildings are smaller than the corresponding values of 4 and 5 story residential buildings. The yield and ultimate spectral accelerations of 3 story RC school buildings are smaller than the corresponding values of residential buildings for X-direction but vice versa for the Y-direction.

The yield and ultimate spectral displacement values of RC school buildings increase gradually with increasing number of stories, but the situation is vice versa for yield and ultimate spectral accelerations. Ultimate spectral displacements for RC school buildings are less than 1 percent of the total building heights for all cases. Effective periods of RC school buildings also increase with increasing number of stories. The ductility values also increase in small amounts with the increasing number of stories, and the average value of ductility is 3.47 for RC school buildings.

The average yield spectral displacements for RC school buildings are found to be 0.23, 0.25, and 0.27 percent of the total height for 3, 4, and 5 story buildings, respectively. The ultimate displacements are found to be 1.11, 2.06, and 2.34 percent of the total height for 3, 4, and 5 story RC school buildings, respectively.

The effective periods of 3, 4, and 5 story RC school buildings are greater than the effective periods of corresponding residential buildings. The reason for high effective periods is using the cracked section rigidities of structural members for analysis. The participation factor (PF) and modal mass factor (α) of the RC school buildings do not show significant variation with the number of stories.

The yield over-strength ratios (γ) were found to decrease significantly with increasing number of stories. The average yield over strength ratios for 5-story RC school buildings is less than 1.0. In the study of Yakut (2004), it is declared that “RC frame buildings that have yield over-strength ratio of less than 1.65 would perform poorly against a devastating EQ in Turkey”. Unfortunately, the average yield over strength ratios are less than 1.65 for 3, 4, and 5 story RC school buildings.

Since all analyses are done using as-built properties of RC school buildings, and the variation of the structural properties are considerable, standard deviation values are high for structural parameters, as is excepted.

The damage sequence of structural members can be observed by pushover analysis. It is observed that for RC school buildings, the damage sequence of structural members "generally" starts by yielding of the beam ends at the lower stories and then yielding propagates to beams of upper stories. The total collapse of the buildings occurs typically by the yielding of the column bases of the ground stories. The damage sequence of RC school buildings is, in fact, the accepted sequence of ductile beam mechanism. That is because RC school buildings are generally regular in plan and do not have vertical irregularity. Unfortunately, the ductilities and the ultimate displacement capacities are not sufficient. The main reason for the low ductilities is the unconfined yielding regions, i.e., insufficient/small rotation capacities.

There are few buildings that the damage sequence starts with the yielding of column bases or both column and beam ends. This is not a desirable sequence.

For all RC school buildings, yielding starts at the lowest story and then propagates to the upper stories.

4.5 Determination of Performance Limit States

The point at which the system is no longer capable of satisfying a performance level is defined as the limit state from the engineering perspective (Ay, 2006). The limit states of the structures are, by definition, directly related to the damage levels of the structure. There are generally four damage levels defined in the negligible, light, moderate, and heavy literature. The immediate occupancy (IO) performance level corresponds to negligible and light damage states. The life safety performance level corresponds to the moderate damage state, and the collapse prevention performance level corresponds to the heavy damage state.

One of the significant steps of F.C. generation is the determination of performance levels, and this is directly related to the realistic and comprehensive limit state determination. These indicators are the key factors and directly affect the resulting F.C.s. (Erberik and Elnashai, 2004; Ay, 2006)

There are two main categories of response parameters, namely local and global. The main response parameters will not be discussed within the scope of this study. The limit states for the RC frame structures considered within the scope of this study are defined in terms of roof drift. The damage limit states in fragility studies may be defined as inter-story drift ratio (maximum lateral displacement between two consecutive stories normalized by the story height), global drift ratio (maximum roof drift normalized by the building height), roof drift, story shear force, etc. (Akkar et al., 2005). In this study, roof drift is chosen for the definition of the damage limit states. It is believed that global roof drift is the simplest and the most frequently used (maybe most meaningful) response parameter for fragility analysis.

FEMA 356 (ASCE 2000) defined the three limit states qualitatively for structural members as tabulated in Table 4-5 below.

Table 4-5 Performance state definitions (FEMA 356, 2000; Ay, 2006)

Performance State	**Definition**
Immediate Occupancy (IO)	-Limited or no damage has occurred after EQ -Strength and stiffness of the structural system and members have remained almost the same as the prior state.
Life Safety (LS)	-Although significant damage has occurred, this has not resulted in the collapse. -The repair of the structure is essential but not always feasible.
Collapse Prevention (CP)	-The structure is on the limits of instability, partial or total collapse. -Strength and stiffness degradation is high; permanent deficiencies have occurred. -Reuse of the structure is not safe.

TEC-2007 defines three limit states, i.e., four regions of performance states for structural members (columns, beams, and shear walls). These limit states are *Minimum Damage Limit* (MN), *Safety Limit* (GV), and *Collapsing Limit* (GÇ). Minimum damage limit defines the beginning of the behavior beyond elasticity. The safety limit defines the limit of the behavior beyond elasticity that the section can safely ensure the strength, and the collapsing limit defines the limit of the behavior before collapsing (Kalem, 2010). This classification is for ductile elements and does not apply to structural members damaged in a brittle behavior. The schematic description of the damage limit states is shown in Figure 4-1 in Section 4.1 above.

The assessment procedures of the TEC-2007 were summarized in Section 4.1, and the requirements given for performance states of buildings in TEC-2007 are tabulated in Table 4-1 of Section 4.1 above. Immediate Occupancy (IO) (Ready for Use), Life Safety (LS), and Collapse Prevention (CP) (Collapse prevention) performance states are defined according to the corresponding damage states MN,

GV, and GÇ, respectively (Yakut et al., 2008). Buildings having deformations beyond CP are in a Collapse performance state.

For school buildings, TEC-2007 requires two different performance states for two different earthquake load levels. School buildings should satisfy the IO performance state for an earthquake loading having a probability of exceedance of 10 % in 50 years lifetime and LS for an earthquake ground motion having a probability of exceedance of 2 % in 50 years lifetime.

As mentioned earlier, the database of RC school buildings used in this study consists of 321 buildings, which are assessed according to TEC-2007 by the contractors of IPCU and under the consultancy of professors of METU. But the 321 RC school buildings were evaluated only for whether they satisfy the requirements of TEC-2007 or not, i.e., the damage and the performance levels of buildings are not determined individually.

In order to be consistent with TEC-2007, three limit states, which are immediate occupancy, life safety, and collapse prevention, are considered for RC school buildings in this study. The limit states of each RC school building are determined individually for precise analysis and generation of precise F.C.s. The limit states are determined by considering the yield (Δ_y) and the ultimate (Δ_u) spectral roof displacements of related equivalent SDOF model of each RC school building. As described by Akkar et al. (2005) and as it is accepted in many other studies, the yield displacement point and the ultimate spectral roof displacement point are taken as the limits for IO and CP performance states, respectively. The buildings with target displacements below the IO limit state are in the IO performance state. The limits for LS performance state are accepted as 75 % (three quarters) of the ultimate roof drift. Performance limits for IO, LS, and CP are denoted as ΔIO, ΔLS, and ΔCP, respectively. ΔCP values are also the ultimate displacements for each building.

Idealized pushover curves for equivalent SDOF systems of each building are used to determine the limit state values. The limit state values in terms of spectral roof drifts are given for each RC school building with respect to EQ direction in Appendix D.

The schematic description of the determination of performance limits for performance states is shown in Figure 4-6 below. The ultimate roof drift is taken as the drift value when the structure became a mechanism. A sudden drop in the load capacity is observed after the system becomes a mechanism. There were no numeric problems that caused the interruption of the analysis. The average values of performance limits for 3, 4, and 5 story RC school buildings are given below in Table 4-6 and the limits are shown in terms of drift raions in Table 4.7.

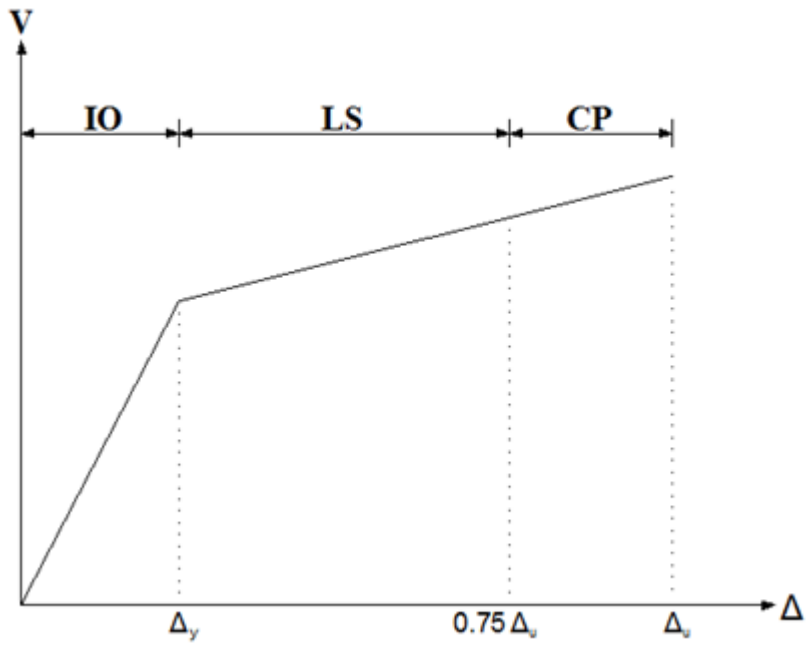

Figure 4-6. Assumed performance limits for performance states (Kalem, 2010)

Table 4-6 The average performance limits for RC school buildings

Number of Stories	ΔIO (cm)		ΔLS (cm)		ΔCP (cm)	
	X	Y	X	Y	X	Y
3	2,82	2.09	6.34	5.96	8.45	7.95
4	3.70	3.61	8.93	9.02	11.90	12.02
5	6.35	5.65.	14.51	12.75	19.35	17.00

Table 4.7 The average drift ratio limits for RC school buildings

Number of Stories	ΔIO (%)		ΔLS (%)		ΔCP (%)	
	X	Y	X	Y	X	Y
3	0.28	0.21	0.63	0.59	0.84	0.79
4	0.29	0.28	0.69	0.69	0.92	0.93
5	0.43	0.38.	0.97	0.84	1.30	1.14

4.6 Assessment Using Target Displacements of Displacement Coefficient Method

There is a continuous demand for using simplified procedures to estimate the inelastic displacement demand of structures since nonlinear response history analysis is time-consuming and analytically complex. The simplified procedures generally use equivalent SDOF representations of MDOF models of structures. One of the most common procedures is the Displacement Coefficient Method (DCM) described in FEMA 356 (FEMA, 2000) and FEMA 440 (FEMA, 2005).

DCM basically depends on the approximate determination of displacement demand under the estimated EQ, i.e., target displacement from the elastic displacement by using modification coefficients. Then the target displacement may be compared with the capacity curve of the structure to determine the seismic performance. In order to apply the DCM procedure, the structure should have a bilinear capacity curve, so the capacity curves of the buildings are needed to be idealized as described in Section 4.3. The effective fundamental periods (T_e) of the structures and the target displacements (δ_t) are determined first by using the idealized bilinear capacity curves through the equations described below.

$$T_e = \sqrt{\frac{K_i}{K_e}} T_i \tag{4-10}$$

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \tag{4-11}$$

The coefficient (modification number) C_0 relates the top roof displacement of the structure to the displacement of an equivalent SDOF system, i.e., the coefficient that converts the SDOF spectral displacement to MDOF (elastic) roof displacement.

The corresponding values of the coefficient C_0 with respect to the number of stories are represented in Table 4-8.

Table 4-8 Values for Coefficient C_0 (FEMA 356, 2000)

Number of Stories	C_0
1	1.0
2	1.2
3	1.3
5	1.4
10+	1.5

The coefficient C_1 is used to modify the elastic displacement to the corresponding inelastic displacement, i.e., the expected inelastic displacement divided by elastic displacement.

$$C_1 = 1 + \frac{R-1}{aT_e^2} \tag{4-12}$$

$$C_1 = \begin{cases} C_1(T = 0.2s), T < 0.2s \\ 1.0, T > 1.0s \end{cases} \quad (4\text{-}13)$$

Where “a” in this equation is 130, 90, and 60 for soil sites B, C, and D, respectively.

$$R = \frac{S_a}{V_y/W} C_m \quad (4\text{-}14)$$

Where C_m is the effective mass factor, and the values for C_m with respect to the number of stories and the structural system are presented in Table 4-9.

Table 4-9 Values for Effective Mass Factor (C_m) (FEMA 356, 2000)

Number of Stories	Concrete Frame	Concrete Shear Wall
1-2	1.0	1.0
≥3	0.9	0.8

The coefficient C_2 incorporates the effects of pinched hysteretic shape, stiffness degradation, and strength deterioration, i.e., depends on the structural system and varies with the hysteretic behavior.

$$C_2 = 1 + \frac{1}{800}\left(\frac{R-1}{T}\right)^2 \quad (4\text{-}15)$$

$$C_2 = \begin{cases} C_2(T = 0.2s), T < 0.2s \\ 1.0, T > 0.7s \end{cases} \quad (4\text{-}16)$$

The coefficient C_3 reflects the increase in the displacements due to dynamic P-Δ effects.

$$C_3 = \begin{cases} 1.0, \alpha \geq 0 \\ 1 + \frac{\alpha(R-1)^{3/2}}{T_e} \end{cases} \tag{4-17}$$

The target (demand) roof displacement (Δ_d) of each RC school building in both orthogonal directions under the estimated EQs of TEC-2007 were computed using the DCM summarized above. Then the target displacements are compared with the performance limit states derived in the previous section to determine the expected seismic performance of each RC school building. The seismic performance levels of each RC school building for both orthogonal directions and the assessments of TEC-2007 are presented in Table 4-10 and Table 4-11 below.

Table 4-10 The comparison of DCM and TEC-2007 10% EQ

ID	DCM Perf.-X	DCM Perf.-Y	TEC-2007 (10 %) Perf.
1	Collapse	LS	Collapse
2	Collapse	Collapse	IO
3	Collapse	Collapse	Collapse
4	Collapse	Collapse	IO
5	Collapse	Collapse	Collapse
6	Collapse	LS	Collapse
7	Collapse	Collapse	Collapse
8	Collapse	LS	Collapse
9	CP	CP	Collapse
10	LS	LS	Collapse
11	LS	LS	IO
12	LS	LS	Collapse
13	Collapse	CP	Collapse
14	CP	Collapse	Collapse
15	Collapse	Collapse	Collapse
16	Collapse	CP	Collapse
17	CP	LS	Collapse
18	Collapse	Collapse	Collapse
19	Collapse	Collapse	IO
20	Collapse	Collapse	IO
21	Collapse	CP	Collapse
22	Collapse	Collapse	IO
23	Collapse	Collapse	Collapse
24	Collapse	Collapse	Collapse
25	Collapse	Collapse	Collapse

Table 4-10 Continued

ID	DCM Perf.-X	DCM Perf.-Y	TEC-2007 (10 %) Perf.
26	Collapse	CP	Collapse
27	CP	CP	Collapse
28	Collapse	Collapse	Collapse
29	Collapse	Collapse	IO
30	Collapse	Collapse	LS
31	Collapse	Collapse	IO
32	Collapse	Collapse	Collapse
33	Collapse	Collapse	IO
34	Collapse	Collapse	Collapse
35	Collapse	IO	LS
36	Collapse	Collapse	LS

Table 4-11 The comparison of DCM and TEC-2007 for 2% EQ

ID	DCM Perf.-X	DCM Perf.-Y	TEC-2007 LS (2%)
1	Collapse	Collapse	Collapse
2	Collapse	Collapse	LS
3	Collapse	Collapse	Collapse
4	Collapse	Collapse	LS
5	Collapse	Collapse	Collapse
6	Collapse	Collapse	Collapse
7	Collapse	Collapse	Collapse
8	Collapse	LS	Collapse
9	Collapse	Collapse	Collapse
10	CP	LS	Collapse
11	CP	CP	LS

Table 4-11 Continued

ID	DCM Perf.-X	DCM Perf.-Y	TEC-2007 LS (2%)
12	LS	LS	Collapse
13	Collapse	Collapse	Collapse
14	Collapse	Collapse	Collapse
15	Collapse	Collapse	Collapse
16	Collapse	Collapse	Collapse
17	Collapse	LS	Collapse
18	Collapse	Collapse	Collapse
19	Collapse	Collapse	LS
20	Collapse	Collapse	LS
21	Collapse	Collapse	Collapse
22	Collapse	Collapse	LS
23	Collapse	Collapse	Collapse
24	Collapse	Collapse	Collapse
25	Collapse	Collapse	Collapse
26	Collapse	Collapse	Collapse
27	Collapse	Collapse	Collapse
28	Collapse	Collapse	Collapse
29	Collapse	Collapse	LS
30	Collapse	Collapse	LS
31	Collapse	Collapse	LS
32	Collapse	Collapse	Collapse
33	Collapse	Collapse	LS
34	Collapse	Collapse	Collapse
35	Collapse	LS	LS
36	Collapse	Collapse	LS

According to DCM assessment results, the RC school buildings perform better in Y (short) directions since the number of continuous frames in Y-directions is more than in X-direction for RC school buildings. 11 (30,5 %) of the buildings are in collapse performance state for both orthogonal directions according to DCM and also according to TEC-2007 assessment results for an EQ of 10 % occurrence probability in 50 years, and 20 (55,5 %) of them are in collapse performance state for an EQ of 2 % exceedance probability in 50 years. For other buildings, there is not a meaningful correlation between the DCM and TEC-2007 assessment results.

4.7 Assessment by ASCE 41 Analysis

One of the well-known and most commonly used detailed assessment procedures is FEMA 356 (FEMA, 2000) or ASCE 41 (ASCE, 2007). The ASCE/SEI 41-06 named as Seismic Rehabilitation of Existing Buildings (ASCE, 2007), commonly known as ASCE 41 standard, was developed from the FEMA 356 named as Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA, 2000).

The procedures of ASCE 41 (ASCE, 2007) are not explained in detail since it is not in the scope of this study. But it is worth to mention that the procedures used by ASCE 41 (ASCE, 2007) and TEC-2007 are basically similar in concept. Unreduced building demands are used, and individual members are assessed using member-specific assessment criteria by ASCE-41 (ASCE, 2007) linear procedures. There is an important difference between the two methods, which is the acceptance value “m” of ASCE-41 (ASCE; 2007) is compared by the demand-to-capacity ratio of a structural member, while the acceptance value “r” of TEC-2007 is derived for the reserve capacity ratio. Both “m” and “r” values represent the demands to capacity ratios of the structural members. The acceptance criteria of the structural members depending on the acceptance value “m,” are listed in Chapter 6 of ASCE-41 (ASCE, 2007). IPCU prefers ASCE-41 (ASCE, 2007) since it is more intuitive than TEC2007 and not as dependent upon initial assumptions (IPCU, 2008).

The nonlinear static procedure of ASCE-41 (ASCE, 2007) uses the plastic rotation values (in radians) of structural members in order to compare with the limit values of performance levels and assess them, respectively. The outcome of comparisons of demand values with the acceptance criteria or performance level criteria depends on whether the structural member satisfies the requirements of the corresponding performance level or not (Yakut and Erduran, 2005).

The major drawback for ASCE-41 is that although there are procedures to assess the structural members individually, no guidelines are provided on how to evaluate the performance level of the building as a whole; i.e., the final decision remains to the judgment of the structural engineer (Yakut and Erduran, 2005).

The nonlinear static procedure of ASCE-41 is applied in both principal directions to all 36 selected RC school buildings within the database. The plastic rotation values of the structural members are determined and compared with the corresponding values of performance level criteria. In order to assess the performance of the whole building, the percentages of the structural members satisfying each performance level are determined. After that, the assessments of the entire buildings are carried on by using the same criteria of TEC-2007, which is described in Table 4-1 above.

The analyzes are performed for the EQ having a probability of occurrence of 10 % in 50 years. The seismic performance levels of each RC school building for both orthogonal directions and the assessments of TEC-2007 are presented in Table 4-12 below.

Table 4-12 The comparison of ASCE-41 and TEC-2007 for 10% EQ

ID	ASCE-41 Perf.-X	ASCE-41 Perf.-Y	TEC-2007 (10 %) Perf.
1	Collapse	Collapse	Collapse
2	Collapse	Collapse	IO
3	IO	Collapse	Collapse
4	Collapse	Collapse	LS
5	Collapse	Collapse	Collapse
6	Collapse	Collapse	Collapse
7	IO	IO	Collapse
8	Collapse	IO	Collapse
9	Collapse	Collapse	Collapse
10	Collapse	Collapse	Collapse
11	Collapse	Collapse	IO
12	Collapse	Collapse	Collapse
13	Collapse	IO	Collapse
14	Collapse	LS	Collapse
15	Collapse	CP	Collapse
16	Collapse	Collapse	Collapse
17	IO	IO	Collapse
18	Collapse	LS	Collapse
19	Collapse	Collapse	IO
20	Collapse	Collapse	IO
21	Collapse	LS	Collapse
22	Collapse	Collapse	IO
23	Collapse	IO	Collapse
24	Collapse	Collapse	Collapse
25	Collapse	IO	Collapse

Table 4-12 Continued

ID	ASCE-41 Perf.-X	ASCE-41 Perf.-Y	TEC-2007 (10 %) Perf.
26	Collapse	LS	Collapse
27	Collapse	CP	Collapse
28	Collapse	Collapse	Collapse
29	Collapse	Collapse	IO
30	Collapse	Collapse	LS
31	Collapse	Collapse	IO
32	IO	IO	Collapse
33	Collapse	Collapse	IO
34	Collapse	CP	Collapse
35	Collapse	CP	LS
36	Collapse	Collapse	LS

It is seen that the RC school buildings perform better in Y (short) directions, according to ASCE-41 (ASCE, 2007) assessment results. 9 (25 %) of the buildings are in collapse performance state in both directions according to ASCE-41 and according to TEC-2007. 12 (33,3 %) of the buildings are in collapse performance state according to TEC-2007 and in collapse performance state at least in one direction, according to ASCE-41 (ASCE, 2007). For other buildings, there is not a meaningful correlation between the ASCE-41 (ASCE; 2007) and TEC-2007 assessment results.

CHAPTER 5

ASSESSMENT BY TURKISH BUILDING EARTHQUAKE CODE

5.1 Introduction to TBEC-2018 for RC School Buildings

Turkey has updated the EQ code with the advances in EQ engineering as well as the increase in social requirements since the first EQ code in 1940. Turkish EQ Code - 2007 (TEC-2007) is no more in use after the new code (TBEC-2018) is in force since January 2019. TBEC-2018 has improved the TEC-2007 to provide a substantially expanded scope.

The primary revision in the new code is in favor of the Turkish hazard map prepared by AFAD (Disaster and Emergency Management Presidency of Turkey). The previous map introduced in 1996 was based on a seismic zonation of the country into five hazard zones. Each zone had a unique value of PGA on a stiff soil as wide as 100's of km.s in some regions within a return period of 475 years. A single PGA value was found to be insufficient to represent the degree of hazard. However, the new seismic hazard map provided contour maps based on geographical coordinates so that the seismic hazard is expressed via spectral acceleration rather than a PGA value. For short periods of T=0.2 and 1.0 sec, site-specific spectral acceleration maps are provided on stiff soils for a given return period of 2475, 475, 72, and 43 years, respectively.

"*A comparison of the 1996 Seismic Zones Map and the 2018 Seismic Hazard Map, both 475-year PGA based, are presented in Figure 1. The mean PGA value in Zone 1 (red zone) of the 1996 map is 0.4g. The total area of the PGA > 0.4g regions is seemingly less in the 2018 map, especially in the Aegean West and along the Eastern Iranian border*." (Sucuoğlu, 2018)

Moreover, the four types of soil conditions (Z1 to Z4) in TEC-2007 are turned into six types from ZA to ZF, in the order of the best to the worst.

Design spectrum and the spectrum parameters are described in a remarkably different fashion in the new code. For a given stiff soil, the seismic hazard map suggests the S_S and S_1, spectral accelerations at 0.2 and 1.0 sec, respectively, while the site condition gives F_S and F_1 values which are used to modify the spectral accelerations to attain the design values, namely, S_{DS} and S_{D1}, respectively. Finally, the design spectrum is constructed in which the corner periods TA and TB are obtained from the associated ratios of S_{DS} and S_{D1}. Design spectrum illustration and equations in TBEC-2018 are shown in Figure 5-2 and Figure 5-2 below.

