

STATE DEPENDENT SEISMIC RESPONSE OF REINFORCED CONCRETE
FRAME BUILDINGS UNDER SEQUENTIAL EARTHQUAKES

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
OF
MIDDLE EAST TECHNICAL UNIVERSITY



BY
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IN PARTIAL FULFILLMENT OF THE REQUIREMENTS
FOR
THE DEGREE OF DOCTOR OF PHILOSOPHY
IN
CIVIL ENGINEERING

MARCH 2025

Approval of the thesis:

**STATE DEPENDENT SEISMIC RESPONSE OF REINFORCED
CONCRETE FRAME BUILDINGS UNDER SEQUENTIAL
EARTHQUAKES**

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ABSTRACT

STATE DEPENDENT SEISMIC RESPONSE OF REINFORCED CONCRETE FRAME BUILDINGS UNDER SEQUENTIAL EARTHQUAKES

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Doctor of Philosophy, Civil Engineering
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March 2025, 153 pages

This study focuses on the seismic damage evolution for reinforced concrete (RC) frame buildings which are subjected to sequential earthquakes. In this study, the most significant sequential earthquakes from different parts of the world are collected from the available strong motion databases. This dataset also includes ground motion records from the recent Kahramanmaraş, Türkiye earthquake sequence that took place in February 2023. The structural modeling is carried out at two different levels of idealization. The first level belongs to a single degree of freedom idealization of RC frame buildings whereas planar frame models have been utilized for the second level. In both cases, the response statistics are obtained through dynamic analyses by employing the developed sequential ground motion dataset, and the evolution of seismic damage is monitored when building models are subjected to multiple earthquakes. The results of this study reveal that sequential earthquakes may create excessive demands on the seismic behavior of RC frame buildings and this issue should be explicitly considered in the seismic design and analysis of structures in the near future.

Keywords: Sequential Earthquake, Damage Evolution, Intensity Measure, Single Degree of Freedom, Planar Building Model



ÖZ

ARDIŞIK DEPREMLER ALTINDA BETONARME ÇERÇEVELİ BİNALARIN DURUMA BAĞLI DEPREM DAVRANIŞLARI

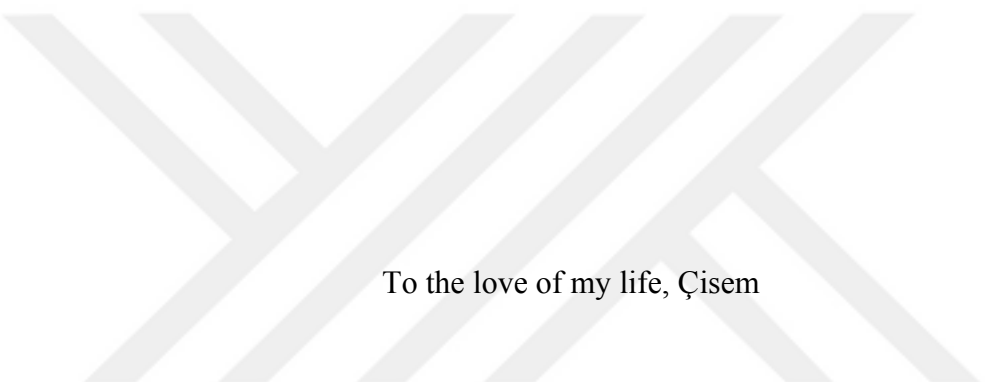
Gökmen, Furkan
Doktora, İnşaat Mühendisliği
Tez Yöneticisi: Prof. Dr. Murat Altuğ Erberik

Mart 2025, 153 sayfa

Bu çalışma, ardışık depremlere maruz kalan betonarme çerçevesel binaların yapısal hasar değişimine odaklanmaktadır. Bu tezde, dünyanın farklı bölgelerinde meydana gelmiş olan en etkili ardışık depremler, mevcut yer hareketi veri tabanlarından alınarak bir araya getirilmiştir. Bu veri tabanı, Şubat 2023'te Türkiye Kahramanmaraş'ta meydana gelmiş olan ardışık depremler sırasında elde edilmiş kuvvetli yer hareketi kayıtlarını da içermektedir. Yapısal modelleme iki farklı detaylandırma seviyesinde gerçekleştirilmiştir. İlk seviyede betonarme binalar tek serbestlik dereceli yapılar olarak ele alınırken ikinci seviyede ise 2 boyutlu çerçeve modelleri olarak dikkate alınmıştır. Her iki durumda da, çalışma için derlenmiş olan ardışık deprem veri tabanı ile dinamik analizler gerçekleştirilerek tepki istatistikleri oluşturulmuş ve ardışık depremlere maruz kalan yapılarda görülen hasar değişimi irdelenmiştir. Bu çalışmanın sonuçları, ardışık depremlerin betonarme binalar üzerinde ek talepler oluşturabileceğini ve bu durumun kısa vadede yapıların deprem tasarımı ve analizi açısından kesin olarak değerlendirilmesi gerektiğini ortaya çıkarmaktadır.

Anahtar Kelimeler: Ardıřık Deprem, Hasar Deęiřimi, řiddet Ölçüsü, Tek Serbestlik Derecesi, İki Boyutlu Bina Modeli





To the love of my life, isem

ACKNOWLEDGMENTS

The author wishes to express his deepest gratitude to his supervisor Prof. Dr. Murat Altuğ Erberik, and thanks to thesis monitoring committee members Prof. Dr. Ayşegül Askan Gündoğan, Assist. Prof. Dr. Fırat Soner Alıcı, for their guidance, advice, criticism, encouragement, and insight throughout the research.

The author would also like to thank Assoc. Prof. Dr. Bekir Özer Ay and Assist. Prof. Dr. Kaan Kaatsız for their suggestions and comments.

The author would especially like to thank his wife Eylül Çisem Uçar Gökmen for everything, nothing would be possible without her.

The author thanks his colleagues Alperen Şenocak and Pınar Arıkoğlu for their support through this journey.

Finally, the author thanks his family for their endless support from the beginning of his undergraduate studies until he got this degree.

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LIST OF ABBREVIATIONS

ABBREVIATIONS

RF: RC frame

1: 1-3 storey buildings

2: 3-8 storey buildings

A: well designed

B: little deficiencies (common type)

C: ill designed

LIST OF SYMBOLS

SYMBOLS

T : Period

η : base shear coefficient

μ : ductility capacity

α_s : strain hardening ratio

λ : residual strength coefficient

α_c : post capping ratio

γ : cyclic deterioration parameter

t_d : duration of an earthquake

$\ddot{u}_g(t)$: ground acceleration

g : gravitational acceleration

CHAPTER 1

INTRODUCTION

1.1 General

There are many cases of sequential earthquakes all over the world that have caused extensive damage and loss to the affected communities. The latest example is the recent 2023 Southeast Türkiye (also known as Kahramanmaraş) earthquakes, which affected 11 highly populated cities in a short period of time. On the other hand, all design and analysis approaches in earthquake engineering are based on single earthquakes. Therefore, this study focuses on the significance of sequential earthquakes on the seismic response of reinforced concrete (RC) frame buildings. The ultimate goals of this study are to assess the significance of sequential earthquakes and to answer questions like what the common properties of events within the sequence cause additional damage if there is an urgent need to revise seismic design and assessment methodologies to account for the effects of sequential events and to generate damage evolution statistics and in turn fragility curves for RC building models. For the first purpose, a special ground motion database regarding 30 different sequential events from different parts of the world has been generated. Since this study concentrates on the effects of sequential earthquakes on building typologies rather than their effects on individual buildings, simplified structural models are employed to represent different classes of RC frame buildings. The structural modeling is carried out at two different levels of idealization. The first level belongs to the single degree of freedom (SDOF) idealization of RC frame buildings whereas planar frame models have been utilized for the second level. In the first level, a multi-parameter hysteretic model is used to carry out dynamic analyses and obtain the response statistics of SDOF structural simulations. Structural variability is also taken into consideration. In the second level of this study, damage

evolution statistics and the corresponding fragility curves are obtained for the most critical combinations of sequential earthquakes by utilizing planar frame models to represent RC building sub-classes. In this part, 20 different RC frame buildings are developed and then they are analyzed with 68 mainshocks and 68 sequences created by scaling or reusing the earthquakes from the previously generated earthquake database. Then, fragility curves are developed to compare the impact of sequential earthquakes on the seismic damage evolution of RC frame buildings. The obtained results reveal that sequential earthquakes, depending on the characteristics of the multiple events within the sequence, do influence the damage state evolution in the considered building classes. Overall, the considerable effect of sequential earthquakes on seismic response and structural damage of conventional building types puts forward the necessity of developing some specific measures for seismic design and analysis of structural systems in the near future.

1.2 Problem Statement

Conventional earthquake engineering research has been focused on the seismic response of building structures subjected to single-ground motions. Moreover, the code-based approaches in many countries generally consider seismic demand calculations based on a unique event or scenario. However, there are cases in which the building stock in a region is affected by multiple earthquakes in a short period of time. The earthquake sequence can be in the form of foreshock+mainshock, mainshock+aftershock, or even more than one main shock occurring in the same region. The common issue in all these cases is the short period of time between these destructive events, which does not enable to take some measures in order to retrofit the already damaged building stock. There are many cases during such sequential events that a damaged building due to the first event collapses during the next event. This is very critical in the sense that the regional damage distribution of the building stock can be changed drastically after the next earthquake, which is also a very

challenging issue for the technical teams in the field that work for building damage assessment as well as rescue operations.

There are many cases of recent sequential earthquakes all over the world that have caused extensive damage and loss to the affected communities like the events that occurred in Southern California USA (1992), Chi Chi Taiwan (1999), Niigata Japan (2004), Sumatra Indonesia (2007), Canterbury New Zealand (2010-2011), Tohoku Japan (2011), Gorkha Nepal (2015), Kumamoto Japan (2016) and Ridgecrest USA (2019).

In Türkiye, the most destructive seismic sequence until the 2023 Kahramanmaraş event was the 1999 Marmara earthquakes, which took place within three months on the west segments of the North Anatolian Fault. The first major event ($M=7.4$) occurred on August 17, 1999, with the epicenter at Golcuk, Kocaeli, and the second major event ($M=7.2$) struck Duzce city, located in the east of the first fault rupture on November 12, 1999. The most affected residential area from these two major earthquakes was the Duzce city center, which was unable to recover from the first event in such a short time (Sucuoglu & Yilmaz 2001). The officially declared number of fatalities in the city during the August and November earthquakes were 264 and 478, respectively. In the downtown area of Duzce city, 4- and 5-story RC buildings that constitute the majority of the building stock had been severely damaged or collapsed. The field observations of Rathje et al. (2006) after the 1999 November earthquake revealed that many of the adversely affected buildings had originally been damaged during the 1999 August earthquake. The field team encountered buildings with 'X' markings (i.e. buildings that had been marked as unsafe to occupy by the local authorities) that had suffered heavy damage or total collapse and they stated that approximately one-third of the damaged buildings in the second earthquake had been already damaged during the first earthquake.

The adverse effects of sequential earthquakes have been experienced in a very intense manner during the recent Kahramanmaraş earthquake sequence in Türkiye that occurred in February 2023 and affected a very wide region containing 11 densely

populated cities. In this sequential event, 2 major ($M=7.7$ and $M=7.6$) and 2 strong ($M=6.6$ and $M=6.4$) destructive earthquakes took place within two weeks, causing significant damage to more than 300,000 buildings and a death toll of more than 50,000 people (Presidency of Strategy and Budget 2023).

Considering the importance of sequential events during past earthquakes from the above discussions, this study concentrates on the effects of sequential earthquakes on building typologies rather than their effects on individual buildings in the first stage of the study. It intends to plot a general picture regarding the significance of sequential earthquakes on the damage evolution of a typical building stock composed of mostly encountered construction typologies. In the second stage, the study focuses on specific RC building types under specific seismic sequences with a higher potential to cause damage evolution between the cascading events. This second stage involves the seismic analysis of more enhanced building simulations, i.e. 2-D RC frame models to generate fragility relationships. Overall, the ultimate goal of this study is to assess the significance of sequential earthquakes and answer questions like what the common properties of events within the sequence cause additional damage, which building types are the most affected ones from sequential earthquakes, and if there is an urgent need to revise seismic design and assessment methodologies to account for the effects of sequential events.

1.3 Objective and Scope

In this dissertation, the significance of sequential earthquakes is examined both at individual and population levels for RC frame buildings. For this purpose, strong ground motion records and their intensity parameters are collected from different databases and gathered together. In the first stage, the significance of sequential earthquakes is examined in the scale of building class by using simple and equivalent structural systems. In this analysis, the Ibarra-Medina-Krawinkler (IMK) deterioration model is employed (Ibarra *et al.*, 2005). By using the results of the first stage, the most critical seismic sequences and building sub-classes are further

examined in more detail by using 2-D frame models. Overall, the obtained results and derived conclusions are limited to the sequential earthquake database used and the simplified or 2D building models considered in this study. In this study, only RC frame buildings are considered in two different levels of structural idealization. Other RC building typologies like dual structures (frame + wall) or shear wall structures are out of the scope of this study. The main reason for excluding these structural types is that the evolution of seismic damage under sequential earthquakes can be observed more frequently for RC frame-type structures when compared with the other types. This trend has been verified in many earthquake sequences including the recent Kahramanmaraş, Türkiye earthquake. Another reason is that the inclusion of other RC construction types or 3-D frame models means a significant increase in the size of this parametric study in terms of modeling strategies, number of analyses, and interpretation of response statistics which has not been possible to handle within the period of this dissertation study.

The thesis report is composed of the following Chapters:

Chapter 2 presents the previous studies in the literature regarding the state-dependent response of RC structures when they are exposed to multiple seismic events.

Chapter 3 introduces the selected ground motion database of sequential events from major earthquake disasters that occurred in the last decades all over the world. It contains 30 earthquake sequences including 105 seismic events. Major intensity parameters of the events are also presented in this chapter.

State-dependent seismic responses of simple structural systems are discussed in chapter 4. Equivalent Single Degree of Freedom (ESDOF) Systems are employed for this purpose in order to simulate major RC building sub-classes. The Chapter also includes the response statistics from dynamic analysis of ESDOF structures and discusses the significance of sequential earthquakes for different typologies.

Chapter 5 is devoted to the state-dependent seismic response of planar RC frame models. Accordingly, these models are employed with appropriate material

characteristics. Both pushover and dynamic analysis are applied to the structural models in order to develop seismic demand and capacity. The obtained results and comparison of cases with main events and sequential earthquakes are given in this chapter in the form of fragility curves.

Chapter 6 gives a summary of the study, states the major findings based on the obtained results, and presents further recommendations.

The general flow chart of the study is provided in Figure 1.1:

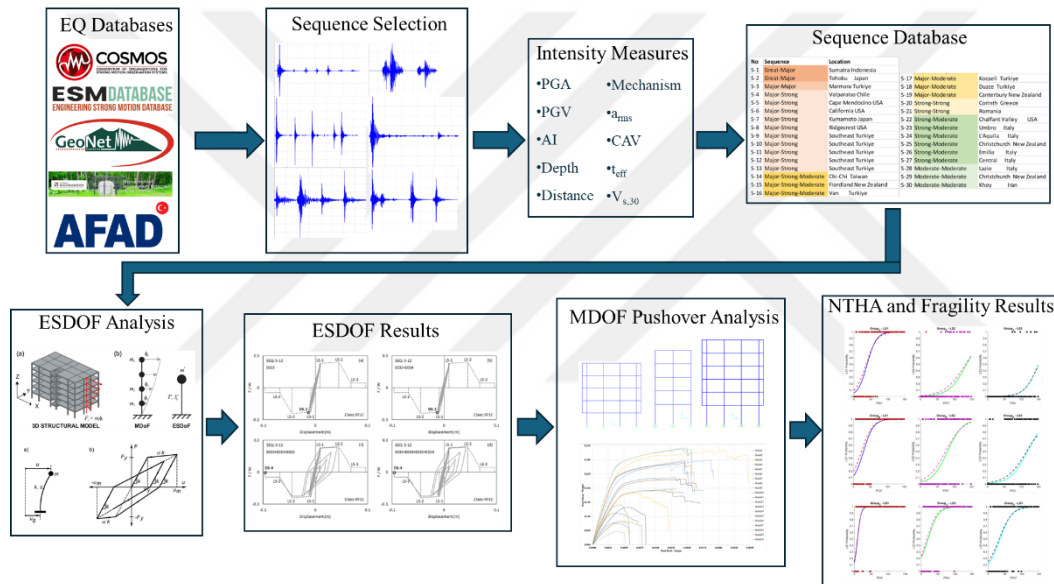


Figure 1.1. Flow Chart of the Study

CHAPTER 2

LITERATURE REVIEW

2.1 General

Analytical studies related to the seismic performance of structural systems under multiple earthquakes date back to the 1980s. Preliminary research belongs to Mahin (1980), who worked on the seismic analysis of elastoplastic SDOF systems during mainshocks and aftershocks. He stated that aftershocks can cause a slight increase in displacement ductility demand of structural models when compared to the performance after the main shock. Elnashai et al. (1998) stated similar conclusions to Mahin, considering the results they obtained from dynamic analysis by utilizing a large ground motion database including records from Europe, the United States, and Japan. They concluded that sequential events cause more critical values in terms of ductility demand when compared to stand-alone earthquakes. There is some other research in the early 2000s with the same motivation but lacks enough ground motion records to provide generalized conclusions.

The relevant research work has increased significantly in the last two decades, partly owing to the advances in the field of computational earthquake engineering and partly to the accumulation of real ground motion data from multiple events. The following sections present this literature by considering major aspects of the research such as ground motion modeling, structural modeling, and analysis approach.

2.2 Literature Work Regarding Ground Motion Modeling

Different approaches exist related to the nature of earthquake input used in these studies. The first decision is based on the form of the employed time history records: they can be real (recorded at the site) or synthetic (numerically generated). Then, the second decision is about the sequential series that has been used during the dynamic analyses. Three alternative approaches are to use scaled or unscaled replicate records within the sequential series, to use uncorrelated and scaled records in the sequence, or as the most convenient approach, to use actual sequential earthquakes.

Regarding the first approach, Hatzigeorgiou and Beskos (2009) constructed 4 groups of recorded motions with 28 events in each. The groups were classified according to different site conditions from hard rock to soft soil by using the USGS site classification system. The collected database is composed of records with a range of PGA values between 0.1g and 0.8g. The researchers applied each of these 112 records one, two, and three times (they called these Cases 1-3) with a time gap in between, which is equal to three times the single event duration. The fourth Case considers a foreshock and an aftershock as the down-scaled versions of the original record (mainshock) for which the scale factor (taken as 0.85) is calculated by using the Joyner-Boore (1982) empirical relationship. The considered cases for a typical record are shown in Figure 2.1. Hatzigeorgiou and Beskos (2009) also stated that the reason for not using real sequential earthquakes is that the analysis becomes complex and it is not easy to arrive at practically useful conclusions.

Regarding the second approach, Manafpour and Moghaddam (2019) selected two sets of uncorrelated and scaled ground motions with near-field and far-field characteristics based on the criteria by FEMA P695 (2009). The records are from earthquakes with magnitudes ranging between 6.5 and 7.9. The near-field ground motion set included records both with and without pulse effects. They combined these records by using scale factors for specified drift ratios and damage levels through incremental dynamic analysis (IDA) results.

Similarly, Mirrashid and Naderpour (2021) utilized three ground motion sets that had been developed by Baker et al. (2011). These sets were abbreviated as Set #1A, #1B, and #2. They can be considered as random seismic sequences of a primary event (mainshock) and a secondary event (aftershock) with a 10-second zero-acceleration interval in between. In total, 180 sequential series are constructed by randomly selecting from the considered sets (see an example in Figure 2.2).

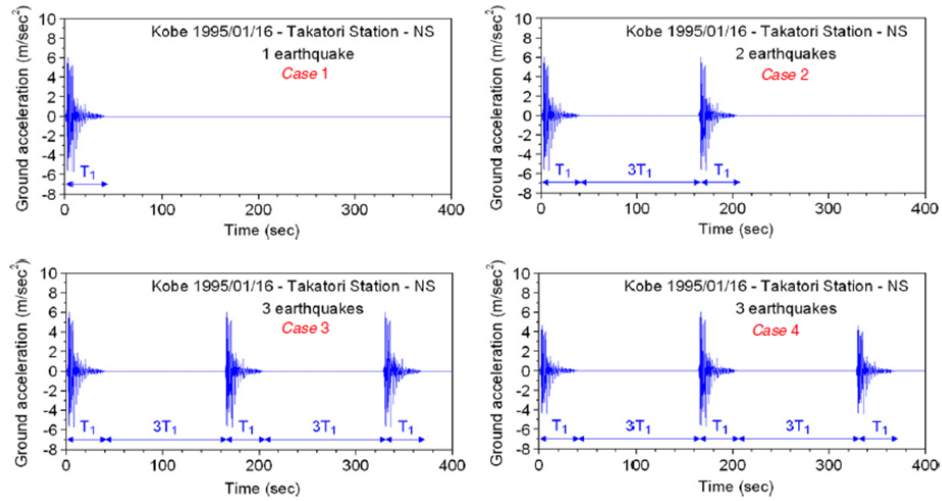


Figure 2.1. Sequential seismic input cases considered by Hatzigeorgiou & Beskos (2009)

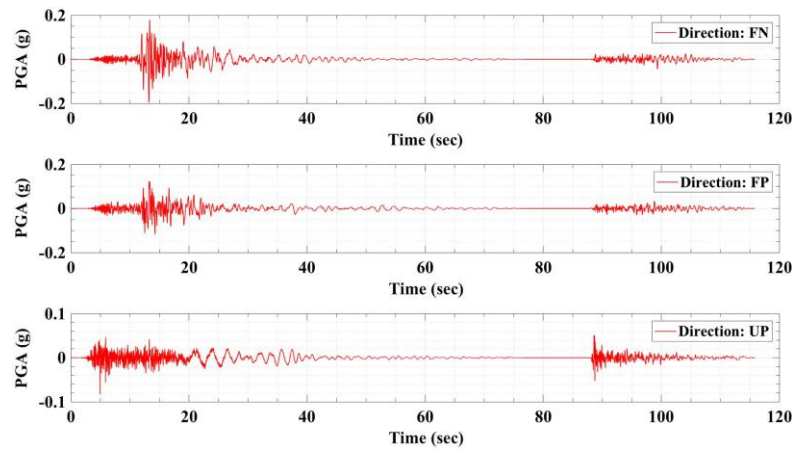


Figure 2.2. An example sequential seismic input from Mirrashid & Naderpour (2021)

Jeon et al. (2015) used the same three sets of Baker et al. (2011) with the fourth set (abbreviated as #3) to derive fragility curves of old RC building frames in the California region by using mainshock-aftershock records. Finally, a suite of 100 mainshock records is employed by using the distribution of PGV for each ground motion set as the intensity parameter. Each mainshock obtained from a scaled ground motion was randomly paired with an aftershock that was presented by an unscaled ground motion. In addition, 10 seconds of zero ground motion acceleration was utilized between the two events (mainshock and aftershock).