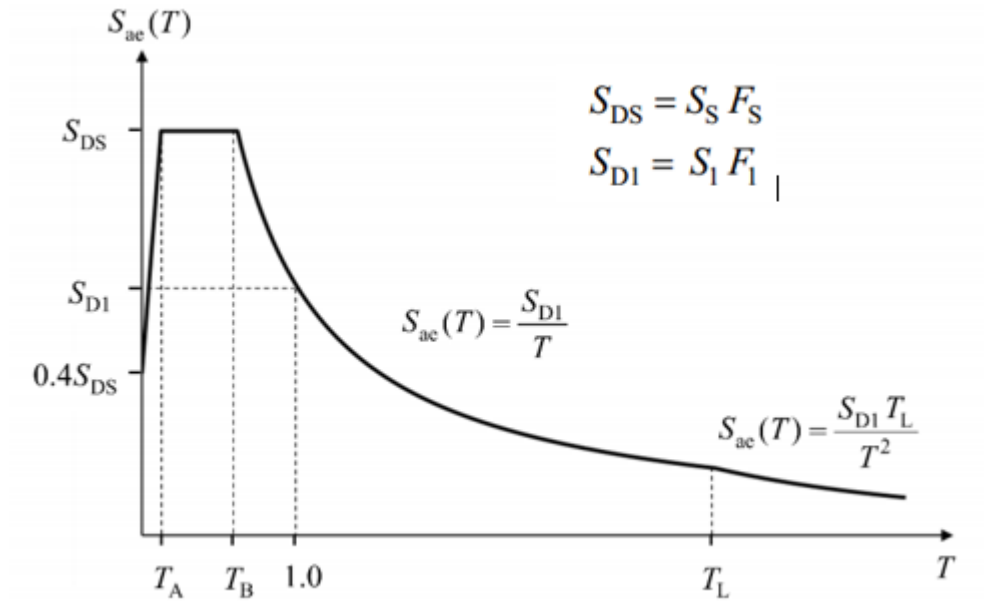

Figure 5-1. Design spectrum illustration in TBEC-2018

$$S_{ae}(T) = \left(0.4 + 0.6\frac{T}{T_A}\right)S_{DS} \qquad (0 \le T \le T_A)$$

$$S_{ae}(T) = S_{DS} \qquad (T_A \le T \le T_B)$$

$$S_{ae}(T) = \frac{S_{D1}}{T} \qquad (T_B \le T \le T_L)$$

$$S_{ae}(T) = \frac{S_{D1}T_L}{T^2} \qquad (T_L \le T)$$

$$T_A = 0.2\frac{S_{D1}}{S_{DS}} \quad ; \quad T_B = \frac{S_{D1}}{S_{DS}}$$

Figure 5-2. Design spectrum equations in TBEC-2018

For the buildings, the probability of exceedance of the design EQ within a period of 50 years is 10 % in TEC-2007. EQs with different probabilities of exceedance are defined in Chapter 7 of TEC-2007 to be considered in evaluating and reinforcing the existing buildings. However, TBEC-2018 proposes four different levels of EQ ground motion, namely DD1, DD2, DD3, and DD4, in the design stage. “DD1” is defined as the “maximum expected EQ ground motion level,” which accounts for a 2 % probability of exceedance in 50 years and a return period of 2475 years. “DD2” is the “standard design EQ ground motion level,” which accounts for a 10 % probability of exceedance in 50 years and a return period of 475 years. “DD3” is defined as the “frequently expected EQ ground motion level,” which accounts for a 50 % probability of exceedance in 50 years and a return period of 72 years. “DD4” is the “service level EQ ground motion,” which accounts for a 50 % probability of exceedance in 30 years and a return period of 43 years.

Seismic performances of the buildings in TEC-2007 are only available in Chapter 7 (evaluation and reinforcement of the existing buildings) of TEC-2007 as defined as IO, LS, CP, and collapse (C) levels. Each of them is determined based on force-based damages of structural elements and relative story drifts. In TBEC-2018, on the other

hand, the performance levels are explicitly defined to specify not only the performance of the existing building but also the design targets of the new building as follows:

Continued Operation Performance (CO): Damage in structural members is negligible.

Limited Damage Performance (LD): Limited damage in structural components leading to minimal inelastic behavior.

Controlled Damage Performance (CD): Damage in structural components is significant but possible to repair.

Collapse Prevention Performance (CP): Damage in structural components is severe; however, a partial or total collapse of the building is prevented.

In the new EQ code, there are many classifications governing the seismic design of the buildings, such as Building Use Category (BKS), Seismic Design Category (DTS), and Building Height Category (BYS). Figure 5-3 shows those categories associated with the corresponding ranks. Based on its critical facility assigned to the building, school buildings have a BKS of 1, which leads to an importance factor of 1.5. It was 1.4 in the previous code. As a new category, in TBEC-2018, the seismic design category (DTS) designates the seismic intensity at the design site under four main categories of DTS1 to DTS4 (or DTS1a to DTS4a) in terms of BKS and S_{DS}. The subscript “a” stands for imposing additional requirements in the design of critical buildings such as schools, hospitals, etc. BYS “building height category” in Figure 5-3 is related to the building height above the foundation or the rigid basement, and it is employed in selecting the design procedure.

Building Use Categories (BKS)

I=1.5 : BKS=1

I=1.2 : BKS=2

I=1.0 : BKS=3

Seismic Design Categories (DTS)

S_{DS} (g)	BKS=1	BKS=2, 3
< 0.33	4a	4
0.33-0.50	3a	3
0.50-0.75	2a	2
> 0.75	1a	1

Building Height Categories (BYS)

Bina Yükseklik Sınıfı	Bina Yükseklik Sınıfları ve Deprem Tasarım Sınıflarına Göre Tanımlanan Bina Yükseklik Aralıkları [m]		
	DTS = 1, 1a, 2, 2a	DTS = 3, 3a	DTS = 4, 4a
BYS = 1	$H_N > 70$	$H_N > 91$	$H_N > 105$
BYS = 2	$56 < H_N \leq 70$	$70 < H_N \leq 91$	$91 < H_N \leq 105$
BYS = 3	$42 < H_N \leq 56$	$56 < H_N \leq 70$	$56 < H_N \leq 91$
BYS = 4	$28 < H_N \leq 42$	$42 < H_N \leq 56$	
BYS = 5	$17.5 < H_N \leq 28$	$28 < H_N \leq 42$	
BYS = 6	$10.5 < H_N \leq 17.5$	$17.5 < H_N \leq 28$	
BYS = 7	$7 < H_N \leq 10.5$	$10.5 < H_N \leq 17.5$	
BYS = 8	$H_N \leq 7$	$H_N \leq 10.5$	

Figure 5-3. Categorization regarding the building use, seismic design, and building height in TBEC-2018 (Sucuoğlu, 2018)

In order to employ an appropriate design procedure so as to achieve the building design targets, target performances, as well as the ground motion levels, are combined as seen in Table 5-1. Then a design procedure is mandated, either the

conventional Force-Based Design (FBD) or the Performance-Based Assessment and re-Design (PBD).

Table 5-1 Performance targets and design procedures for new buildings (TBEC-2018)

EQ GM Level	DTS = 1,1a[(1)], 2, 2a[(1)], 3, 3a, 4, 4a		DTS = 1a[(2)], 2a[(2)]	
	Ordinary Performance Target	Design Procedure	Advanced Performance Target	Design Procedure
DD-3	—	—	LD	PBD
DD-2	CD	FBD	CD	FBD[(3)]
DD-1	—	—	CD	PBD

A "Controlled Damage" under DD-2, or the 475-year design ground motion performance target, is set for all buildings, excluding the tall buildings. However, the seismic design category (DTS) of 1a or 2a (critical buildings under high-intensity ground motions) necessitates applying the advanced target performances, which are the performance-based procedures under DD-1 and DD-3, force-based preliminary design under the DD-2 design spectra. The advanced performance targets are Limited Damage under the service earthquake DD-3, and Controlled Damage under the maximum expected earthquake DD-1. The associated relations between the ground motion level, target performance, and the must-use design procedure are given in Table 5-1.

TBEC-2018 provides Table 5-2 for the performance of the existing buildings where the ordinary performance target is controlled damage (CD) under the 475-year DD-2 earthquake, and the assessment/design procedure is performance-based. The school buildings with DTS of 1a or 2a require advanced performance targets accompanied by limited damage and controlled damage under DD3 and DD1 design spectra, respectively. A new displacement-based linear elastic procedure is developed for existing buildings, whereas if the method being invalid, the nonlinear analysis procedures have to be utilized.

Table 5-2 Performance targets and design procedures for existing buildings (TBEC-2018)

EQ GM Level	DTS = 1, 2, 3, 3a, 4, 4a		DTS = 1a, 2a	
	Ordinary Performance Target	Design/Assessment Procedure	Advanced Performance Target	Design/Assessment Procedure
DD3	—	—	LD	PBD
DD2	CD	PBD	—	—
DD1	—	—	CD	PBD

Forced-based analysis and design procedures implemented with the capacity design principles are similar to the provisions of TEC-2007, with marked improvement, however, in the definition of force reduction factors (R) and the over strength factors (D). The former ranges from 4 to 8, and the latter ranges from 2 to 3. Both values are assigned to be 1.0 for the assessment of the existing buildings.

TBEC-2018 describes the inter-story drift limits for new designs as follows:

If infill walls and frames are rigidly connected: $\lambda \frac{\delta imax}{hi} \leq 0.008\ \kappa$

If infill walls and frames have flexible connections: $\lambda \frac{\delta imax}{hi} \leq 0.016\ \kappa$

Where λ is the spectral acceleration ratio of DD-3 to DD-2 as in the range of 0.4 to 0.5 range and κ is 1.0 for concrete.

Sucuoğlu (2018) briefly summarized the new improvements in the new seismic code TBES-2018. Sucuoğlu (2018) pointed to nonlinear analysis procedures gaining wider acceptance as performance-based EQ engineering increases its popularity.

Dalyan and Şahin (2019) compared TEC-2007 and TBEC-2018 and told that significant differences between the two codes are the description of the effective cross-section stiffness and damage limits of RC sections. Dalyan and Şahin (2019)

concluded that effective periods of structures are calculated by TBEC-2018 are longer than the ones calculated by TEC-2007.

Nemutlu and Sarı (2018) compared TEC-2007 and TBEC-2018 and concluded that base shear force demands are calculated as 6 % greater by TBEC-2018. Nemutlu and Sarı (2019) compared TBEC-2018 and IBC-2018 and concluded that base shear force demands are calculated as 8 % greater by TBEC-2018.

5.2 Assessment of Selected RC School Buildings by TBEC-2018

A total of 36 RC school buildings presented in Appendix-B have been tested for the performance criteria of existing buildings as per TBEC-2018. According to TBEC-2018, all the buildings have a BKS of 1.0, leading to an importance factor of 1.5, whereas it is reduced to unity in the analysis of existing buildings. Then, Ss and S1, spectral accelerations at 0.2 and 1.0 sec, are acquired from the Turkish Seismic Hazard Map by entering the geographical coordinates of each building for the return periods of 2475, 475, and 72 years, respectively. Site classification of the buildings as per the 2007 code is transformed to the corresponding category of the TBEC-2018 code. Out of 36 school buildings, therefore, ZA, ZC, and ZD soil classes are assigned to 9, 25, and 2 of the buildings. Since the calculated design spectral accelerations (SDS) range between 0.50 and 0.75, DTSs of all the buildings are found to be 2a. As mentioned above, DTS of 2a indicates the necessity of employing a performance-based assessment approach as well as advanced performance.

For a given DTS of 2a building height category (BYS) are found to be 7, 6, and 5 for the school buildings with 3, 4, and 5 floors as seen in Figure 5-3. The new seismic code suggests only two information levels, namely limited and comprehensive. The comprehensive information level is utilized with a coefficient of 1.0 in the assessment of the school buildings as the code does not allow the application of the former to such buildings of critical facilities. In-situ material properties of the

concrete and steel are assigned to the buildings. It should be noted that both the R and D values are 1.0.

In order to make a comparison between the outcomes of the analysis with respect to the 2007 and 2018 codes, the same analysis method is applied. Thus, the linear elastic method in terms of mod combination is used for seismic analysis of all school buildings. The buildings are expected to satisfy the following target performance levels simultaneously, as suggested in TBEC-2018 and 2007, respectively.

TBEC-2018:

- LD (limited damage) under service earthquake DD3 for which the probability of exceedance in 50 years is 50 % within a return period of 72 years.
- CD (Controlled Damage) under the maximum expected earthquake DD1 for which the probability of exceedance in 50 years is 2 % within a return period of 2475 years.

TEC-2007:

- IO under the earthquake for which the probability of exceedance in 50 years is 10 % within a return period of 475 years.
- LS under the earthquake for which the probability of exceedance in 50 years is 2 % within a return period of 2475 years.

Seismic analyzes of 36 school buildings have been carried out in line with the conditions above by means of a commercially available software, so-called ProtaStructure-2019. Table 5-3 also presents the validity of the application of the linear analysis method according to the new code. Although the linear analysis method is found to be valid for 8 of the buildings, all having shear walls, the linear procedure is used for all buildings. The outcomes of this program are presented in tabular forms in Table 5-4, and Table 5-5. The direction of earthquake loading is designated by +X, -X, +Y, and –Y letters.

Table 5-3 Control of the linear analysis method as per TBEC-2018 (I: Invalid and V: Valid)

Building No	TURKISH EQ CODE 2018											
	EQ level: DD1 (2 % in 50 years)				EQ level: DD2 (10 % in 50 years)				EQ level: DD3 (50 % in 50 years)			
	+X	-X	+Y	-Y	+X	-X	+Y	-Y	+X	-X	+Y	-Y
1	I	I	I	I	I	I	I	I	V	V	V	V
2	I	I	I	I	I	I	I	I	I	I	I	I
3	I	I	I	I	I	I	I	I	I	I	I	I
4	I	I	I	I	I	I	I	I	I	I	I	I
5	V	V	V	V	V	V	V	V	V	V	V	V
6	I	I	I	I	I	I	I	I	I	I	I	I
7	V	V	V	V	V	V	V	V	V	V	V	V
8	I	I	I	I	I	I	I	I	I	I	I	I
9	I	I	I	I	I	I	I	I	I	I	I	I
10	V	V	V	V	V	V	V	V	V	V	V	V
11	V	V	V	V	V	V	V	V	V	V	V	V
12	V	V	V	V	V	V	V	V	V	V	V	V
13	V	V	V	V	V	V	V	V	V	V	V	V
14	I	I	I	I	I	I	I	I	I	I	I	I
15	I	I	I	I	I	I	I	I	I	I	I	I
16	I	I	I	I	I	I	I	I	I	I	I	I
17	I	I	I	I	I	I	I	I	V	V	V	V
18	I	I	I	I	I	I	I	I	I	I	I	I
19	I	I	I	I	I	I	I	I	V	V	V	V
20	I	I	I	I	I	I	I	I	I	I	I	I
21	V	V	V	V	V	V	V	V	V	V	V	V
22	I	I	I	I	I	I	I	I	I	I	I	I
23	I	I	I	I	I	I	I	I	I	I	I	I
24	I	I	I	I	I	I	I	I	I	I	I	I

Table 5-3 Continued

Building No	TURKISH EQ CODE 2018											
	EQ level: DD1 (2 % in 50 years)				EQ level: DD2 (10 % in 50 years)				EQ level: DD3 (50 % in 50 years)			
	+X	-X	+Y	-Y	+X	-X	+Y	-Y	+X	-X	+Y	-Y
25	I	I	I	I	I	I	I	I	I	I	I	I
26	I	I	I	I	I	I	I	I	I	I	I	I
27	I	I	I	I	V	V	V	V	V	V	V	V
28	I	I	I	I	I	I	I	I	I	I	I	I
29	V	V	V	V	V	V	V	V	V	V	V	V
30	I	I	I	I	I	I	I	I	I	I	I	I
31	I	I	I	I	I	I	I	I	I	I	I	I
32	I	I	I	I	I	I	I	I	I	I	I	I
33	I	I	I	I	I	I	I	I	I	I	I	I
34	I	I	I	I	I	I	I	I	V	V	V	V
35	I	I	I	I	I	I	I	I	I	I	I	I
36	I	I	I	I	I	I	I	I	I	I	I	I

According to TBEC-2018, the linear analysis method is found to be valid for 8 of the 36 buildings, all having shear walls. The linear analysis method is not valid according to TBEC-2018 for RC school buildings having no shear walls. Buildings are assessed by TBEC-2018 using linear analysis since they are all analyzed by linear methods for TEC-2007. In order to compare them properly, the analysis methods should be similar.

As it is well observed in Table 5-4, detailed seismic assessment of the buildings according to TBEC-2018 revealed that none of the 36 RC school buildings do satisfy the LD and CD performance levels. Indeed, in the case of DD1, the maximum expected earthquake having a probability of occurrence of 2 % in 2475 years, all of the school buildings appear to be within the collapse level (C). Similar findings were

seen under the DD3 - the service earthquake in that none of the buildings achieve the expected limited damage stage, even entering the collapse zone. As seen in Table 5-4, few buildings seem to have controlled damage (CD) or collapse prevention (CP) performance states only in Y directions. Therefore, usage of the buildings under these circumstances poses substantial threats towards life safety, according to TBEC-2018.

Table 5-4 Performances of the RC school buildings as per TBEC-2018

Bldng. No	TURKISH BUILDING EQ CODE 2018							
	Target Performance level: CD				Target Performance level: LD			
	EQ level: DD1 (2 % in 50 years)				EQ level: DD3 (50 % in 50 years)			
	+X	-X	+Y	-Y	+X	-X	+Y	-Y
1	C	C	C	C	C	C	C	C
2	C	C	C	C	C	C	C	C
3	C	C	C	C	C	C	C	C
4	C	C	C	C	C	C	C	C
5	C	C	C	C	C	C	CD	CP
6	C	C	C	C	C	C	C	C
7	C	C	C	C	C	C	C	C
8	C	C	C	C	C	C	C	C
9	C	C	C	C	C	C	C	C
10	C	C	C	C	C	C	C	C
11	C	C	C	C	C	C	C	C
12	C	C	C	C	C	C	C	C
13	C	C	C	C	C	C	C	C
14	C	C	C	C	C	C	C	C
15	C	C	C	C	C	C	C	C
16	C	C	C	C	C	C	C	C
17	C	C	C	C	C	C	C	C
18	C	C	C	C	C	C	C	C

Table 5-4 Continued

Bldng. No	TURKISH BUILDING EQ CODE 2018							
	Target Performance level: CD				Target Performance level: LD			
	EQ level: DD1 (2 % in 50 years)				EQ level: DD3 (50 % in 50 years)			
	+X	-X	+Y	-Y	+X	-X	+Y	-Y
19	C	C	C	C	C	C	C	C
20	C	C	C	C	C	C	C	C
21	C	C	C	C	C	C	CD	CD
22	C	C	C	C	C	C	C	C
23	C	C	C	C	C	C	C	C
24	C	C	C	C	C	C	C	C
25	C	C	C	C	C	C	C	C
26	C	C	C	C	C	C	C	C
27	C	C	C	C	C	C	C	C
28	C	C	C	C	C	C	C	C
29	C	C	C	C	C	C	C	C
30	C	C	C	C	C	C	CP	CP
31	C	C	C	C	C	C	C	C
32	C	C	C	C	C	C	C	C
33	C	C	C	C	C	C	C	C
34	C	C	C	C	C	C	C	C
35	C	C	C	C	C	C	C	C
36	C	C	C	C	C	C	C	C

The performances of 36 RC school buildings as per TEC-2007 were summarized in Appendix B. The EQ spectra applied in the analysis differ in 2007 and 2018 provisions. The design EQ, which has the probability of occurrence of 10% in 50 years within a return period of 475 years, is not in the scope of TBEC-2018 for RC school buildings. Although it is not mandatory, 36 selected RC school buildings are

also analyzed by the design EQ by using the same software so that a direct comparison can be made between the assessment results of EQ codes for the design EQ According to TEC-2007, RC school buildings are expected to satisfy immediate occupancy performance level under the design EQ The performances of selected RC school buildings with respect to TBEC-2018 under the design EQ is given in Table 5-5.

Table 5-5 Performances of the RC school buildings as per TBEC-2018 under design EQ

Building No	TURKISH BUILDING EQ CODE 2018			
	Target Performance level: LD			
	EQ level: DD2 (10 % in 50 years)			
	+X	-X	+Y	-Y
1	C	C	C	C
2	C	C	C	C
3	C	C	C	C
4	C	C	C	C
5	C	C	C	C
6	C	C	C	C
7	C	C	C	C
8	C	C	C	C
9	C	C	C	C
10	C	C	C	C
11	C	C	C	C
12	C	C	C	C
13	C	C	C	C
14	C	C	C	C
15	C	C	C	C
16	C	C	C	C
17	C	C	C	C
18	C	C	C	C
19	C	C	C	C
20	C	C	C	C
21	C	C	C	C
22	C	C	C	C
23	C	C	C	C
24	C	C	C	C
25	C	C	C	C

Table 5-5 Continued

Building No	TURKISH BUILDING EQ CODE 2018			
	Target Performance level: LD			
	EQ level: DD2 (10 % in 50 years)			
	+X	-X	+Y	-Y
26	C	C	C	C
27	C	C	C	C
28	C	C	C	C
29	C	C	C	C
30	C	C	C	C
31	C	C	C	C
32	C	C	C	C
33	C	C	C	C
34	C	C	C	C
35	C	C	C	C
36	C	C	C	C

The assessment reports of all selected 36 RC school buildings performed by ProtaStructure software are presented in Appendix E.

5.3 Discussion of Results

Linear calculation methods for determining the building performance in EQ have a limited application on the buildings given in Section-15.5 of TBEC-2018. Otherwise, a non-linear analysis method is employed. For this reason, the validity of the mod-superposition method is checked in Table 5-3, in which “V” and “I” stand for “valid” or “invalid” for linear analysis procedure. Irrespective of the EQ level of DD1, DD2, and DD3, the linear analysis is found to be accurately applicable for 8 buildings out of 36. Moreover, 4 buildings fulfilled those constraints in DD2 and DD3 ground motions. It is evident that the linear analysis method seems to be invalid in the case of buildings with no shear walls. However, the method is mostly applicable, provided

that the buildings have shear walls cross-section of higher than 0.5% by the land area in both directions.

The density of shear walls in a given principal direction is believed to have a prominent role in the seismic performance of the buildings (Gülkan and Sözen, 1999; Yakut, 2004), but unfortunately, most of the buildings in this study have no sufficient shear walls in either direction to fulfill the code requirements of TEC-2007 as well as TBEC-2018. Also, the combined effects of low concrete quality and reinforcing steel help the buildings exhibit these unexpectedly poor performances, irrespective of the seismic code.

The performances of 36 selected RC school buildings are summarized as per TEC-2007 in Appendix B and TBEC-2018 in the previous section. While 20 % of the buildings satisfy the performance requirements of TEC-2007, it is not the same for TBEC-2018. Excluding a few buildings that have shown satisfactory performance levels in either X or Y directions for DD-3 EQ, there is no such building that has fulfilled both DD-1 and DD-3 EQ performance requirements simultaneously for TBEC-2018.

The base shear forces determined by TEC-2007 and TBEC-2018 are compared and tabulated below in Table 5-6.

Table 5-6 Comparison of base shear force ratios

	TBEC-2018 / TEC-2007	
Number of Stories	X-direction	Y-direction
3	1.075	1.064
4	1.070	1.070
5	1.078	1.097
Overall	1.074	1.077

Nemutlu and Sarı (2018) compared TEC-2007 and TBEC-2018 and concluded that base shear force demands are calculated as 6 % greater by TBEC-2018. Nemutlu and

Sarı (2019) compared TBEC-2018 and IBC-2018 and concluded that base shear force demands are calculated as 8 % greater by TBEC-2018.

Consistently, it is determined that the base shear forces of TBEC-2018 are 7.5% larger than the base shear forces of TEC-2007.

The assessment results reveal that there is not much difference between the codes for the buildings in this study owing to the deficiency of the shear walls in the buildings to promote either Turkish Codes. It is evident that TBEC-2018 is more conservative than TEC-2007.

CHAPTER 6

ASSESSMENT BY PROVISIONS UNDER URBAN RENEWAL LAW

6.1 Assessment by Provisions Published in 2013

Urban Renewal Law (URL) (no.6306) was accepted by the Turkish Parliament on 16 May 2012 (URL-2013) and in force since then. The primary focus of the law is to reduce the expected seismic risk due to the vulnerability of existing RC and masonry buildings having a total height of less than 25 meters and at most 8 stories, which comprises the majority of the existing vulnerable buildings in Turkey (Binici et al., 2015).

Detailed provisions (ordinance) for applying the law were accepted by the Turkish Cabinet and published in the Official Newspaper on 15 December 2012. The 2nd annex of the ordinance consists of the technical procedures to determine the vulnerable existing buildings, i.e., critical or tagged buildings, as high risk. The 2nd annex of the ordinance is generally known as the provisions/code for determining the critical (or high risk) buildings.

According to the provisions, a building is tagged as highly critical if the building's performance state under the estimated EQ is neat collapse or CP , i.e., the building is expected to experience heavy damage or collapse. The estimated number of buildings to be examined in the next decades is up to millions, which means assessing these buildings with TEC-2007 or TBEC-2018 would need a tremendous amount of time and budget (Binici et al., 2015). In order to overcome these constraints, the new provisions suggest faster and cheaper procedures with acceptable consistency with the EQ codes.

The provisions are not explained in detail since it is not in the scope of this study. But it is worth to mention that the provisions are not to be used for seismic retrofit

purposes but to determine whether a single building has a risk for heavy damage or collapse. The assessment is based on the critical floor, which is generally the ground floor. The search for the irregularities of the building is decisive for whether there is a need for a detailed survey of upper stories (Binici et al., 2015).

In this study, linear elastic analysis is employed for 36 selected RC school buildings for two orthogonal directions, and bending moment demand capacity ratios (DCR) at member ends and inter-story drift deformations are determined as stated by the provisions. The design EQ described by TEC-2007 having a 10 % probability of occurrence in 50 years is employed without using any response modification (i.e., R=1). The determined DCR values and inter-story drift deformations are then compared with the performance limit values declared by the provisions. The columns or structural walls that are not satisfying the limits are named as unacceptable. The performance limits defined by the provisions are intermediate levels between the commonly accepted life safety and collapse. Buildings are then classified as critical or not critical depending on the number of unacceptable members. All analyses within this study are conducted as declared by this provision and as explained by Binici et al. (2015) using the “protastructure software” (2019). The performances of the buildings are summarized in Table 6-1; “R” designating for “Risky” and “UR” for “UnRisky.”

Table 6-1 Risk assessment of the buildings as per URL-2013

Building No.	Provisions of Urban Renewal Law (2013)			
	EQ level: DD2 (10 % in 50 years)			
	+X	-X	+Y	-Y
1	R	R	R	R
2	UR	UR	UR	UR
3	UR	UR	R	R
4	R	R	R	R
5	UR	UR	UR	UR
6	R	R	UR	UR
7	UR	UR	UR	UR
8	R	R	UR	UR
9	R	R	R	R
10	UR	UR	UR	UR
11	UR	UR	UR	UR
12	UR	UR	UR	UR
13	UR	UR	UR	UR
14	UR	UR	R	R
15	R	R	R	R
16	UR	UR	R	R
17	R	R	R	R
18	R	R	R	R
19	UR	UR	R	R
20	R	R	R	R
21	R	R	R	R
22	R	R	R	R
23	R	R	R	R
24	R	R	R	R
25	R	R	R	R
26	R	R	R	R
27	R	R	R	R
28	R	R	R	R
29	UR	UR	UR	UR
30	R	R	R	R
31	R	R	R	R
32	R	R	R	R
33	R	R	R	R
34	R	R	R	R
35	R	R	R	R
36	R	R	UR	UR

Assessment reports of all selected 36 RC school buildings performed by ProtaStructure software under URL provisions published in 2013 are provided in Appendix F.

6.2 Assessment by Provisions Published in 2019

After TBEC-2018 has been officially enforced since January 1, 2019, Urban Renewal Law (URL) (no.6306) is revised in accordance with the new code and issued on February 16, 2019, and it is so-called URL-2019 hereafter in this study. Like the URL-2013, URL-2019 aims to reduce the expected seismic risk of the buildings but with expanded content. The former involves solely "the other buildings, namely, houses, hotels, etc.," as being identified in Section-7 of the TEC-2007. The design spectrum given in TEC-2007 corresponding to the design earthquake having a 10% probability of occurrence in 50 years is employed to determine the elastic earthquake forces. The latter, however, provides to determine the seismic risk of a much broader spectrum of the buildings, including "The buildings that should be used after earthquakes," "The buildings that people stay in for a long time period," "The buildings that people visit densely and stay in for a short time period," "The buildings containing hazardous materials," as well as the other buildings mentioned in TEC-2007. Conforming to the TBEC-2018, in URL-2019 DD1 (2 % probability of being exceeded in 50 years) or DD2 (10 % probability of being exceeded in 50 years), EQ ground motions are employed as the design earthquake.

Moreover, the number of stories and the total height of the building are limited to 8 floors and 25 m, respectively. URL-2019 categorizes the buildings into low-rise, mid-rise, and high-rise according to the height and number of stories so that it enables to seismically analyze a wide range of RC and masonry buildings. Both URL-2013 and URL-2019 provisions allow using Linear Calculation Methods for seismic risk assessment.

In this part of the study, seismic assessments of 36 selected RC school buildings have been conducted under the DD1 design earthquake having a 2% probability of being exceeded in 50 years as per URL-2019 via “Protastructure-2019”. The program outcomes designating the buildings as risky “R or Un-risky “UR” are tabulated below in Table 6-2.

Assessment reports of all selected 36 RC school buildings performed by ProtaStructure software under URL provisions published in 2019 are provided in Appendix G.

Table 6-2 Risk assessment of the buildings as per URL-2019

Building No.	Urban Renewal Law (2019)			
	EQ level: DD1 (2 % in 50 years)			
	+X	-X	+Y	-Y
1	R	R	R	R
2	R	UR	R	R
3	R	R	R	R
4	R	R	R	R
5	UR	UR	UR	UR
6	R	R	R	R
7	R	R	R	R
8	R	R	UR	UR
9	R	R	R	R
10	R	UR	UR	UR
11	R	R	R	R
12	R	UR	UR	UR
13	R	R	R	R
14	R	R	R	R
15	R	R	R	R
16	R	R	R	R
17	R	R	R	R
18	R	R	R	R
19	R	R	R	R
20	R	R	R	R
21	R	UR	R	R
22	R	R	R	R
23	R	R	R	R
24	R	R	R	R
25	R	R	R	R
26	R	R	R	R
27	R	R	R	UR
28	R	R	R	R
29	UR	UR	R	UR
30	R	R	R	R
31	R	R	R	R
32	R	R	R	R
33	R	R	R	R
34	R	R	R	R
35	R	R	R	R
36	R	R	R	UR

6.3 Discussion of Results

Table 6-1 presents the performance of the school buildings according to the URL-2013. 8 buildings out of 36 satisfy the URL-2013 in all directions so that they are found to be seismically having a low risk. However, this finding does not necessarily mean that they also satisfy the target performance specified in TEC-2007. Most of the buildings in this category have three stories. The lower number of stories associated with the lower building height, the buildings become less vulnerable to EQs. Almost all five and four-story buildings failed the risk assessment as per URL-2013.

Binici et al. (2015), in their study, analyzed 10 RC buildings as a case study and concluded that the new provisions seem to provide fast and easy-to-apply procedures and to produce results that are as acceptable as the results of nonlinear procedures of TEC-2007 with much less effort; however, linear elastic procedures of TEC-2007 may result in either way. Although the selected 36 RC school buildings in this study are analyzed and assessed by linear elastic procedures of TEC-2007, there is reasonable consistency between the provisions of URL and TEC-2007 assessment results.

The status of the buildings appears to be much worse when URL-2019 is considered. Indeed, all of the buildings except for only one are found to be risky, as seen in Table 6-2. Even the satisfactory buildings in the case of URL-2013 have failed the risk assessment in this case. The main reason for this is based on the DD1 earthquake level, which has as high as 1.5 times design spectral acceleration in the elastic spectrum.

In order to compare URL-2013 and URL-2019, the buildings are tested for the same design earthquake of a 10% probability of occurrence in 50 years. As seen in Table 6-3, among the 36 buildings, only four of them satisfy both provisions. Although the selected 36 RC school buildings in this study are analyzed and assessed by linear

elastic procedures of either TEC-2007 or TEC-2018, there is reasonable consistency between the assessment results of URL provisions and the seismic codes.

TEC-2007, TBEC-2018, URL-2013, URL-2019, DCM, and ASCE-41 assessment results are all presented in the same table in order to compare them simultaneously. The same EQ level DD' is used for comparison. The assessment results for orthogonal directions are not given, and the riskiest, i.e., the conservative result, is mentioned for the whole building. The comparison table is provided in Table 6.4. There is no RC school building, which is not risky for all assessment procedures.

Table 6-3 Comparing the risk assessments as per URL 2013 and 2019

Building No.	Urban Renewal Law (2013)				Urban Renewal Law (2019)			
	EQ level: DD2 (10 % in 50 years)				EQ level: DD2 (10 % in 50 years)			
	+X	-X	+Y	-Y	+X	-X	+Y	-Y
1	R	R	R	R	R	R	R	R
2	UR	UR	UR	UR	R	UR	R	R
3	UR	UR	R	R	R	R	R	R
4	R	R	R	R	R	R	R	R
5	UR	UR	UR	UR	UR	UR	UR	UR
6	R	R	UR	UR	R	R	R	R
7	UR	UR	UR	UR	R	R	R	R
8	R	R	UR	UR	R	R	UR	UR
9	R	R	R	R	R	R	R	R
10	UR	UR	UR	UR	UR	UR	UR	UR
11	UR	UR	UR	UR	R	R	R	R
12	UR	UR	UR	UR	UR	UR	UR	UR
13	UR	UR	UR	UR	R	R	R	R
14	UR	UR	R	R	R	R	R	R
15	R	R	R	R	R	R	R	R
16	UR	UR	R	R	UR	UR	R	R
17	R	R	R	R	R	R	R	R
18	R	R	R	R	R	R	R	R
19	UR	UR	R	R	R	R	R	R
20	R	R	R	R	R	R	R	R
21	R	R	R	R	R	UR	UR	UR
22	R	R	R	R	R	R	R	R
23	R	R	R	R	R	R	R	R
24	R	R	R	R	R	R	R	R
25	R	R	R	R	R	R	R	R
26	R	R	R	R	R	R	R	R
27	R	R	R	R	R	R	R	UR
28	R	R	R	R	R	R	R	R
29	UR	UR	UR	UR	UR	UR	UR	UR
30	R	R	R	R	R	R	R	R
31	R	R	R	R	R	R	R	R
32	R	R	R	R	R	R	R	R
33	R	R	R	R	R	R	R	R
34	R	R	R	R	R	R	R	R
35	R	R	R	R	R	R	R	R
36	R	R	UR	UR	R	R	R	UR

Table 6.4 Comparison of assessment results for EQ level DD2

Building No	TEC-2007	TBEC-2018	URL-2013	URL-2019	DCM	ASCE-41
1	C	C	R	R	C	C
2	IO	C	UR	R	C	C
3	C	C	R	R	C	C
4	IO	C	R	R	C	C
5	C	C	UR	UR	C	C
6	C	C	R	R	C	C
7	C	C	UR	R	C	IO
8	C	C	R	R	C	C
9	C	C	R	R	CP	C
10	C	C	UR	UR	LS	C
11	IO	C	UR	R	LS	C
12	C	C	UR	UR	LS	C
13	C	C	UR	R	C	C
14	C	C	R	R	C	C
15	C	C	R	R	C	C
16	C	C	R	R	C	C
17	C	C	R	R	CP	IO
18	C	C	R	R	C	C
19	IO	C	R	R	C	C
20	IO	C	R	R	C	C
21	C	C	R	R	C	C
22	IO	C	R	R	C	C
23	C	C	R	R	C	C
24	C	C	R	R	C	C
25	C	C	R	R	C	C
26	C	C	R	R	C	C
27	C	C	R	R	CP	C
28	C	C	R	R	C	C
29	IO	C	UR	UR	C	C
30	LS	C	R	R	C	C
31	IO	C	R	R	C	C
32	C	C	R	R	C	IO
33	IO	C	R	R	C	C
34	C	C	R	R	C	C
35	LS	C	R	R	C	C
36	LS	C	R	R	C	C

CHAPTER 7

DEVELOPMENT OF FRAGILITY CURVES

7.1 Introduction to Fragility Curves

When an EQ strikes a populated urban region, a large number of residential and public buildings may suffer damages of various states, and some of these buildings may collapse totally. This section of the study aims to determine the F.C.s for 3, 4, and 5 story RC school buildings in Turkey. A realistic description of the fragilities and generation of F.C.s for these buildings is essential since vulnerable school buildings affect society more than other types of buildings after a severe EQ

The term fragility may be defined as "the quality of being easily broken or damaged" in common, but in structural engineering, we define fragility as a typical expression of damageability of an asset as a function of excitation (Porter, 2014). Fragility is not vulnerability. Fragility measures probability, vulnerability measures loss. The fragility of structures is a performance measure in most basic terms (Ay, 2006).

There may be two different analytical approaches for the assessment of the seismic vulnerability of a specific building stock. The first one is to assess each building in the stock individually, and the fragility information related to building stock is obtained by combining the assessment results of each building. Although the results are accurate in this approach, it is economically and practically not feasible. The second and much more practical and economical approach is using the fragility studies of equivalent and simplified analytical models. The obtained results are not as accurate as in the first approach, but decision-makers need rapid estimates of losses in order to execute their EQ mitigation plans (Özün, 2007).

F.C.s express conditional cumulative distribution functions that define the attainment or exceedance probability of a damage state for a given ground motion intensity level

(PGV, PGA, SA, etc.). In most cases, the probability distribution function is the standard lognormal distribution. (Shinozuka et al. 2000, Zentner et al., 2008, Kircher et al. 1997).

F.C.s provide graphical information on structural damage distribution. Damage distribution is shown by a damage/performance state curve, and every point on the curve represents a probability of attaining or exceeding the related damage state for the related ground motion intensity. It is a unique curve because every curve is developed depending on related structural parameters and related ground motions.

The mathematical expression of the fragility function is given in Equation 7-1.

$$PF_{i,j} = P(D \geq DLS_i \,|IM_j) \tag{7-1}$$

In EQ 7-1, D represents the damage (response) of the structure under the ground motion level with an Intensity Measure of j (IM_j). Hence, $PF_{i,j}$ expresses the probability of attainment or exceedance of Damage Limit State i (DLS_i) under the ground motion level IM_j.

F.C.s can be generated for one specific structural system or a component or a class of structural systems and components. Fragility information should be integrated with seismic hazard levels to estimate seismic risk at an acceptable level of accuracy (Özün, 2007).

The reliability of the SVA by F.C.s is directly dependent on the reliability of the F.C.s. The derived F.C.s are highly sensitive to the choices made for the structural idealization, analysis method, seismic hazard, and performance/damage state definitions (Avşar, 2009). One must keep in mind that F.C.s are inherently blind: they reveal percentages of damage for building groups but not their addresses (Yakut et al., 2005).

A schematic description of generating F.C.s are shown in Figure 7-1 (Avşar, 2009). The x-axis represents the seismic intensity measure (in terms of PGV, PGA, SA, etc.) of the EQ, and the y-axis represents the probability of attainment or exceedance of related damage limit state. The resulting F.C. is the best fit for the probability points.

The probability of attaining or exceeding a certain damage limit state is generally modeled as a cumulative lognormal probability distribution in most of the recent studies (HAZUS, 2003; Karim and Yamazaki, 2003; Elnashai et al., 2004; Nielson, 2005; Banerjee and Shinozuka, 2007). The lognormal probability distribution is also used in this study.

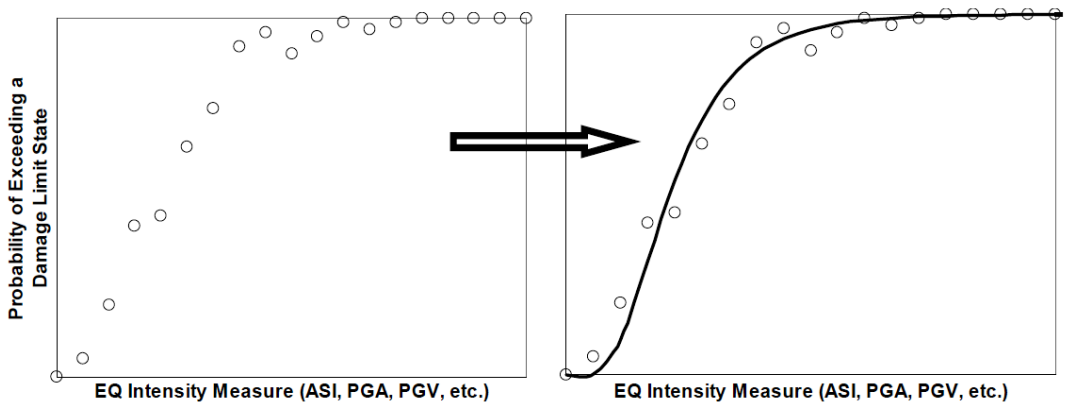

Figure 7-1. Schematic representation of generating an F.C. (Avşar, 2009)

The probability of attaining or exceeding the limit states can also be expressed by the Damage Probability Matrix (DPM), which is the tabular presentation of the same information. DPM representation is out of the scope of this study.

F.C.s can be used in many ways, and most of them are somehow related to the assessment of structures. Some of the uses of F.C.s are listed below:

i. As a screening tool to determine structure types and urban regions that are more likely to be damaged in future EQs (Miranda, 2005),

ii. As a tool to perform parametric studies in order to identify structural or ground motion parameters increasing seismic demands on structures (Miranda, 2005),
iii. As a planning tool for emergency management authorities by using F.C.s in order to generate possible damage scenarios (Miranda, 2005),
iv. As a tool for loss estimation of extensive inventories of structures,
v. As a useful tool for insurance or reinsurance companies to determine the seismic risk, and the related optimum insurance costs,
vi. As a tool to provide early performance estimates just a few minutes after an EQ,
vii. As a planning tool for hospitals and highways network (Mohsen, 2012),
viii. As a tool for analysis, evaluation, and improvement of seismic performance of structures (Mohsen, 2012),
ix. As a tool for prioritizing of retrofit of structures (Güneyisi, 2007; Kirçil and Polat, 2006),
x. As a tool for cost-benefit analysis of retrofit of structures (Güneyisi, 2007), etc.

There are various ways to obtain F.C.s. Existing F.C.s are often classified into four major groups called empirical, judgmental, analytical, and hybrid. Each type of F.C. has some advantages and disadvantages.

Empirical F.C.s are obtained by using the damage data observed after real EQs. Structural and non-structural damage to buildings is documented by reconnaissance reports after different EQs recorded by accelerograms. F.C.s for different structure types can be determined by using the observational damage data by statistical analysis easily. It is the most realistic approach among all types of F.C.s, but the reliability is questionable since there exists objectivity in the observed damage data. Besides, the number of damaged structures of the same type with similar properties affected by the EQs are usually inadequate. For example, the empirical F.C.s developed for European countries are based on very few damage surveys carried out

for single locations or EQ events (Orsini, 1999). As a result, empirical F.C.s are specific, and the application is minimal (Avşar, 2009).

To obtain more accurate empirical F.C.s, damage data from post-EQ surveys carried out for large populations of buildings of similar construction over areas of uniform soil conditions in close proximity to ground motion recording stations should be used (Orsini, 1999).

Judgmental F.C.s are based on expert opinions on damage data of related structure types. Experts are asked to estimate the performance states or damage distribution of structure populations subjected to different intensities of EQs. Then F.C.s are obtained by fitting probability functions to the expert opinions with respect to EQ intensities and damage states. Delphi method and panels may be used for collecting expert opinions. This way is one of the most straightforward methods of obtaining F.C.s; however, the reliability of these curves is questionable due to their dependence on the individual experience of the experts consulted (Güneyisi, 2007).

Applied Technology Council (ATC) conducted a study using the judgmental method and obtained F.C.s of 40 different structural types in California (USA) are received with the opinion of 58 experts, reporting the results in ATC-13 (1985) thereafter. Several studies (Cardona and Yamin, 1997; King et al., 1997; HAZUS, 1999) followed the judgmental method and used the F.C.s given in ATC–13 (Ay, 2006).

Using analytical models and structural simulations are the most common way of obtaining F.C.s due to developments in analysis techniques that enable the generation of damage data at very high speeds. The possibility of executing big numbers of structural analysis is the primary advantage of this method. Lack of inspection of real structural response is the primary disadvantage of the analytical method. Performance limit state definitions have a direct influence on the analytical F.C.s. Modeling assumptions, material models, ground motion parameters, analysis techniques, and the response parameter are the necessary factors affecting the accuracy of analytical F.C.s.

Elastic or inelastic time–history analysis has gained popularity due to efficiency and reliable results. ESDOF system, which is the simplest model, enables the computation of numerous analyses in a short period of time. As a result, many researchers have used ESDOF models (Ibarra, 2003; Jeong and Elnashai, 2004; Erberik and Elnashai, 2004). Akkar et al. (2005), Erberik and Çullu (2006), Baykal and Kırçıl (2006), Kırçıl and Polat (2006) are some of the analytical fragility studies conducted in Turkey.

Hybrid F.C.s are generated by combining data from different sources, generally involving the modification of analytical or judgmental F.C.s with observational data. The curves proposed in ATC-13 (1985) and ATC-40 (1996) are typically based on expert opinion but also involve limited observational data from the San Fernando EQ (1971) and Northridge EQ (1994), respectively (Güneyisi, 2007).

Hybrid vulnerability curves aim to compensate for the scarcity of observational data, the subjectivity of judgmental data, and modeling deficiencies of analytical procedures by combining data from different sources (Rossetto and Elnashai, 2003). The primary disadvantage of hybrid F.C.s is the lack of proper supplementary data.

Experimental data can also be used for the generation of F.C.s, usually with other sources of data. Although there is an interest to shift toward experimental studies, it is not practically feasible because of both the time and economic limitations of big-scale realistic experiments. Experimental studies are generally performed to define the vulnerabilities of structural elements (columns, beams, etc.) rather than the complete structures (Chong and Soong, 2000; Constantinou et al., 2000).

The type of method to choose for F.C. development depends not only on the objective of the assessment but also on the availability of data and technology (Ay, 2006). Kwon and Elnashai (2006) summarized the basic features and limitations of four methods of generating F.C.s, which are also given in Table 7-1. In this study, analytical F.C.s are developed for RC school buildings in Turkey.

Table 7-1 Features and Limitations of F.C.s (Kwon and Elnashai, 2006)

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

7.2 Ground Motion Data

As expressed in Nielson and Pang (2011), a fragility curve, conditioned on an intensity measure (IM), represents the probability of exceedance of a prescribed damage state for a structure or a type of structure.

The IM, i.e., the ground motion (GM) intensity, expresses the ability of the EQ to cause damage to structures. Therefore, selecting the proper GM is a challenging task for EQ engineering. Particularly for fragility studies, the correlation between the selected GM intensity and the structural deformation (response) demand requires elaborate consideration (Ay, 2006). The selection of GM intensity and the GM characteristics have a considerable effect on the resulting F.C.s.

The ground motion intensities in the F.C.s can be peak ground motion values, spectral quantities, modified Mercalli scale, etc. (Akkar et al., 2005). Peak Ground Velocity (PGV), Peak Ground Acceleration (PGA), Spectral Acceleration (Sa), and Spectral Displacement (Sd) are among the most frequently used IMs for fragility studies in the literature. PGV appears to be a suitable GM intensity parameter for describing deformation demands in structures that deform beyond the elastic range, and it is more indicative for defining the correlation between structural damage and GM intensity (Akkar et al., 2005). PGV usually reflects the effect of soil conditions very well during a large magnitude EQ (Wald et al., 1999). Akkar and Özen (2005) mentioned that PGV correlates well with the SDOF deformation demands. Besides, PGV primarily influences the seismic spectral response of medium period systems, approximately in the period range $0.5<T<2.0$ seconds (Sucuoğlu et al., 1999), which corresponds the most of the RC school buildings in this study. Therefore, based on the above discussion, PGV is chosen as the reference GM intensity parameter in this study.

PGV fits well for the RC school buildings, but one of the major drawbacks is that PGV does not consider the structural properties of the buildings. PGVconsiders the

EQ and site properties but not the structural properties of buildings. It is necessary to include an IM considering structural properties.

PSA is an IM considering both EQ and structural properties. Dhakal et al. (2006) mentioned that PSA is a more efficient intensity measure in comparison with PGA. Using Sa as the GM intensity parameter would give more confidence in the result or would require less number of records to generate results with the same level of confidence (Avşar, 2009). Since it also plays a vital role in the traditional seismic design approach (Akkar and Özen, 2005), PSA is chosen as the second GM intensity parameter in this study.

In order to generate reliable F.C.s, a set of GM records from different EQs with different magnitudes should be used. In this study, a group of 100 selected GMs having different seismological characteristics from different parts of the world were used in order to generate the F.C.s. The selected and used GM data consists of dense to firm soil records with surface magnitudes (Ms), ranging from 5,1 to 7,8. Detailed information about the GM data used in this study is given in Appendix H. The information in Appendix H contains the name, the date, the location of the recording station, component, the surface wave magnitude, distance to fault, scaling factor, peak ground acceleration (PGA), peak ground velocity (PGV), V/A ratio, effective peak acceleration (EPA), energy index (EI) and effective duration (Δt_{eff}) of the corresponding EQs.

Besides strong shaking, the characteristics of near-fault ground motions are linked to the fault geometry and the orientation of the traveling seismic waves (Sommerville, 2000). The primary characteristics of near-fault ground motions are the forward directivity and fling step effects which have caused severe structural damage in recent major EQs (Mavroeidis and Papageorgiou, 2003). As can be seen in Appendix H, none of the ground motions have near-fault effects.

The statistical properties of the selected GM set are given in Table 7-2 below.

Table 7-2 Statistical Properties of Selected GM Set

Property	**Range**	**Mean (μ)**	**COV.**
M_s	5,1-7,8	6,9	0,1
Distance (km)	1-234	25,6	1,7
PGA (g)	0,011-1,778	0,41	0,7
PGV (cm/s)	1,09-113,11	50,50	0,6
V/A (s)	0,032-0,375	0,143	0,6
EPA (g)	0,012-1,366	0,338	0,7
EI	2,92-228,28	106,49	0,6
Δt_{eff} (s)	3,19-47,57	13,53	0,6

49 of the selected 100 GM records are from the USA, and 17 of them are from Turkey. 85 of them have a surface wave magnitude of 6.5 or larger. The closest distance to the fault is 20 km or less for 75 of the GM records. 65 of them have PGA, and 64 of them have an EPA between 0.2 and 0.6 g. 81 of the GM records have an effective duration between 5 and 20 seconds. It can clearly be stated that the selected GM records for this study have a wide range of characteristics, and they are able to excite the selected RC school buildings in all hazard levels.

The histogram of Ms of the original 100 GM records is shown in Figure 7.2. The histogram of CD (closest distance) of the original 100 GM records is shown in Figure 7.3. Detailed information about the GM data is given in Appendix H.