The most convenient approach is to use real sequential earthquakes to obtain response statistics of building structures. Hosseinpour and Abdelnaby (2017a) emphasized the significance of utilizing real sequential earthquakes and they constituted an extensive ground motion database of 464 actual records of sequential earthquakes from different locations around the world including New Zealand, Japan, and Chile. Changhai et al. (2018) also employed real sequences with one or two aftershocks from Imperial Valley 1979, Northridge 1994, and Chi-Chi 1999 earthquakes as the mainshock. The authors did not consider employing cascading earthquakes that had been recorded at the same station. Therefore, the total number of ground motion combinations that they utilized was 218 with one aftershock and 91 with two aftershocks. Moreover, Hatzigeorgiou and Liolios (2010) constructed a strong ground motion database with five real sequential earthquakes (five events from Mammoth Lakes 1980, two events from Chalfant Valley 1986, two events from Coalinga 1983, two events from Imperial Valley 1979 and two events from Whittier Narrows 1987) recorded within three days, at the same station with comparable distances to the fault. Hatzivassiliou and Hatzigeorgiou (2015) employed the same set of real sequential earthquakes as Hatzigeorgiou and Liolios (2010) in order to evaluate the seismic performance of 3D RC frame buildings. There are also studies that have employed simulated ground motion records instead of real ones. As an example, Trevlopoulos and Guéguen (2016) utilized groups of simulated ground motion records with varying magnitude-distance pairs. The time histories were generated by employing the spectrum-fit technique, for which the target spectrum

was selected as the mean response spectrum by considering suitable ground motion prediction equations (GMPEs). In their study, Hatzigeorgiou and Liolios (2010) also employed simulated earthquake records in addition to real records. They used 10 simulated records to generate 20 sequences with two events and another 20 sequences with three events. Tolentino and Ruiz (2015) constructed multiple sets of simulated ground motion records by taking reference to the acceleration time series recorded on 19 September 1985 during the Mexico City earthquake at the SCT site with soft soil conditions.

2.3 Literature Work Regarding Structural Modeling

Based on the modeling approaches of building structures under sequential earthquakes, different levels of structural idealization have been employed from simple single degree of freedom (SDOF) systems to complex 3D simulations. Among the researchers that conducted analyses with SDOF models, Di Sarno (2013) utilized three basic hysteretic models, which are the elastoplastic model, bilinear model, and stiffness degrading (Clough) model to calculate the inelastic structural response under sequential earthquakes. He carried out dynamic analyses with an equivalent viscous damping ratio of 0.5% and ductility ratios of 2, 4, and 6. Zhai et al. (2014) employed four different inelastic SDOF models (elastoplastic model, modified Clough model, pinching model, and degrading hysteresis model with both stiffness and strength deterioration) for periods of vibration between 0.1 and 5.0 seconds and strength reduction factors of 2, 3, 4, 5 and 6. The authors assumed a constant damping ratio of 5%. Zhang et al. (2017) employed elastoplastic SDOF systems with periods between 0.1 and 3.0 seconds and a constant damping ratio of 5%. Ductility-based and damage-based strength reduction factors were calculated with varying ductility ratios of 2, 4, 6, 8, and 10 and damage ratios according to the damage index proposed by Park and Ang (1985). A cantilever beam-column member with a concentrated mass at the free end was used by Manafpour and Moghaddam (2019) in order to idealize the SDOF response. For the considered model, the

inelastic action was provided by the help of a rotational spring at the fixed end. For the hysteretic response parameters of the rotational spring, a multi-parameter degrading hysteretic model by Lignos (2008), which had been embedded to the OPENSEES (McKenna *et al.*, 2000) analysis platform, was employed. The optimum time gap between mainshock and aftershock events was studied by Pirooz *et al.* (2021), who used linear elastic and inelastic (bilinear) SDOF models with periods between 0.05 and 7.0 seconds. They also employed three damping ratios 2.5%, 5%, and 7.5% during the seismic analyses.

There exist many studies in the literature that considered the seismic analyses of 2D structural models that had been subjected to multiple earthquakes. Hatzigeorgiou and Liolios (2010) conducted a parametric study on eight RC planar frames in two groups to demonstrate the increased displacement demands and damage caused by multiple earthquakes compared to single events. The first group (Class A buildings in Figure 2.3) was designed for seismic and vertical loads according to European codes while the second group (Class B buildings in Figure 2.3) was designed only for vertical loads to represent RC buildings, which have been constructed before the introduction of adequate seismic design code provisions. Frames in A and B classes do not differ in geometrical properties of their structural members or loading conditions applied, but the reinforcement details only. The reinforcement ratio of the members in Class A buildings is higher than the ones in Class B buildings and they satisfy the capacity design rule of having strong columns and weak beams in accordance with Eurocode 8 (2005). Typical floor height was taken as 3.0 m, except the first floor of 8-story buildings, which was taken as 4.0 m. Brittle shear failure of members was not considered by assuming sufficient lateral reinforcement for structural members. As for the material properties, concrete strength was taken as 20 MPa whereas steel yield strength (for both longitudinal and transverse reinforcement) was considered as 500 MPa.

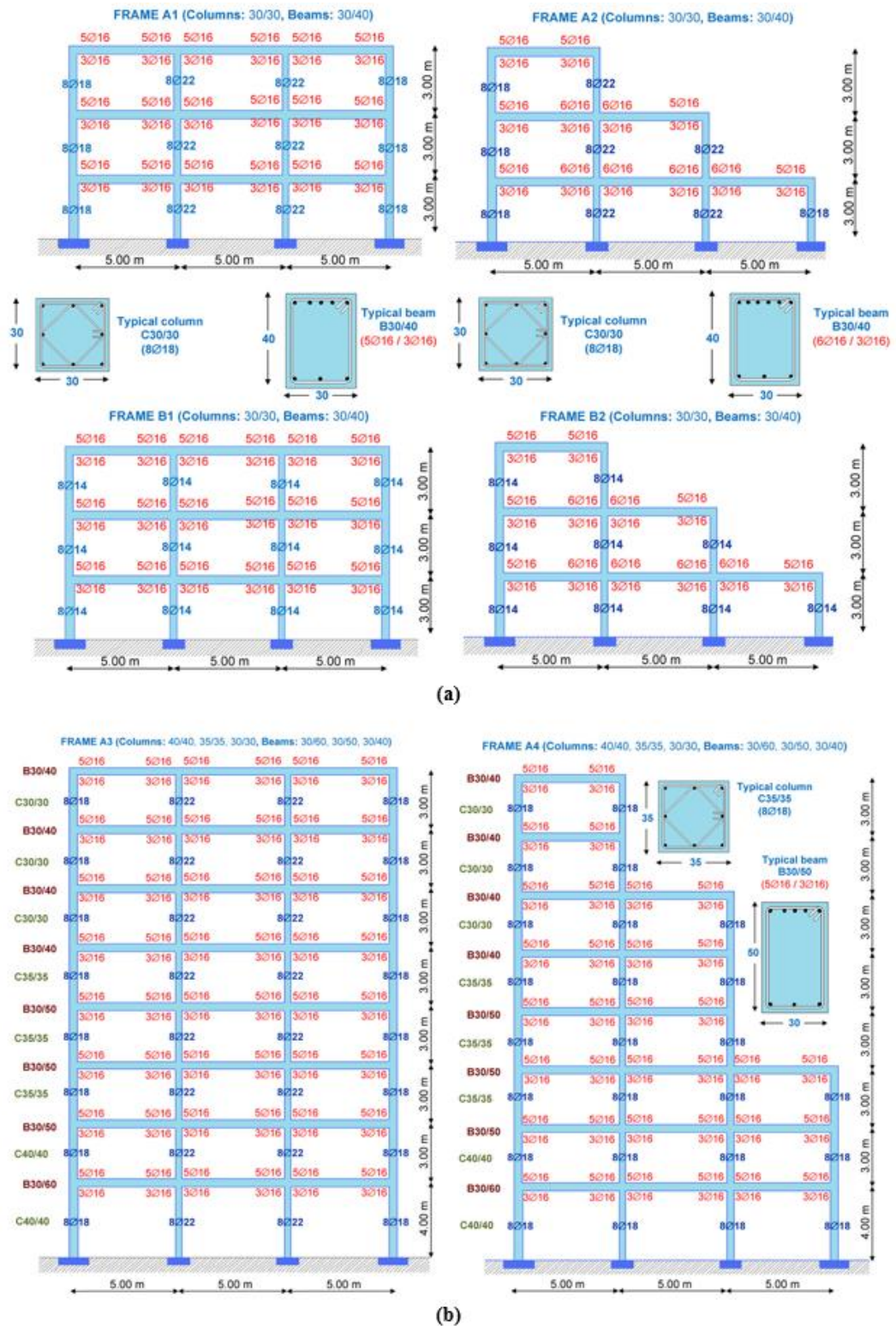


Figure 2.3. Planar frame models (Classes A and B) used by Hatzigeorgiou & Liolios (2010), a) 3 story buildings, b) 8 story buildings.

A 3-story RC frame building having two bays in the long direction and four bays in the short direction was designed by Abdelnaby and Elnashai (2015) for a particular location in California, United States. There are three versions of conceptual design regarding the considered building: gravity, direct, and capacity design. An intermediate frame in the transverse direction was selected by the authors as a planar model (Figure 2.4).

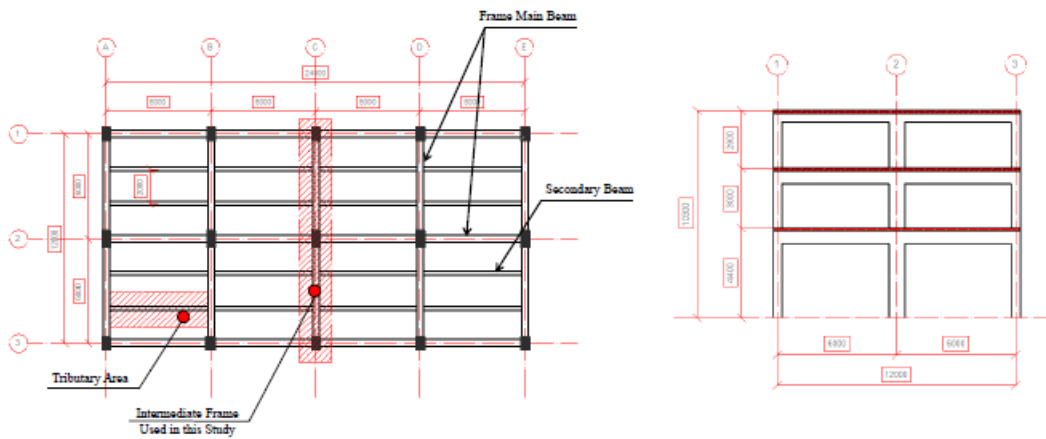


Figure 2.4. The selected intermediate RC frame by Abdelnaby & Elnashai (2015)

The gravity frame was designed by considering only factored vertical dead and live loads defined by the ASCE-07 Standard (2005). For direct and capacity-designed frames, seismic requirements of ACI 318-08 (2008) for ordinary moment frames ($R=3$) and special moment frames ($R=8$) were used, respectively. Then the designed frames were subjected to pushover analysis in order to determine their capacities in terms of stiffness, strength, and ductility. Figure 2.5 presents the comparison of the capacity curves of the considered frames. It is clear that the capacity-designed frame has the highest capacity in all aspects whereas the gravity-designed frame possesses the lowest values. The hinging mechanism of the frames that represents the local failure modes also exhibits the same trends as seen in Figure 2.6. In the gravity-designed frame, all plastic hinges seem to have formed at the column ends whereas a soft story mechanism seems to have caused the early failure of the direct-designed frame as seen in Figure 2.5. However, the capacity-designed frame is observed to

satisfy the strong column-weak beam concept with the hierarchy of hinging first at the ends of beams and finally at columns ends on the verge of collapse.

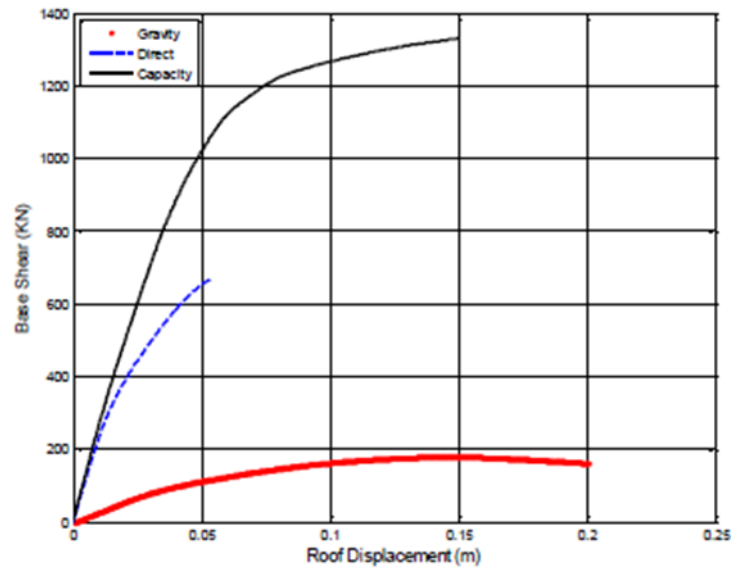


Figure 2.5. Comparison of the capacity curves for the three frames designed by Abdelnaby & Elnashai (2015)

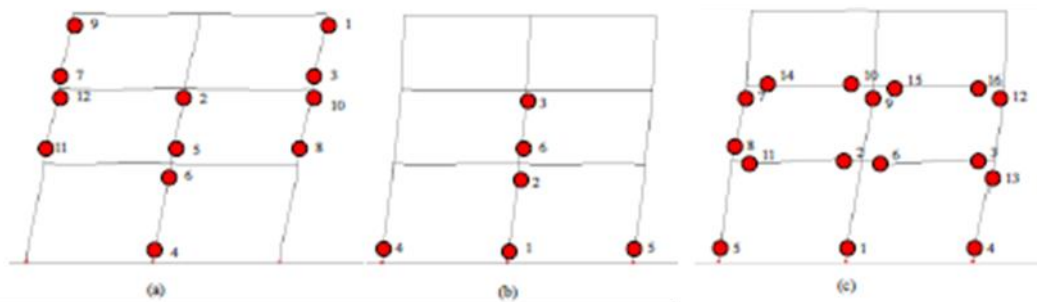


Figure 2.6. Comparison of the plastic hinge formation for the three frames designed by Abdelnaby & Elnashai (2015)

Hosseinpour and Abdelnaby (2017a) used two 8-story planar RC frame buildings that had been used by Hatzigeorgiou and Liolios (2010), previously to assess the seismic performance of structures when subjected to sequential earthquakes (see Figure 2.3b). The same authors extended the study by employing 3, 7, and 12-story planar RC buildings (Figure 2.7) to generate fragility curves of these structures under sequential earthquakes (Hosseinpour & Abdelnaby, 2017b). These buildings were designed per the ASCE-07 Standard (2005). The frames were considered as intermediate moment resisting frames with $R=5$. As the major material properties, the compressive strength of concrete and yield strength of steel were assumed as 30 MPa and 414 MPa, respectively.

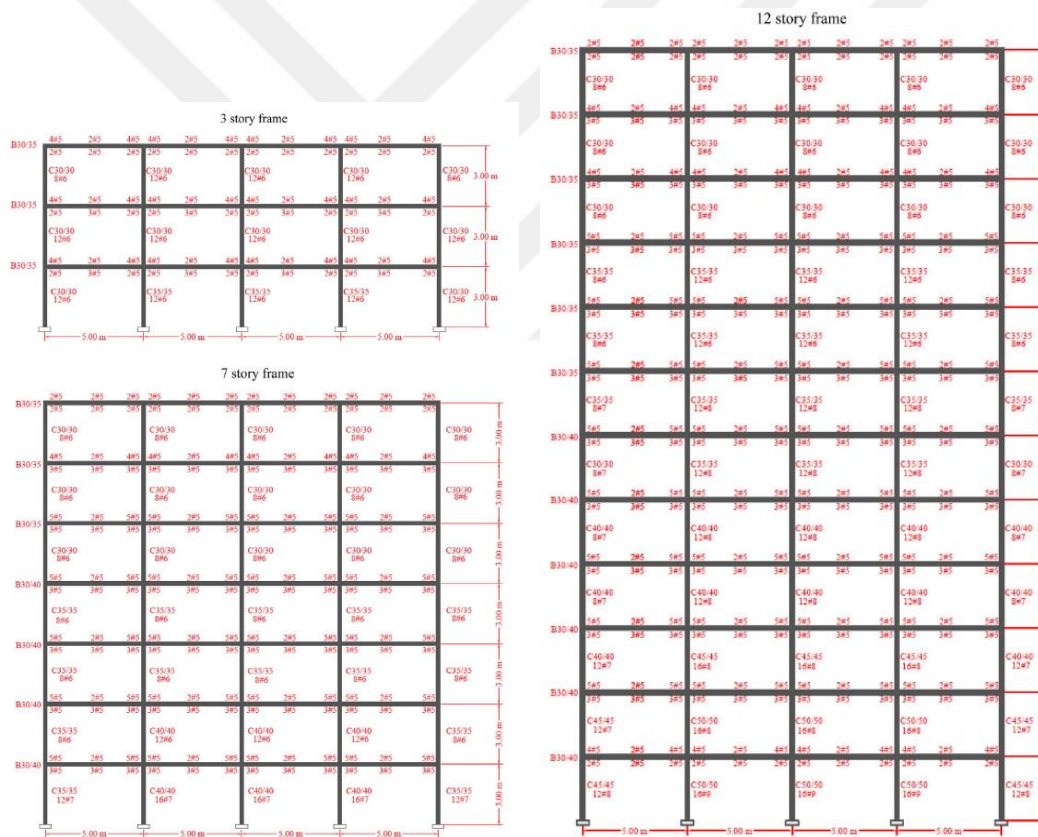


Figure 2.7. RC frames designed by Hosseinpour & Abdelnaby (2017b)

There are also some researchers that utilized 3D structural models to obtain seismic responses of RC buildings when subjected to sequential earthquakes. For instance, Hatzivassiliou and Hatzigeorgiou (2015) examined four different RC buildings with 3 and 5 stories that possess regular and irregular vertical framing (see Figure 2.8). The building models have two bays in each principal direction. The concrete grade used for the buildings is C20/25 whereas the steel grade for both longitudinal and transverse reinforcement is B500C. The structures were designed for vertical loads (dead and live) and earthquake loads ($PGA=0.24g$) per Eurocode-2 (2004) and Eurocode-8 (2005).

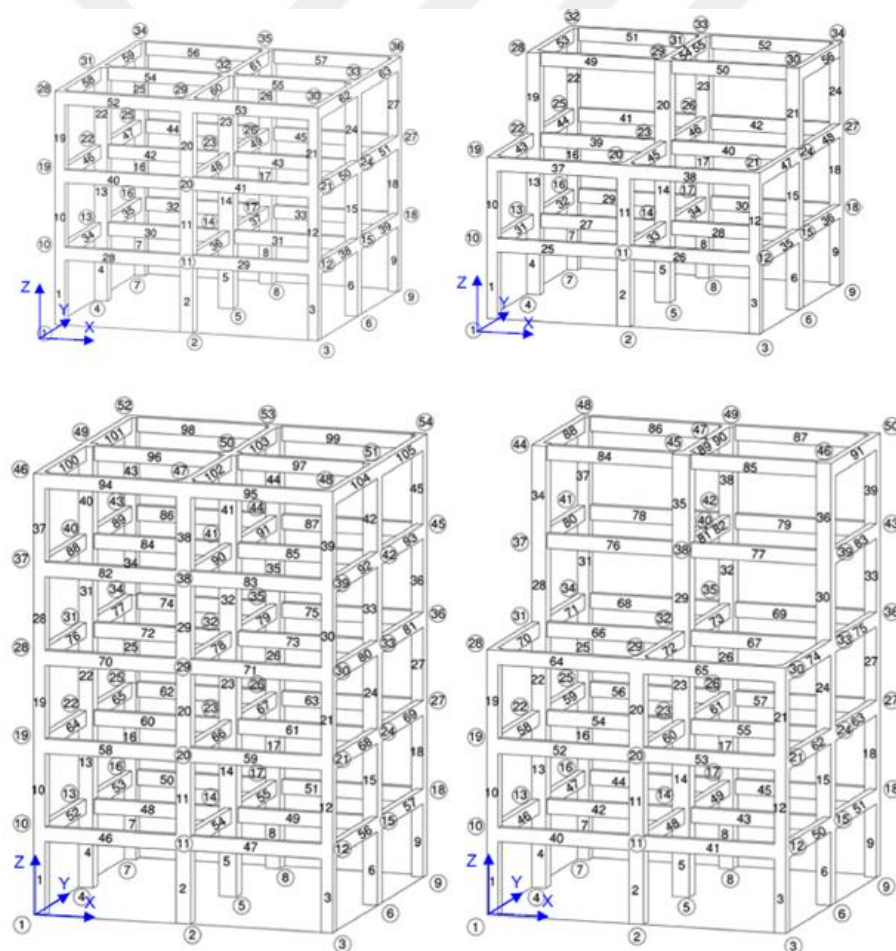


Figure 2.8. The demonstration of the 3D RC frame buildings used by Hatzivassiliou & Hatzigeorgiou (2015)

Three RC frame buildings with irregular plan layouts were selected by Oyguc et al. (2018). These were the SPEAR building, the ICONS building, and an existing school building from Van, Türkiye. The first two buildings represent structures designed with no seismic detailing. The third building was designed according to the Turkish earthquake code.

Four RC frame buildings with masonry infill walls were considered by Oggu and Gopikrishna (2020). The seismic design of these 4-story buildings was carried out for a specific location in India. In order to reflect the local characteristics of the construction practice in the region, construction features like setbacks along the height of the building and open ground story were implemented into these structural models.

2.4 Literature Work Regarding Analysis Approach

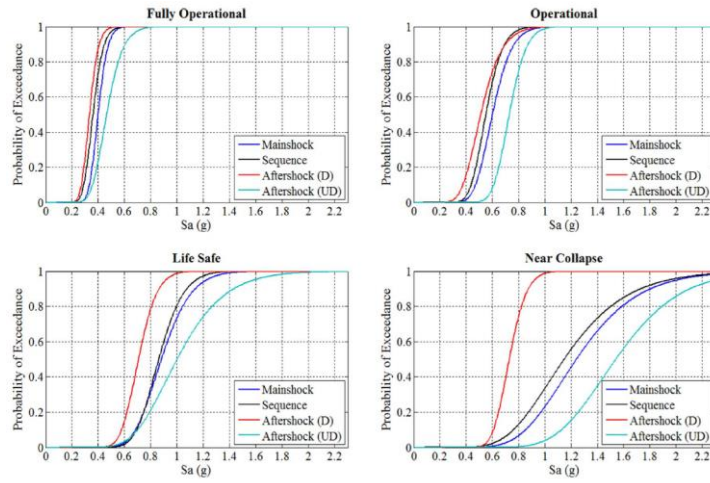
This section focuses on the analysis methods used, response parameters employed, and the interpretation formats of the response statistics obtained.

In terms of selected analysis methods, conventional nonlinear time history analysis of the structural models under sequential ground motions is the most common approach used by the researchers in previous studies (Hosseinpour & Abdelnaby 2017b; Tesfamariam & Goda, 2015; Han et. al., 2014; and Wen et. al., 2017). There are also studies that used Incremental Dynamic Analysis (IDA) (Hatzigeorgiou & Liolios 2010, Raghunandan et al. 2014)

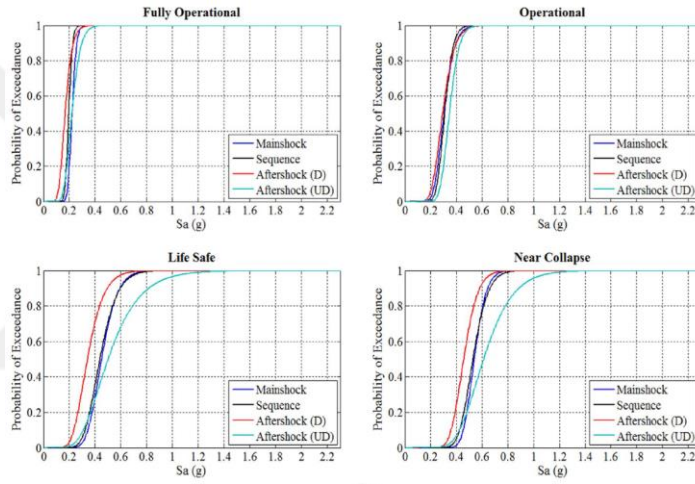
When it comes to response parameters monitored during the analyses, alternatives like (interstory) drift ratio, permanent drift, residual displacement, ductility (curvature, displacement, or cumulative), and damage indices (like Park and Ang index) can be provided as a short list. Among these alternatives, it should be mentioned that drift ratio is observed to be the most commonly used response parameter (Hatzigeorgiou & Liolios, 2010; Di Trapani et al., 2018; Shokrabadi et. al., 2018; Tesfamariam & Goda, 2015). There are also studies in which the

permanent drift ratio was utilized to represent the initial condition of an already damaged building during multiple earthquakes (Reveillere *et al.*, 2012). Noting that residual displacement is another good indicator during the post-earthquake decision-making process, Yazgan and Dazio (2011) stated that this parameter can be used to assess the existing condition of previously damaged buildings which may be exposed to aftershocks after a major earthquake. The use of damage indices was recommended by some researchers (Amadio *et al.*, 2003; Hatzigeorgiou & Liolios, 2010) to evaluate the damage state evolution of structural systems under sequential earthquakes.

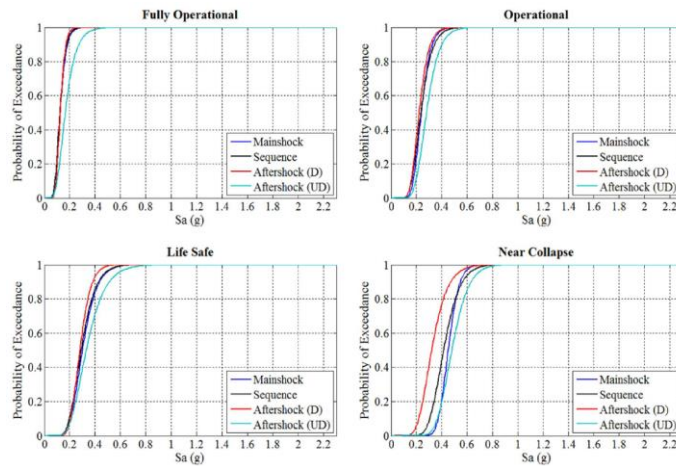
Considering the style of interpretation of the response statistics obtained during the analyses with sequential earthquakes, different approaches have been employed by the researchers. A common way to present the results is in terms of fragility functions. Figure 2.9 presents the fragility curves of 3, 7, and 12-story RC frame buildings for 4 performance states (fully operational, operational, life safety, near collapse) and also for the cases of mainshock (only), mainshock and aftershock (sequence), aftershock only without previous damage (aftershock, UD) and aftershock only with previous damage (aftershock, D) (Hosseinpour & Abdelnaby 2017b). The figures reveal that the effect of sequential earthquakes on the seismic performance of RC buildings is significant, especially for low-rise buildings. As the number of stories increases, this effect seems to diminish. The authors explain that taller buildings have higher displacement demands even under a single event so the increase during sequential events becomes relatively small.



(a)



(b)



(c)

Figure 2.9. Fragility curves for 4 performance states for a) low-rise, b) mid-rise, c) high-rise buildings (Hosseinpour and Abdelnaby 2017b)

2.5 Literature Work Regarding Significance of Sequential Earthquakes

Regardless of the application pattern of sequential earthquakes, structural modeling strategy, employed analysis approaches, and the way the response statistics have been interpreted, nearly all the related studies in the literature mentioned above emphasize the significance of sequential earthquakes on the seismic performance of RC buildings. Some of the remarkable findings from these studies have been stated in the below paragraphs.

Many researchers claimed that the structural deformations are observed to be increased due to sequential earthquakes. The deformations can be quantified in terms of lateral displacement (Di Sarno & Pugliese, 2021), drift (Abdelnaby & Elnashai, 2015; Hosseinpour & Abdelnaby, 2017b; Abdelnaby, 2018), ductility (Hatzigeorgiou & Liolios, 2010).

The development and distribution of plastic hinges throughout the structure can be different under single and multiple ground motions. This is a critical comment by (Hatzigeorgiou & Liolios, 2010) to point out how the collapse mechanism for the same building can vary due to sequential earthquakes. Furthermore, Abdelnaby and Elnashai (2015) stated that capacity-designed frames have superior performance when subjected to sequential earthquakes due to ideal plastic hinge mechanisms that allow stress transfer between members in a ductile manner and limited drifts.

According to the findings of some researchers (Hosseinpour & Abdelnaby, 2017a), irregular buildings (in plan and elevation) have been observed to be affected more by sequential earthquakes with increased seismic demands. According to these researchers, earthquake direction (especially for irregular buildings), vertical components of ground motions (especially for buildings with heavy overhangs on the first floor or the ones with setbacks), and polarity of the aftershocks can affect the structural response under sequential events.

Another important statement is related to the realization of the material properties during sequential earthquake analysis and implementation of material degradation

into the numerical models. Some researchers like Abdelnaby and Elnashai (2015) state that sophisticated degrading material models should be used during structural modeling and dynamic analysis in order to predict reliable nonlinear behavior (and also damage) of RC structures when subjected to multiple events.

As the final comment, a considerable number of researchers stated that conventional seismic design procedures that consider single earthquakes should be revised by considering the effects of multiple earthquakes. Finally, according to some researchers, increased seismic demands can be taken into consideration, especially for displacement-based design approaches (Hatzigeorgiou & Liolios, 2010).

2.6 Discussion of Literature Work

The review of existing studies on sequential earthquakes highlights key approaches and findings that directly relate to this research. The literature underscores the importance of considering the cumulative impact of multiple events on RC structures, an approach that aligns well with the methodology adopted in this study. Mahin (1980) and later Elnashai et al. (1998) established that aftershocks contribute to increased ductility demand, a phenomenon further supported by subsequent research. Unlike many earlier studies that relied on simplified ground motion sequences or synthetic records, this study employs real recorded sequences from the same stations, ensuring a more realistic representation of seismic effects.

A notable aspect of existing literature is the classification of ground motion modeling approaches. Studies such as Hatzigeorgiou and Beskos (2009) and Mirrashid and Naderpour (2021) have demonstrated the variability in seismic response based on if the sequential records are real, scaled, or artificially generated. While some studies, such as Jeon et al. (2015), have used PGV distributions as intensity measures for fragility analysis, this study advances the methodology by incorporating both scaled and unscaled records to generate a more comprehensive PGV distribution. This

ensures that the fragility curves produced here capture a broad spectrum of seismic intensities.

Structural modeling strategies in the literature range from simple SDOF systems to complex 3D models. Studies like Di Sarno (2013) and Zhang et al. (2017) have utilized hysteretic SDOF models, whereas Hatzigeorgiou and Liolios (2010) extended their analysis to multi-story 2D frames. This study integrates both SDOF and MDOF analyses, benefiting from the efficiency of SDOF models in parametric studies while ensuring the realism of MDOF nonlinear time history analysis. The adoption of nonlinear analysis is consistent with the leading studies (e.g., Hosseinpour & Abdelnaby, 2017a), which have emphasized its necessity in capturing sequential earthquake effects on RC frames.

In terms of seismic demand parameters, previous studies have commonly focused on drift ratios, residual displacements, and damage indices (Hatzigeorgiou & Liolios, 2010; Tesfamariam & Goda, 2015). This study follows the same convention, using drift ratio values to evaluate structural responses under cascading earthquakes. Furthermore, the findings of Abdelnaby and Elnashai (2015) regarding the significance of degradation effects in sequential earthquake analyses justify the nonlinear modeling approach used in this research.

Finally, fragility analysis has been extensively used in the literature to quantify the probability of damage states under sequential seismic loading (Hosseinpour & Abdelnaby, 2017b). The methodological consistency with prior studies validates this research, while the emphasis on PGV-based fragility functions provides an alternative perspective for evaluating seismic vulnerability. The literature confirms the need for multiple-earthquake considerations in seismic design, reinforcing the relevance and significance of this study's findings.



CHAPTER 3

SEQUENTIAL EARTHQUAKE DATABASE

3.1 Introduction

The selection of ground motion records for the purpose of dynamic analysis is an important issue in the field of earthquake engineering research. This selection process becomes more critical in the case of constructing a ground motion database with sequential events. As explained in Chapter 2, different approaches exist related to ground motion modeling in sequential earthquake databases. Most of the studies do not employ actual sequential events during the analysis or they use a very limited set of records. However, it is necessary to utilize a sufficiently large set of actual sequential events in order to obtain more reliable results from seismic analyses. In this study, 30 sequential events from different parts of the world containing 105 ground motion records have been selected to examine the significance of sequential earthquakes on the seismic response of building structures. More importantly, the dataset includes events from the recent 2023 Kahramanmaraş, Türkiye earthquake sequence, which stands as one of the most significant, sequences recorded in the instrumental era of modern earthquake engineering. Consequently, the sequential ground motion database utilized in this study represents a key novelty of this research.

The chapter starts with the presentation of the sequential ground motion database used in this study, emphasizing the most remarkable events within the database. Then the classification of the sequences, ground motion intensity parameters of earthquakes, and their role within the sequences are discussed in detail.

3.2 Sequential Ground Motion Database

There seems to be no consensus in the literature about the definition of sequential earthquakes (also called mainshock-aftershock series or repeated earthquakes in some references) due to the ambiguity of defining a stand-alone earthquake as a foreshock, aftershock or mainshock among a cluster of events in a time frame. This study defines sequential earthquakes as the events that have been recorded at the same station (i.e., same site conditions) within 4 months of time period between any two events, with magnitudes $M > 5$ for significance with respect to engineering concern (i.e., with the potential of structural damage). The reason for setting 4 months of interval is that generally the governments or municipalities try to demolish the damaged structure or retrofit it. Therefore, it is not possible to track the cumulative effects of main events and subsequent events on the buildings. Hence, the sequential ground motion database employed in this study is composed of records from 30 sequences with 105 earthquakes that fit the above criteria for each event. Concerning that, different ground motion databases (COSMOS, PEER, AFAD, ESM, KNET, GEONET) have been considered to generate a special ground motion database regarding sequential events. The ground motion database is provided in Appendix A with the main parameters of each station and the earthquake that the records were taken from. The first column presents the abbreviations of each sequence as S-1, S-2, up to S-30. The earthquake sequences consist of a different number of events which vary in magnitude. Hence, the second column gives the classification of each sequence according to the magnitude of events that build up that sequence.

For the magnitude classification, the conventional earthquake magnitude scale that correlates with the expected effects on the built environment is considered as presented in Table 3.1. In this respect, there are 2 Great-Major, 1 Major-Major, 10 Major-Strong, 3 Major-Strong-Moderate, 3 Major-Moderate, 2 Strong-Strong, 6 Strong-Moderate and 3 All-Moderate sequences within the database. The majority of the available sequences are from Major-Strong and Strong-Moderate sets that also

include 5 different stations from the 2023 Kahramanmaraş, Türkiye earthquake sequence.

Table 3.1 Classification of earthquakes in terms of magnitude

Magnitude Range	Descriptive Term	Typical Effects
5.0-5.9	Moderate	Can cause major damage to poorly constructed buildings, and slight damage to well-designed buildings.
6.0-6.9	Strong	Destructive in populated areas, major damage to most buildings.
7.0-7.9	Major	Serious damage over larger areas, widespread damage, and ground fissures.
8.0-8.9	Great	Severe destruction, major damage in large areas.
9.0 and greater	Extreme	Devastating in populated areas, total destruction and permanent changes to landscape.

The next three columns in the ground motion database present the location of the earthquake, the name (or code) of the station, and the site class for each station. Site classes are attained from the available Vs30 values, by using the classification system in the ASCE Standard (ASCE 2017).

The following column gives the abbreviation of each earthquake with a sequence (that are called events in this study) as E-001, E-002, up to E-105. As mentioned above, the number of events in each sequence may vary according to the selection criteria. Thus, there are sequences with only two events as well as sequences with up to six events.

The next three columns present the date, time, and magnitude (M_w) of each event. Noting that magnitude is a simple, yet essential parameter to quantify the severity of

an earthquake, a parameter denoted as MBRI (that means “magnitude-based relative intensity”) has been introduced in the next column of the database. The reason for introducing MBRI is to provide a relative intensity measure for a pre- or post-event with respect to the largest magnitude main event. It is based on the simple assumption that the intensity levels of two earthquakes measured on a seismograph can be compared by Equation 3.1.

$$\log(I_2/I_1) = M_2 - M_1 \quad (3.1)$$

In this equation, I_1 and M_1 stand for the relative intensity and magnitude of the main event, I_2 denotes MBRI and M_2 represents the corresponding magnitude of the event for which MBRI is calculated. For the sake of consistency, I_1 is taken as unity in all the calculations. Moreover, Modified Mercalli Intensities (MMI) of the earthquakes are also added to the database to see how destructive the events are. Besides that, shake maps of each event are also presented in Appendix E. The last three columns of the database are devoted to the seismological parameters of each event as epicentral distance (in km), event depth (in km), and the faulting mechanism.

The sequences and the related events within the database have been collected from significant disasters that caused damage and loss to the built environment. In order to justify the reasons that these sequences have been selected for this study, the following sub-sections provide some information in chronological order about the seismic sequences selected in this study, also including the devastating 1999 Marmara (Kocaeli and Düzce) earthquakes and the extraordinary 2023 Kahramanmaraş, Türkiye sequence.

3.2.1 Gulf of Corinth, Greece (1981) Earthquake Sequence

The earthquakes with magnitudes $M=6.7$, 6.4 , and 6.4 took place on February 24, 25, and March 4, 1981, in the eastern region of the Gulf of Corinth, Greece. Some researchers worked on the complex faulting mechanism that caused three strong earthquakes and a vast number of aftershocks with smaller magnitudes and claimed

that the sequence is associated with the active extension of the Gulf of Corinth (King *et al.*, 1985). The first two events with a time interval of only 6 hours had very close epicenters, which caused severe shaking, especially in the city of Corinth. The third event, which occurred 10 days later, mainly affected the villages in the region. Since the events happened in a very short period of time, it was not possible to distinguish the damage distribution after each event. A total evaluation of earthquake loss reveals that over 7000 buildings were damaged with a death toll of 22 (Papanikolaou *et al.*, 2009). Among these three events, only the first two events are included in the database as E-065 and E-066 in S-20 at station KORA, since it has not been possible to find a station with all three recordings in any of the available databases searched in this study.

3.2.2 Lazio, Italy (1984) Earthquake Sequence

An earthquake with $M=5.9$ occurred on May 7, 1984 in the Central-Southern Italy region within an area at the borders of Latium, Abruzzo, and Molise. The first event was followed by a second one (which may be regarded as an aftershock with $M=5.5$) 4 days later on May 11, 1984. The first event killed 7 people whereas the second one was reported to cause 3 deaths. According to Graziani *et al.* (2017), the first event caused widespread moderate damage, and worsened, in some cases, by the second event of 11 May. The earthquakes were reported to damage particularly small mountain villages with old and poorly constructed building stock with widespread moderate damage (Westaway *et al.*, 1989). These authors also complained about the lack of field data to constitute seismic intensity information individually for each event due to the very short time period between these two events.

Both events are available in the database as E-099 and E-100 in sequence S-29 recorded at the station with the code ATN.

3.2.3 Valparaiso, Chile (1985) Earthquake Sequence

Santiago, Chile was struck by a major earthquake with $M=7.8$ on March 3, 1985, followed by a strong magnitude aftershock with $M=6.3$ within one hour. Since the events were very close to each other, the total destruction was reported as 177 people dead, 2575 people injured, more than 80,000 houses damaged or destroyed, and also a million people homeless. (Booth & Taylor 1988) The main event was felt with a maximum intensity of VIII on the Mercalli intensity scale (Wood *et al.*, 1987). Both earthquakes from this sequence exist in the database as E-010 and E-011 in sequence S-4 recorded at the station with code 4407.

3.2.4 Chalfant Valley, USA (1986) Earthquake Sequence

The Chalfant Valley earthquake sequence is composed of a foreshock ($M=5.9$), a mainshock ($M=6.4$), and two large aftershocks ($M=5.6$ and 5.8), which took place between July 20 and July 31, 1986 in Southern California, USA. This strong-moderate seismic sequence injured 2 people and caused economic losses in the region with an approximate amount of 2.7 million dollars. The earthquake mostly affected mobile homes as they were toppled from their foundations. Only two structures collapsed during the mainshock. Also, some damages were reported in the potable water and wastewater pipeline networks (Brewer, 1989).

All four events from this sequence (S-22) are present in the database as E-069 to E-072 at the station coded as 54171. This is one of the sequences for which the main event (i.e., the highest magnitude event) is not the first one within the sequence.

3.2.5 Vrancea, Romania (1990) Earthquake Sequence

Two strong earthquakes with magnitudes $M=6.9$ and $M=6.6$ occurred within 13 hours on May 30-31, 1990 that affected Vrancea county of Romania as well as neighboring countries like Moldova, Ukraine, and Bulgaria. Although the

earthquakes were quite strong, fortunately, they did not cause extensive damage to the built environment, only killing 9 people. The reason for this uncorrelated damage with magnitude is due to the extreme focal depths of these earthquakes (approximately 90 km), which is typical of the seismicity and faulting mechanism in the region (Pomonis *et al.*, 1990). This is also the reason why the earthquakes were felt in a very wide area including several countries.

The aforementioned strong events exist within the sequential earthquake database with abbreviations E-067 and E-068 in sequence S-21 recorded at the station with the code VRI.

3.2.6 California (1992) Earthquake Sequence

This major-strong California earthquake doublet also known as Landers (M=7.3) and Big Bear (M=6.4) doublet occurred within 4 hours in Southern California on June 28, 1992. The epicenters of the earthquakes observed in a station are shown in Figure 3.1. Since these earthquakes affected the sparsely populated Mojave region of California, structural damage after the earthquakes was not extensive despite the high magnitudes. The Landers earthquake had a maximum Mercalli intensity of IX with 3 people deaths whereas the Big Bear earthquake had a maximum intensity of VIII with no casualties but 63 injuries. The Big Bear earthquake was originally considered as an aftershock, but then, it was declared as a separate but related earthquake by the United States Geological Survey (USGS). Therefore, these two events (E-015 and E-016) are included in the database of this study as a regional earthquake sequence (S-6) in Southern California. The strong motion station selected from the database is with the code 12149.

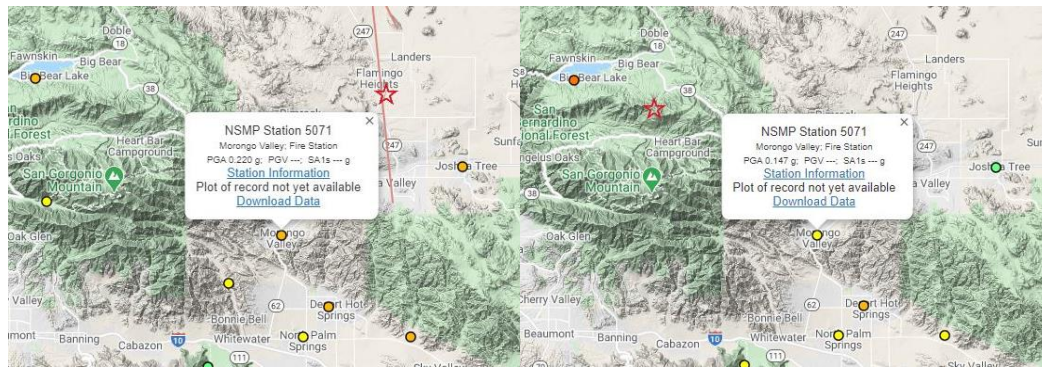


Figure 3.1. Epicenters of Landers (left) and Big Bear (right) earthquakes

3.2.7 Cape Mendocino (1992) Earthquake Sequence

This sequence took place at one of the triple junctions, where North America, Pacific, and Gorda plates interact (Oppenheimer *et al.*, 1993). The first event ($M=7.0$) occurred on April 25, 1992, close to the Petrolia town in California. The next day, two more events took place with $M=6.6$ within hours. Damage and loss in the affected area had been reported to be caused by the first event. However, fires were triggered by the cascading events due to the collapse of already damaged structures. Oppenheimer *et al.* (1993) also stated that the structural damage caused by this seismic sequence was rather low due to the low population density as well as the dominance of small-sized, wood-frame buildings in the affected area.

All these three events are within the gathered database as E-012, E-013, and E-014 in the sequence S-5 which was recorded at the station with the code 89156.

3.2.8 Umbria Marche, Italy (1997) Earthquake Sequence

Multiple events took place between September 26 and October 14, 1997, with strong and moderate magnitudes in Central Italy, close to the regions of Umbria and Marche. The second event in the series has the highest magnitude ($M=6.0$) and has been regarded as the main event in this study. The remaining 5 events have

magnitudes between 5.2 and 5.7. These multiple events and the related structural damage have been extensively discussed by Spence and D'Ayala (1999). The authors stated that most of the structural damage took place during the first two events on September 26, 1997, with failures of traditional stone masonry buildings. 11 people died during these multiple events. Four of them were killed tragically during the inspection of the damage, experienced by the Basilica of St. Francis in Assisi during the first event as the roof of the structure collapsed during the second event (Figure 3.2). Spence and D'Ayala (1999) also stated that the number of collapsed buildings and the death toll were not correlated. They concluded this due to the fact that more and more deficient masonry buildings collapsed during multiple events while the people had already been warned by these events and emptied their dwellings.

In this sequence (S-23), which has also been added to the database, there are all 6 events (E-073 up to E-078), where the second one (E-074) is the main event with the highest magnitude. All ground motions had been recorded at the station with the code CLF.



Figure 3.2. (a) The instant that the collapse occurred during the second event (left),
(b) the collapsed vault (right)

3.2.9 Marmara (1999) Earthquake Sequence

This sequence refers to one of the most destructive disasters in the history of the Turkish Republic, which contains the August 17, 1999, Kocaeli earthquake ($M=7.6$) as the first event and the November 12, 1999, Düzce earthquake ($M=7.1$) as the second event that took place 3 months later on the North Anatolian Fault Zone. Both earthquakes affected the Marmara region of Türkiye intensively. During the Kocaeli earthquake, 17000 people were killed and more than 44000 people were injured. The earthquake cost approximately \$3-8 billion (Parliamentary Commission of Inquiry, 2010). In the Düzce earthquake, 845 people were killed and more than 4900 people were injured and it cost around \$1 billion. Epicenters of these two events are shown in Figure 3.3.



Figure 3.3. The epicenters of the Kocaeli (top) and the Düzce (bottom) earthquakes

The most affected residential area by these two major events was the city of Düzce which could not recover from the first event in a short period of time. Sucuoğlu and Yılmaz (2001) reported the structural damage in the city of Düzce after these two earthquakes. They stated that the ratio of damaged households increased after the

second earthquake although it had a lower magnitude. The damage statistics in Table 3.1 provide physical evidence regarding the cascading effects of sequential earthquakes. After the second event, almost all the building stock in Düzce had experienced different levels of damage. Therefore, the city was demolished almost completely and reconstructed.

Table 3.2 Percent of damaged households in two earthquakes (Sucuoğlu and Yılmaz, 2001)

Earthquake	Damage States			
	None	Light	Moderate	Severe/Collapsed
17 August 1999	70.1	9.2	13.8	6.9
12 November 1999	22.5	19.6	28.7	29.2

Rathje et al. (2006) conducted field observations following the November 1999 earthquake and found that many of the severely affected buildings had already sustained damage during the August 1999 earthquake. The team came across buildings marked with ‘X’ markings (indicating they had been deemed unsafe for occupancy by local authorities) that had experienced significant damage or collapse (Figure 3.4). They estimated that about one-third of the buildings damaged in the second event had previously been affected by the first event.