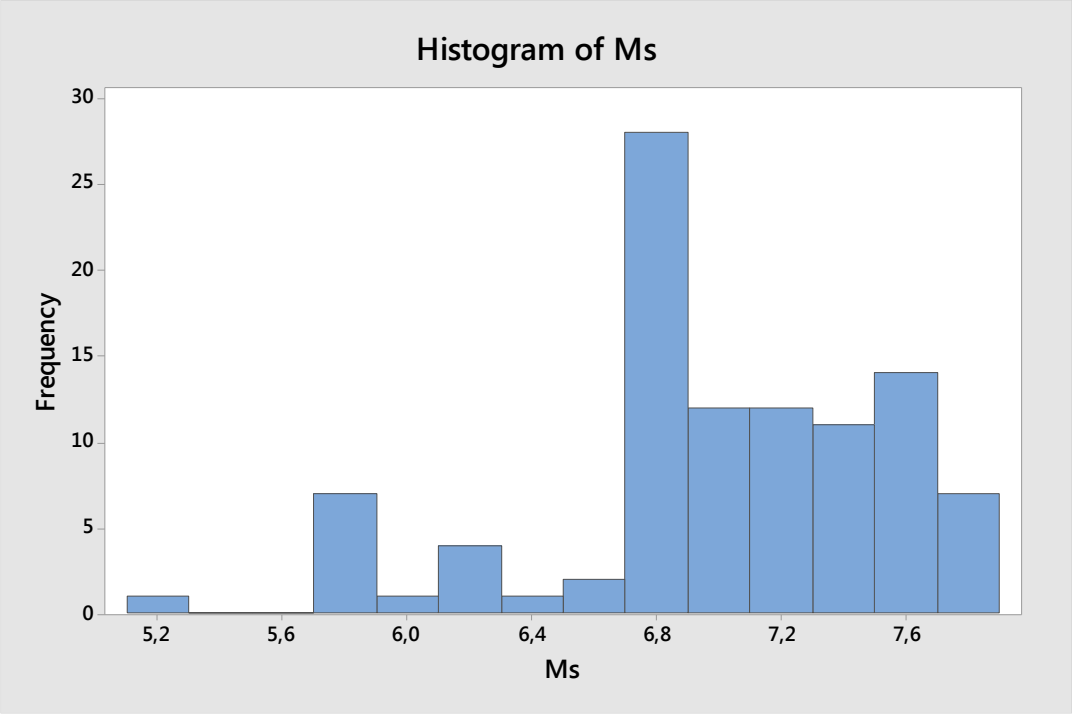

Figure 7.2. Histogram of Ms of GM records

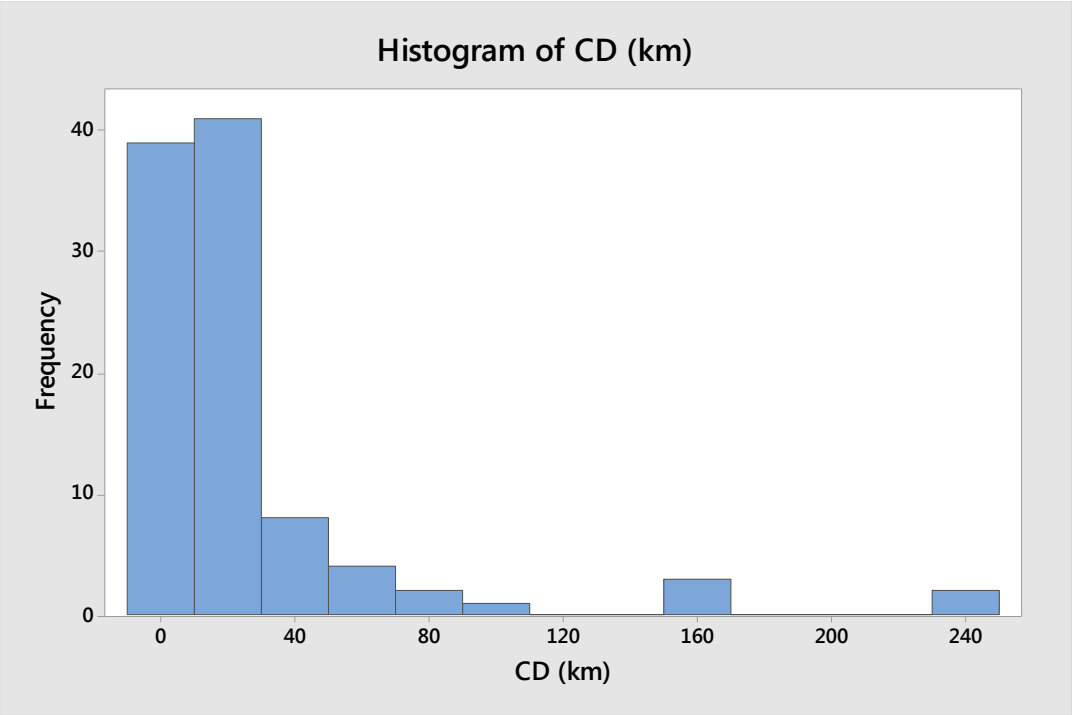

Figure 7.3. Histogram of CD of GM records

The dataset classification of selected 100 GM records is made by dividing the ground motion data into 12 bins with PGV intervals of 10 cm/s. The original PGV distribution of the data set can be seen in Figure 7-4 below.

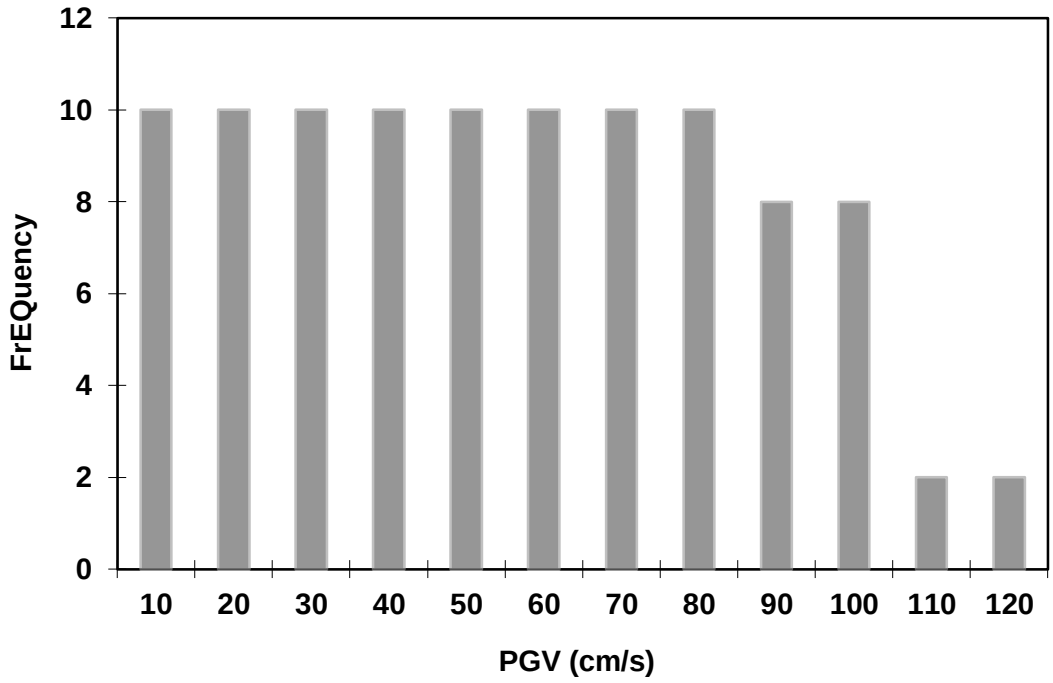

Figure 7-4. Original PGV distribution of the GM records

The purpose of such a classification is to observe an even distribution of response, so 20 new records are generated by scaling from the nearest bin in the time domain in order to compose an even distribution between the bins. The scaling of the records is formed from the nearest bin not to cause any distortion of the original characteristics of the GM records. In this way, the GM set contains 120 records totally covering a PGV range of 0-120 cm/s with PGV intervals of 10 cm/s. Then, this set was applied to both orthogonal directions of the selected 36 RC school buildings.

It is not possible to define a single PSA value for each GM record since PSA values also depend on the periods of the RC school buildings. In order to define the PSA values, 100 selected GM records are processed by Seismosignal software, and the

corresponding PSA values are compiled for every 0.02-second interval between 0-to-5 seconds period. Thereafter, with a simple code, the corresponding PSA values are determined for each RC building consistent with the period of the structure for each selected GM record. It is not possible to form even distributions of bins for fragility analysis by PSA; therefore, 12 bins are formed by considering the number of nonlinear time history analysis outcomes, as explained in the next chapter.

7.3 Generation of Fragility Curves

F.C.s and their types were described shortly in Section 7.1 above, and it was mentioned that in this study, analytical F.C.s are developed for RC school buildings in Turkey. The details of the generation of the F.C.s for RC school buildings are described in this section.

In order to generate F.C.s, structural, ground motion, and response parameters should be described and involved in the analysis. The construction practice may differ from each other in different countries, and these differences may cause significant deviations between the generated F.C.s. Therefore, obtaining the country-specific/local characteristics of Turkey and generating F.C.s in accordance with these local properties has substantial importance for the precise estimation of EQ damage and loss in Turkey (Ay, 2006).

Another critical point is that obtaining the structural parameters directly from the design parameters and layouts of related buildings for the generation of F.C.s may lead to unexpected deviations from the real response characteristics. Like many other countries, the construction practice in Turkey is far beyond complying with the designed structural requirements.

The statistical properties of RC school buildings in Turkey have been obtained. The structural and capacity curve parameters were calculated for 3-, 4- and 5-story buildings throughout the above chapters of this study. It should be mentioned that all structural input data included for the generation of F.C.s in this study are local and

as-built data. Structural variability for the generation of F.C.s is taken into account by using as-built properties and by using a large number of RC school buildings within the analysis process. So the building samples of each height category include real buildings with varying properties providing uncertainity in typical building properties such as span length, height, material property, member sizes, etc.

Generation of F.C.s generally compose two basic steps, which are pushover analysis in order to determine the structural capacity and the performance state limits; then nonlinear time history analysis in order to compute the responses of the related structures. By following these two basic steps, it is possible to determine the probability of exceedance of limit states for the related structural system.

The methodology used for the generation of F.C.s in this study is described step by step in Figure 7-5 below. Steps 1-5 of the methodology were described throughout the above chapters and sections.

Time history analysis (step 6) has been performed using both bilinear and Clough material models for equivalent SDOF, which will be described shortly below. The seismic load is given by ground motion time histories. F.C.s are derived separately for different number of stories, i.e., 3-, 4- and 5- story RC school buildings. As mentioned before, the number of stories is a prominent parameter influencing the vulnerability of existing RC buildings.

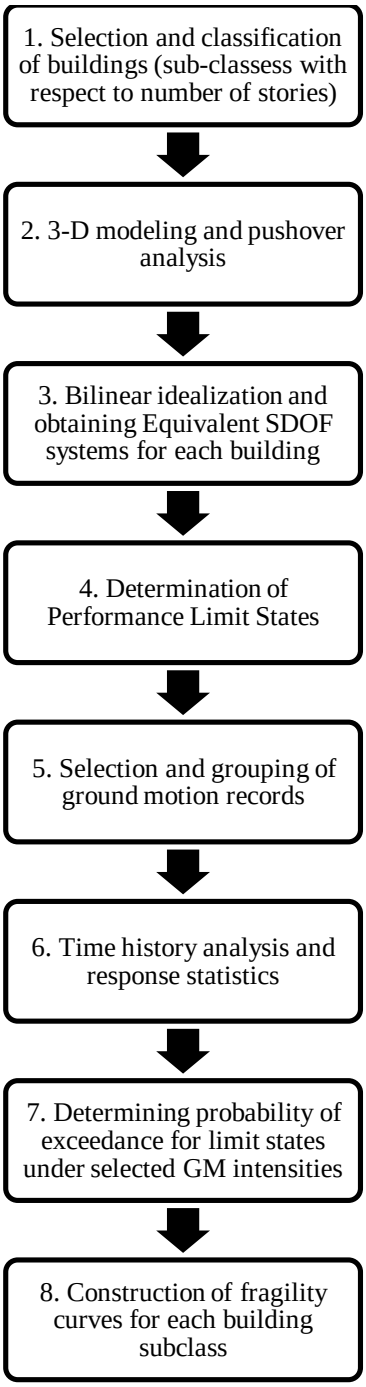

Figure 7-5. The methodology used for the generation of F.C.s

7.3.1 F.C.s wrt. PGV Using Bilinear Material Model

First, F.C.s for 3, 4- and 5-story RC school buildings have been derived with respect to PGV utilizing the above-described set of 120 GM records for nonlinear time history analysis with bilinear models. To derive the fragility curves for 3 limit states (IO, LS, and CP) for 3-, 4- and 5-story structures, an analytical approach assuming log-normally distributed capacity and demand was followed utilizing the probit method, which was codified by Baker (2015).

Utilizing derived parameters for models of selected RC school buildings, equivalent SDOF systems (with 5% damping ratio) were modeled in OpenSees software and nonlinear time history analyzes were performed employing the GM set. The GM recordset was applied to both two orthogonal directions of the selected 36 buildings. Accordingly, maximum displacement results were obtained from 8640 equivalent SDOF-based nonlinear time history analysis, and these results have been processed to determine the observed fractions of 3 performance states (IO, LS, and CP) in each bin. Afterward, a MATLAB script (Baker, 2015) was utilized to derive the fragility curves fitting to the observed fractions.

The inelastic dynamic characteristics of each building are represented by a bilinear equivalent SDOF system. The seismic displacement demand is obtained, subjecting such an SDOF system to time history analysis under the selected ground motions. The ESDOF displacement demands are then converted into building displacement demands by multiplying the first mode participation factor, Γ_1. Following displacement demand estimates, the seismic performance of each school building was evaluated using its own performance state limits.

Extracting maximum roof/top drift demands from the analysis results, each structure was marked to exhibit a particular damage state under each ground motion record. While defining the damage level of the structure, following Akkar et al. (2005), yield spectral displacement of each structure was taken as Immediate Occupancy (IO) performance level, whereas ultimate spectral displacement value was taken as

Collapse Prevention (CP) limit state. Three-quarters of the ultimate spectral displacement value was taken as Life Safety (LS) performance level as described in Section 4.5 above (Mazılıgüney et al., 2013).

There are 12 RC school buildings, i.e.,24 ESDOF models in each subclass. There exist 10 GM records in each bin, which makes a total of 240 response data for each bin. There are 12 bins, so the F.C.s for each subclass of RC school buildings are generated by using a total of 2880 response data.

The performance limit states of each ESDOF model of each building is determined for each GM record (i.e., for each EQ intensity). Afterward, the number of buildings reached or exceeded a specific performance state limit can be determined. The percentage of the buildings reached or exceeded (probability of exceedance) a specific performance state limit is determined by dividing the number determined number of response data by the total number of response data in each bin. The mean PGV values of each bin are plotted against the corresponding probability of exceedance values. Finally, the log-normal cumulative distribution function by employing the method of least squares is fitted to the scattered data in order to obtain the final F.C. for the related performance limit state. The perfoemance limit states were determined separately for each RC school building in Section 4.5 and are presented in Appendix D.

Figure 7-6, Figure 7-7, and Figure 7-8 demonstrate the fragility curves wrt. PGV, developed for the RC school buildings generated by using bilinear material models for 3, 4, and 5 story subclasses, respectively. The three curves in each figure represent the probability of exceeding the IO (red), LS (blue), and CP (green) performance limit states. Any point on the F.C. of a particular RC school building subclass indicates the probability of attaining or exceeding the corresponding performance/damage limit state due to the corresponding EQ intensity (Avşar, 2009).

As it is seen from F.C.s, damage probability for each damage state increases with increasing seismic demand.

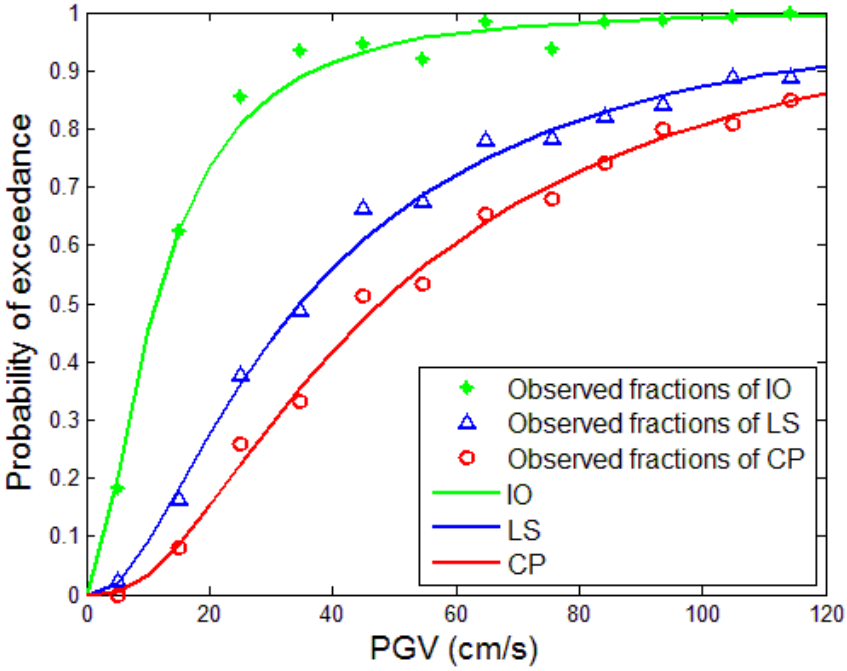

Figure 7-6. F.C. for 3 story RC school buildings using bilinear material model wrt. PGV

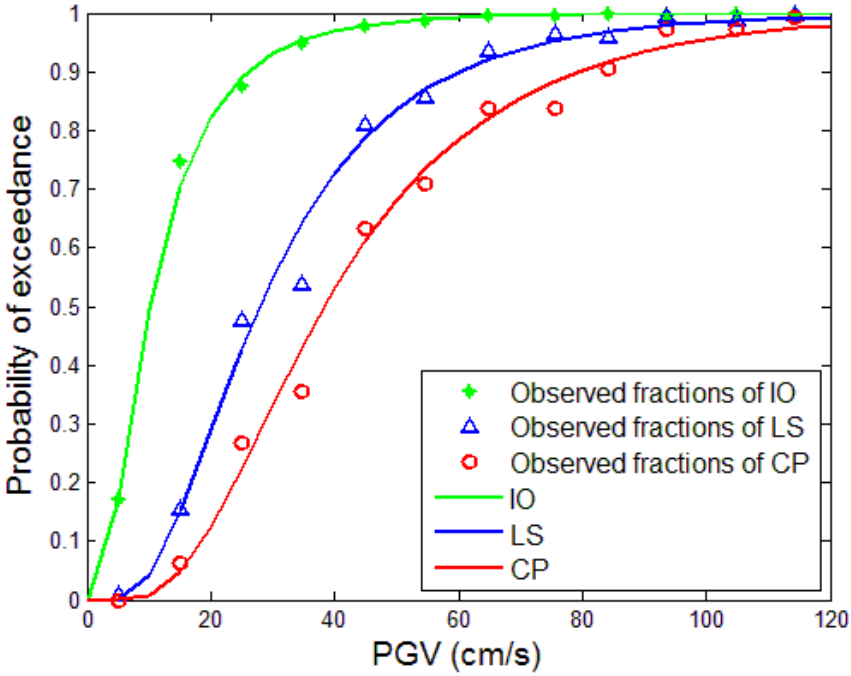

Figure 7-7. F.C. for 4 story RC school buildings using bilinear material model wrt. PGV

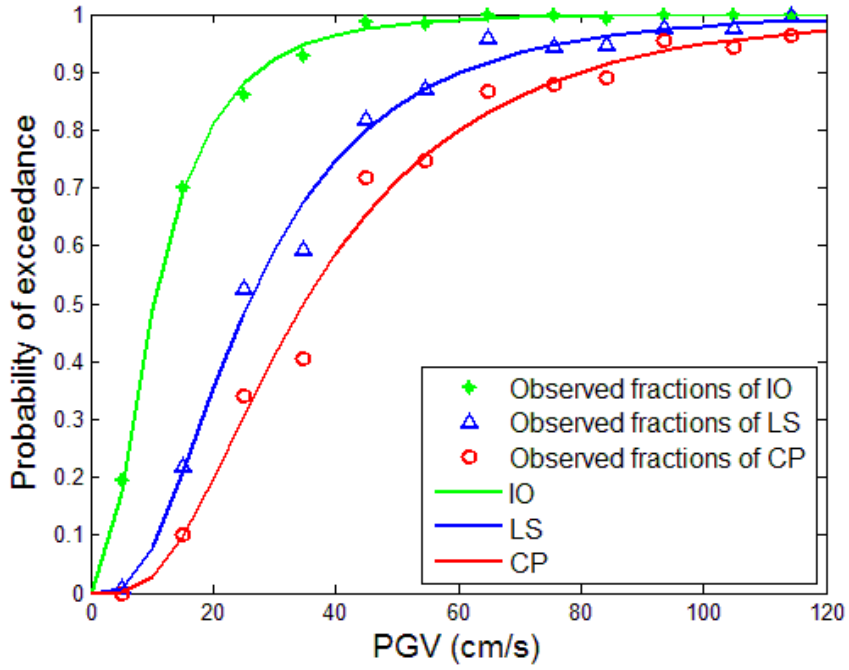

Figure 7-8. F.C. for 5 story RC school buildings using bilinear material model wrt. PGV

7.3.2 F.C.s wrt PSA Using Bilinear Material Model

As described in Section 7.2, it is not possible to define a single PSA value for each GM record since PSA values also depend on the periods of the RC school buildings. In order to define the PSA values, 100 selected GM records are processed by Seismosignal software, and the corresponding PSA values are compiled for every 0.02-second interval between 0-to-5 second periods. Thereafter, with a simple code, the corresponding PSA values are determined for each RC building consistent with the period of the structure for each selected GM record. 2 models for each 36 selected RC school buildings and 100 GM records form 7200 PSA values totally (i.e.,2400 for each subclass).

Since it is not possible to form even distributions of bins for fragility analysis by PSA; therefore, 12 bins are formed by considering the number of nonlinear time history analysis outcomes.

The same procedure described in section 7.3 above is applied to form F.C.s wrt. PSA and Figure 7-9, Figure 7-10, and Figure 7-11 demonstrate the F.C.s developed for the RC school buildings generated using bilinear material models for 3, 4, and 5 story subclasses, respectively.

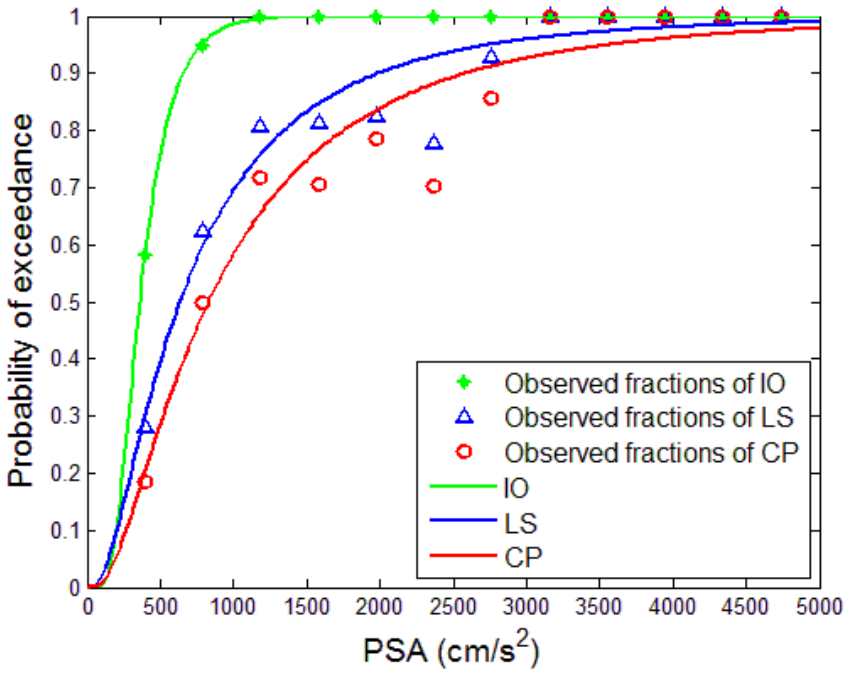

Figure 7-9. F.C. for 3 story RC school buildings using bilinear material model wrt. PSA

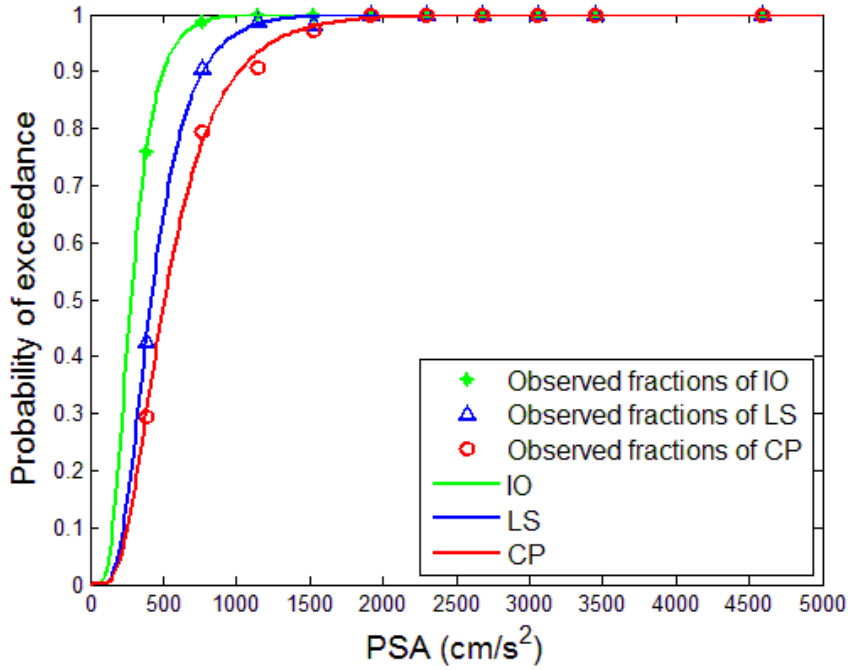

Figure 7-10. F.C. for 4 story RC school buildings using bilinear material model wrt. PSA

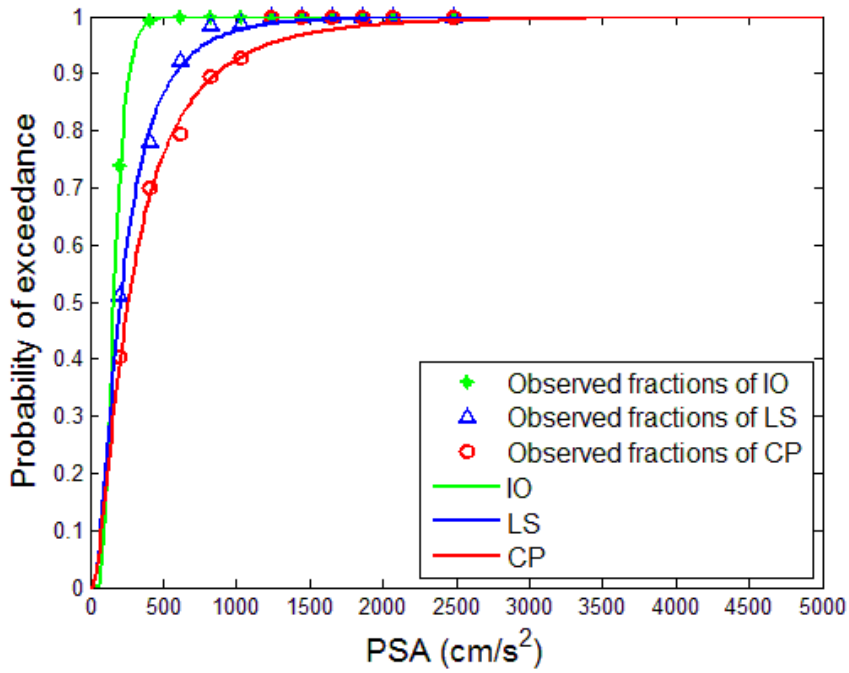

Figure 7-11. F.C. for 5 story RC school buildings using bilinear material model wrt. PSA

7.3.3 F.C.s wrt. PGV using Modified Clough Material Model

In his dissertation, Ay (2006) emphasized that strength degradation determines the stability of response and the rate of approaching failure. Strength degradation depends on many parameters such as the dissipated energy per cycle, the number of inelastic excursions (or low-cycle fatigue), and the plastic deformation range. The elastic stiffness of the structures degrades due to the yielding of the members. The cracking of concrete basically causes stiffness degradation in RC members under repeated cyclic loading. Increasing ductility gives rise to the degrading of the elastic stiffness (Ay, 2006).