Figure 3.4. An example of a collapsed building during the Duzce earthquake that had already been damaged and tagged during the Kocaeli earthquake (Rathje *et al.*, 2006)

1999 Marmara earthquakes are represented by 3 different sequences in the database. Sequence S-3 contains one ground motion recorded from each event (E-008 and E-009) at the station with the code 8101. The second sequence belongs to the 17 August Kocaeli earthquake (E-056) and two aftershocks (E-057 and E-058) recorded on August 31, 1999 (M=5.1) and September 13, 1999 (M=5.8) at station 5401. The third sequence belongs to the 12 November Düzce earthquake (E-059) and two aftershocks (E-060 with M=5.6 and E-061 with M=5.3) recorded on the same day at station 1401.

3.2.10 Chi Chi, Taiwan (1999) Earthquake Sequence

A major earthquake ($M=7.6$) took place in the central region of Taiwan on September 21, 1999. It was officially declared that there were about 2400 dead, 10,002 injuries, and more than 100,000 building collapses (D'Amico *et al.*, 2010). As Ma *et al.* (1999) mentioned, this earthquake is unique with many aftershocks of magnitude greater than 6.0 in the sense that the aftershock sequence is more energetic than average for an earthquake of this size. They also stated that large aftershocks caused panic, damage, and some additional casualties.

Within the ground motion database in this study, Chi Chi, Taiwan (1999) sequence (S-14) is presented by the main event (E-041) and 5 cascading events (E-042 up to E-046) with magnitudes varying between 5.9 and 6.3. The ground motions for all the events had been recorded at station TCU068.

3.2.11 Fiordland, New Zealand (2003) Earthquake Sequence

This sequence (S-15) includes a major earthquake with $M=7.1$ (E-047) on August 21, 2003, and also two following events (E-048 and E-049) on the same day with $M=5.9$ and $M=6.1$. Since the main event had struck the remote region of Fiordland in the South Island of New Zealand, the maximum Mercalli intensity was VII (very strong) and the physical damage was generally minor (McGinty and Robinson 2007). The ground motions that have been included in the database of this study from the aforementioned 3 events had been recorded at station MANS.

3.2.12 Sumatra, Indonesia (2007) Earthquake Sequence

Sumatra, Indonesia serves as the first recorded example of megathrust earthquake sequences in the ground motion database. In this region, a powerful earthquake with a magnitude of 8.4 was followed by a 7.9-magnitude earthquake within 11 hours along the Sunda Trench, off the coast of Sumatra. The following day, a third

significant earthquake occurred with a magnitude of 7.0 (Haridhi *et al.*, 2018). Although the epicenters were located far from densely populated areas, these seismic events resulted in 25 fatalities. Additionally, the largest earthquake triggered a tsunami.

These three great/major events have been added to the database as E-001, E-002, and E-003, and the sequence is coded as S-1. The ground motion records had been recorded at PSKI.

3.2.13 L'Aquila Italy (2009) Earthquake Sequence

An earthquake with $M=6.1$ occurred on April 6, 2009, close to L'Aquila, the capital city of the Abruzzo region of Italy, which caused over 300 deaths and thousands of homeless people. The main event was followed by several cascading events with $M>5.0$ (Aydan *et al.*, 2009). Among these, the event with $M=5.5$, which took place one day later and was strong enough to be felt in Rome, caused further damage. Aftershocks created safety hazards for rescue teams searching for injured victims trapped in unstable structures in L'Aquila's historic center, a medieval city. Crews used cranes and backhoes to clear loose bricks and broken timbers, fully aware that any aftershock could cause the collapse of severely damaged walls or parapets. Additionally, the aftershocks inflicted ongoing psychological distress on young children and the elderly, who had already been traumatized by the main earthquake on April 6, 2009. In response, the Italian government temporarily relocated thousands of residents from the epicentral area. The aftershocks also led to the near-total collapse of the Anime Sante Basilica's dome, which had already suffered severe damage from the main event. Further structural collapses occurred in L'Aquila and nearby towns (Chiaraluce *et al.*, 2011).

In the database, there exist 5 events in this sequence (S-24). The first one (E-079) is the main event whereas the others (E-080 up to E-083) are the cascading events (i.e., aftershocks) with $M>5.0$ that occurred within 3 days of the main event.

3.2.14 Canterbury (2010) and Christchurch (2011) Earthquake Sequences in New Zealand

A series of earthquakes occurred near the city of Christchurch and the Canterbury region of New Zealand, starting from early September 2010 up to late December 2011. These events provide valuable contributions to the sequential earthquake database. The largest magnitude event ($M=7.2$) took place on September 3, 2010, with an epicenter located 40 km to the west of Christchurch city and cost around 2.1–3.5 billion dollars due to damage in several fields of service. During this event (also known as the “Darfield earthquake”), 500 buildings were damaged to be demolished and approximately 150,000 buildings experienced moderate and low damage. There were no casualties, but 100 people were injured (Bannister & Gledhill 2012).

After 6 months, another earthquake with $M=6.3$ occurred close to the same region, i.e., near Christchurch city center. Although the magnitude was lower than the 2010 earthquake, 185 people were killed and 2000 people were injured, meaning that it caused more significant damage. The cost was estimated to be 12 billion dollars. After the earthquake, 45% of the inspected buildings were classified as red (high) or yellow (moderate), and more than 800 buildings were marked to be demolished. In total, 170,000 buildings were damaged slightly or moderately. (Kaiser *et al.*, 2012). The epicenters of the 2010 and 2011 earthquakes are shown in Figure 3.5.

In the database, the 2010 sequence (S-19) includes the $M7.2$ main event (E-062) and two aftershocks that took place on the same day with moderate magnitudes ($M=5.0$ and $M=5.5$). There are 2 sequences related to the 2011 New Zealand events in the ground motion database. The first one (S-25) belongs to the 2011 sequence with the main event (E-084) and four cascading events (from E-085 up to E-088) with magnitudes varying between 5.3 and 6.0. The first three events occurred in February 2011 whereas the remaining two events took place in June 2011. The second sequence (S-29) is composed of three moderate events (E-101, E-102, and E-103) that occurred in December 2011.

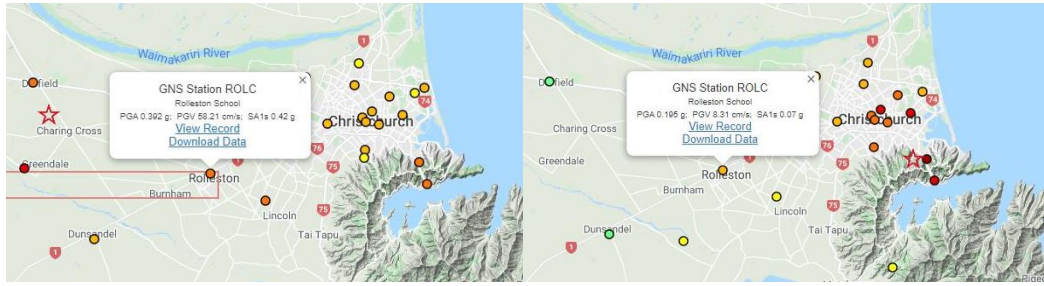


Figure 3.5. Epicenters of 2010 (left) and 2011 (right) New Zealand earthquakes

3.2.15 Van, Türkiye (2011) Earthquake Sequence

An earthquake with $M=7.3$ occurred in eastern Türkiye, close to the town of Erciş on October 23, 2011, and as a result, 2200 buildings were damaged or destroyed (Akansel *et al.*, 2011). In this earthquake with the highest Mercalli intensity IX, 604 people were killed, and 4152 people were injured. After 17 days, another earthquake with $M=5.6$ occurred and 25 buildings were destroyed in the city of Van due to the damage accumulation of these two cascading events. 40 people were killed and 260 people were injured after this aftershock although many people were still outside due to ongoing events (Akkar *et al.*, 2012). One of the buildings destroyed in this aftershock was a six-story hotel, where many national and international reporters and rescue teams had been staying. A Japanese aid worker died due to a collapse. This unfortunate event revealed the importance of structural safety during cascading events.

In the ground motion database, the Van sequence (S-16) is composed of 6 events. The first one (E-050) is the $M=7.3$ earthquake, which is followed by 3 aftershocks (E-051, E-052, E-053). The fifth event (E-054) is another aftershock with $M=5.3$ on October 25, 2011. The last event (E-055) is the $M=5.6$ event on November 9, 2011, in which there were collapses and physical losses as mentioned above. The recordings are from the station coded as 1302.

3.2.16 Tohoku, Japan (2011) Earthquake Sequence

This is another megathrust earthquake sequence in the ground motion database. Tohoku earthquake sequence took place with $M=7.3$ first event, then $M=9.0$ main event, which is followed by two major events with $M=7.9$ and $M=7.7$ within 3 days (Zhao 2015). Most of the damage and loss was due to the megathrust mainshock that generated a major tsunami along the Pacific coastline of Japan's northern islands. According to Carvalho et al. (2020), the Tohoku seismic sequence caused a considerable decline in Japan's real GDP growth in the year following the disaster.

In the database, this sequence is abbreviated as S-2 and the aforementioned events have been coded from E-004 up to E-007. The records have been obtained from the station MYG015. It should be mentioned that since the earthquakes took place under the ocean, the epicentral distances for all events are more than 100 km.

3.2.17 Emilia, Italy (2012) Earthquake Sequence

A series of events with moderate and strong magnitudes took place in Northern Italy between May 20 – May 29, 2012 (Lauciani *et al.*, 2012). During the first event ($M=6.1$) on May 20, 2012, 7 people were dead and some historical structures in the region were damaged (Acito *et al.*, 2014). On May 29, 2012, there was another strong earthquake ($M=6.0$) that caused 20 more casualties and the collapse of already damaged buildings.

In this sequence (S-26), there are five events. The first one (E-089) is the main event with $M=6.1$. Then comes an aftershock (E-090) on the same day with $M=5.1$. The next three events seem to have occurred on May 29, including an $M=6.0$ event (E-091) and two $M=5.5$ events (E-092 and E-093). The recordings are from the station coded as MRN.

3.2.18 Kumamoto, Japan (2016) Earthquake Sequence

Kumamoto earthquake sequence is composed of one strong ($M=6.2$) and one major ($M=7.0$) earthquake that occurred on the 14th and 16th of April, 2016 in southern Japan. In the first event, there were 9 deaths and 1108 injuries. After 38 hours, the second earthquake took place. Its effects were more catastrophic. 41 people were killed and 2021 people were injured. This earthquake sequence was different from the others since the second event was stronger than the first one. As a result, the earthquake loss cost around 5.5–7.5 billion dollars, 90 buildings were destroyed and more than 1000 buildings were damaged in this sequence (Kato *et al.*, 2016).

Kumamoto sequence in the ground motion database (S-7) has two events as mentioned above with $M=6.2$ (E-017) and $M=7.0$ (E-018). The earthquakes were recorded in station KMMH16.

3.2.19 Central Italy (2016) Earthquake Sequence

A series of events occurred in Central Italy between 24 August and 30 October 2016. The first event took place on 24 August 2016 with $M=6.0$, severely damaging tens of villages and causing 300 fatalities. This earthquake was followed by an aftershock on the same day with $M=5.3$. Then on October 26, 2016, two moderate events occurred with $M=5.5$ and $M=5.9$, shaking the area again. Finally, an earthquake with $M=6.6$ happened 4 days later, causing more damage to the built environment (Michele 2016).

This sequence (S-27) is composed of 5 events, starting with the $M=6.0$ event (E-094) and ending up with $M=6.6$ (E-098). The station that recorded these events was selected as AMT.

3.2.20 Ridgecrest, USA (2016) Earthquake Sequence

During the Ridgecrest sequence, the first event had a magnitude of 6.4 and happened in the Los Angeles region. This earthquake had been followed by many events, including a $M=7.1$ earthquake that happened 33 hours later. As a result, the earthquake loss due to damage was 5 billion dollars. Only one person was killed and 25 were injured during the sequence (Shcherbakov, 2021). This decrease in damage and casualties was due to actions taken after the 1994 Northridge earthquake. It is interesting to note that the evolution of the sequence was similar to the Kumamoto 2016 earthquake sequence.

This is sequence S-8 within the database in the study and the events with magnitudes 6.4 and 7.1 are represented as E-019 and E-020.

3.2.21 Khoy, Iran (2020) Earthquake Sequence

In the Khoy Iran earthquake sequence, there are two moderate events both with $M=5.9$. The epicenters of these earthquakes were close to the Türkiye-Iran border. These two events occurred in February 2020 within 10 hours. In this sequence, 10 people were killed and 141 people were injured in total on both sides of the border. This sequence is represented as S-30 with the events as E-104 and E-105. The recording station was selected as 6508.

3.2.22 Kahramanmaraş, Türkiye (2023) Earthquake Sequence

In February 2023, Türkiye faced a series of powerful earthquakes, making it a significant sequential event in global seismic history. Four major earthquakes above magnitude 6 struck the densely populated southeast region within a fortnight. The first two events, close in time and location, had overlapping impact areas, complicating the distinction. The first event on February 6, 2023, at 4:17 a.m. (with $M=7.7$) caused the most devastation. The second event happened 10 minutes later

with $M=6.6$. Then 9 hours later, a third quake hit, toppling already damaged structures but causing fewer casualties due to people being outdoors after the first quake. Lastly, a 6.4 magnitude earthquake struck two weeks later, centered in Defne, Antakya, causing further destruction to already weakened buildings, injuring or killing those inside attempting to retrieve belongings. A simple sketch showing the epicenters of the earthquakes, ruptured fault lines, and strong ground motion stations in the region is demonstrated in Figure 3.6.

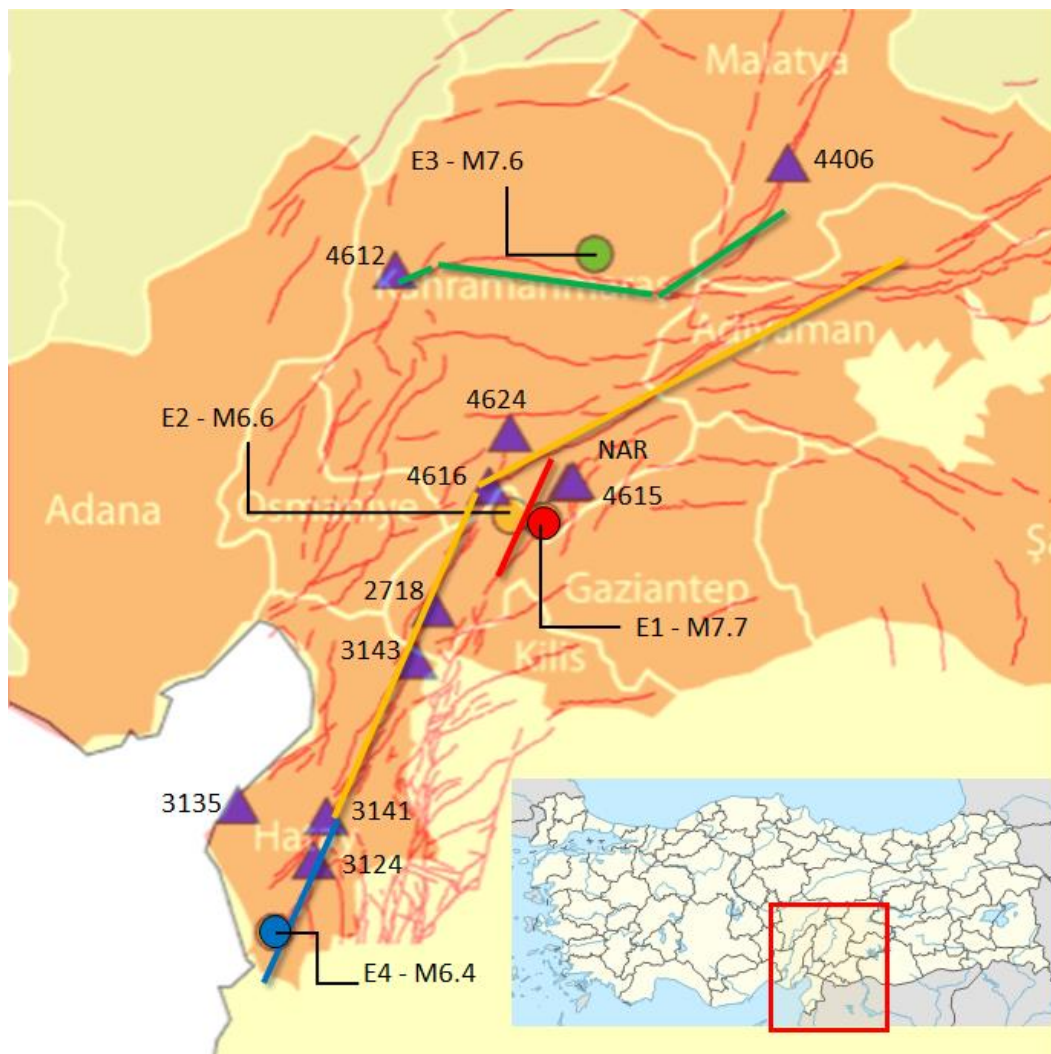


Figure 3.6. Simple sketch indicating the epicenters, ruptured faults and nearby stations



(a)



(b)



(c)

Figure 3.7. Two-story stone masonry building (a) before the earthquake, (b) after the February 6 events, (c) after the M6.4 event and the other M5+ aftershocks

For two weeks, this sequence of events included two major ($7.0 < M < 7.9$) and two strong ($6.0 < M < 6.9$) destructive earthquakes, leading to extensive damage to over 300,000 buildings and a death toll exceeding 50,000 people (Presidency of Strategy and Budget 2023). Figure 3.7 illustrates the structural damage progression of a representative building among thousands affected by these earthquakes. Figure 3.7.a depicts the pre-earthquake condition of a two-story historical masonry building in the city center of Antakya, Hatay. Following the February 6 earthquakes, some upper-story piers suffered an out-of-plane failure, though the roof remained intact (Figure 3.7.b). However, the M6.4 earthquake on February 20, along with

subsequent M5+ aftershocks, ultimately led to the collapse of the entire upper story and the roof (Figure 3.7.c).

Although many strong motion stations recorded this extraordinary earthquake sequence, 5 stations have been selected for this study: 3 from Kahramanmaraş, 1 from Gaziantep, and 1 from Antakya. The sequence numbers in the database are from S-9 to S-13. The locations of the selected stations can be traced from the map in Figure 3.6. Each sequence includes all four earthquakes that happened in February 2023. The events are listed from E-021 up to E-040.

3.3 Response Spectra of the Events in the Database

It is also important to demonstrate the spectral properties of the multiple events within each sequence in order to assess the relative intensities of these shocks in a wide period range. Accordingly, the 5% damped elastic acceleration spectra for all 30 sequences and 105 events are presented in Appendix B. The spectra reveal which periods have amplified demands, which is a piece of crucial information to understand the damage evolution in different building classes.

3.4 Intensity Measures of the Events in the Database

The final part of this chapter is devoted to the presentation of the damaging potentials of the selected events within the sequences. This task can only be achieved by employing ground motion intensity measures (GMIMs). There are a vast number of GMIMs, proposed by different researchers in the literature, but none of them can be a stand-alone candidate to define the potential damage of a ground motion record. Each has its strengths and weaknesses. Therefore, in this study, the commonly used and well-known GMIMs are utilized to express the damaging potential of the 105 selected events. These GMIMs are explained in the following paragraphs. The values of the selected GMIMs for the 105 events are presented in Appendix C.

Peak ground acceleration (PGA) is the maximum ground acceleration experienced at a site during seismic activity. It is one of the simplest GMIMs that can immediately be obtained after an earthquake. Another advantage is that it is directly related to the force exerted on the structure through Newton's Second Law. However, the disadvantage is that the correlation is relatively low, especially for RC structures.

Peak Ground Velocity (PGV) is a unit of measurement for the severity of shaking; more precisely, it is the fastest rate of movement that the earth's surface, or ground, can experience as a result of an earthquake. PGV is also a directly obtained single-valued GMIM and it is more related to the energy input to and dissipated from a structure, as well as the damage conceived by the structure.

Velocity-to-acceleration (V/A) ratio is obtained by dividing PGV by PGA for a specific ground motion record. It has a physical unit (i.e. sec), and it reveals the vibrational properties of the record. For instance, low V/A ratios are indicative of ground motions with pulse recorded at stiff site conditions whereas high V/A ratios stand for ground motions with repetitive cycles recorded at soft soil conditions.

Arias Intensity (AI) is another important GMIM in earthquake engineering, representing the cumulative energy per unit weight carried by the seismic wave. It quantifies the strength of shaking experienced during an earthquake and is particularly useful in assessing the potential for damage to structures and infrastructure (Arias, 1970). It can be calculated as

$$AI = \frac{\pi}{2g} \int_0^{t_d} [\ddot{u}_g(t)]^2 dt \quad (3.2)$$

Root Mean Square of Acceleration (a_{rms}) is a GMIM that represents the average intensity of ground shaking during an earthquake (Boore, 2005). It is calculated as the square root of the mean of the squared ground acceleration values over a specific time interval. The equation is as follows

$$a_{rms} = \sqrt{\frac{1}{t_d} \int_0^{t_d} [\ddot{u}_g(t)]^2 dt} \quad (3.3)$$

Cumulative Absolute Velocity (CAV) is a GMIM that has gained prominence in earthquake engineering due to its potential to better correlate with structural damage than traditional intensity measures like peak ground acceleration (PGA) (Reed & Kassawara, 1990). It is calculated by integrating the absolute value of ground acceleration throughout an earthquake as

$$CAV = \int_0^{t_d} |\ddot{u}_g(t)| dt \quad (3.4)$$

Among many different effective duration definitions in the literature, the oldest and the most commonly used one is the effective duration (t_{eff}) by Trifunac and Brady (1975), which denotes the time interval between 5% and 95% of total Arias Intensity, as measured through the time history of the considered record. In other words, it is the time interval of strong ground motion where the damage is expected to occur in the structure.

Housner Intensity (I_H), which was proposed by Housner in 1952, connects forces imposed on structures and the corresponding earthquake intensity. The formulation in given Equation 3.5 that computes the area under 5% damped pseudo-velocity spectrum within the period range between 0.1 s and 2.5 s.

$$I_H = \int_{0.1}^{2.5} P S_v dT \quad (3.5)$$

Sucuoglu et al. (1999) proposed Input Energy Intensity (I_E), which contains input energy equivalent velocity (v_{eq}) computed from input energy (E_I) as shown in Equations 3.6 and 3.7. The upper bound of the integral is generally taken as 4.0 seconds.

$$I_E = \frac{1}{T} \int_0^T v_{eq} dT \quad (3.6)$$

where

$$v_{eq} = \sqrt{\frac{2E_I}{m}} \quad (3.7)$$

To assess the damaging potential of events based on the given GMIMs, the IM values in the columns of the table in Appendix C can be interpreted in various ways. In this study, each event was ranked individually according to its GMIM values (except for the V/A ratio), and the ranks obtained from each IM were combined to determine an overall ranking for each event.

Examining the top 10 most damaging events based on this combined GMIM ranking, the first five events originate from the devastating earthquake sequence that struck Southeast Türkiye (Kahramanmaraş) in 2023. Specifically, these are events E-025, E-021, E-029, E-037, and E-035. The first four events were recorded at different stations during the February 6 earthquake (M7.7), highlighting the extreme destructiveness of the initial event in the sequence. The fifth event corresponds to the M7.6 earthquake that occurred nine hours later on the same day, marking the third event in the sequence. The remaining events in the top 10 include earthquakes from various global locations as follows:

- Tohoku, Japan (E-005) – M9.0, second in the sequence
- Cape Mendocino, USA (E-012) – M7.0, second in the sequence
- Kumamoto, Japan (E-018) – M7.0, second in the sequence
- Chi-Chi, Taiwan (E-041) – M7.6, first in the sequence
- Marmara, Türkiye (E-009) – M7.1, second in the sequence

A closer analysis of these earthquakes reveals that all had magnitudes greater than 7.0. Among them, six were the first events in their respective sequences; three were the second events, and one was the third. When considering earthquake sequences with the most damaging events, the following order is obtained:

- S-3 (Marmara, Türkiye)

- S-5 (Cape Mendocino, USA)
- S-6 (California, USA)
- S-7 (Kumamoto, Japan)
- S-8 (Ridgecrest, USA)
- S-12 and S-13 (Southeast Türkiye, 2023 earthquakes)

These sequences demonstrate significant destructive potential, distinguishing them from others.



CHAPTER 4

STATE-DEPENDENT SEISMIC RESPONSE OF SINGLE DEGREE OF FREEDOM SYSTEMS

4.1 General

This study examines the impact of sequential earthquakes on the seismic response of conventional construction typologies rather than individual buildings. To achieve this task, simplified and idealized models are used to simulate the inherent seismic behavior of various building classes. Each typology is represented by an inelastic ESDOF system, incorporating a multi-parameter hysteretic model. For dynamic analyses under sequential earthquakes, this study employs the Modified Ibarra-Medina-Krawinkler (MIMK) deterioration model, which features a pinched hysteretic response (Lignos, 2008). The characteristics of this model are explained in the following section. Next, the classification of RC frame buildings is explained in terms of ESDOF system parameters. The variability in each building class is carried out by considering some model parameters as random variables. After the idealized structural simulation set has been constructed, inelastic dynamic analyses are carried out by employing the sequential ground motion database in the previous chapter. Then the response statistics are discussed in terms of the evolution of seismic damage for different RC building classes.

4.2 Idealized SDOF Model Parameters

There exist previous studies regarding the idealization of RC frame structures in Türkiye by considering the inherent structural characteristics and deficiencies. These studies have been conducted after obtaining field data in different parts of the country, such as Düzce, Bursa, Erzincan, Gaziantep, and Kahramanmaraş (Erberik, 2008; Un *et al.*, 2015; Karimzadeh *et al.*, 2018; Arslan Kelam *et al.*, 2022; Askan *et*

al., 2025). These studies reveal that Turkish RC buildings have their properties regardless of the location of the structure. Hence, the classification of RC buildings can be carried out accordingly by considering these properties in terms of construction type, building height, and level of conformity of buildings to seismic design principles.

As mentioned above, the Modified Ibarra-Medina-Krawinkler (MIMK) deterioration model with pinched hysteretic response has been employed to conduct time-history analyses with sequential events. This vertex-oriented, piecewise linear hysteretic model was initially implemented within the OPENSEES analysis platform (McKenna *et al.*, 2000). Its cyclic behavior is depicted in Figure 4.1, incorporating parameters that capture key hysteretic characteristics such as stiffness degradation, strength degradation, and pinching. The multi-parameter nature of the MIMK model makes it a highly suitable choice for accurately simulating the global seismic response of various building typologies.

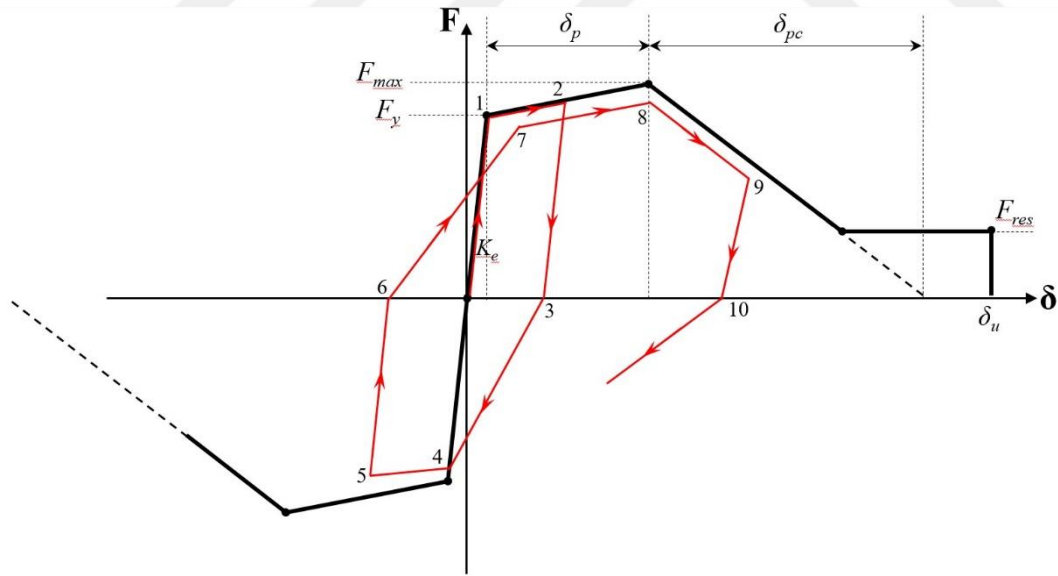


Figure 4.1. The cyclic response of the considered hysteretic response (Ibarra *et al.* 2005)

The main parameters of the selected hysteretic model can be listed as period (T), base shear coefficient (η), ductility capacity (μ), strain hardening ratio (α_s), residual strength coefficient (λ), post-capping ratio (α_c), and cyclic deterioration parameter (γ). Among these parameters, three of them (i.e., T , η , and μ) have the most significant influence on the global seismic behavior of ESDOF systems. Therefore, these parameters are treated as random variables to account for the inherent variability in the seismic response of each building class while the remaining parameters are assigned constant values.

There are six sub-classes representing RC frame buildings (RF classes). In the RF abbreviation, the number “1” or “2” indicates the building height category, with “1” representing low-rise structures (1–3 stories) and “2” denoting mid-rise structures (4–8 stories). Additionally, the letters “A,” “B,” or “C” signify the level of compliance with conventional seismic design principles as follows:

- "A" represents buildings that largely or fully adhere to modern earthquake codes, with only minor deficiencies, if any.
- "B" denotes buildings that partially comply with seismic codes but have structural deficiencies that may affect their performance during a strong earthquake.
- "C" refers to buildings constructed without consideration of seismic design principles; often older structures designed primarily for gravity loads.

The mean values of all hysteretic model parameters are provided in Table 4.1. For the random variables, the coefficient of variation (c.o.v.) is also included in the same table. Considering constant parameters, the strain hardening ratio (α_s) is taken as 4%. Furthermore, the post-capping ratio (α_c) is taken as -%20, -%25, and -%30 for A, B, and C-type buildings, respectively. The residual strength coefficient (λ) is taken as 0.2. The cyclic deterioration parameter (γ) is selected according to the studies of Ibarra and Krawinkler (2005), Ibarra et al. (2005), Lignos and Krawinkler (2011) and Lignos and Krawinkler (2012). This parameter typically ranges between 25 and 200 for structural elements and systems with inadequate design and performance

while it reaches values of 400 and above for those designed according to regulations and expected to perform well. Theoretically, an infinite value of this parameter would indicate no damage to the system. Considering these factors and the RC frame building subclasses in this study, the parameter is assumed to be 800 for RF1A and RF2A subclasses, 400 for RF1B and RF2B subclasses, and 200 for RF1C and RF2C subclasses.

Table 4.1 Building Parameters (Askan *et al.*, 2015)

Building Class	T(s)		η		μ		α_s (%)	α_c (%)	λ	γ
	mean	c.o.v.	mean	c.o.v.	mean	c.o.v.				
RF1A	0.40	0.47	0.40	0.20	9.00	0.35	4	-20	0.2	800
RF1B	0.40	0.47	0.30	0.37	7.30	0.28	4	-25	0.2	400
RF1C	0.40	0.47	0.23	0.27	4.90	0.30	4	-30	0.2	200
RF2A	0.70	0.38	0.34	0.31	7.10	0.32	4	-20	0.2	800
RF2B	0.70	0.38	0.26	0.34	6.10	0.29	4	-25	0.2	400
RF2C	0.70	0.38	0.17	0.37	5.10	0.27	4	-30	0.2	200
MU1A	0.06	0.30	0.86	0.20	3.53	0.20	0	-20	0.2	600

In order to quantify the variability in response statistics, 20 building samples were generated using Latin Hypercube Sampling (LHS) for each class by varying the random variables (i.e., period, base shear coefficient, and ductility capacity) with respect to the mean and standard deviation values. LHS is a statistical sampling method that is used to generate a near-random sample of parameter values from a multidimensional distribution. It is widely employed in earthquake engineering due to its efficiency in exploring the variability of input parameters in seismic hazard and risk assessments (McKay *et al.*, 1979). In total, 126 samples were obtained with the mean samples. A detailed table of all 120 building samples is presented in Appendix D.

4.3 Execution of Inelastic Dynamic Analyses

The inelastic dynamic analysis for the specified SDOF simulations is carried out within the OPENSEES software (McKenna *et al.*, 2000). OPENSEES, or the Open System for Earthquake Engineering Simulation, is a versatile software framework renowned for its capability to simulate the behavior of structures subjected to seismic events.

In this analysis platform, a nonlinear cantilever column with a uniaxial inelastic section is simulated by a material model known as the Modified Ibarra-Medina-Krawinkler (IMK) deterioration model with peak-oriented hysteretic response (ModIMKPeakOriented Material). The reason for using a cantilever column instead of the zero-length member is that the material model incorporates only beam or column elements. It is not designed for zero-length members. The Modified Ibarra-Medina-Krawinkler (IMK) deterioration model with peak-oriented hysteretic response (ModIMKPeakOriented Material) is an advanced material model implemented in OPENSEES. It is a refined version of the original IMK model, specifically designed to capture the peak-oriented hysteretic behavior observed in reinforced concrete (RC) structures under cyclic loading. This model considers various deterioration mechanisms, including strength and stiffness degradation, pinching, and damage accumulation. It is particularly useful for simulating the seismic response of RC elements, such as beams and columns, where peak-oriented behavior is prevalent.

The ModIMKPeakOriented material model in OPENSEES takes several parameters that appoint its behavior. These parameters include:

- ePd, nPd, rDispP, rForceP: These control the post-capping strength deterioration, considering displacement and force-based criteria.
- ePf, nPf, rForceF: These assign the unloading stiffness deterioration based on force criteria.

- e_{Pi} , n_{Pi} , r_{ForceI} : These control the reloading stiffness deterioration based on force criteria.
- Λ_S , Λ_C , Λ_A , Λ_K : These are damage parameters for strength, pinching, cyclic loading, and unloading stiffness, respectively.
- c_{Factor} , β_{Fact} , Res_{fac} , c_S , c_C , c_K : These are parameters used to calculate the damage index and its components.

The ModIMKPeakOriented material model was selected for this study due to its ability to accurately represent the hysteretic behavior of reinforced concrete (RC) frame structures subjected to sequential earthquake loading. Unlike the conventional Ibarra-Krawinkler (IMK) model, which does not explicitly account for peak-oriented reloading effects, ModIMKPeakOriented directs the reloading path toward the maximum deformation previously experienced. This feature enhances the model's capability to capture stiffness and strength degradation, which are critical for assessing structural performance under cascading earthquakes. Additionally, the model provides a more realistic representation of pinching effects, commonly observed in RC structures due to crack opening and closing, thereby improving the accuracy of cyclic energy dissipation modeling. Given that this research focuses on the fragility assessment of RC frame buildings under multiple seismic events, it is essential to adopt a hysteretic model that effectively simulates cumulative damage and progressive deterioration. The ModIMKPeakOriented model offers significant advantages over the conventional IMK model in this regard, making it a more suitable choice for the sequential ground motion analyses conducted in this study.

In order to facilitate the input process and the analyses and to get fast results from huge amount of earthquake sets, an open-source program II-DAP is utilized for SDOF analyses. The Interactive Interface for Dynamic Analysis Procedures (II-DAP) is a MATLAB-based software tool developed by Elkady and Lignos (2019) to facilitate the dynamic analysis of single-degree-of-freedom (SDOF) systems. It provides a user-friendly graphical interface for performing various types of dynamic analyses, making it accessible to both researchers and students.

The software includes a diverse library of SDOF system models, ranging from simple linear and bilinear models to more sophisticated ones that account for deteriorating behavior, such as pinched, peak-oriented, and self-centering responses including the ModIMKPeakOriented material model. Additionally, II-DAP incorporates a seismic hazard module for quantifying the collapse risk of structures.

In Figure 4.2, the main console of II-DAP can be seen. It consists of four input parts. First one is the SDOF part where mass, period, and stiffness parameters can be defined in Figure 4.3. Here, a 5 % damping coefficient is given to the system to ensure that Rayleigh damping parameters are utilized. Furthermore, one can easily define p-delta effects in this section. Secondly, the user can choose the force deformation hysteretic model from the System Model part. There are 7 models, and after each selection, the parameter input interface changes accordingly (Figure 4.4). As a third input, a single ground motion or set of ground motions can be defined. Also, one can create harmonic waves and include free vibration. The input page for ground motions is shown in Figure 4.5. Finally, the user should choose the analysis type as in Figure 4.6. After selecting each analysis type, a pop-up window opens and shows the related settings module. After finishing the input face, the analysis can be done, and the results can be viewed and saved.

Since the II-DAP program creates automatic .mat files that include several parameters, and this hand input process takes too much time, a MATLAB code is created to alter .mat files with our parameters automatically. According to the building parameters in Appendix D, 126 input files were created and 105 ground motions were defined inside each of them. The results were saved in .txt format. Each result file includes 8 columns of data. The first column contains the ground motion time value while the remaining columns contain the response quantities for the ground acceleration, relative displacement/velocity/acceleration, pseudo velocity/acceleration, absolute acceleration, and force.

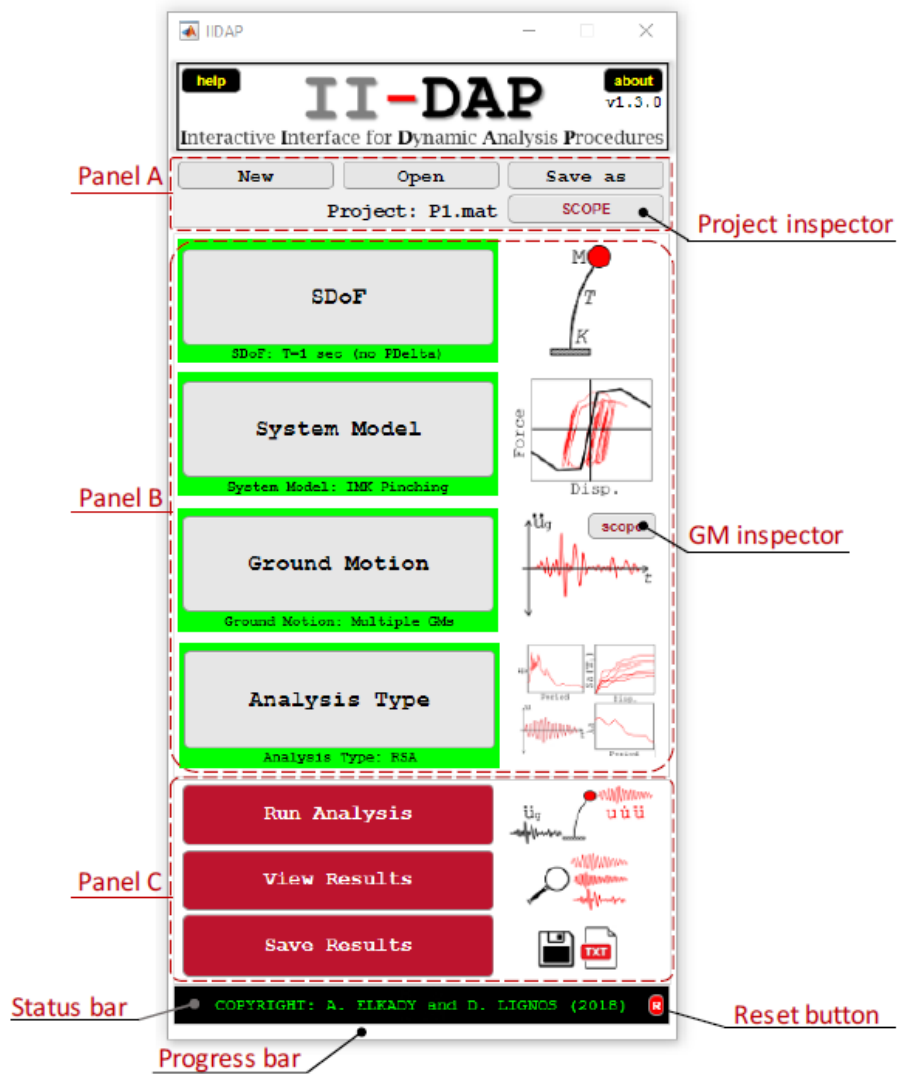


Figure 4.2. II-DAP main console

SDoF Parameters

Period and Mass

M: Mass [kN.sec²/m]
2

K: Stiffness [kN/m]
123.37

T: Period [sec]
0.8

ζ: Damping coefficient [%]
2

Optional: P-Delta

☒ Include System P-Delta

Specify Weight and Height

P: 19.62 kN **H:** 3.1806 m

Specify Stability Coeff.

θ: 0.0499999

$\theta = \frac{P \cdot \delta}{H \cdot F}$

Submit

Figure 4.3. SDOF parameters input interface

System Model

Choose System Model

☒ **Linear**
Non-deteriorating linear elastic system

☐ **BiLinear**
Non-deteriorating bilinear system with symmetric response

☐ **IMK Bilinear**
Deteriorating multi-linear model based on the modified I-M-K model with smooth loading/hardening transition

☐ **IMK Pinching**
Deteriorating multi-linear model based on the modified I-M-K model with pinching response

☐ **IMK Peak-Oriented**
Deteriorating multi-linear model based on the modified I-M-K model with peak-oriented response

☐ **Flag-Shaped**
Non-deteriorating multi-linear model with flag-shaped response

Next

IMK Pinching Model

Backbone Parameters

K_0 78.9568 kN/m

δ_y 0.0372735 m

	Positive	Negative
δ_p/δ_y	3	3
δ_{pc}/δ_y	10	10
δ_u/δ_y	20	20
F_y/W	0.15	0.15
F_y/F_{max}	1.05	1.05
F_{res}/F_y	0.2	0.2

☒ Symmetric Backbone

K_F 0.2 K_δ 0.4

Deterioration Parameters

	λ	ϕ
Basic Strength	10.0	1.0
PostCap Strength	10.0	1.0
Lateral Stiffness	10.0	1.0
Acc. Stiffness	10.0	1.0
Asymmetric Rate	D+ 1.0	D- 1.0

Submit

Backbone Parameters Definition

Force vs. Displacement plot showing F_{max} , F_y , K_e , δ_y , δ_p , δ_{pc} , δ_u , and F_{res} .

Backbone Plot

Force [kN] vs. Displacement, δ [m]. Legend: Backbone (w/o P- δ), Backbone (w P- δ), test response.

Figure 4.4. System Model parameters input interface

Ground Motion Mo...

Panel A: Single Record

Select file

Scale factor

Time step [sec]

Panel B: Multiple Records

Select folder

Scale factor

Panel C: Harmonic Wave

Wave shape

Amplitude [g]

Period [sec]

Duration [sec]

Panel D: Optional: Free Vibration

☒ Additional Time [sec]

Figure 4.5. Ground motion input interface

Analysis Type Options

Choose Analysis Type

☒ **Response History Analysis (RHA)**
Run dynamic response-history analysis at a single GM scale factor

☐ **Incremental Dynamic Analysis (IDA)**
Run response-history analysis for a set of ground motions through collapse. Get the mean annual frequency of collapse

☐ **Response Spectrum Analysis (RSA)**
Get the response-spectrum analysis for a range of periods at a single scale factor

☐ **Collapse Hazard Analysis (CHA)**
Get the collapse acceleration spectrum and the mean annual frequency of collapse

Figure 4.6. Analysis Type module

4.4 Verification of II-DAP

The program uses the OPENSEES system even if it seems like different software, it only helps the input process with a user interface controlled with MATLAB runtime executable code. The program is verified with the examples in the book by Chopra (2012). In Section 6.4 of the book, maximum displacements were obtained for the El Centro 1940 earthquake, acting on generic SDOF systems. In Table 4.2, it can be seen that the software catches the maximum response with less than 1% error. Therefore, it can be stated that the software yields accurate results for dynamic analysis of SDOF systems.

Table 4.2 Earthquake Response Comparison of II-DAP and Chopra (2012)

Period	Damping Ratio (%)	Maximum Response (in)		Absolute True Error (%)
		IIDAP	Chopra (2012)	
0.50	0.02	2.69	2.67	0.749
1.00	0.02	5.96	5.97	0.168
2.00	0.02	7.48	7.47	0.134
2.00	0.00	9.92	9.91	0.101
2.00	0.02	7.48	7.47	0.134
2.00	0.05	5.38	5.37	0.186

4.5 Evaluation of Response Statistics After the Analyses

Using the obtained results, the following response parameters have been generated with a MATLAB code from each analysis: damage state at the end of the sequence, maximum displacement, input energy, and hysteretic energy. In total, 126 buildings are analyzed with 105 earthquake sequences, which results in 13,230 analyses. These analyses took around 25 hours to compile. To demonstrate the general analysis

results, Figure 4.7 and Figure 4.8 show the sequence of the earthquakes (S-12) and how the sequence affects the structure (RF1C) step by step.