The energy dissipation capacity of a building is computed as the area under its pushover curve. Unfortunately, the capacity curves may overestimate the ultimate ductility of the columns, since the pushover analysis cannot take the strength degradation due to cyclic loading into account.

Strength and stiffness degradation are generally not considered by the F.C.s developed in the existing studies, and their use for the low strength RC buildings stock of developing countries is highly questionable. In most seismic demand studies employing hysteresis models, the models generally have a non-degrading (ND) backbone (bilinear model) curve and/or ignore stiffness deterioration (Ibarra et al., 2005) due to the fact that the elapsed time, computational difficulty, and versatility of the models increase with increasing complexity.

The FEMA-440 (2005) publication on nonlinear seismic analysis has given significant importance to strength and stiffness degradation and emphasizes that the current procedures for accounting degradations are not clear. Then, a FEMA-440 (2005) connected project, FEMA-P440A (2009), concentrated on the effects of strength and stiffness degradation on seismic response. FEMA-P440A (2009) summarized the previous works and examined the phenomena in detail by using analytical studies. Researchers may refer to FEMA-P440A (2009) for further and detailed information. Altoontash (2004), Kurtman (2007), Kale (2009), and Demirci

(2014) are some of the researchers who studied hysteresis and degrading behavior, and their dissertations are valuable references for other researchers.

One of the well-known and commonly used material models including strength and stiffness degrading behavior of RC is the Clough (Clough and Johnston, 1966; Mahin and Bertero, 1976) Model, which is also mentioned by FEMA-P440A (2009). The Clough (stiffness degrading, SD) material model and the modified (stiffness and strength degrading, SSD) versions are used commonly for seismic response studies. The Clough model uses a bilinear backbone curve that represents the stiffness degradation during the reloading stage (Clough and Johnston, 1966). Mahin and Bertero (1976) modified the Clough model by adding stiffness degradation in the unloading stage (Kale, 2009).

In this study, the strength and stiffness degrading model is employed for the generation of following F.C.s in order to take into account the inelastic behavior and compare it with the bilinear F.C.s. Modified Clough Peak Oriented material model, which is available in OPENSEES (coded by Arash Altoontash and Gregory Deierlein) material model library, is used in order to define and employ the strength and stiffness degradation of RC within the analysis. The model was studied by Altoontash (2004) in his dissertation and then coded for OPENSEES in 2012. Since it is not the scope of this study, the material model is not explained in detail. Researchers may refer to the dissertation of Altoontash (2004) and online OPENSESS publications. The loading envelope for peak oriented models are shown in Figure 7-12 (Ibarra, 2003). The hysteresis curve of the material model can be seen in Figure 7-13 (Altoontash, 2004) below.

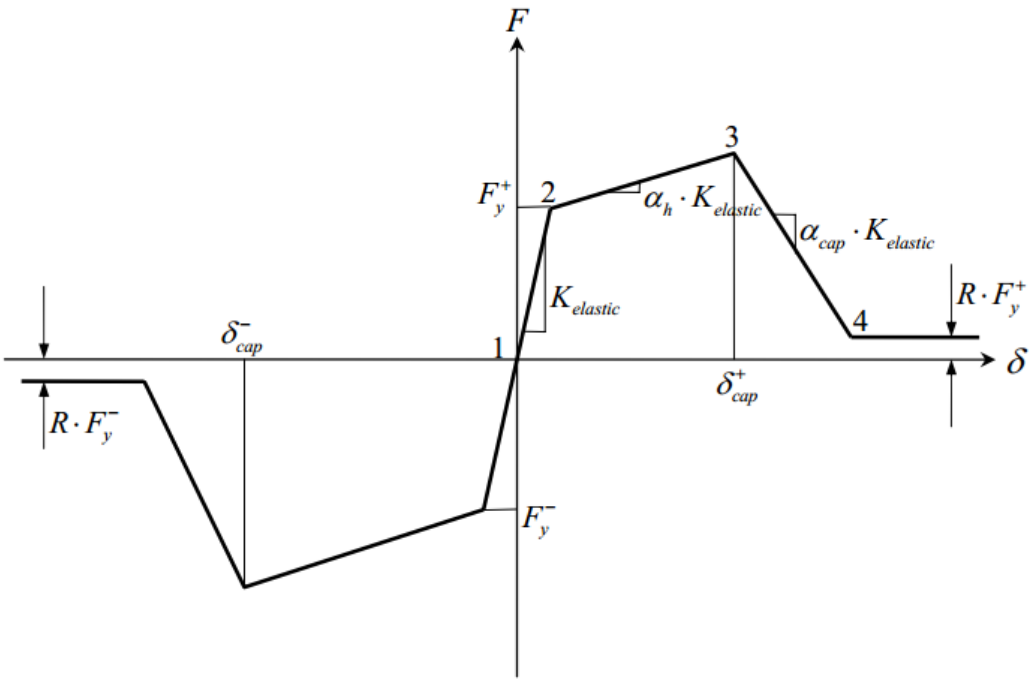

Figure 7-12. Loading envelope for peak oriented models (Ibarra, 2013)

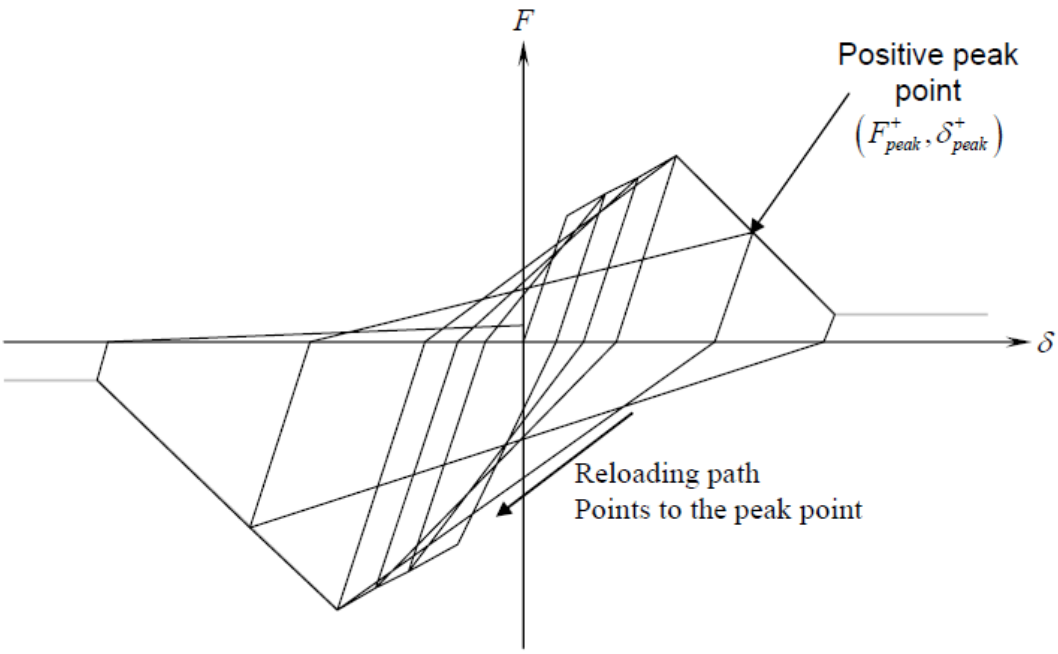

Figure 7-13. Modified Clough peak oriented material model (Altoontash, 2004)

The envelope key points which can be seen in Figure 7-12, are determined by user-defined inputs, and the corresponding equations are listed in Table 7-3 (Altoontash, 2004) below.

Table 7-3 Key points of force-deformation envelope of peak oriented models (Altoontash, 2004)

Point ID	**Deformation/Strain**	**Force/stress**	**Stiffness**
1	0	0	
2	$\delta_y^+ = \frac{F_y^+}{K_e}$	F_y^+	K_e
3	δ_{cap}^+	$F_{cap}^+ = F_y^+ + \alpha_h . K_e . (\delta_{cap}^+ - \delta_y^+)$	$\alpha_h \cdot K_e$
4	$\delta_{res}^+ = \delta_{cap}^+ + \frac{(R \cdot F_y^+ - F_{cap}^+)}{\alpha_{cap} \cdot K_e}$	$F_{res}^+ = R \cdot F_y$	$\alpha_{cap} \cdot K_e$
5	$> \delta_{res}^+$	$F_{res}^+ = R \cdot F_y$	0

The parameters used in the Modified Clough peak oriented material model within the OPENSEES software are user-defined or depends on the ESDOF model. The user-defined parameters are determined after several trials, and the most suitable parameters contributing to meaningful hysteresis curves and consistent with previous studies (Altoontash, 2004; Kurtman, 2007; Kale, 2009 and Demirci, 2014) are selected for analysis. The parameters used in the OPENSEES software are described in Table 7-4 below.

Table 7-4 Parameters used in the OPENSEES software for Modified Clough peak oriented material model

Symbol	Defined/Used
tag	1
K_e	Model
F_y^+	Model
F_y^-	Model
α_h	Model
R	0,12
α_{cap}	0,012
δ_{cap}^+	Model
δ_{cap}^-	Model
λ_S	0,04
λ_K	0,04
λ_A	0
λ_D	0
c_S	1
c_K	1
c_A	0
c_D	0

The same procedure described in section 7.3 above is applied to form F.C.s wrt. PGV and Figure 7-14, Figure 7-15, and Figure 7-16 demonstrate the F.C.s developed for the RC school buildings generated by using Modified Clough peak oriented material model for 3, 4, and 5 story subclasses, respectively.

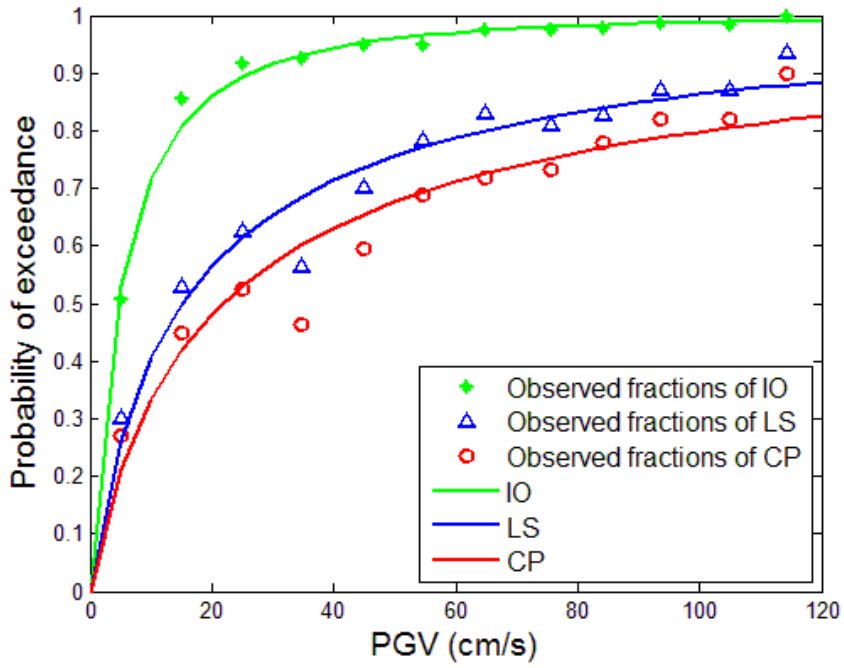

Figure 7-14. F.C. for 3 story RC school buildings using Modified Clough peak oriented material model wrt. PGV

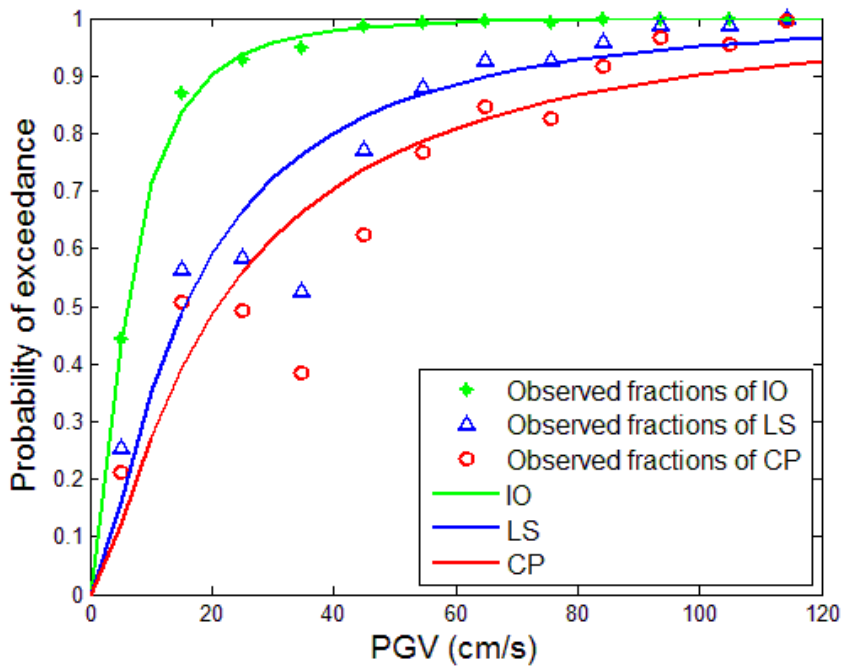

Figure 7-15. F.C. for 4 story RC school buildings using Modified Clough peak oriented material model wrt. PGV

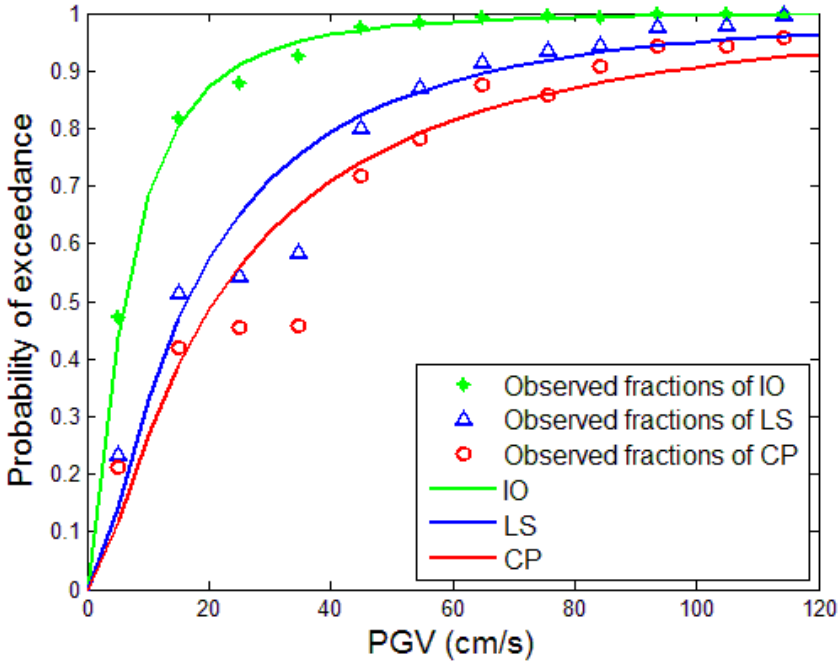

Figure 7-16. F.C. for 5 story RC school buildings using Modified Clough peak oriented material model wrt. PGV

7.3.4 F.C.s wrt. PSA Using Modified Clough Material Model

The same procedure described in section 7.3 above is applied to form F.C.s wrt. PSA and Figure 7-17, Figure 7-18, and Figure 7-19 demonstrate the F.C.s developed for the RC school buildings generated by using Modified Clough peak oriented material model for 3, 4, and 5 story subclasses, respectively.

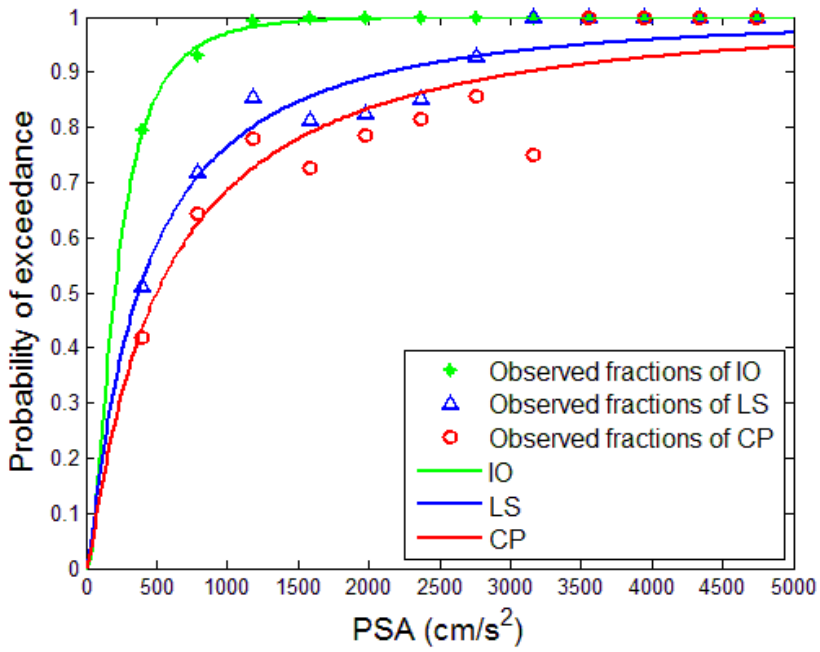

Figure 7-17. F.C. for 3 story RC school buildings using Modified Clough material model wrt. PSA

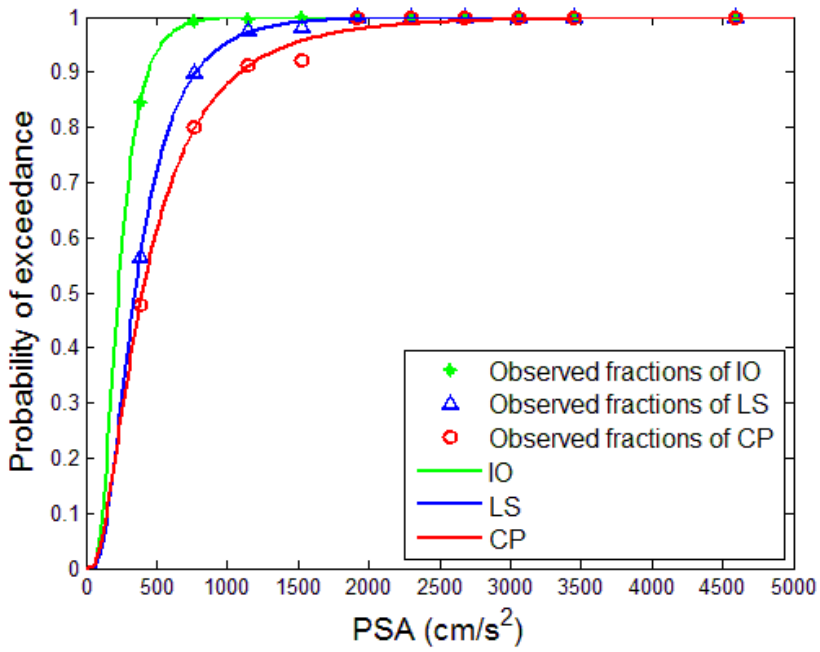

Figure 7-18. F.C. for 4 story RC school buildings using Modified Clough material model wrt. PSA

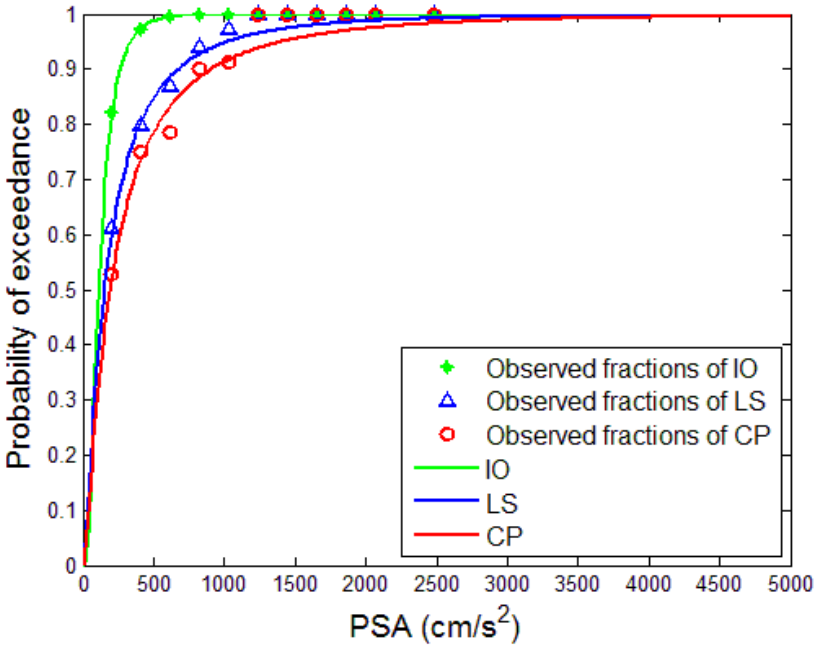

Figure 7-19. F.C. for 5 story RC school buildings using Modified Clough material model wrt. PSA

7.3.5 Discussion and Comparison of F.C.s

F.C.s are generated for 3, 4, and 5 story RC school buildings wrt. EQ intensity and also wrt. Bilinear and Modified Clough material models. In order to compare the F.C.s generated wrt. Bilinear and Modified Clough material models, both curves are shown on the same graph wrt. number of stories. The dashed curves represent F.C.s generated by the Modified Clough material model, while the continuous curves represent the F.C.s generated by bilinear material models.

F.C.s generated by both Bilinear and Modified Clough material models wrt. PGV are shown in Figure 7-20, Figure 7-21, and Figure 7-22 for 3, 4, and 5 story RC school buildings, respectively.

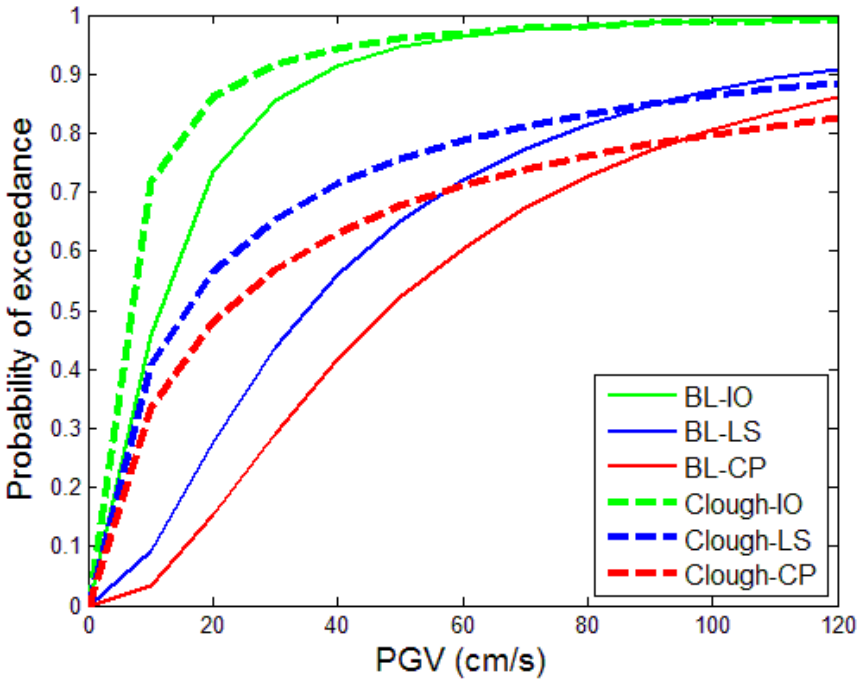

Figure 7-20. Comparison of F.C.s generated by Bilinear and Modified Clough Material Models wrt. PGV for 3 story RC school buildings

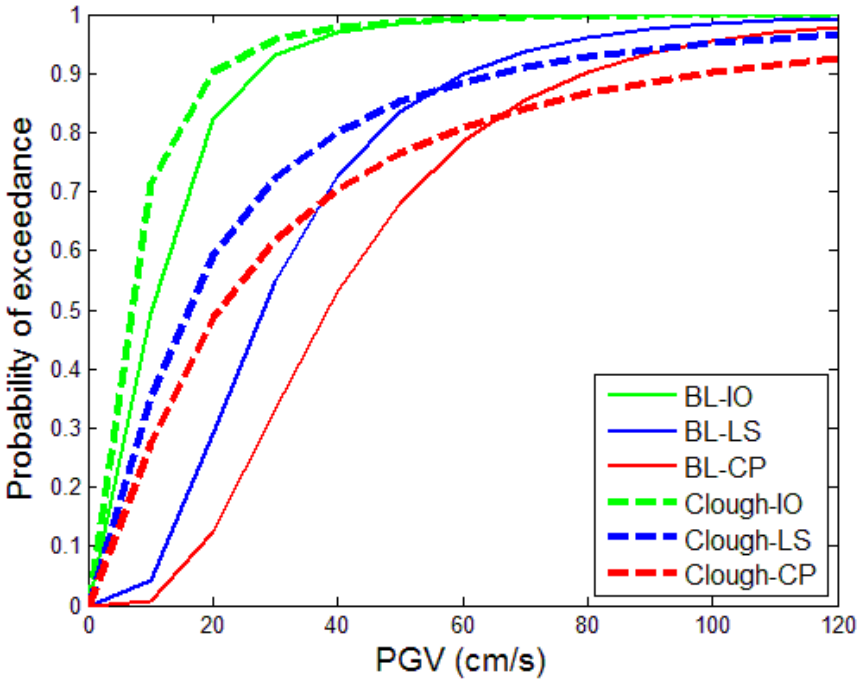

Figure 7-21. Comparison of F.C.s generated by Bilinear and Modified Clough Material Models wrt. PGV for 4 story RC school buildings

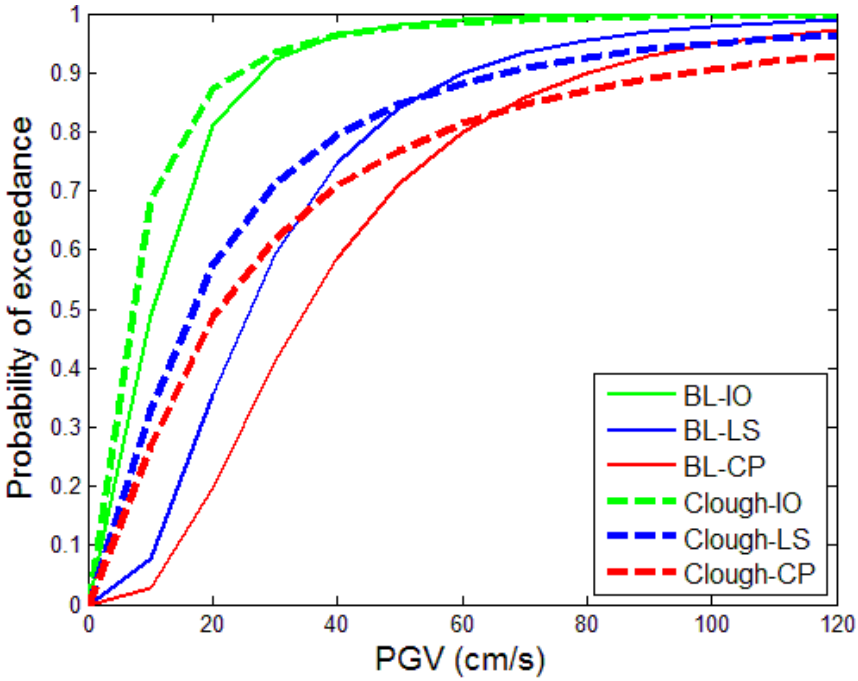

Figure 7-22. Comparison of F.C.s generated by Bilinear and Modified Clough Material Models wrt. PGV for 5 story RC school buildings

F.C.s generated by both bilinear and Modified Clough material models wrt. PSA are shown in Figure 7-23, Figure 7-24, and Figure 7-25 for 3, 4, and 5 story RC school buildings, respectively.