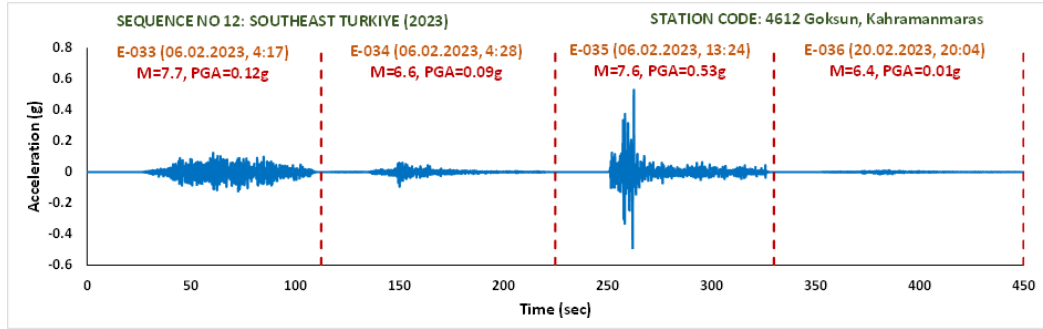


Figure 4.7. An example of sequential earthquakes to be used in dynamic analyses:
Sequence S-12 with four events (E-033, E-034, E-035, E-036)

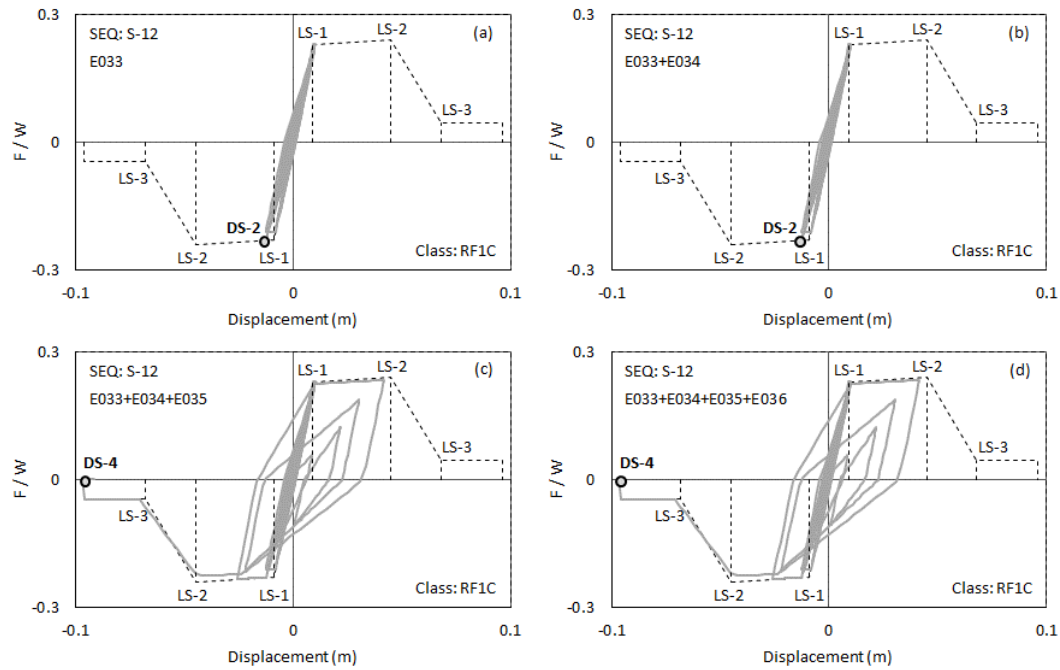


Figure 4.8. Illustration of damage evolution process for building class RF1C when
subjected to the earthquake sequence S-12

An example of processed results, the following Figure 4.9 shows some indications. Arrows imply the change of damage state. As presented in Figure 4.9, a discrete damage state definition is employed in this study, in which the limit states are selected as the specific performance thresholds (at yield, maximum, and residual capacity) on the skeleton curve of the hysteresis model for the considered building class with respect to force (F) and displacement (δ). Since there are 3 limit states (LS-1, LS-2, and LS-3), there eventually exist 4 damage states (DS-1, DS-2, DS-3, and DS-4). These damage states can be defined as “none/slight”, “moderate”, “severe” and “collapse”, respectively. At the end of each dynamic analysis, the damage state of the SDOF simulation is obtained by comparing the maximum calculated displacement with the predefined thresholds (FEMA, 2000).

	res_41_DS	res_41_MaxD	res_41_IE	res_41_HE	res_42_DS	res_42_MaxD	res_42_IE	res_42_HE	res_43_DS	res_43_MaxD	res_43_IE	res_43_HE	res_44_DS	res_44_MaxD	res_44_IE	res_44_HE	res_45_DS	res_45_MaxD	res_45_IE	res_45_HE
66	2	23.188	0.319	0.098	2	11.452	0.247	0.084	2	20.402	0.243	0.029	2	18.909	0.269	0.104	2	32.933	0.369	0.052
67	2	23.188	0.325	0.100	2	11.452	0.253	0.087	2	20.402	0.249	0.030	2	18.909	0.274	0.106	2	32.933	0.374	0.052
68	4	91.187	0.361	0.049	4	45.115	0.182	0.026	4	162.720	0.844	0.305	4	44.308	0.184	0.019	2	120.493	1.419	0.560
69	4	90.244	0.404	0.050	4	44.948	0.228	0.026	4	162.921	0.902	0.310	4	44.313	0.230	0.019	3	142.799	2.936	1.120
70	4	90.242	0.425	0.050	4	44.876	0.249	0.026	4	163.357	0.927	0.310	4	44.315	0.251	0.019	3	142.285	3.410	1.288
71	2	17.070	0.227	0.062	2	16.888	0.239	0.089	1	16.995	0.289	0.000	2	15.262	0.226	0.087	1	19.900	0.178	0.000
72	2	30.866	0.517	0.171	4	42.538	0.409	0.148	2	27.622	0.563	0.065	4	41.361	0.411	0.147	2	34.541	0.516	0.070
73	4	97.483	0.410	0.087	4	49.329	0.595	0.073	4	169.133	1.019	0.266	4	41.786	0.272	0.018	4	281.455	1.701	0.452
74	4	101.031	0.504	0.087	4	52.388	0.770	0.073	4	172.647	1.193	0.266	4	41.831	0.447	0.018	4	281.965	1.976	0.454
75	1	7.410	0.029	0.000	1	3.192	0.010	0.000	1	5.865	0.019	0.000	2	4.132	0.011	0.001	1	11.076	0.012	0.000
76	2	30.582	0.271	0.095	3	19.558	0.311	0.124	2	25.874	0.387	0.119	3	22.013	0.255	0.101	2	33.751	0.240	0.034
77	1	3.180	0.011	0.000	1	2.291	0.015	0.000	1	3.612	0.014	0.000	1	2.617	0.015	0.000	1	5.329	0.009	0.000
78	1	8.457	0.101	0.000	2	6.237	0.151	0.033	1	9.499	0.155	0.000	2	5.904	0.135	0.038	1	8.922	0.051	0.000
79	1	8.457	0.103	0.000	2	6.237	0.152	0.033	1	9.499	0.156	0.000	2	5.904	0.137	0.038	1	8.922	0.052	0.000
80	1	4.974	0.007	0.000	1	3.659	0.014	0.000	1	4.802	0.013	0.000	1	3.699	0.011	0.000	1	6.214	0.006	0.000
81	4	154.292	1.119	0.362	4	42.628	0.463	0.070	4	216.459	3.395	1.186	4	41.427	0.324	0.036	2	117.212	4.664	1.694
82	4	354.420	1.175	0.361	4	226.039	0.507	0.076	4	417.038	3.464	1.193	4	227.540	0.375	0.033	2	119.023	4.687	1.700
83	4	421.897	1.189	0.355	4	293.377	0.513	0.076	4	482.972	3.448	1.184	4	295.003	0.383	0.034	2	118.653	4.702	1.703
84	4	94.381	0.405	-0.022	4	46.351	0.278	0.050	2	25.244	0.357	0.092	4	44.463	0.284	0.048	2	44.707	0.613	0.209
85	4	94.381	0.423	-0.022	4	46.351	0.296	0.050	2	25.244	0.410	0.109	4	44.463	0.312	0.048	2	44.707	0.689	0.338
86	4	94.381	0.425	-0.022	4	46.351	0.298	0.050	2	25.244	0.420	0.111	4	44.463	0.314	0.048	2	44.707	0.716	0.346
87	4	94.381	0.425	-0.022	4	46.351	0.298	0.050	2	25.244	0.421	0.111	4	44.463	0.314	0.048	2	44.707	0.718	0.347
88	4	92.327	0.712	0.226	4	42.412	0.380	0.093	2	44.007	1.003	0.347	4	41.189	0.316	0.088	2	62.409	0.925	0.333
89	4	92.454	0.725	0.226	4	42.412	0.393	0.093	2	44.007	1.110	0.390	4	41.189	0.329	0.088	2	62.409	1.067	0.388
90	4	92.454	0.727	0.226	4	42.412	0.396	0.093	2	44.007	1.129	0.398	4	41.189	0.332	0.088	2	62.409	1.085	0.394
91	4	92.454	0.727	0.226	4	42.412	0.396	0.093	2	44.007	1.130	0.398	4	41.189	0.332	0.088	2	62.409	1.085	0.394
92	4	93.958	0.386	0.054	4	43.132	0.210	0.026	2	30.901	0.552	0.184	4	44.461	0.187	0.018	2	51.459	0.725	0.288
93	4	93.958	0.389	0.054	4	43.132	0.213	0.026	2	30.901	0.578	0.192	4	44.461	0.190	0.018	2	51.459	0.737	0.293
94	4	93.958	0.392	0.054	4	43.132	0.216	0.026	2	30.901	0.592	0.196	4	44.461	0.193	0.018	2	51.459	0.748	0.297
95	4	93.958	0.392	0.054	4	43.132	0.216	0.026	2	30.901	0.592	0.196	4	44.461	0.193	0.018	2	51.459	0.748	0.297
96	2	19.956	0.348	0.086	2	10.075	0.352	0.119	1	16.608	0.255	0.000	2	15.219	0.398	0.156	1	26.625	0.535	0.000
97	2	19.956	0.434	0.113	2	10.075	0.391	0.133	1	16.608	0.278	0.000	2	15.219	0.461	0.180	1	26.625	0.643	0.000
98	4	92.863	0.982	0.278	4	46.093	0.560	0.167	2	46.803	1.215	0.365	4	46.210	0.671	0.213	2	82.251	2.052	0.570
99	4	92.863	0.982	0.278	4	46.093	0.560	0.167	2	46.803	1.217	0.366	4	46.210	0.671	0.213	2	82.251	2.055	0.571
100	4	91.889	0.614	0.159	4	46.224	0.504	0.109	3	74.794	1.364	0.524	4	43.131	0.335	0.055	2	95.109	1.359	0.506
101	4	91.889	0.615	0.159	4	46.224	0.505	0.109	3	74.794	1.374	0.529	4	43.131	0.337	0.055	2	95.109	1.372	0.511
102	4	91.889	0.616	0.159	4	46.224	0.506	0.109	3	74.794	1.378	0.530	4	43.131	0.337	0.055	2	95.109	1.377	0.513
103	4	91.889	0.654	0.159	4	46.224	0.544	0.109	3	74.794	1.504	0.589	4	43.131	0.375	0.055	2	95.109	1.471	0.561
104	4	91.459	0.450	0.150	4	47.009	0.184	0.043	2	38.471	0.391	0.150	4	44.789	0.106	0.012	2	63.089	0.454	0.176
105	4	93.214	0.520	0.150	4	48.162	0.274	0.043	3	84.406	1.643	0.672	4	45.800	0.195	0.012	2	108.630	2.049	0.818

Figure 4.9. Partial presentation of the analysis results

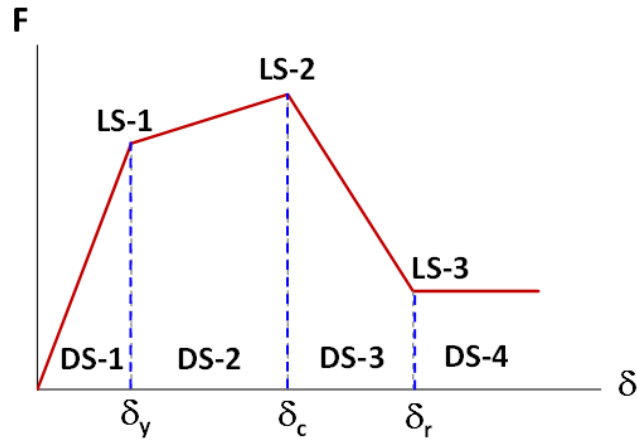


Figure 4.10. The definitions of limit and damage states on the skeleton curve of the hysteresis model (FEMA, 2000)

The response statistics obtained by considering the variability in structural parameters within each building class are given in Table 4.3 for all building classes. The tables present the number of times the damage state either remains as it is after the first event (i.e., columns with headers DS-1, DS-2, DS-3, and DS-4) or shifts to a higher level due to sequential events (i.e., columns with headers 1→2, 1→3, 1→4, 2→3, 2→4, and 3→4). The last column of the table presents the number of all damage state changes within the selected sequences for the given building class.

Table 4.3 Damage state changes in the building classes

RC Frame	DS-1	1→2	1→3	1→4	DS-2	2→3	2→4	DS-3	3→4	DS-4	DS Change
RF1A	198	61	3	10	234	7	6	14	9	57	96
RF2A	235	55	2	2	256	4	4	12	5	25	72
RF1B	134	59	1	14	238	12	14	11	7	109	107
RF2B	183	55	1	6	243	14	8	15	9	64	93
RF1C	112	41	3	22	190	7	22	13	10	177	105
RF2C	125	47	1	12	217	10	20	24	16	124	106

Table 4.3 includes damage state statistics for all simulations subjected to 30 sequences. The inclusion of variability in structural parameters seems to yield more scattered data among all cases of change/no change in damage states within sequences. The observations can be listed as follows:

- For the RF1A and RF2A classes, the structural models generally remain in DS-1 or DS-2 throughout the entire sequence, with no significant shifts in the damage state. The percentage of time spent in DS-1 or DS-2 is 72% for RF1A and nearly 82% for RF2A. This is expected, as these classes represent reinforced concrete (RC) building structures with proper seismic design, which typically ensures sufficient strength and ductility. Most damage state transitions occur from DS-1 to DS-2, with other shifts being relatively rare, as indicated in Table 4.3. This is due to the high ductility capacity of these structural simulations, which prevents significant damage progression.
- For the RF1B and RF2B classes, the trends are similar to those of the RF1A and RF2A classes, but with a slight shift towards higher damage states. This shift is due to these classes representing RC buildings with some structural deficiencies. There are more incidents of buildings experiencing DS-4 (collapse) or transitioning from DS-2 to higher damage states during multiple events. However, it is still observed that the general trend of remaining in either DS-1 or DS-2 throughout the entire sequence, without significant shifts in the damage state, persists to some extent for these RC classes. A notable finding is that the number of collapsed structures (remaining in DS-4) has doubled in these RF classes compared to the RF1A and RF2A classes.
- Failure occurs during the first event in 30% of the cases for RF1C and 21% of the cases for RF2C classes, which is expected given the vulnerability of these RC structures. Additionally, there is a notably higher number of cases where the building model, initially in one of the first three damage states (DS-1, DS-2, or DS-3), directly shifts to collapse (DS-4) after experiencing a subsequent event. Therefore, the general trend for these two RC classes indicates poor seismic performance after the first event, leaving no

opportunity for further progression to higher damage states as the building is already in a severe damage state.

- For RF classes, most of the cases for damage state change seem to occur from DS-1 to DS-2. For inferior RF classes (i.e., RF1C and RF2C) the number of shifts from DS-1 to DS-4 or from DS-2 to DS-4 are significantly high. This shows that the adverse effects of sequential events on vulnerable RC classes are critically significant.
- The comparison of low-rise (1-story) and mid-rise (2-story) structures revealed that mid-rise buildings experience more damage and greater changes in damage states. This is attributed to higher damage accumulation, increased seismic forces due to greater mass, and a higher likelihood of construction imperfections as the number of stories increases.

Table 4.4 Damage state changes for each earthquake sequence

No	Sequence	1→2	1→3	1→4	2→3	2→4	3→4	count	% in 120
S-2	Great-Major	58	3	54	0	1	1	117	97.50
S-8	Major-Strong	76	5	1	3	5	0	90	75.00
S-12	Major-Strong	35	1	9	7	36	1	89	74.17
S-22	Strong-Moderate	64	1	1	0	2	0	68	56.67
S-3	Major-Major	0	0	0	12	12	12	36	30.00
S-1	Great-Major	34	0	1	0	0	0	35	29.17
S-26	Strong-Moderate	2	0	0	5	13	10	30	25.00
S-6	Major-Strong	21	0	0	3	4	1	29	24.17
S-27	Strong-Moderate	8	0	0	5	0	11	24	20.00
S-7	Major-Strong	0	0	0	10	0	11	21	17.50
S-5	Major-Strong	0	0	0	8	1	5	14	11.67
S-29	Moderate-Moderate	8	0	0	0	0	0	8	6.67
S-22	Strong-Moderate	7	0	0	0	0	0	7	5.83
S-5	Major-Strong	0	0	0	0	0	4	4	3.33
S-23	Strong-Moderate	2	0	0	0	0	0	2	1.67
S-25	Strong-Moderate	2	0	0	0	0	0	2	1.67
S-25	Strong-Moderate	1	1	0	0	0	0	2	1.67

The next question is which specific sequences or corresponding events are responsible for these shifts in damage states. When considering all the sequences (see Appendix A), it is observed that 17 out of 30 sequences caused at least one damage state shift in any of the building class simulations with varying structural parameters. These 17 sequences are listed in Table 4.4, ranked in descending order based on the percentage of cases with damage state changes across the 6 building classes. The most critical sequences are S-2 (Tohoku, Japan), S-8 (Ridgecrest, USA), S-12 (Station 4612 from the 2023 earthquake in Southeast Türkiye), and S-22 (Chalfant Valley, USA). A common feature of these four sequences is that the first event has a smaller magnitude (MBRI values under 0.3) compared to the second event, which is responsible for the damage state shift. From a structural perspective, the Tohoku Japan Great-Major sequence (S-2) stands out as one of the most critical, where the second event, with $M=9.0$, strikes just two days after a devastating $M=7.3$ event. In 78% of all building class simulations, this sequence caused a drastic shift from DS-1 to DS-4. It is important to note that S-2 is a particularly unique sequence, being a mega-thrust earthquake with exceptionally high magnitudes. While such earthquakes often occur in deep ocean basins, far from populated areas, the other three sequences involved ruptures crossing land and were much closer to populated cities.

As expected, most of the Great-Major, Major-Major, and Major-Strong sequences lead to damage state changes across all the considered building classes. Additionally, 5 Strong-Moderate, 2 Major-Strong-Moderate, and 1 Moderate-Moderate sequences also contribute to damage state shifts, particularly in the more vulnerable building classes, such as those with RC construction (i.e., RF1C, RF2C) as shown in the statistical results in Table 4.3. This highlights that even moderate-magnitude events can trigger significant structural damage and collapse in these vulnerable and deficient RC buildings.

It is also worth investigating the 13 sequences in which no damage state change has occurred. The first trend belongs to the sequences in which the first event is the major one and the following events are strong or moderate earthquakes with relatively

smaller magnitudes ($MBRI < 0.10$). This trend also includes exceptional cases from the 2023 Kahramanmaraş, Türkiye earthquake in which the first damaging event (M7.7) and the third event (M7.6) are major ones while the second (M6.6) and the fourth (M6.4) are strong earthquakes. There are two reasons for these outlier statistics. The first one is that the first event already causes collapse for vulnerable building classes, so there is no damage state to change after all. Second, for building classes with better performance, when other intensity measures are considered, the first event of the Kahramanmaraş sequence seems to surpass the third one in terms of potential damage.

Due to the complex relationship between the variable structural characteristics of the building classes and the unique nature of each strong ground motion record within the related sequences, it is a challenging task to arrive at generalized conclusions for the state-dependent response under sequential earthquakes. However, this parametric study serves as a guideline to identify the critical issues from the obtained response statistics.

When the sequences are examined, it is observed that there are two major patterns regardless of the number of events within the sequence: 1) the main event (i.e., with the highest magnitude) is the first event, and 2) the main event is not the first event. In both cases, the secondary criterion is the closeness of the other events to the main event in terms of magnitude. This can be checked by the MBRI value practically. But mostly, MBRI by itself falls short of explaining the damaging potential of the sequential events since other factors like the epicentral distance and absolute magnitude of the events in the sequence play important roles as well as the relative values of the IMs. Among the 30 sequences considered in this study, 23 of them are in the first group (i.e., the first event is the main event) however only 10 out of 23 sequences (i.e., 43%) have the potential to cause damage state changes in the simulated building population. When these 10 sequences are examined thoroughly, it is revealed that the magnitudes of the main (first) and the proceeding events exceed $M=6.5$, the MBRI values of the proceeding events are more than 0.2 and epicentral distances for the proceeding events are closer than main events and absolutely within

the 70 km range. This also means that the other sequences with no damage state change do not satisfy at least one of the above criteria (examples are S-15, S-18, and S-24 from Appendix C in which the events other than the main event generally have low damaging potential, not capable of causing severe structural response even for the weak structural classes) or the first event is so destructive that the structural models directly fail. Hence, there is no chance of damage state change for the proceeding events regardless of their damaging potential (Examples are the sequences from the 2023 Southeast Türkiye earthquake, S-9, S-10, S-11, and S-13 in Appendix C).

For the second group of sequences (i.e., in which the main event is not the first event), the five have the main event as the second one whereas there is one case for each where the main event is either the third or the fifth in the sequence. Overall, it is remarkable to observe that all of the seven sequences considered in this study exhibit a potential for damage state change in the simulated structural models. When these sequences are examined in detail, it is revealed that either the preceding event and the main event have large magnitudes (like E-004 and E-005 in S-2, see Appendix C) or $MBRI > 0.2$ for the preceding event (E-019 in S-8, E-069 in S-22 and E-073 in S-23, see Appendix C). Another important criterion is that in all seven cases, the epicentral distances of both preceding and main events are in close vicinity (i.e., shorter than 30 km).

It should also be pointed out that among the first 10 most damaging events according to the combined IM value ranking results, as stated previously, only four of them appear in Table 4.4 as the events with the potential of damage state change. This poor correlation can be attributed to two reasons: 1) the other six earthquake records are the first events in the sequence with very high seismic intensity and causing the structural models to reach DS-4 immediately, 2) the proceeding events are so small that they cannot change the damage state of the structural model. As a result, the relative IM values of the events are more important than the absolute values to identify the potential for damage state change during the sequence.

If the statistics about damage state changes for different building classes are considered, it can be observed from Table 4.3 that the percent of minimum and maximum number of damage state changes per building class is 4% and 19% respectively, with an average percent of 14% for all building classes. Most of the damage state shifts are for the case of change from DS-1 to DS-2.

Finally, Figure 4.11 presents the percent values of damage states and the related shifts for two different cases: 1) damage state at the end of the first event, and 2) damage state at the end of the sequence. In this way, it becomes possible to monitor the effects of all considered sequences on building classes with variable structural characteristics. The plots in Figure 4.11 are for RF classes. The arrows in the plots show the increase or the decrease in each limit state by the application of the sequence. No change in the percent of a damage state is indicated by an equal sign.

When the damage state statistics of RF classes in Figure 4.11 are examined, firstly, it is observed that DS-1 and DS-2 generally dominate. For all RF-classes, there seems to be a shift from lower limit states (i.e. DS-1 and DS-2) to higher ones (i.e. DS-3 to DS-4), especially in the case of inferior C-classes. The effect of sequential earthquakes is reflected by the percent shifts in all damage states. When the structural models are exposed to multiple events rather than a single one, DS-1 shifts to higher damage states in large percent (20% to 38%). The probability of experiencing DS-4 due to multiple earthquakes is noticeably increased, even reaching values like 45% when compared to statistics for single events.

Overall, these statistics state that due to sequential earthquakes considerable number of buildings that have not been damaged or slightly damaged in the first earthquake (i.e. at state DS-1) experience damage, and for deficient RC classes (i.e. C-classes), a significant number of the buildings that have shifted to higher damage states get collapsed. It is noteworthy to observe that a small percentage of the well-designed (i.e. A-class) buildings experience collapse under a single earthquake and this percentage is increased by approximately 45% during multiple events. These results show that some design implications should also be developed for structures under

sequential earthquakes in future seismic design codes. This may include revising capacity design principles to account for cumulative damage, introducing reduction factors for strength and stiffness degradation, and updating performance-based design frameworks to reflect the increased vulnerability of structures subjected to multiple shocks. Additionally, new detailing requirements could enhance the resilience of critical structural elements against successive ground motions.

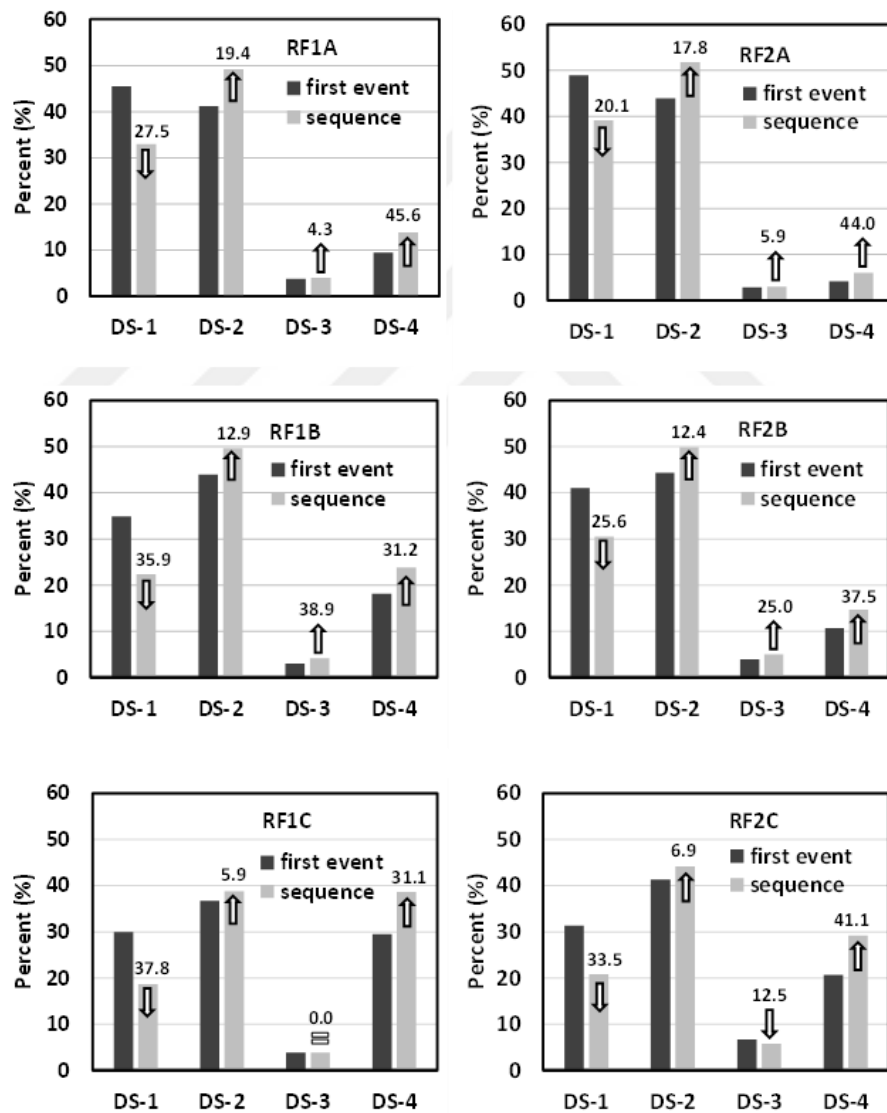


Figure 4.11. Statistics regarding the changes in damage states for RF-classes of simulations

CHAPTER 5

STATE-DEPENDENT SEISMIC RESPONSE OF MULTI DEGREE OF FREEDOM STRUCTURES

5.1 Introduction

The previous chapter that focused on idealized SDOF models revealed that the damage evolution is significant for mid-rise RC frame structures. The field observations after the recent earthquakes also support this statement. Hence, this chapter is devoted to the seismic analysis of multi degree of freedom systems when subjected to sequential earthquakes in order to have a more detailed inspection of damage evolution in the case of repeated earthquakes. 20 planar (2D) RC frame models are employed for the analyses, assuming that the considered structures are symmetrical in plan. The building classification for mid-rise RC frames is also taken into consideration in this chapter.

The workflow of this chapter is presented in Figure 5.1. In the first stage, structural modeling is carried out. Sections 5.2 and 5.3 explain the material and structural properties of the developed frames. The next section presents how the nonlinear static (pushover) analyses have been carried out and the limit state definitions attained from the pushover analysis. Then, time history analysis is conducted for mainshock only and sequential cases separately for each building model and the response statistics are obtained in terms of damage states that have been obtained during the pushover analyses. Damage state statistics are presented both for individual buildings and for mid-rise RC building classes in the form of fragility curves that have been obtained by using binary logistic regression. According to the obtained results, final remarks on sequential earthquakes are presented at the end of the chapter.

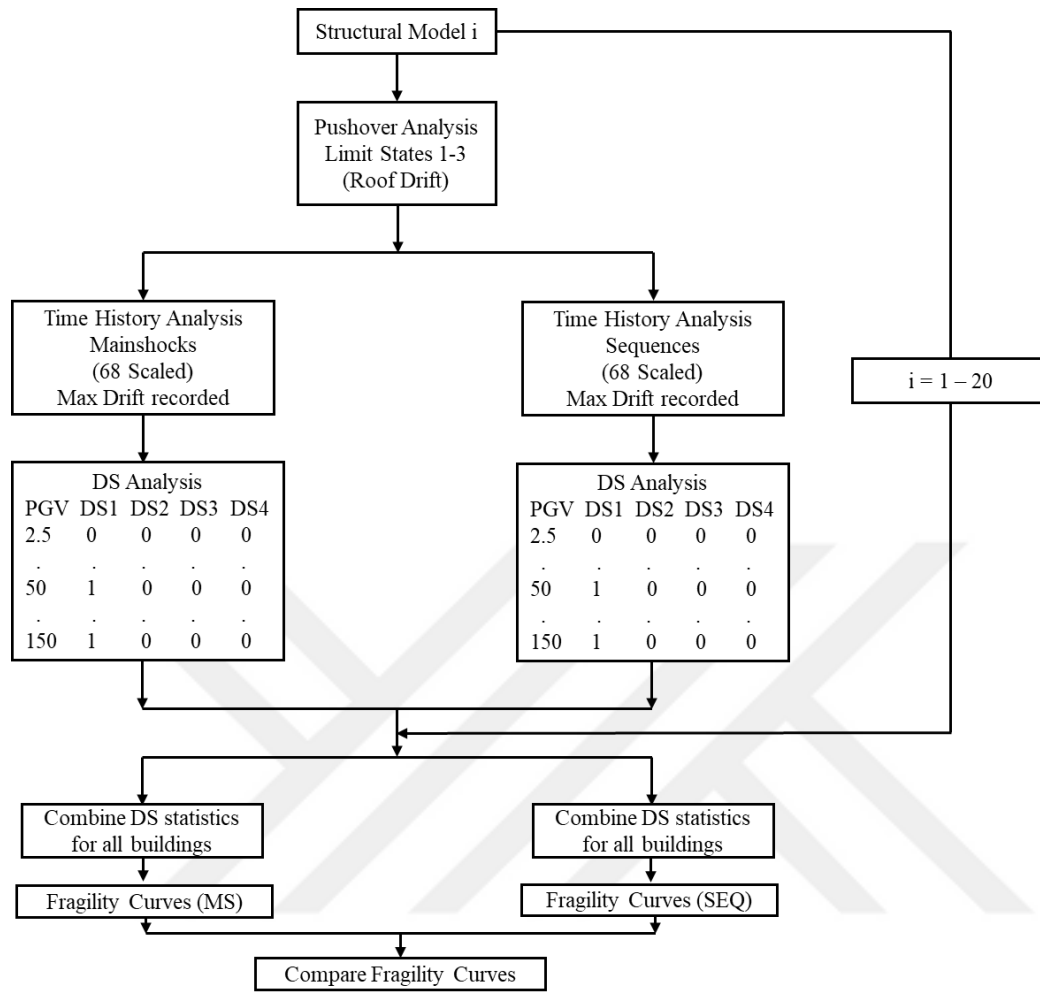


Figure 5.1. Seismic response analysis of 2D RC frame buildings under sequential earthquakes

5.2 Explanation of the RC Frame Properties

In this study, The building models have been designed in accordance with 3 different releases of the Turkish earthquake code, namely TEC-75 (1975), TEC-98 (1998), and TEC-07 (2007). In order to generate in-class variability in terms of structural form and material and sectional properties for mid-rise RC frame simulations, the number of stories varied from 4 to 8 whereas the number of bays varied from 2 to 4.

Also, two very common building irregularities are implemented in the structural models, which are soft story formation and overhangs (i.e. cantilevered first floors). In total, 20 RC frame samples are generated. Some sketches of typical samples from the building database are demonstrated in Figure 5.2.

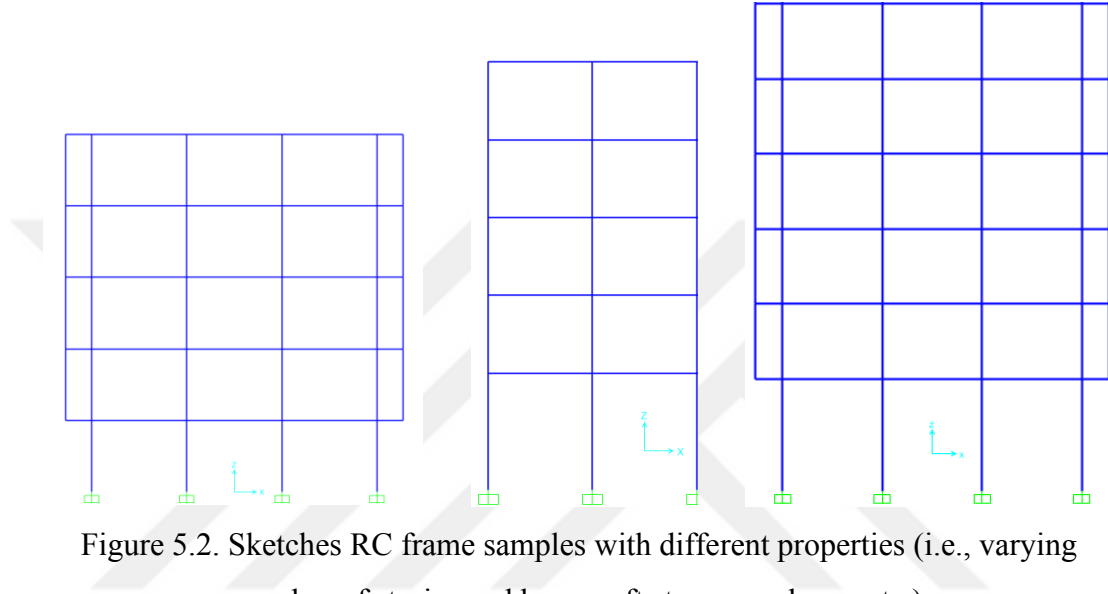


Figure 5.2. Sketches RC frame samples with different properties (i.e., varying number of stories and bays, soft story, overhang, etc.)

5.3 Material and Sectional Properties of Frame Members

There are some constraints to the selection process of structural parameters. It is a fact that the mean strength values of concrete and steel materials exhibit variations according to design and construction practices during the time of construction. In this study, these mean strength values were obtained from the study of Ozmen et al. (2015). Furthermore, the mean values for reinforcement ratios, beam height, and width have been taken from the same study. The bay width and story heights are determined according to the study of Azak et al. (2014). Accordingly, the mean values for span length and story heights have been assumed as 3.6 m and 2.7 m, respectively.

The study of Meral (2018) has also been investigated and the findings of this study also agree with the other studies. The author summarized the details of the buildings constructed after the year 2007. Furthermore, the study indicated that still 70% of the building stock is composed of RC frame structures. Therefore, in this study, dual structures and reinforced concrete wall structures are not considered.

Among the 20 frame simulations, 6 of them have steel reinforcement with 220 MPa yield strength (S220), which are also TEC-75 designs. The rest of the simulations have a yield strength of 420 MPa (S420). With the TEC-98, it was prohibited to use S220 reinforcements in the building construction. Therefore, the use of S420 belongs to the TEC-98 and TEC-07 designs. Moreover, the concrete strength value has a relationship with the design and construction practice of the time considered. The building samples after the year 1975 had 17-18 MPa compressive strength on average. After the year 1998, the concrete compressive strength values commonly varied between 25-30 MPa.

The dimensions of the structural members also vary according to the earthquake codes. Beam dimensions change in height and width from 0.45 m to 0.55 m and from 0.25 m to 0.30 m, respectively. Column dimensions in this study are taken from 0.2 m to 0.4 m in a square section. The reason for using square dimensions is that when an earthquake hits the building from the weak direction of the column, it only behaves as if it is a square.

The reinforcement ratios are determined based on three factors: the minimum values required by earthquake codes, the average values for different story numbers, and the general selection of reinforcement bar sizes. In both beams and columns, the reinforcement ratios exceed the code-prescribed minimums and are nearly double the required values. Additionally, two types of confinement are used, with stirrup spacings of 100 mm and 200 mm. Although there were some little differences for each section, for simplicity, they were grouped according to their required reinforcements. As a result of these variables, there exist 6 types of beam sections

and 8 types of column sections in 20 frame simulations. Typical representations for beam and column sections are shown in Figure 5.3.

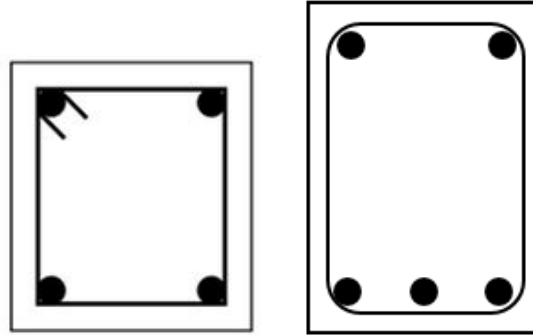


Figure 5.3. Typical Column and Beam Cross-Sections

In Table 5.1, all the aforementioned frame parameters are summarized. The explanations for the abbreviation as follows:

- No: Simulation number
- z: Number of stories
- x: Number of spans
- Code: Release year of the earthquake code
- SS: Soft Story Status (1 = Yes, 0 = No)
- HC: Cantilever Status (1 = Yes, 0 = No)
- tslab: Slab thickness (m)
- fyk: Characteristic steel yield strength (MPa)
- fck: Characteristic concrete strength (MPa)
- hg: Ground story height (m)
- h: Typical story height (m)
- dx: Span length (m)
- bh: Beam height (m)
- bw: Beam width (m)
- cw: Column width and height (square section) (m)
- rho_b: Beam reinforcement ratio (%)
- rho_c: Column reinforcement ratio (%)

Table 5.1 Main properties of the building simulations

No	z	x	Code	SS	HC	tslab	fyk	fck	hg	h	dx	bh	bw	cw	rho_b	rho_c
1	4	2	1975	0	0	0.13	220	18	2.7	2.7	3.6	0.55	0.25	0.2	0.603	1.131
2	4	2	1975	1	1	0.13	220	18	4	2.7	3.6	0.55	0.25	0.2	0.603	1.131
3	4	3	1998	0	0	0.13	420	25	2.7	2.7	3.6	0.45	0.3	0.25	0.510	0.985
4	5	2	1975	1	0	0.13	220	18	4	2.7	3.6	0.55	0.25	0.25	0.603	0.985
5	5	3	1975	0	1	0.13	220	18	2.7	2.7	3.6	0.55	0.25	0.25	0.603	0.985
6	5	3	1998	0	0	0.13	420	25	2.7	2.7	3.6	0.45	0.3	0.3	0.510	1.131
7	5	4	2007	0	0	0.13	420	25	2.7	2.7	3.6	0.45	0.3	0.3	0.510	1.131
8	6	2	1975	1	0	0.12	220	17	4	2.7	3.6	0.55	0.3	0.3	0.674	1.131
9	6	3	1975	0	1	0.12	220	17	2.7	2.7	3.6	0.55	0.3	0.3	0.674	1.131
10	6	3	1998	1	1	0.13	420	25	4	2.7	3.6	0.45	0.3	0.3	0.510	1.131
11	6	3	1998	0	0	0.13	420	25	2.7	2.7	3.6	0.45	0.3	0.3	0.510	1.131
12	6	3	2007	0	0	0.13	420	25	2.7	2.7	3.6	0.45	0.3	0.3	0.510	1.131
13	6	4	2007	0	0	0.13	420	25	2.7	2.7	3.6	0.45	0.3	0.3	0.510	1.131
14	7	2	1998	1	0	0.12	420	29	4	2.7	3.6	0.5	0.3	0.4	0.741	1.131
15	7	3	1998	0	1	0.12	420	29	2.7	2.7	3.6	0.5	0.3	0.4	0.741	1.131
16	7	3	1998	0	0	0.12	420	29	2.7	2.7	3.6	0.5	0.3	0.4	0.741	1.131
17	7	4	2007	0	0	0.12	420	29	2.7	2.7	3.6	0.5	0.3	0.4	0.741	1.131
18	8	3	1998	0	0	0.12	420	29	2.7	2.7	3.6	0.5	0.3	0.4	0.741	1.131
19	8	4	2007	0	0	0.12	420	29	2.7	2.7	3.6	0.5	0.3	0.4	0.741	1.131
20	8	4	2007	0	1	0.12	420	29	2.7	2.7	3.6	0.5	0.3	0.4	0.741	1.131

5.4 Structural Modeling Details of the Building Simulations

In this part, similar to SDOF modeling in the previous chapter, the OPENSEES analysis platform has been utilized for structural modeling and analysis. The base code of the study has been provided by Alici and Sucuoglu (2015). Concrete material is selected as “Concrete01” from the library of the OPENSEES. This material module implements the Kent and Park material model with linear reloading and unloading stiffness and zero tensile strength. Reinforcing steel material is modeled with “Steel01” to obtain uniaxial bilinear behavior with a linear strain hardening feature. Column sections are modeled with fiber sections including concrete core and cover properties. In order to get cracked section behavior correctly, the uncracked moment of inertia of beams is multiplied by 0.4, and similarly, for columns, the coefficient is 0.6 by using the formula in ACI 318 (ACI 2019). Hysteretic behavior is obtained with the Hysteretic Material model as presented in Figure 5.4. This model is a piece-wise linear realization of the force-deformation relationship with pinching,

damage, and degradation based on the ductility concept. The hysteretic material model parameters are defined after moment-curvature analysis of each section.

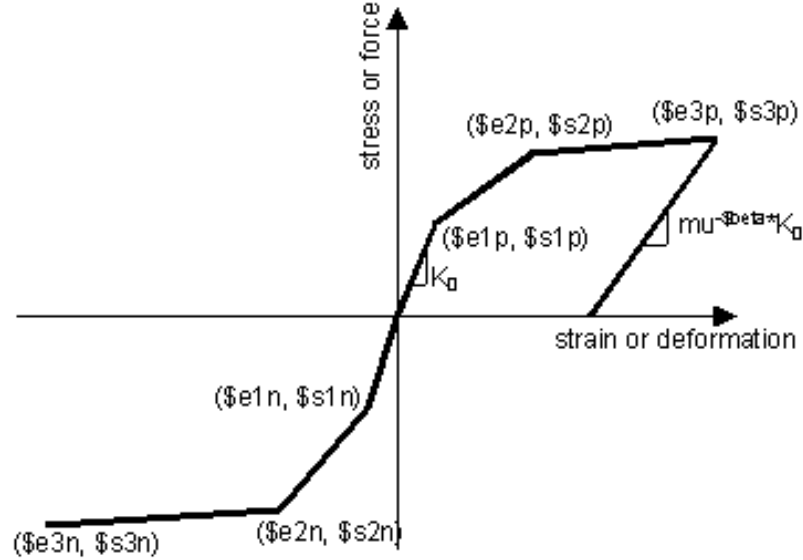


Figure 5.4. Hysteretic material model in OPENSEES library used in this study

The beam elements are modeled by using the “beam with hinges” element model. This model constructs two plastic hinge regions at both ends of the beam. The interior part is modeled with elastic material properties with cracked inertia in order to decrease the analysis duration. The plastic hinge length is selected as half the dimension of the beam height for simplicity since many of the plastic hinge length equations use similar values. Hysteretic material is utilized at hinge regions in order to get nonlinear behavior. The column elements are also generated by using the “beam with hinges” model; however, their hinge regions are considered fiber sections. Similar to beams, their interior sections also have elastic behavior with cracked inertia. P-delta effects are implemented into columns. In other words, the beam and columns are modeled with the limited plasticity approach.

5.5 Seismic Analyses of the Building Simulations

Seismic analyses are carried out in two stages. In the first stage, pushover analyses are carried out in order to attain the limit states of each building simulation to be used during the fragility analysis. In the second stage, dynamic analyses are conducted by considering only the main shock in each sequence (stand-alone response) and by also considering all the events as a combined time-history (sequential response). The following sections explain both stages of analysis in detail.

5.5.1 Pushover Analysis of the Building Simulations

The generated building models are analyzed under pushover analysis to obtain 3 limit states (LS) and in turn, 4 damage states (DS) as defined in the previous chapter. The pushover analysis curves of all building models are presented in Figure 5.5. The horizontal axis shows the top drift of the building (in meters), and the vertical axis demonstrates the base shear coefficient η (i.e., force divided by weight). From pushover curves, it can be observed that there exists a great variability in terms of strength and deformation capacities. This is expected since different design and construction practices have been employed in the generation of the building simulations on purpose.

The sample buildings are created with respect to the limitations of TEC-75, TEC-98, and TEC-07 and also some of the models have been developed by imposing structural deficiencies like soft story or excessive overhang. Considering all these parameters, the buildings are classified under 3 groups in accordance with the A, B, and C-classification of mid-rise RC frame buildings in the previous chapter for SDOF idealizations. These groups can be presented as follows:

- Group A: Well-designed buildings that satisfy TEC-07 with no structural deficiencies (i.e., high-code buildings). The codes of the simulations in this group are 14, 15, 16, 17, 18, 19, and 20 (7 Buildings).

- Group B: Buildings designed according to TEC-98 or TEC-07, but with some structural deficiencies due to design or construction stages. The codes of the simulations in this group are 3, 6, 7, 11, 12, and 13 (6 Buildings).
- Group C: Poorly designed buildings that partly satisfy TEC-75 and possess many structural deficiencies. The codes of the simulations in this group are 1, 2, 4, 5, 8, 9, and 10 (7 Buildings)

The pushover curves of each group are given in Figure 5.6. Force and displacement capacities are observed to be consistent within each group. As expected, Group C has the lowest values whereas Group A has the highest capacities. These trends are also in accordance with the RC frame classes in SDOF models, i.e. RF2A-B-C.

After obtaining these pushover curves, trilinear curves are fitted to each pushover curve in order to attain 3 limit states. The first limit state, i.e., LS1 denotes the drift ratio when yielding capacity in members has been exceeded so that the softening of the initial slope takes place. The second limit state, i.e., LS2 stands for the drift ratio with the highest strength capacity (in meters). Finally, the third limit state, i.e., LS3 denotes the drift ratio just before the failure with a significant drop in strength capacity (i.e. 75%-80% of the maximum capacity). The tri-linearization process is demonstrated in Figure 5.7. The values of limit states for all the building simulations are provided in Table 5.2 together with height to width ratio of the buildings and ductility capacity (μ_{cap} , defined as the ratio of drift at LS2 to LS1) and ductility at collapse (μ_{col} , defined as the ratio of drift at LS3 to LS1) values. The range of η , μ_{cap} and μ_{col} values for A, B, and C classes are as follows: For Class A: $0.28 < \eta < 0.35$, $2.50 < \mu_{cap} < 5.04$, $3.79 < \mu_{col} < 6.46$; for Class B: $0.18 < \eta < 0.24$, $1.58 < \mu_{cap} < 2.84$, $2.38 < \mu_{col} < 4.62$; for Class C: $0.05 < \eta < 0.15$, $1.31 < \mu_{cap} < 1.68$, $1.77 < \mu_{col} < 2.80$. This range of values seems to be consistent with the structural characteristics of each group.