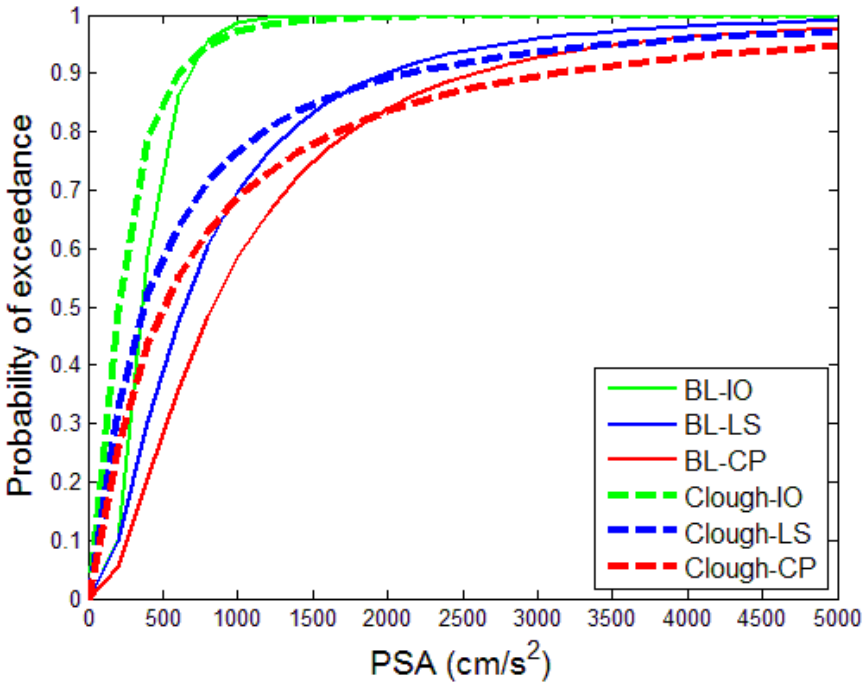

Figure 7-23. Comparison of F.C.s generated by Bilinear and Modified Clough Material Models wrt. PSA for 3 story RC school buildings

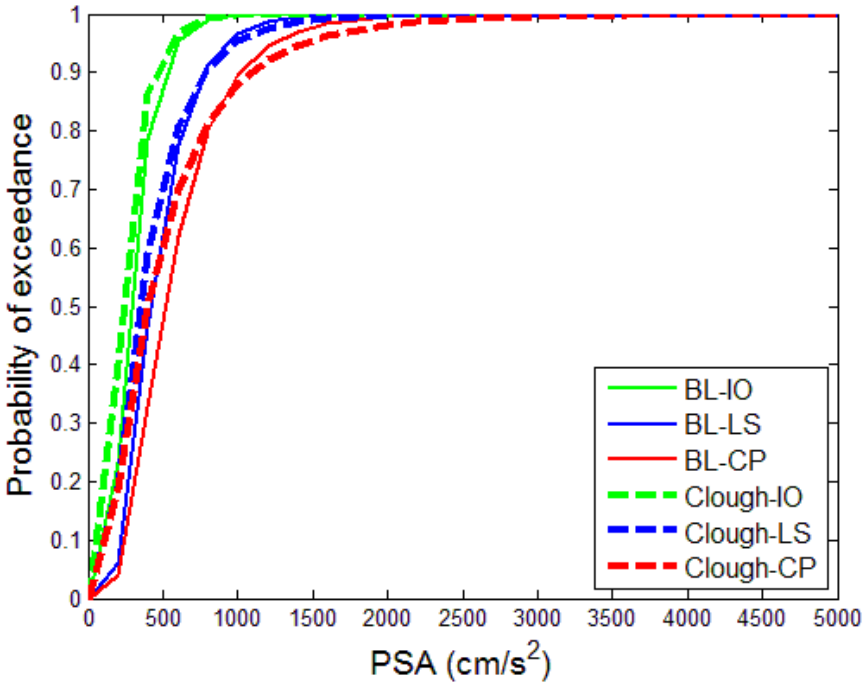

Figure 7-24. Comparison of F.C.s generated by Bilinear and Modified Clough Material Models wrt. PSA for 4 story RC school buildings

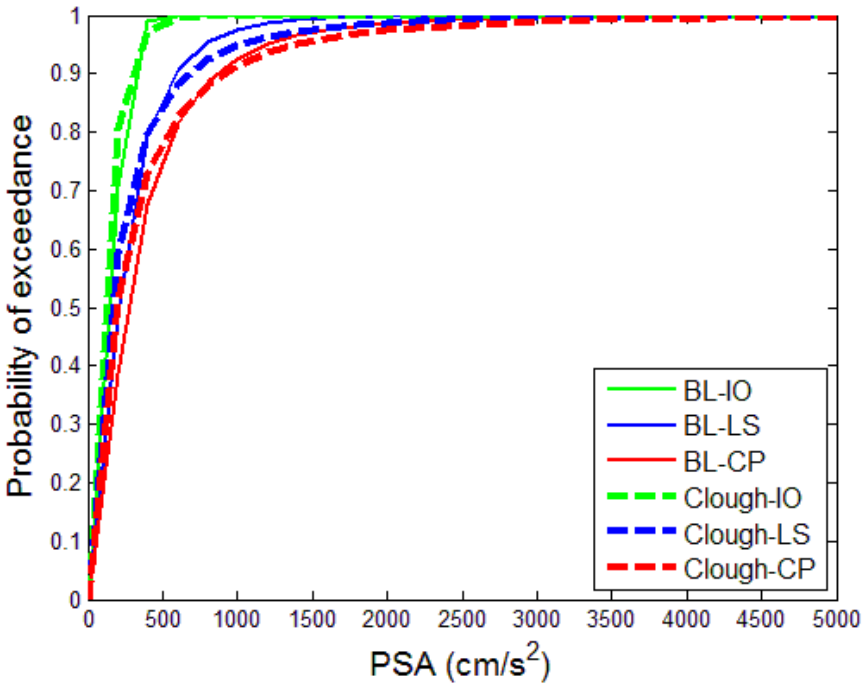

Figure 7-25. Comparison of F.C.s generated by Bilinear and Modified Clough Material Models wrt. PSA for 5 story RC school buildings

The number of stories of RC school buildings was selected as the primary parameter of the seismic performance, and the F.C.s were generated wrt. number of stories. The generated F.C.s revealed that, number of stories affect the probability of exceedance of all performance states for all cases. EQ vulnerability of RC school buildings increases with increasing number of stories.

The probability of exceedance curves for IO performance state is generally distant from the curves of LS and CP performance states, whereas the probability of exceedance curves for LS and CP performance states are close to each other for RC school buildings. That is why the performance limit definitions are likewise. IO and LS performance limits are distant from each other, while LS and CP limits are close.

Bilinear and Modified Clough F.C.s have significant differences until the probability of exceedance of 80 %, and Modified Clough-based curves indicate a larger probability of exceedance values. Two curves nearly coincide at and after 80% of probability of exceedance. It is evident that there are significant differences between

the Bilinear and Modified Clough material models based on probability of exceedance curves. Hence, the degradation characteristics of the structural model seem to have a significant influence on the final F.C.s.

It is important to note here that maximum values of the IMs considered and the number of records utilized or available in each bin throughout GM set formation and F.C. generation would undoubtedly affect the outcomes of the risk study, which should be further investigated as it is stated by Mazılıgüney et al., 2013.

7.4 Comparison of F.C.s with Bingöl Database

On May 1, 2003, an EQ of moment magnitude 6.4 and with a depth of 6 km occurred in Bingöl at 03:27 (A.M.) local time. The collapse of the Çeltiksuyu Primary School Dormitory building in Bingöl EQ, which killed 84 students and a teacher, is an obvious and tragic evidence of the seismic vulnerability of RC school buildings in Turkey. A photograph of the Çeltiksuyu Primary School taken the following morning and published by nearly all of the newspapers of Turkey can be seen in Figure 7-26 below.

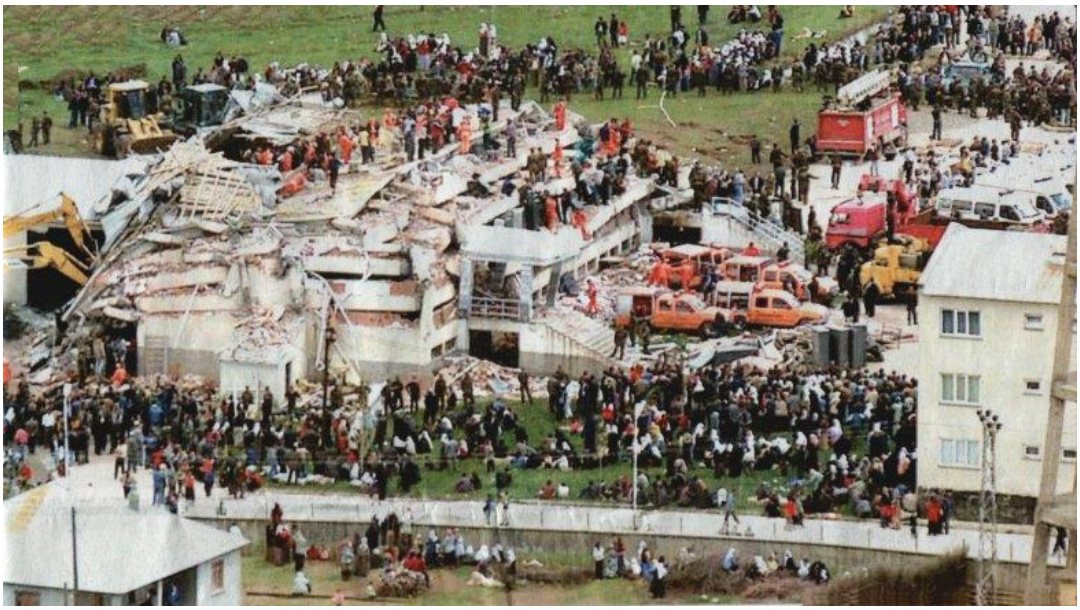

Figure 7-26. Photo of the Çeltiksuyu Primary School after the May 1, 2003, Bingöl EQ

Bingöl is located almost at the intersection of the North Anatolian Fault and East Anatolian Fault, and the total number of fatalities is 168 wrt. official authorities (Özcebe et al., 2003). The active faults of the region can be seen in Figure 7-27.

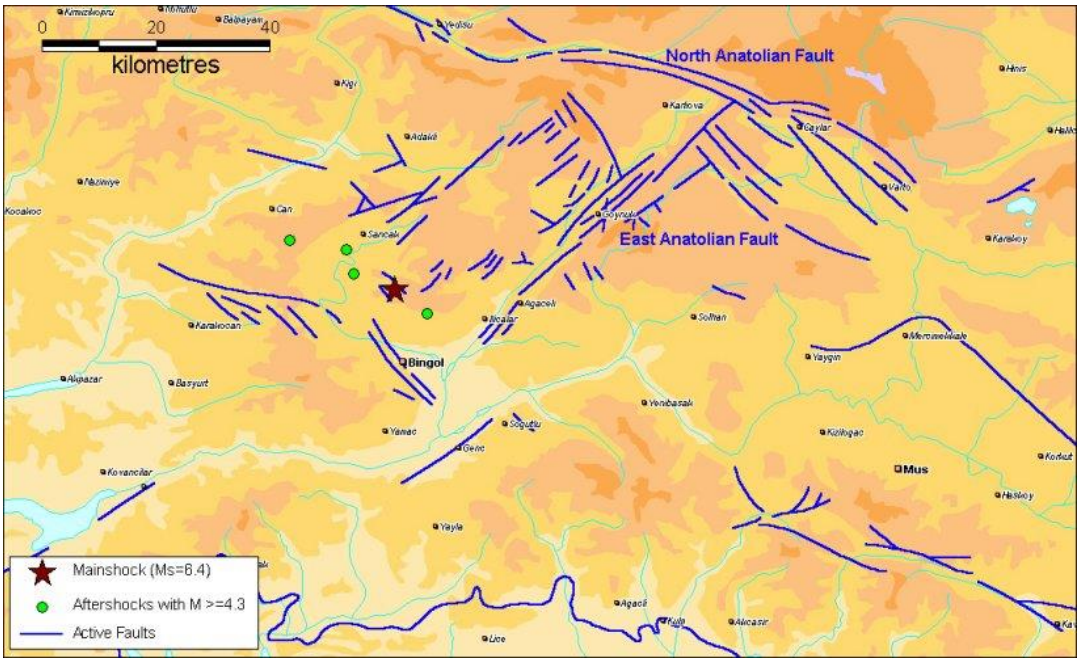

Figure 7-27. Active faults of the Bingöl region and the centers of main and aftershocks of May 1, 2003 EQ (Özcebe et al., 2003)

Immediately after the Bingöl EQ, technical reconnaissance teams composed of academics, researchers, and practicing engineers arrived at the disaster site in order to examine the impacts of the event. The teams were coordinated with each other and studied different buildings, including RC school buildings. The METU team studied 23 (13 of them 3 stories and 10 of them 4 stories) RC school buildings and tabulated the related structural data and the observed damage for the buildings.

The flowchart of the work done for the building set in Bingöl is described below:

- Consider the major Bingöl (1 May 2003, M_w=6.4) EQ that affected the reinforced concrete school buildings under consideration.
- Find the corresponding PGV and Sa values for each building by using appropriate attenuation relationships (the distances of school buildings to the related fault and the soil properties are considered within attenuation relationships).

- Obtain damage probability values for each RC school building by using the generated F.C.s and related PGV and Sa values.
- Compare the obtained results with the actual (observed) performance states of the RC school buildings.

Since the observed damage data was not enough to make a comprehensive comparison with obtained fragility curves, 3- and 4-story buildings were compared together. Among 23 RC school buildings, 2 of them have negligible or light damage corresponding to IO performance state, while 6 of them have moderate and 12 of them heavy damage level corresponding to LS and CP performance states, respectively. 3 of the buildings had been observed to be totally collapsed. The geographical coordinates of the observed RC school buildings in Bingöl were obtained, and corresponding site PGV values were calculated by using attenuation relations of Akkar et al. (2014). The probability of exceedance values for each building from each fragility curve were obtained, and then, the average values are calculated. The comparison data can be seen in Table 7-5. (Mazılıgüney et al., 2013)

Table 7-5 Comparison of observed and derived fragility data

Performance State	Probability Of Exceedance (%)	
	Observed	Obtained from Fragility Curves
IO	91.3	83.5
LS	65.2	39.2
CP	13.0	22.9

The difference between the observed and the derived probability of exceedance values are reasonable for IO and CP performance states, but the difference for LS performance state is not tolerable and not on the safe side. The difference can be attributed to the varying construction practices of different regions of Turkey.

The differences between predicted and observed fragilities are less than 10 percent at both damage limit states in the 3 and 4 story buildings. Considering the uncertainties inherent in the fragilities and randomness of ground intensities, these results can be accepted as satisfactory, suggesting that the proposed procedure can be implemented to large building stocks of RC school buildings in Turkey for loss estimation.

7.5 Fragility Curves under Repeated Earthquakes wrt. PGV

The buildings damaged under an EQ may additionally be exposed to the aftershocks or different earthquakes. In most of cases, these buildings are not retrofitted until the second EQ occurrence. Damaged buildings need to be assessed separately because they will soften, and the response under the next EQ will not be identical to the undamaged ones. Like the main shock-aftershock, sequential EQs are observed in the medium strongly active seismic regions such as; Turkey, Mexico, Italy, Japan, and California. (Mazılıgüney et al., 2017)

Many researchers investigated these phenomena for different types of structures. Moustafa and Takewaki (2010 and 2011) studied the critical ground motion sequences for modeling the inelastic response of SDOF structures. Jalayer and Ebrahimian (2017) studied cumulative damage due to aftershocks for existing reinforced concrete moment-resisting frames. Ruiz-García and Negrete-Manriques (2011) evaluated drift demand for mainshock and aftershock sequential EQs for existing steel frames. Jeon (2013) studied the aftershock vulnerability assessment for RC structures in California. Hatzigeorgiou and Beskos (2009), Zhai et al. (2014), and Kimura et al. (2010) studied the inelastic response of SDOF structures subjected to repeated EQs. Uma et al. (2011) compared mainshock and aftershock fragility curves of New Zealand and US building stock. (Mazılıgüney et al., 2017)

Rehabilitation of the critical structures such as RC school buildings after an EQ is usually not rapid. Most of these buildings are subjected to the second EQ without

any retrofitting and are collapsed under the next EQ Consequently, there is also a need to derive the F.C.s for such damaged structures to determine the ones that need an urgent retrofit. This section of the study concentrates on the derivation of F.C.s for RC school buildings in Turkey due to repeated EQs. (Mazılıgüney et al., 2017)

To investigate the effect of repeated EQ on fragility curves, the ground motions were sequentially applied to the SDOF models described above. After the first record, a 40-second noise (i.e., zero acceleration) was imposed on the damaged SDOF model to force the vibration of the structure to get damped (i.e., the velocity of the structure is approaching zero) and making the SDOF model ready for the second EQ effects. This process has been repeated for 120x120 ground motion records yielding 14400 nonlinear time history analyses for each SDOF. The maximum top drift values were extracted from these cumulative actions, and corresponding limit states have been determined. After this step, PGV-based fragility surfaces obtained from the interpolated values of probability of exceedance values determined by frEQuency analysis have been generated. (Mazılıgüney et al., 2017)

The outcomes of this section are illustrated by the below figures, whereas fragility surfaces display the variation of the probability of exceedance values with repeated EQ action more clearly. The “x” and “y” axes of the following figures of this section correspond to the intensity of the first and second earthquakes in terms of PGV, respectively.

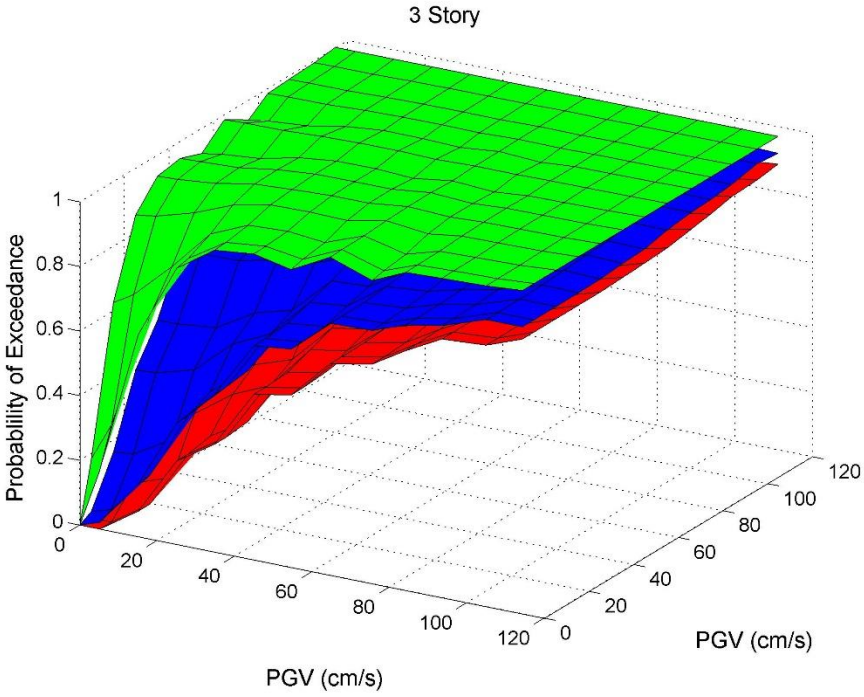

Figure 7-28. Fragility surfaces for 3 story RC school buildings

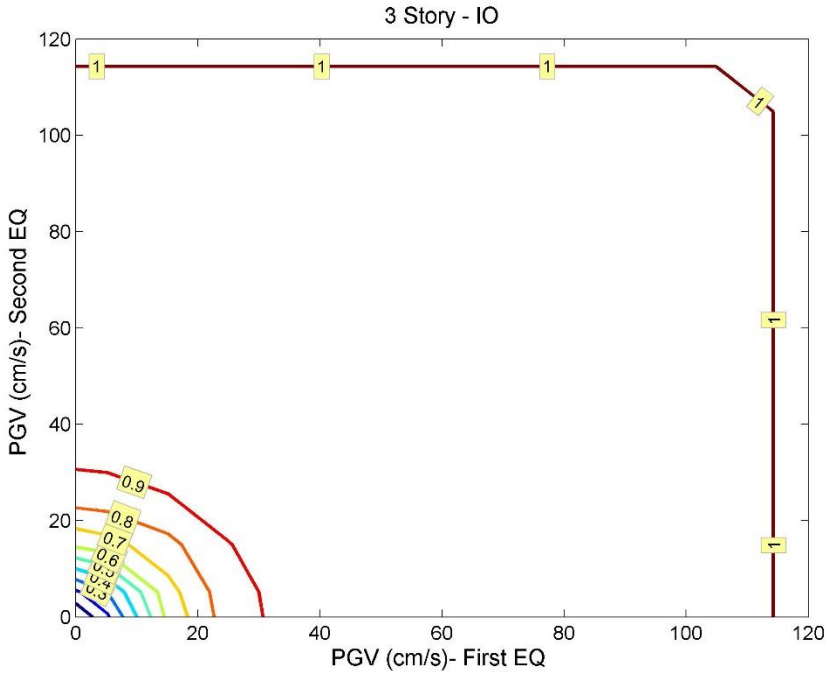

Figure 7-29. Contour plots of the limit state IO (for 3 story RC school buildings)

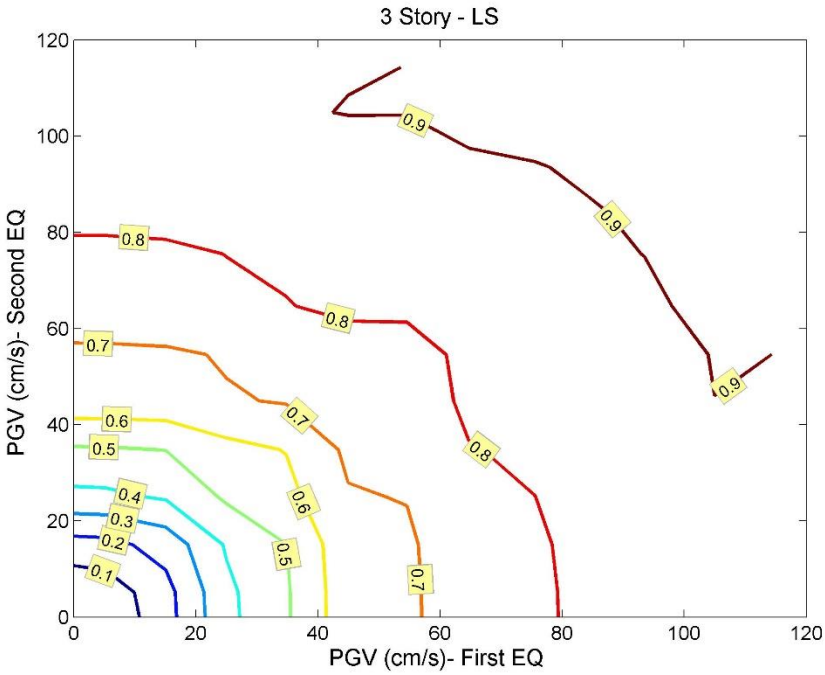

Figure 7-30. Contour plots of the limit state LS (for 3 story RC school buildings)

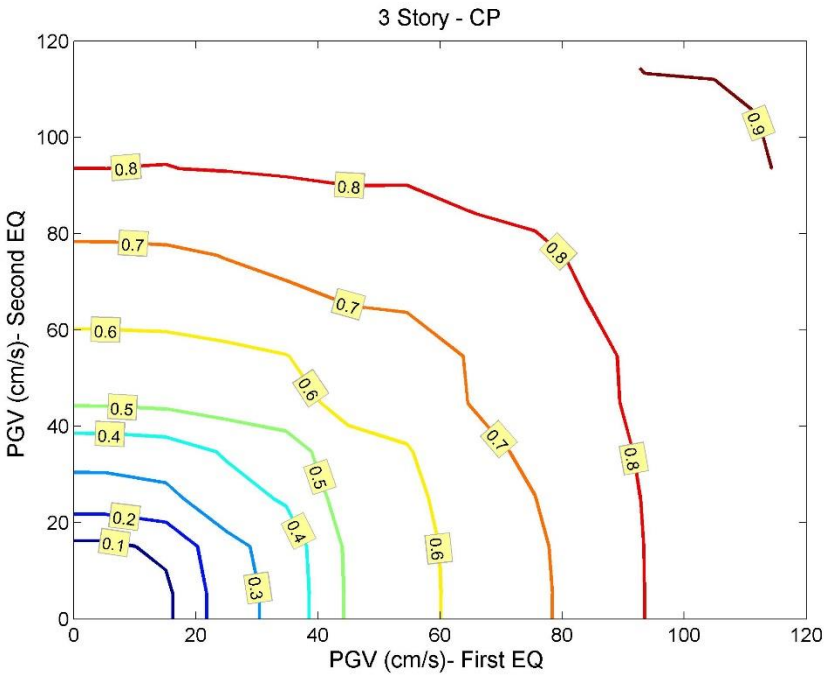

Figure 7-31. Contour plots of the limit state CP (for 3 story RC school buildings)

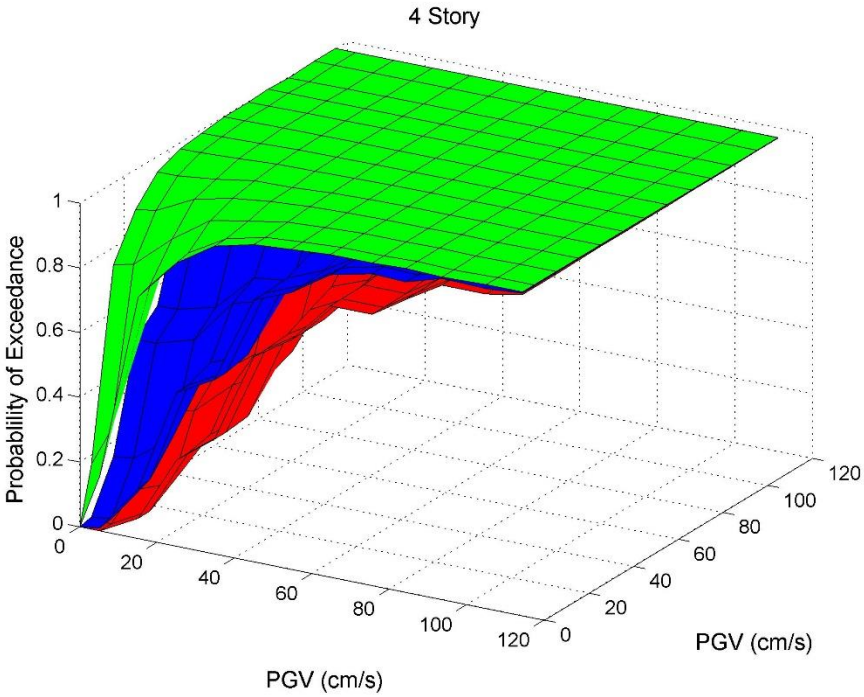

Figure 7-32. Fragility surfaces for 4 story RC school buildings

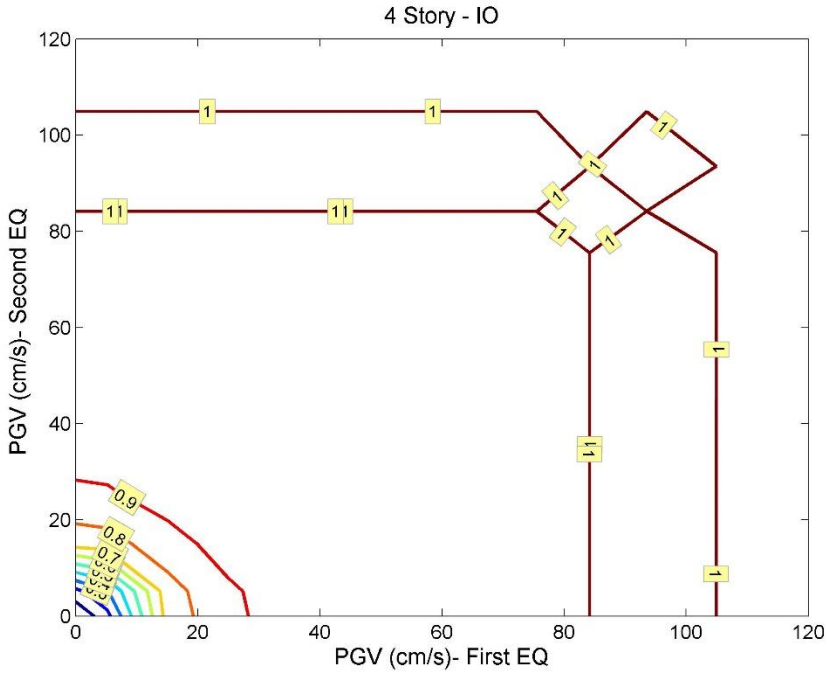

Figure 7-33. Contour plots of the limit state IO (for 4 story RC school buildings)

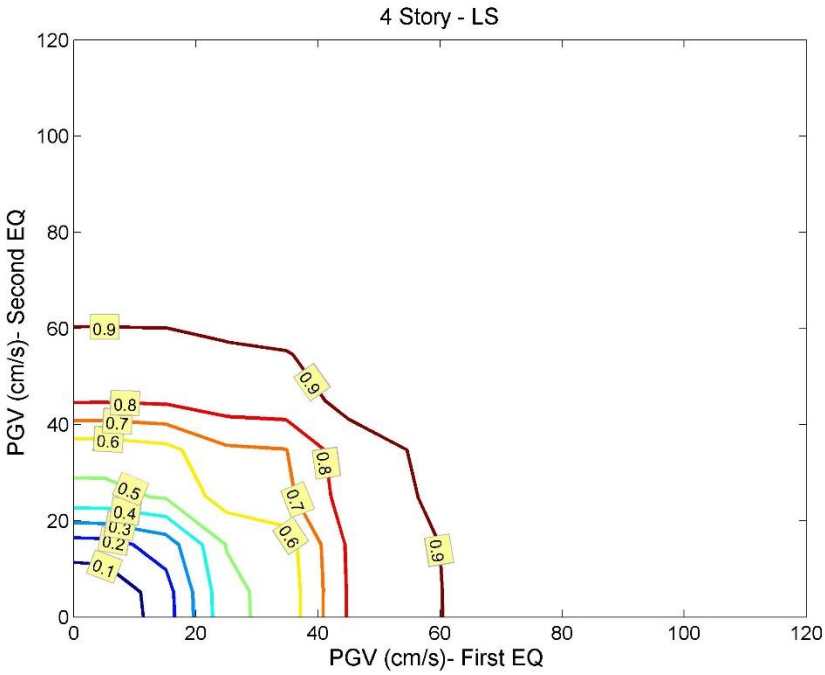

Figure 7-34. Contour plots of the limit state LS (for 4 story RC school buildings)

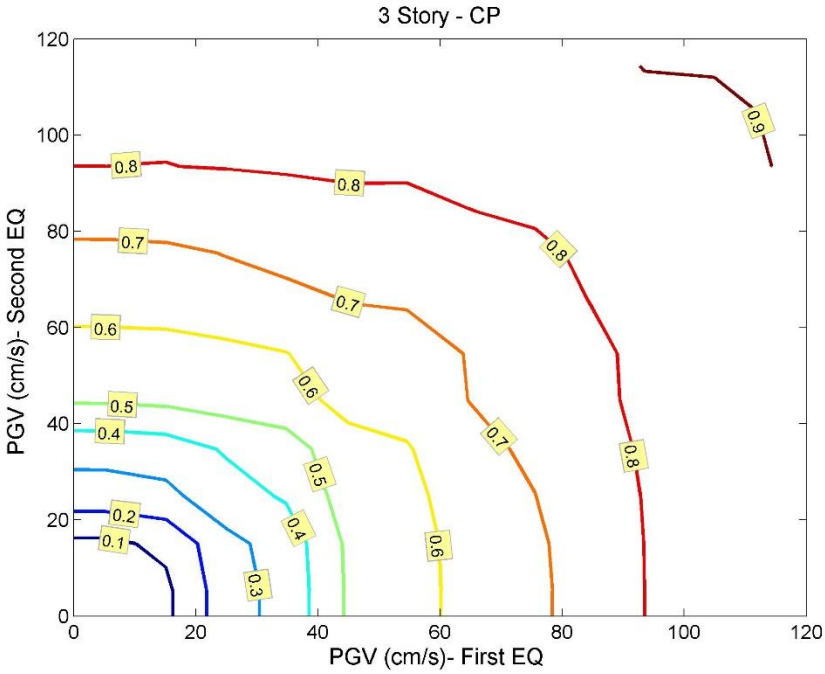

Figure 7-35. Contour plots of the limit state CP (for 4 story RC school buildings)

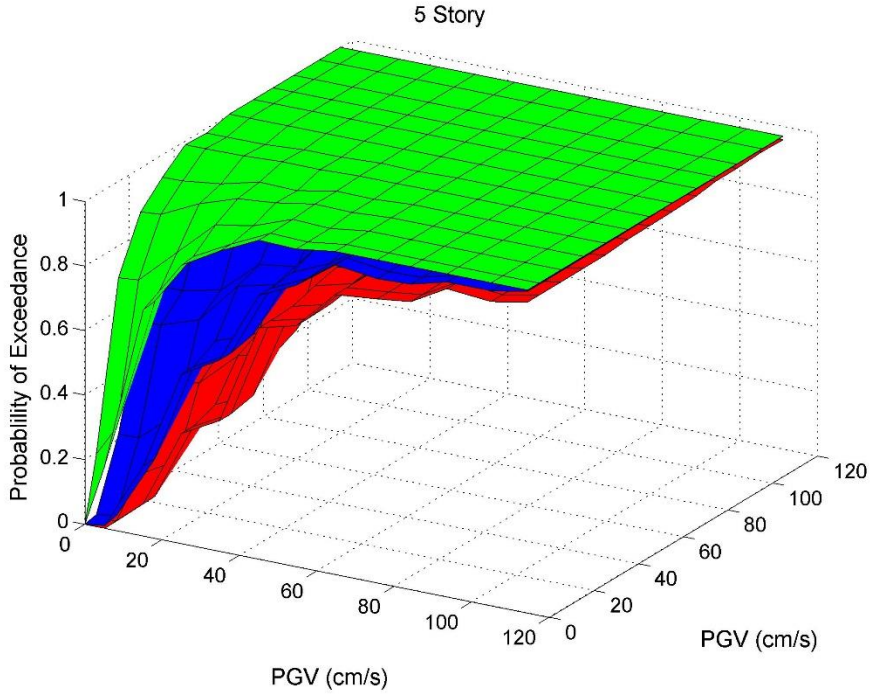

Figure 7-36. Fragility surfaces for 5 story RC school buildings

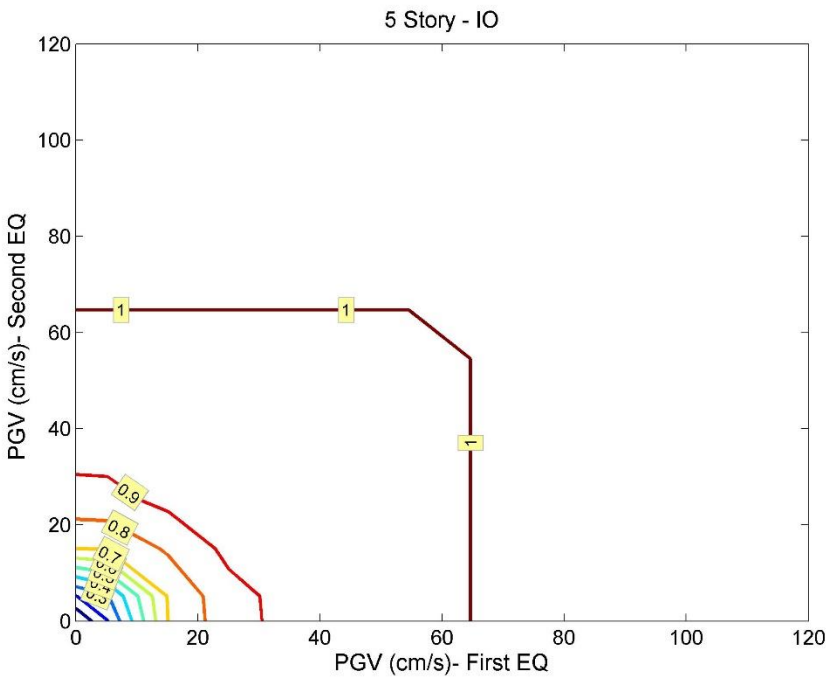

Figure 7-37. Contour plots of the limit state IO (for 5 story RC school buildings)

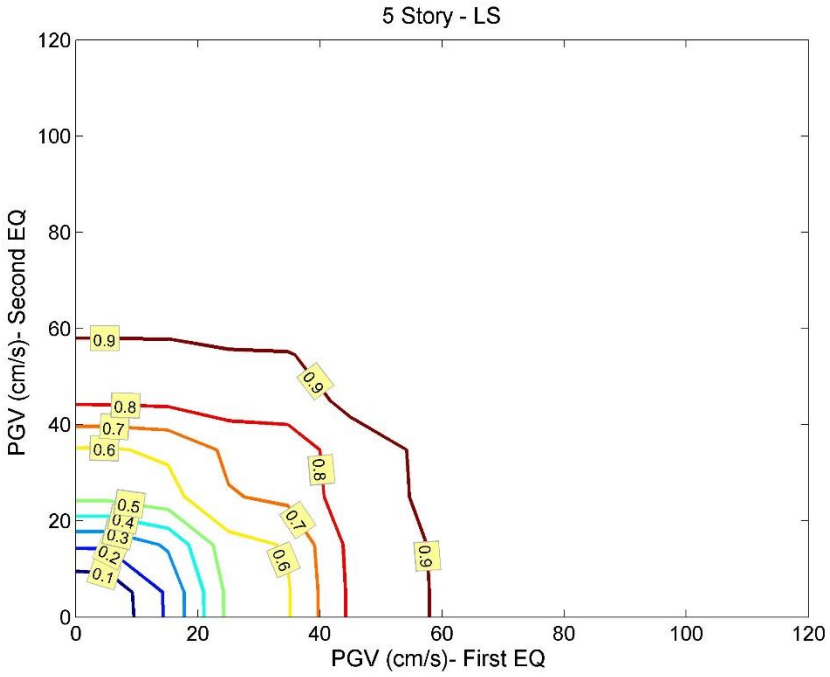

Figure 7-38. Contour plots of the limit state LS (for 5 story RC school buildings)

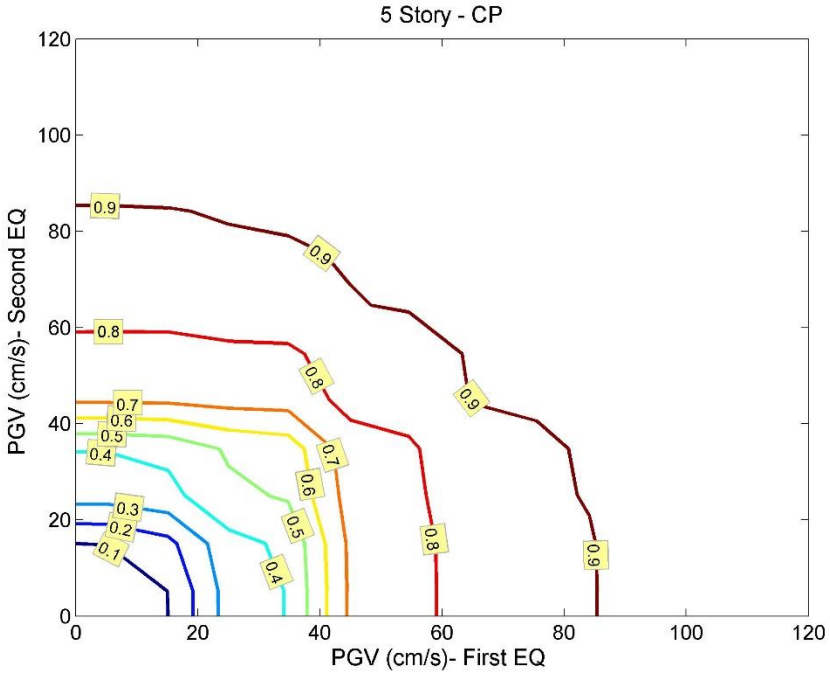

Figure 7-39. Contour plots of the limit state CP (for 5 story RC school buildings)

An example can be useful at this stage to understand F.C.s under repeated EQs. Considering a 3-story school building experienced an EQ which has a PGV= 50 cm/s and was not retrofitted after the EQ, and then, excited to a second EQ which has a 30 cm/s of PGV value. According to Figure 7-6, the building will have a probability of exceedance of approximately 95% for IO, 65% for LS, and 50% for CP for the first EQ, and a probability of exceedance of approximately 85% for IO, 45% for LS and 30% for CP for the second EQ On the other hand, according to Figure 7-28 and corresponding contour plots (Figure 7-29, Figure 7-30, and Figure 7-31), this 3-story building will have a probability of exceedance of approximately 98% for IO, 70% for LS, and 57% for CP after the second EQ considering the combined action. It is clearly observed that combined action yielded more critical values in terms of probability of exceedance, although the second EQ was less critical in terms of PGV. (Mazılıgüney et al., 2017)

7.6 Discussion of Results

The probability of exceedance curves for LS and CP performance states are close to each other for RC school buildings. Fragility curves obtained for selected RC school buildings are roughly consistent with similar research studies (Bilgin, 2013) concentrating on other public buildings. The number of stories seems to affect the probability of exceedance of LS and CP performance states. The probability of exceedance of CP performance state is sensitive in the range of 20-100 cm/s.

The second part of the chapter dealt with the derivation of the F.C.s (or surfaces) due to repeated EQs where such critical structures might encounter without being retrofitted. The fragility surfaces and corresponding contour plots display the accumulation of damage on the structures analyzed and increasing probability of exceedance for a specific limit state, as expected for such seismically vulnerable structures encountering catastrophic seismic activities.

Due to the accumulation of structural damage and increasing risk of costly damage or collapse, decision-makers for retrofitting such structures are advised to consider the resulting fragility surfaces under repeated EQs.

CHAPTER 8

CONCLUSIONS AND RECOMMENDATIONS

8.1 Summary of the Work Done

The dissertation is composed of 8 chapters.

In the 1st chapter, a brief introduction gives a general overview of the background and research significance, literature survey, object, and scope of the study. The previous studies on the seismic vulnerability of RC school buildings are discussed in the literature survey section of the 1st chapter.

The 2nd chapter is devoted to the description of the database composed of 321 RC school buildings used in the study. Seismic performance and general properties of RC buildings in Turkey are mentioned in the first section of the second chapter. Detailed statistical and structural properties of RC school buildings in the database are described by related figures and tables in this chapter. The selection procedure of the buildings for 3, 4, and 5 story sub-classes is also described in this chapter.

In the 3rd chapter, the assessments of RC school buildings in the database by existing walk-down and preliminary seismic assessment methods and the proposed preliminary assessment method are discussed. The comparisons of assessment results with the results of TEC-2007 are also mentioned and discussed in this chapter. Proposed (Modified ATC-21) Preliminary Assessment Method is also described and compared with the assessment results of TEC-2007 in the 3rd chapter.

In the 4th chapter, the work done for 3-D modeling of the selected 36 RC school buildings, pushover analysis, bilinear idealization of capacity curves, ESDOF model formation, determining of performance limit states for each building, assessment by

using DCM target displacements, by ASCE-41 and the discussion of the assessment results are described. Detailed assessment results of selected 36 RC school buildings are also compared with TEC-2007 results within the chapter.

In the 5th chapter, the new EQ code of Turkey, TBEC-2018, is examined for RC school buildings, and selected 36 RC school buildings are analyzed accordingly. The assessment results are compared by the results of other methods and the results of TEC-2007.

The 6th chapter is devoted to the assessment of 36 RC school buildings under the provisions of the Urban Renewal Law. Both the 2013 and 2019 versions of the provisions are used for assessment, and the results between the versions are compared as well as with the results of TEC-2007 and TBEC-2018.

The 7th chapter is devoted to the generation of the F.C.s for 3, 4, and 5 story RC school buildings wrt. PGV and PSA using both bilinear and stiffness and strength degradation (Modified Clough) material models, the validation of the F.C.s, and the assessment of related buildings in the database. F.C.s generated are validated by site observations of the Bingöl database for RC school buildings. F.C.s under repeated EQs wrt. PGV were also developed and compared with the F.C.s under single EQs in this chapter.

The 8th and the last chapter is devoted to conclusions and recommendations, and the work done throughout the dissertation is described briefly.

8.2 Conclusions

Recent EQs in Turkey have shown that the structural performance of reinforced concrete (RC) buildings play crucial role in terms of EQ losses. RC school buildings have been observed among the most vulnerable and severely damaged buildings after

17 August 1999 Marmara (Mw=7,4), 12 November 1999 Düzce (Mw=7,2), 1 May 2003 Bingöl (Mw=6,4) and 23 October 2011 Van (Mw=7,2) EQs.

A database consisting of 321 RC school buildings are examined for their seismic vulnerabilities throughout the thesis. A total of 36 RC school buildings, 12 of each 3, 4, and 5 story ones selected from the database were analyzed in detail.

First, the statistical and structural properties of RC school buildings are determined. Although general parameters of RC school buildings are summarized in Table 2-1, important parameters are mentioned again.

School buildings have an average concrete compressive strength of 10.22 MPa, which is consistent with the building stock of Turkey but slightly better than other public buildings. The average steel tensile strength of the steel used for the buildings in the database is 253 MPa since STIIIa is used only in 16.5% of the RC school buildings, and STI is used for the remaining 83.5% of buildings.

36.7% of the school buildings have basement stories. Factory brick was used in 99.4% of the school buildings, and local brick was used in only 0.6% of them, which are not selected for subclasses.

Only 0.6% of the school buildings have an apparent building quality of "good", while 35.5% of them have "average" and 63.9% of them have "poor" apparent building quality. The two buildings having apparent building quality "good" responses both cases of EQs satisfactorily according to TEC-2007.

The irregularities observed on the school buildings are listed in Table 2-4. As can be seen from Table 2-4, 37.1% of the RC school buildings in the database have topographic effects, 13.4% of them have soft-story, 2.2% have a weak story, 1.9% have vertical irregularity, 2.8% have plan irregularity, 31.5% have short columns, 13.4% have heavy overhangs, 2.2 % have torsion, and 57.9% have pounding effect.

The database consisting of 321 RC school buildings are all analyzed by walk-down and preliminary assessment methods, and the results are compared by TEC-2007. Comparisons of assessment procedures with TEC-2007 for school buildings are tabulated in Table 3-15. ATC-21/FEMA-154 (1998) SVA method is evaluated to be approximately 80 percent relatively and 75 % overall consistent with TEC-2007 for RC school buildings in Turkey.

Hassan and Sözen (1997), and Yakut (2004) procedures are found to be approximately 50 percent relative consistent with TEC-2007 for RC school buildings in Turkey. Since there are usually big openings on the masonry walls, including or excluding the masonry walls of RC school buildings, do not cause a significant change in the assessment results of Yakut (2004) procedure.

Sucuoğlu and Yazgan (2003), Sucuoğlu et al. (2007), and Özcebe et al. (2003) procedures that rely mostly on visual properties of buildings such as irregularities are not consistent with TEC-2007 for RC school buildings in Turkey.

The differences in assessment results are due to different parameters taken into consideration in different assessment procedures.

Although none of the procedures were found to adequately determine the performance in conformance with TEC-2007, the ATC-21/FEMA-154 (1998) procedure is the most suitable one to assess approximately the performance of existing RC school buildings consistent with TEC-2007.

Since the Turkish EQ Code is in force and obligatory for all RC school buildings, the method which has the best consistency with TEC-2007 is advised to be preferred for rapid assessment purposes, which is ATC-21/FEMA-154 (1998).

A new preliminary assessment method is proposed by merging Yakut's Procedure and ATC-21/FEMA-154. The basic capacity index of the Yakut's Procedure is used, and the penalty scores for ATC-21/FEMA-154 are modified. The proposed preliminary assessment method is described in section 3.3 of the dissertation.

Evaluation of proposed procedure with TEC-2007 for the LS performance state for the EQ having a probability of exceedance of 2 % in 50 years lifetime can be seen in Table 3-17. The unsatisfactory buildings may be assessed by a 91.09 % relative consistency ratio, and the method is evaluated to be 81.00 % consistent with TEC-2007.

Evaluation of proposed procedure with TEC-2007 for the IO performance state for the EQ having a probability of exceedance of 10 % in 50 years lifetime can be seen in Table 3-18. The unsatisfactory buildings may be assessed by a 91.78 % relative consistency ratio, and the method is evaluated to be 71.96 % consistent with TEC-2007.

Although TEC-2007 has been modified, and TBEC-2018 is in effect since January 2019, the proposed method may give insight to structural engineers about the seismic vulnerabilities of RC school buildings.

Since the consistency of the proposed method with TEC-2007 is more than 90 % for unsatisfactory buildings, it is advised to be used for preliminary assessment of RC school building groups. Since there are more than 55,000 RC school buildings in Turkey, it is advised that the determination of a classification for prioritization of vulnerable structures can be done by the proposed method, before applying TEC-2007 or TBEC-2018, which will be time-consuming, uneconomical and impractical.

36 RC school buildings, 12 of each 3, 4, and 5 story ones are selected from the database, and structural parameters are determined. Structural parameters of the selected RC school buildings are tabulated in Appendix A. The equivalent SDOF system parameters are also given for all selected buildings in Appendix D. Statistics of capacity curve parameters are listed in Table 4-2, Table 4-3, and Table 4-4 for 3, 4, and 5 story RC school buildings, respectively.

The average value of ductility is 3.47 for RC school buildings. The average yield spectral displacements for RC school buildings are found to be 0.23, 0.25, and 0.27

percent of the total height for 3, 4, and 5 story buildings, respectively. The ultimate displacements are found to be 1.11, 2.06, and 2.34 percent of the total height for 3, 4, and 5 story RC school buildings, respectively.

The yield over-strength ratios (γ) were found to decrease significantly with increasing number of stories. The average yield over strength ratios for 5 story RC school buildings is less than 1.0. In the study of Yakut (2004), it is declared that "RC frame buildings that have yield over-strength ratio of less than 1.65 would perform poorly against a devastating EQ in Turkey". The average yield over strength ratios are less than 1.65 for 3, 4, and 5 story RC school buildings.

It is observed that for RC school buildings, the damage sequence of structural members "generally" starts by yielding of the beam ends at the lower stories and then yielding propagates to beams of upper stories. The total collapse of the buildings occurs typically by the yielding of the column bases of the ground stories. The damage sequence of RC school buildings is, in fact, the accepted sequence of ductile beam mechanism. That is because RC school buildings are generally regular in plan and do not have vertical irregularity. Unfortunately, the ductilities and the ultimate displacement capacities are not sufficient. The main reason for the low ductilities is the unconfined yielding regions, i.e., insufficient/small rotation capacities.

There are few buildings that the damage sequence starts with the yielding of column bases or both column and beam ends. This is not a desirable sequence.

For all RC school buildings, yielding starts at the lowest story and then propagates to the upper stories.

The average performance limits for RC school buildings are determined and listed in Table 4-6.

36 selected RC school buildings are assessed by using DCM target displacements, and the assessment results are tabulated and compared with TEC-2007 in Table 4-10 and Table 4-11. It is seen that the RC school buildings perform better in Y (short)

directions according to DCM assessment results. 11 (30,5 %) of the buildings are in collapse performance state for both orthogonal directions according to DCM and also according to TEC-2007 assessment results for an EQ of 10 % occurrence probability in 50 years, and 20 (55,5 %) of them are in collapse performance state for an EQ of 2 % occurrence probability in 50 years. For other buildings, there is not a meaningful correlation between the DCM and TEC-2007 assessment results.

36 selected RC school buildings are assessed by ASCE-41, and the assessment results are tabulated and compared with TEC-2007 in Table 4-12. It is seen that the RC school buildings perform better in Y (short) directions, according to ASCE-41 (ASCE, 2007) assessment results. 9 (25 %) of the buildings are in collapse performance state in both directions according to ASCE-41 and according to TEC-2007. 12 (33,3 %) of the buildings are in collapse performance state according to TEC-2007 and in collapse performance state at least in one direction, according to ASCE-41 (ASCE, 2007). For other buildings, there is not a meaningful correlation between the ASCE-41 (ASCE; 2007) and TEC-2007 assessment results.

36 selected RC school buildings are assessed by TBEC-2018. First, the control of the usability check of linear analyzes are performed for all selected 36 RC school buildings. According to TBEC-2018, the linear analysis method is found to be valid for 8 of the 36 buildings, all having shear walls. The linear analysis method is not valid according to TBEC-2018 for RC school buildings having no shear walls. However, the method is broadly applicable, provided that the buildings have shear walls cross-section of higher than 0.5% by the land area in either direction.

As it is well observed in Table 5-4, detailed seismic assessment of the buildings according to TBEC-2018 revealed that none of the 36 RC school buildings do satisfy the LD and CD performance levels. Indeed, in the case of DD1, the maximum expected earthquake having a probability of occurrence of 2 % in 2475 years, all of the school buildings appear to be within the collapse level (C). Similar findings were seen under the DD3 - the service earthquake in that none of the buildings achieve the

expected limited damage stage, even entering the collapse zone. As seen in Table 5-4, few buildings seem to have controlled damage (CD) or collapse prevention (CP) performance states only in Y directions. Therefore, usage of the buildings under these circumstances poses substantial threats to life safety, according to TBEC-2018.

While approximately 20 % of the buildings satisfy the performance requirements of TEC-2007, it is not the same for TBEC-2018. Excluding a few buildings that have shown satisfactory performance levels in either X or Y directions for DD-3 EQ, there is no such building that has fulfilled both DD-1 and DD-3 EQ performance requirements simultaneously for TBEC-2018.

The density of shear walls in a given principal direction is believed to have a prominent role in the seismic performance of the buildings (Gülkan and Sözen, 1999; Yakut, 2004), but unfortunately, most of the buildings in this study have no sufficient shear walls in either direction to fulfill the code requirements of TEC-2007 as well as TBEC-2018. Also, the combined effects of low concrete quality and reinforcing steel help the buildings exhibit these unexpectedly poor performances, irrespective of the seismic code.

It is determined that the base shear forces of TBEC-2018 are 7.5% larger than the base shear forces of TEC-2007.

The assessment results reveal that there is not much difference between the codes for the buildings in this study owing to the deficiency of the shear walls in the buildings to promote either Turkish Codes. It is evident that TBEC-2018 is more conservative than TEC-2007.

36 RC school buildings are assessed under the provisions of Urban Renewal Law. Both 2013 and 2019 versions of the provisions are used for assessment, and the results between the versions are compared as well as with the results of TEC-2007 and TBEC-2018.

Table 6-1 presents the performance of the school buildings according to the URL-2013. 8 buildings out of 36 satisfy the URL-2013 in all directions so that they are found to be seismically having a low risk. However, this finding does not necessarily mean that they also satisfy the target performance specified in TEC-2007. Most of the buildings in this category have three stories. The lower number of stories associated with the lower building height, the buildings become less vulnerable to EQs. Almost all five and four-story buildings failed the risk assessment as per URL-2013.

The status of the buildings appears to be much worse when URL-2019 is considered. Indeed, all of the buildings except for only one are found to be risky, as seen in Table 6-2. Even the satisfactory buildings in the case of URL-2013 have failed the risk assessment in this case. The main reason for this is based on the DD1 earthquake level, which has as high as 1.5 times design spectral acceleration in the elastic spectrum.

In order to compare URL-2013 and URL-2019, the buildings are tested for the same design earthquake of a 10% probability of occurrence in 50 years. As seen in Table 6-3, among the 36 buildings, only four of them satisfy both provisions. Although the selected 36 RC school buildings in this study are analyzed and assessed by linear elastic procedures of either TEC-2007 or TEC-2018, there is reasonable consistency between the assessment results of URL provisions and the seismic codes.

F.C.s are generated for 3, 4, and 5 story RC school buildings wrt. PGV and PSA using both bilinear and stiffness and strength degradation (Modified Clough) material models. 100 ground motion records are used for F.C. generation. F.C.s generated are validated by site observations of the Bingöl database for RC school buildings.

The generated F.C.s revealed that the number of stories affects the probability of exceedance of all performance states for all cases. EQ vulnerability of RC school buildings increases with increasing number of stories.

The probability of exceedance curves for IO performance state is generally distant from the curves of LS and CP performance states, whereas the probability of exceedance curves for LS and CP performance states are close to each other for RC school buildings. That is why the performance limit definitions are likewise. IO and LS performance limits are distant from each other, while LS and CP limits are close. The probability of exceedance of CP performance state is sensitive in the range of 20-100 cm/s.

Bilinear and Modified Clough F.C.s have significant differences until the probability of exceedance of 80 %, and Modified Clough-based curves indicate a larger probability of exceedance values. Two curves nearly coincide at and after 80% of probability of exceedance. It is evident that there are significant differences between the Bilinear and Modified Clough material models-based probability of exceedance curves. Hence, the degradation characteristics of the structural model seem to have a major influence on the final F.C.s.

It is important to note here that maximum values of the IMs considered and the number of records utilized or available in each bin throughout GM set formation, and F.C. generation would certainly affect the outcomes of the risk study, which should be further investigated as it is stated by Mazılıgüney et al., 2013.

Comparison of observed from Bingöl database and derived fragility data are tabulated in Table 7-5. The difference between the observed and the derived probability of exceedance values are reasonable for IO and CP performance states, but the difference for LS performance state is not tolerable and not on the safe side. The difference can be attributed to the limited number of school buildings in the Bingöl database and varying construction practices of different regions of Turkey.

The differences between predicted and observed fragilities are less than 10 percent at both damage limit states in the 3 and 4 story buildings. Considering the uncertainties inherent in the fragilities and randomness of ground intensities, these results can be accepted as satisfactory, suggesting that the proposed F.C.s can be implemented to large building stocks of RC school buildings in Turkey for loss estimation studies.

F.C.s under repeated EQs wrt. PGV are also generated and compared with the F.C.s under single EQs in section 7.5. It is clearly observed that combined action yielded more critical values in terms of probability of exceedance, although the second EQ was less critical in terms of PGV (Mazılıgüney et al., 2017). Due to the accumulation of structural damage and increasing risk of costly damage or collapse, decision-makers for retrofit of such structures are advised to consider the resulting fragility surfaces under repeated EQs. It is believed that the F.C.s generated in the dissertation can be used for school buildings in Turkey.

8.3 Recommendations

There are more than 55.000 RC school buildings in Turkey, and the seismic vulnerability of them is a significant risk for the whole society. Determining seismic vulnerabilities of RC school buildings, and retrofitting or reconstructing the risky ones is an urgent task for Turkey.

The extension of the study to develop more precise preliminary assessment methods and F.C.s will help to determine the seismic vulnerabilities of RC school buildings in a shorter time. Undoubtedly, most of the effort should be devoted to determining the vulnerabilities of RC school buildings and taking urgent precautions to reduce the risks.

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APPENDICES

A. Determined Properties of the Database Consisting of 321 RC School Buildings

The determined properties of the database consisting of 321 RC school buildings are tabulated in both Microsoft Excel (.xls) and .pdf format file and presented in the CD, supplemented by the dissertation.

B. Properties of Selected 36 RC School Buildings

All determined parameters of the selected RC school buildings are tabulated in Appendix A. The selected RC school buildings are labeled with red ID numbers, and all parameters are written with bold characters in Appendix A.

Likewise, the selected 36 RC school buildings are extracted and tabulated separately in Appendix B. The determined properties of selected 36 RC school buildings are tabulated in both Microsoft Excel (.xls) and .pdf format file and presented in the CD, supplemented by the dissertation.

Table C-1 Structural Parameters of 12 selected 3 story RC school buildings (X-direction)

Building ID	Sd_y (cm)	Sa_y (g)	Sd_u (cm)	Sa_u (g)	C_s	T_e	PF	α	γ	l	μ
1	2,8672	0,0682	8,1576	0,0840	0,1	1,3010	1,2835	0,9102	0,6205	1,2315	2,8451
2	3,9082	0,0953	8,4723	0,1242	0,1	1,2846	1,2976	0,7640	0,7281	1,3028	2,1678
3	0,4742	0,1519	1,3448	0,2329	0,1	0,3545	1,4128	0,7627	1,1585	1,5332	2,8358
4	6,8132	0,0810	17,9989	0,1113	0,1	1,8395	1,2476	0,8667	0,7022	1,3731	2,6418
5	0,1299	0,2983	0,5349	0,4240	0,1	0,1324	1,3086	0,7509	2,2398	1,4215	4,1176
6	1,2286	0,1322	4,1833	0,1494	0,1	0,6116	1,2861	0,8967	1,1852	1,1302	3,4051
7	0,4344	0,2760	0,8905	0,4033	0,1	0,2516	1,3813	0,7869	2,1723	1,4609	2,0500
8	1,3310	0,1486	3,5702	0,1811	0,1	0,6004	1,2772	0,8690	1,2914	1,2183	2,6824
9	1,9066	0,0931	10,1601	0,1179	0,1	0,9079	1,3375	0,8048	0,7491	1,2667	5,3290
10	2,9523	0,2938	9,8361	0,3547	0,1	0,6359	1,3549	0,7392	2,1718	1,2074	3,3316
11	6,7709	0,4512	14,8545	1,0061	0,1	0,7771	1,2966	0,7662	3,4573	2,2297	2,1939
12	1,3109	0,4545	7,8189	0,5656	0,1	0,3407	1,1442	0,6437	2,9253	1,2445	5,9644

Table C-2 Structural Parameters of 12 selected 3 story RC school buildings (Y-direction)

Building ID	Sd_y (cm)	Sa_y (g)	Sd_u (cm)	Sa_u (g)	C_s	T_e	PF	α	γ	l	μ
1	2,0518	0,1226	7,7814	0,1425	0,1	0,8207	1,2915	0,7768	0,9523	1,1627	3,7925
2	3,0312	0,1458	8,1126	0,2253	0,1	0,9147	1,2969	0,7769	1,1326	1,5456	2,6763
3	1,3389	0,0900	2,2761	0,1181	0,1	0,7739	0,9710	0,6125	0,5511	1,3127	1,7000
4	3,5278	0,0712	13,7005	0,0873	0,1	1,4122	1,2501	0,8697	0,6192	1,2260	3,8835
5	0,1496	0,1406	0,6881	0,2990	0,1	0,2069	1,3371	0,7434	1,0452	2,1264	4,6000
6	1,9193	0,1086	8,9649	0,1287	0,1	0,8433	1,2504	0,8314	0,9029	1,1846	4,6708
7	0,3064	0,3090	0,7369	0,4537	0,1	0,1998	1,3706	0,7908	2,4438	1,4681	2,4048
8	0,3527	0,5378	1,1909	0,8311	0,1	0,1625	1,3184	0,7714	4,1483	1,5455	3,3763
9	3,1798	0,1386	14,4553	0,1744	0,1	0,9610	0,7862	0,5006	0,6936	1,2590	4,5460
10	2,7856	0,4639	13,5304	0,6088	0,1	0,4916	0,7898	0,4513	2,0935	1,3124	4,8572
11	4,3908	0,4194	13,9826	0,5852	0,1	0,6491	1,2992	0,7697	3,2280	1,3955	3,1845
12	2,4225	0,4931	12,7829	0,7096	0,1	0,4446	0,8256	0,4563	2,2502	1,4390	5,2767

Table C-3 Structural Parameters of 12 selected 4 story RC school buildings (X-direction)

Building ID	Sd_y (cm)	Sa_y (g)	Sd_u (cm)	Sa_u (g)	C_s	T_e	PF	α	γ	l	μ
13	0,7605	0,3124	1,3930	0,4073	0,1	0,3130	1,4070	0,7474	2,3348	1,3037	1,8318
14	1,7537	0,1456	6,9400	0,1557	0,1	0,6963	1,3400	0,7193	1,0470	1,0695	3,9574
15	1,9014	0,0538	4,6146	0,0547	0,1	1,1930	1,4042	0,7332	0,3942	1,0183	2,4270
16	0,5975	0,1190	1,6413	0,1889	0,1	0,4495	1,3892	0,7214	0,8585	1,5869	2,7470
17	0,9728	0,1693	4,3096	0,19641	0,1	0,48082	1,27157	0,85723	1,45163	1,15984	4,43007
18	2,9457	0,0710	11,5620	0,0811	0,1	1,2918	1,3138	0,8349	0,5931	1,1423	3,9251
19	12,1010	0,1150	28,5882	0,1742	0,1	2,0575	1,3383	0,7516	0,8646	1,5145	2,3625
20	7,5441	0,0978	18,7919	0,1490	0,1	1,7622	1,2593	0,8559	0,8368	1,5241	2,4910
21	3,5319	0,0618	11,2865	0,0683	0,1	1,5162	1,2741	0,9159	0,5663	1,1045	3,1956
22	2,2788	0,0260	19,8644	0,0329	0,1	1,8797	1,0971	0,7471	0,1939	1,2695	8,7172
23	2,1529	0,0778	8,6641	0,0886	0,1	1,0550	1,3377	0,8961	0,6976	1,1375	4,0243
24	2,3722	0,0279	6,7798	0,0320	0,1	1,8502	1,3068	0,8508	0,2373	1,1488	2,8581

Table C-4 Structural Parameters of 12 selected 4 story RC school buildings (Y-direction)

Building ID	Sd_y (cm)	Sa_y (g)	Sd_u (cm)	Sa_u (g)	C_s	T_e	PF	α	γ	l	μ
13	0,7445	0,5130	1,3826	0,6493	0,1	0,2417	1,4104	0,7396	3,7940	1,2657	1,8571
14	1,9928	0,1335	4,7017	0,1679	0,1	0,7750	1,4803	0,7142	0,9535	1,2575	2,3593
15	1,7532	0,0496	4,5787	0,0670	0,1	1,1929	1,4174	0,7265	0,3602	1,3517	2,6117
16	4,0144	0,0816	11,9813	0,0927	0,1	1,4068	1,2928	0,8317	0,6789	1,1351	2,9846
17	1,7081	0,1265	11,6387	0,14919	0,1	0,7371	1,25873	0,82396	1,04243	1,17922	6,81395
18	3,8148	0,1299	10,0389	0,1494	0,1	1,0871	1,2452	0,6876	0,8932	1,1502	2,6316
19	7,3031	0,0824	20,6622	0,1182	0,1	1,8885	1,2826	0,8283	0,6826	1,4342	2,8292
20	3,9906	0,1110	12,0111	0,1495	0,1	1,2031	1,3134	0,7884	0,8748	1,3470	3,0098
21	5,0368	0,1558	15,2010	0,1768	0,1	1,1405	0,8835	0,4968	0,7741	1,1347	3,0180
22	3,2706	0,0317	24,6381	0,0452	0,1	2,0374	1,0957	0,7339	0,2327	1,4241	7,5333
23	2,6711	0,1446	6,8354	0,1831	0,1	0,8621	1,0782	0,6175	0,8932	1,2658	2,5590
24	3,1175	0,0661	8,7029	0,0882	0,1	1,3775	0,9238	0,6049	0,4000	1,3345	2,7917

Table C-5 Structural Parameters of 12 selected 5 story RC school buildings (X-direction)

Building ID	Sd_y (cm)	Sa_y (g)	Sd_u (cm)	Sa_u (g)	C_s	T_e	PF	α	γ	l	μ
25	2,4899	0,0868	9,5022	0,1069	0,1	1,0743	1,3334	0,7509	0,6519	1,2318	3,8163
26	3,3707	0,0596	11,4089	0,0693	0,1	1,5088	1,3647	0,8257	0,4920	1,1635	3,3848
27	2,5960	0,0750	12,6865	0,0774	0,1	1,1804	1,3290	0,8090	0,6066	1,0326	4,8870
28	3,7413	0,0109	7,7474	0,0113	0,1	3,7175	1,4166	0,7223	0,0787	1,0343	2,0708
29	4,1383	0,0123	12,6041	0,01758	0,1	3,68588	1,36529	0,82423	0,10104	1,43379	3,0457
30	0,5575	0,0348	4,1837	0,0525	0,1	0,8030	1,7936	0,6858	0,2386	1,5095	7,5040
31	1,6219	0,0375	6,1451	0,0509	0,1	1,3191	1,3873	0,7502	0,2814	1,3572	3,7889
32	0,5630	0,0079	3,4133	0,0087	0,1	1,6954	2,0427	0,8234	0,0649	1,1093	6,0628
33	5,1686	0,0290	11,8633	0,0327	0,1	2,6792	1,3737	0,7325	0,2123	1,1279	2,2953
34	11,1779	0,0273	44,7061	0,0284	0,1	4,0599	1,3419	0,7771	0,2121	1,0402	3,9995
35	13,1563	0,0312	24,9587	0,0336	0,1	4,1194	1,3682	0,6399	0,1996	1,0774	1,8971
36	6,8993	0,0322	18,8770	0,0364	0,1	2,9364	1,3480	0,7692	0,2477	1,1298	2,7361

Table C-6 Structural Parameters of 12 selected 5 story RC school buildings (Y-direction)

Building ID	Sd_y (cm)	Sa_y (g)	Sd_u (cm)	Sa_u (g)	C_s	T_e	PF	α	γ	l	μ
25	4,0532	0,1084	15,6005	0,1265	0,1	1,2264	1,1102	0,6011	0,6519	1,1662	3,8489
26	3,9632	0,1359	15,2042	0,1703	0,1	1,0834	1,1102	0,6011	0,8167	1,2535	3,8364
27	2,6372	0,1594	9,6297	0,1848	0,1	0,8159	1,2513	0,7232	1,1529	1,1592	3,6515
28	3,8774	0,0188	8,8411	0,0202	0,1	2,8827	1,3798	0,7724	0,1450	1,0754	2,2802
29	4,4229	0,0129	14,9952	0,018086	0,1	3,719259	1,35659	0,821394	0,10569	1,405624	3,390383
30	0,5223	0,0563	5,8655	0,0804	0,1	0,6111	1,6275	0,7333	0,4127	1,4278	11,2311
31	1,0678	0,0391	3,3283	0,0495	0,1	1,0481	1,5452	0,7385	0,2889	1,2648	3,1169
32	1,2844	0,0163	5,1531	0,0190	0,1	1,7814	1,8685	0,8256	0,1345	1,1694	4,0120
33	4,2415	0,0404	11,1651	0,0486	0,1	2,0550	1,3439	0,7233	0,2924	1,2012	2,6324
34	6,6172	0,0498	23,5762	0,0570	0,1	2,3130	1,3148	0,8107	0,4035	1,1450	3,5629
35	10,4253	0,2365	22,9818	0,3157	0,1	1,3318	1,4388	0,6087	1,4397	1,3348	2,2044
36	7,0985	0,0261	15,4809	0,0308	0,1	3,3077	1,4087	0,6723	0,1755	1,1782	2,1809

D. Equivalent SDOF Parameters of Selected Buildings

Equivalent SDOF parameters for all selected 36 RC school buildings are provided by excel sheets for each building for both X and Y-directions separately. All excel sheets are presented in the CD, supplemented by the dissertation.

E. Assessment Reports of Selected RC School Buildings Performed by ProtaStructure Software under TBEC-2018

The assessment reports of the selected 36 RC school buildings performed by ProtaStructure software under TBEC-2018 are provided in the CD, supplemented by the dissertation. All assessment reports are in .pdf file format.

F. Assessment Reports of Selected RC School Buildings Performed by ProtaStructure Software under URL Provisions Published in 2013

Assessment reports of all selected 36 RC school buildings performed by ProtaStructure software under the URL provisions published in 2013 are provided in the CD, supplemented by the dissertation. All assessment reports are in .pdf file format.

G. Assessment Reports of Selected RC School Buildings Performed by ProtaStructure Software under URL Provisions Published in 2019

Assessment reports of all selected 36 RC school buildings performed by ProtaStructure software under the URL provisions published in 2019 are provided in the CD, supplemented by the dissertation. All assessment reports are in .pdf file format.

H. Ground Motion Data

Detailed information about the ground motion records set list used for the F.C. generation is presented in .xls file format, and also the records and OPENSEES formats are provided in the CD, supplemented by the dissertation.

CURRICULUM VITAE

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EDUCATION

Degree	Institution	Year of Graduation
MS	Anadolu Unv. Civil Engineering (Construction Management)	2008
MS	METU Civil Engineering (Materials of Construcyion)	2007
MS	Gazi Unv. Civil Engineering (Transportation)	2006
BE	Anadolu Unv. Economics	2017
LLB	Ankara Unv. Faculty of Law	2014
BS	METU Civil Engineering	2000
High School	Science High School, Kırıkkale	1996

WORK EXPERIENCE

Year	Place	Enrollment
2018 December-Present	Mazılıgüney Consultancy	Owner-Lawyer
2017 August-2018 August	Ankara Bar Association	Trainee Lawyer
2001-2017 April	Ministry of Defense	Officer (Civil Engineer and Internal Auditor)
2000 June-2001 November	Private Engineering and Construction Firms	Civil Engineer

FOREIGN LANGUAGES

Advanced English

PUBLICATIONS

1. Mazılıgüney, L., Yakut, A., Kadaş, K., Kalem, İ., (2012), "Evaluation of Preliminary Assessment Procedures for Reinforced Concrete School Buildings in Turkey", 10th International Congress on Advances in Civil Engineering, ACE2012, October 17-19, Middle East Technical University, Ankara, Turkey

2. Mazılıgüney, L., Yakut, A., Kadaş, K., Kalem, İ., (2013), "Fragility Analysis of Reinforced Concrete School Buildings Using Alternative Intensity Measure-Based Ground Motion Sets", 2nd Turkish Conference on Earthquake Engineering and Seismology – TDMSK -2013, Hatay, Turkey.

3. Mazılıgüney, L., Azılı, F., Yaman, İ.Ö. (2008), "In-Situ Concrete Compressive Strengths of Residential, Public and Military Structures", Eighth International Congress on Advances in Civil Engineering, ACE2008, September 15-17, Vol.3, Famagusta, Turkish Republic of Northern Cyprus, pp. 273-280.

4. Mazılıgüney, L., Kadaş, K., Akansel, V. H., Yakut, A., (2017), "Fragility Curves for Reinforced Concrete School Buildings due to Repeated Earthquakes", 4th International Conference on Earthquake Engineering and Seismology, Eskişehir, Turkey, 468-471

5. Mazılıgüney, L., Akansel, V. H., Soysal, B. F., Kadaş, K., Yakut, A., (2019), "Seismic Risk Assessment of School Buildings on the Basis of Recent Turkish Seismic Hazard Map", VI. International Earthquake Symposium Kocaeli 2019, IESKO 2019, September 25-27, Kocaeli, Turkey

Table Tennis, Swimming, Digital Forensics, Photographs, Movies, Literature