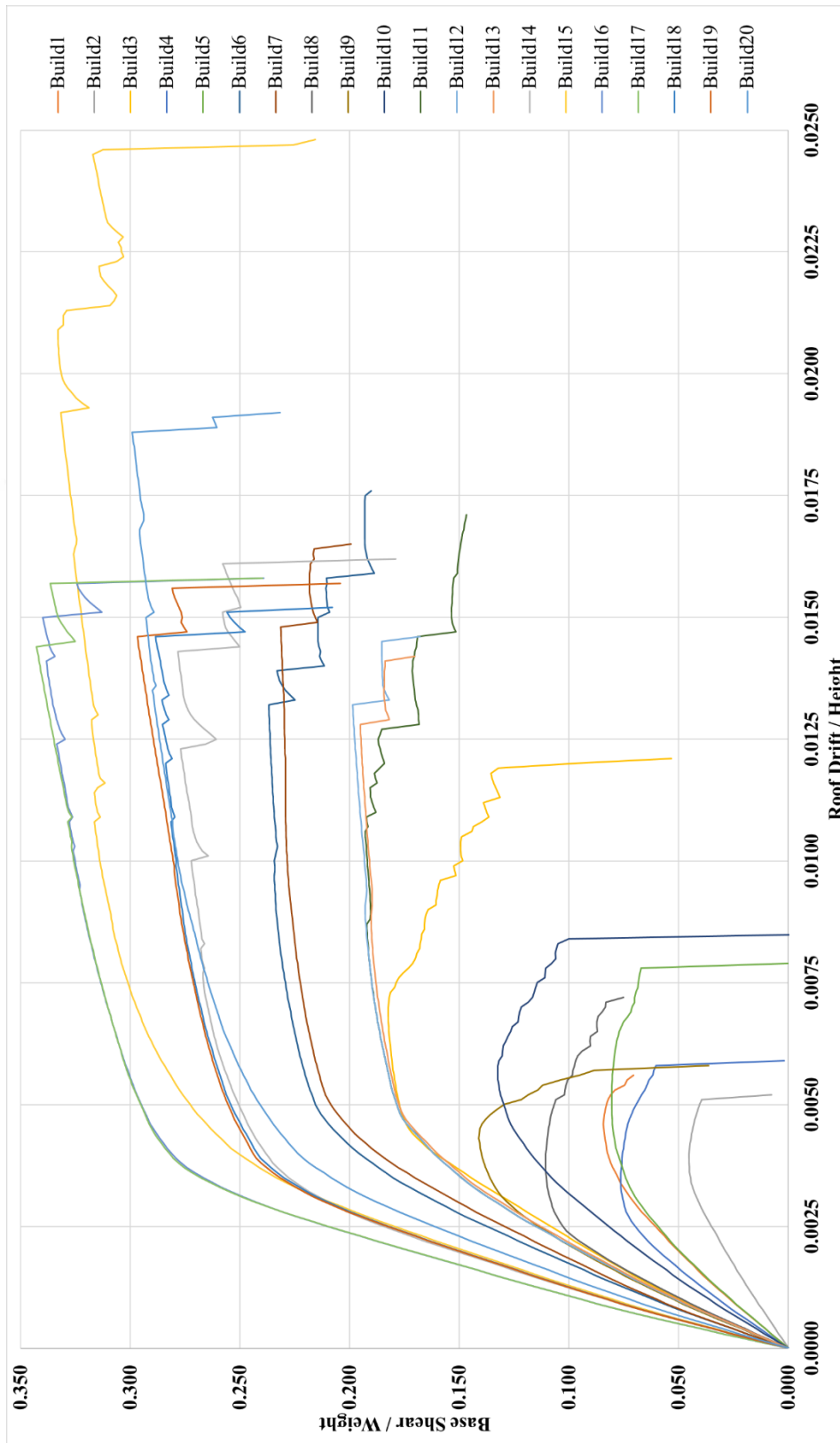


Figure 5.5. Pushover curves for all building simulations

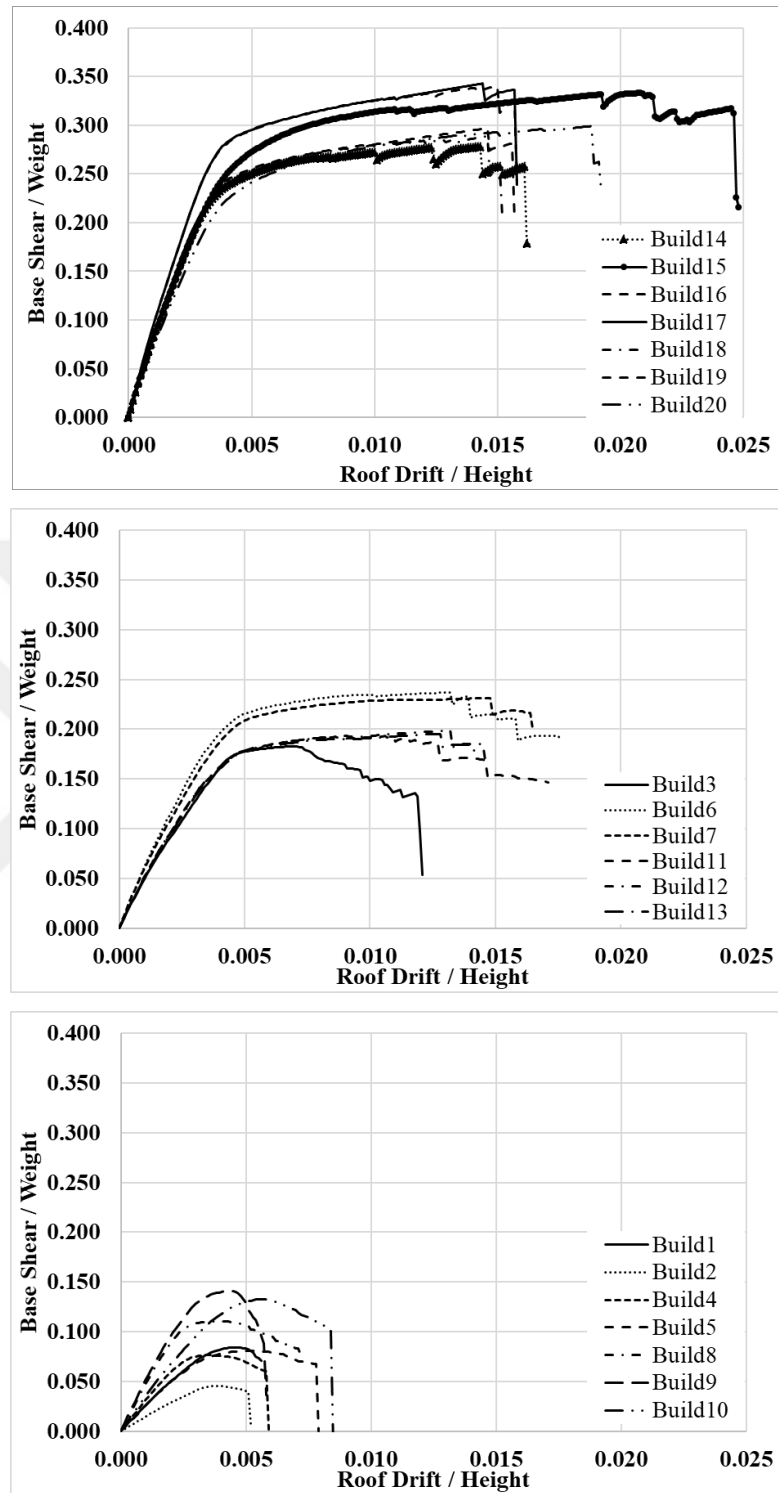


Figure 5.6. Pushover curves for groups A (top), B (middle) and C (bottom)

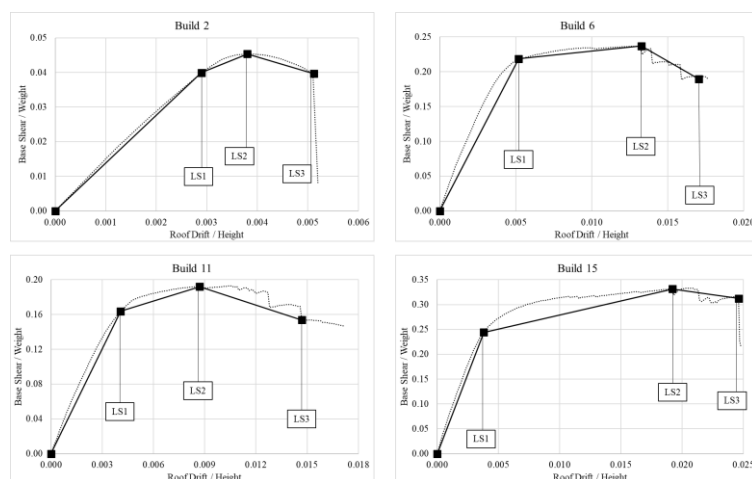


Figure 5.7. Attainment of limit states from sample pushover curves

Table 5.2 Limit state values for all considered building simulations

NO	H (m)	LS1 (m)	LS2 (m)	LS3 (m)	LS1 (%)	LS2 (%)	LS3 (%)	μ_{cap}	μ_{col}
1	10.8	0.032	0.050	0.061	0.30	0.46	0.56	1.56	1.91
2	12.1	0.035	0.046	0.062	0.29	0.38	0.51	1.31	1.77
3	10.8	0.048	0.076	0.114	0.44	0.70	1.06	1.58	2.38
4	14.8	0.036	0.050	0.086	0.24	0.34	0.58	1.39	2.39
5	13.5	0.042	0.069	0.105	0.31	0.51	0.78	1.64	2.50
6	13.5	0.070	0.179	0.230	0.52	1.33	1.70	2.56	3.29
7	13.5	0.072	0.199	0.320	0.53	1.47	2.37	2.76	4.44
8	17.5	0.040	0.067	0.112	0.23	0.38	0.64	1.68	2.80
9	16.2	0.042	0.070	0.092	0.26	0.43	0.57	1.67	2.19
10	17.5	0.061	0.098	0.145	0.35	0.56	0.83	1.61	2.38
11	16.2	0.066	0.141	0.238	0.41	0.87	1.47	2.14	3.61
12	16.2	0.076	0.213	0.345	0.47	1.31	2.13	2.80	4.54
13	16.2	0.073	0.207	0.337	0.45	1.28	2.08	2.84	4.62
14	20.2	0.073	0.250	0.325	0.36	1.24	1.61	3.42	4.45
15	18.9	0.072	0.363	0.465	0.38	1.92	2.46	5.04	6.46
16	18.9	0.078	0.206	0.296	0.41	1.09	1.57	2.64	3.79
17	18.9	0.072	0.205	0.296	0.38	1.08	1.57	2.85	4.11
18	21.6	0.080	0.233	0.326	0.37	1.08	1.51	2.91	4.08
19	21.6	0.082	0.205	0.337	0.38	0.95	1.56	2.50	4.11
20	21.6	0.084	0.294	0.415	0.39	1.36	1.92	3.50	4.94

Table 5.3 Earthquake Set

No	Seq	Event	Scale	PGA (g)	PGV (cm/s)	No	Seq	Event	Scale	PGA (g)	PGV (cm/s)
1	S-21	E-067	1.00	0.03	2.36	35	S-3	E-008	1.00	0.36	56.00
2	S-30	E-105	0.55	0.02	2.45	36	S-17	E-056	1.00	0.40	57.47
3	S-28	E-099	1.00	0.11	3.77	37	S-26	E-089	1.30	0.34	60.23
4	S-30	E-105	1.00	0.02	4.40	38	S-17	E-056	1.10	0.44	63.22
5	S-1	E-001	1.25	0.05	5.25	39	S-3	E-008	1.15	0.41	64.40
6	S-2	E-005	0.80	0.33	6.40	40	S-13	E-037	1.00	1.40	65.43
7	S-2	E-005	1.00	0.42	8.00	41	S-3	E-008	1.20	0.43	67.20
8	S-16	E-050	1.00	0.10	8.61	42	S-17	E-056	1.20	0.48	68.96
9	S-15	E-047	1.00	0.17	9.50	43	S-5	E-012	0.80	0.53	71.60
10	S-29	E-103	1.20	0.12	11.84	44	S-13	E-037	1.12	1.57	73.28
11	S-4	E-010	1.00	0.18	14.60	45	S-18	E-059	1.10	0.90	73.50
12	S-12	E-033	1.10	0.14	15.81	46	S-9	E-021	0.90	0.46	75.72
13	S-23	E-074	1.00	0.23	17.10	47	S-13	E-037	1.20	1.68	78.52
14	S-22	E-070	1.00	0.25	18.00	48	S-18	E-059	1.20	0.98	80.18
15	S-6	E-015	1.00	0.17	19.00	49	S-13	E-037	1.30	1.82	85.06
16	S-6	E-015	1.20	0.20	22.80	50	S-5	E-012	1.00	0.66	89.50
17	S-25	E-084	1.00	0.23	23.88	51	S-10	E-025	1.00	0.59	92.97
18	S-20	E-065	1.00	0.30	24.62	52	S-18	E-059	1.45	1.19	96.89
19	S-25	E-084	1.10	0.26	26.27	53	S-5	E-012	1.10	0.73	98.45
20	S-25	E-084	1.20	0.28	28.66	54	S-10	E-025	1.10	0.65	102.27
21	S-20	E-065	1.20	0.36	29.54	55	S-9	E-021	1.25	0.64	105.16
22	S-24	E-079	0.87	0.31	31.14	56	S-5	E-012	1.20	0.79	107.40
23	S-20	E-065	1.35	0.40	33.24	57	S-9	E-021	1.32	0.68	111.05
24	S-24	E-079	1.00	0.35	35.79	58	S-11	E-029	1.00	0.66	113.16
25	S-27	E-098	1.00	0.53	37.92	59	S-7	E-018	1.00	1.18	116.00
26	S-24	E-079	1.10	0.39	39.37	60	S-10	E-025	1.30	0.77	120.86
27	S-27	E-098	1.15	0.41	41.16	61	S-11	E-029	1.10	0.72	124.48
28	S-8	E-020	1.00	0.48	42.35	62	S-7	E-018	1.10	1.30	127.60
29	S-8	E-020	1.05	0.51	44.47	63	S-10	E-025	1.40	0.71	130.16
30	S-19	E-062	1.00	0.19	45.95	64	S-11	E-029	1.20	0.79	135.79
31	S-19	E-062	1.05	0.20	48.25	65	S-7	E-018	1.20	1.42	139.20
32	S-26	E-089	1.10	0.29	50.96	66	S-11	E-029	1.25	0.82	141.45
33	S-19	E-062	1.13	0.22	51.92	67	S-7	E-018	1.25	1.47	145.00
34	S-26	E-089	1.15	0.30	53.28	68	S-14	E-041	0.70	0.35	149.31

5.5.2 Time History Analysis of the Building Simulations

After obtaining the limit states from the pushover analysis, a nonlinear time history analysis procedure has been implemented in the building simulations in order to develop fragility curves for mainshock only and sequence cases by using the ground motion database in this study. During the analyses, Rayleigh damping parameters were used to maintain a 5% damping ratio. The damping parameters are calculated in both mass and stiffness proportionally from modal analyses.

Finally, a selection of single earthquakes and their sequences are represented in Table 5.3. This table has been formed according to the results of the SDOF analysis in the previous chapter and it is the refined and scaled version of the earthquake sequences in Appendix C with reference to PGV values. In order to obtain a ground motion set with evenly distributed PGV values from 2.36 cm/s to 149.31 cm/s., some records have been scaled.

In total, 68 single earthquakes (mainshocks from sequences) and 68 sequential records have been developed for the time history analysis procedure. Each building was analyzed with 136 earthquakes and a total of 2720 analyses were carried out with the OPENSEES platform. This process took around 28 hours to complete and resulted in 550 GB of data.

5.6 Seismic Fragility Analysis

Following the completion of the analyses, extensive post-processing was conducted on the large dataset of structural responses. A MATLAB script was developed to organize and process this data for fragility analysis. The results for each building were systematically categorized into two groups: responses to single earthquake events and responses to sequential earthquake events.

Figures 5.8 – 5.27 show the scatter of global drift ratio (%) in terms of PGV for each building simulation individually. Since the drift ratio values cover a wide range, a

log scale was used to visualize them on the plots. Also, the limit states were shown on the figures to indicate the portion of data exceeding each limit state. Crosses (X) represent the results from the main event only cases whereas squares show the results from sequential cases. When all the figures are examined, it is observed that in the case of deficient building simulations (i.e., class-C and some of class-B), drifts from mainshock analysis are very high so that they largely exceed even the most severe limit state, i.e., LS3. This means these buildings directly collapse under the main event without being subjected to sequential earthquakes. This was also the case during the response analysis carried out for the SDOF simulations in Chapter 4.

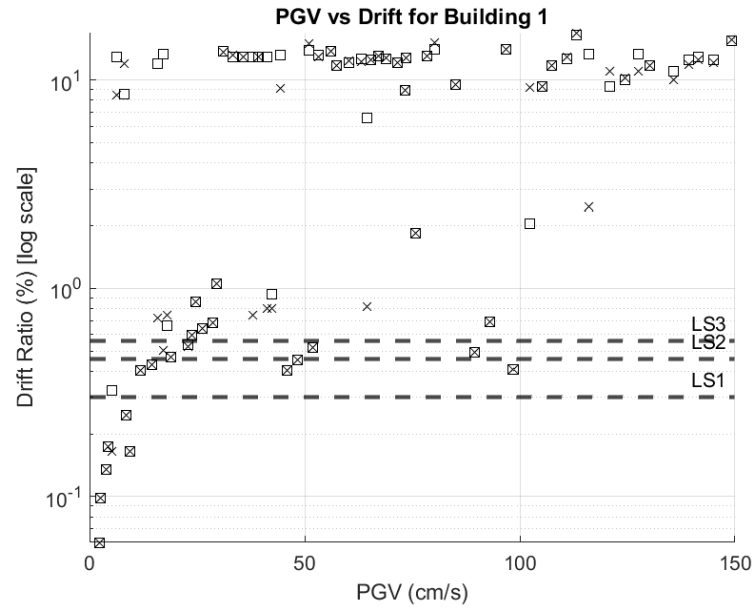


Figure 5.8. PGV vs drift ratio scatter plots for building simulation no. 1

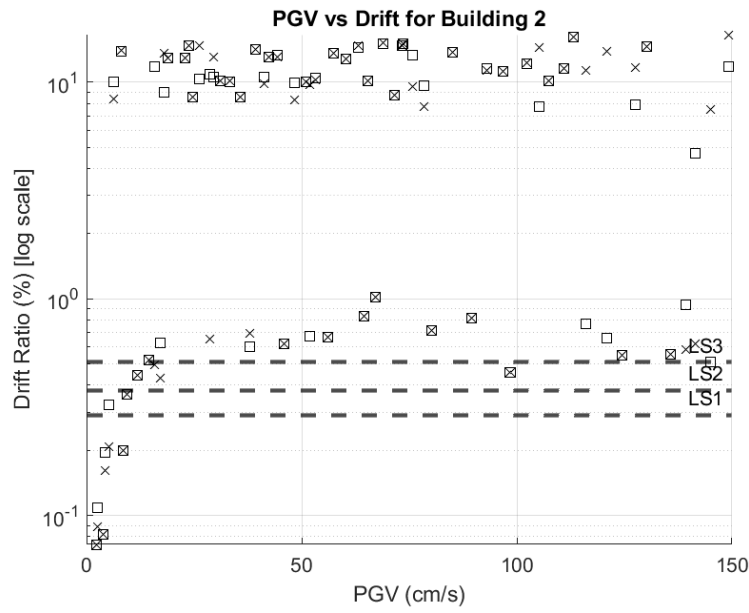


Figure 5.9. PGV vs drift ratio scatter plots for building simulation no. 2

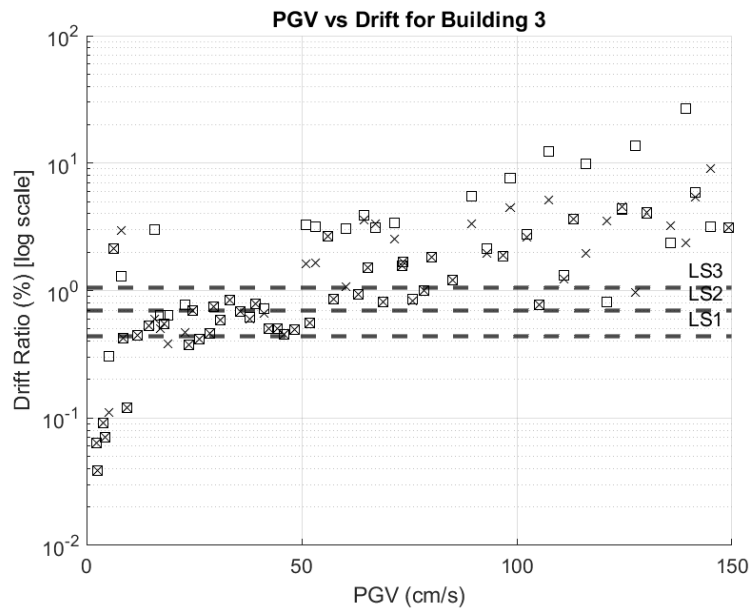


Figure 5.10. PGV vs drift ratio scatter plots for building simulation no. 3

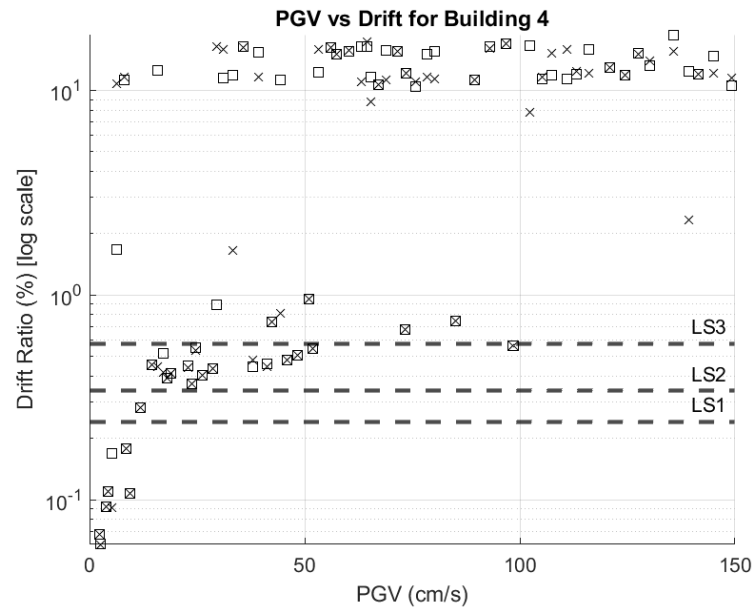


Figure 5.11. PGV vs drift ratio scatter plots for building simulation no. 4

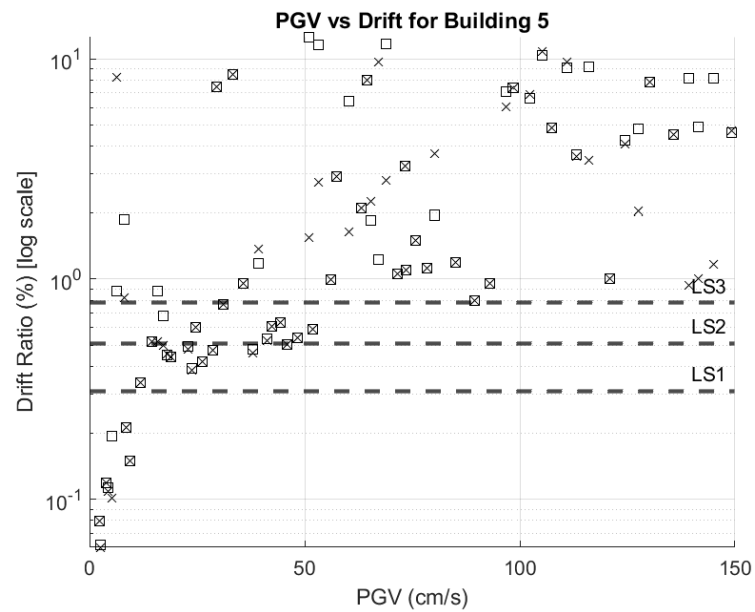


Figure 5.12. PGV vs drift ratio scatter plots for building simulation no. 5

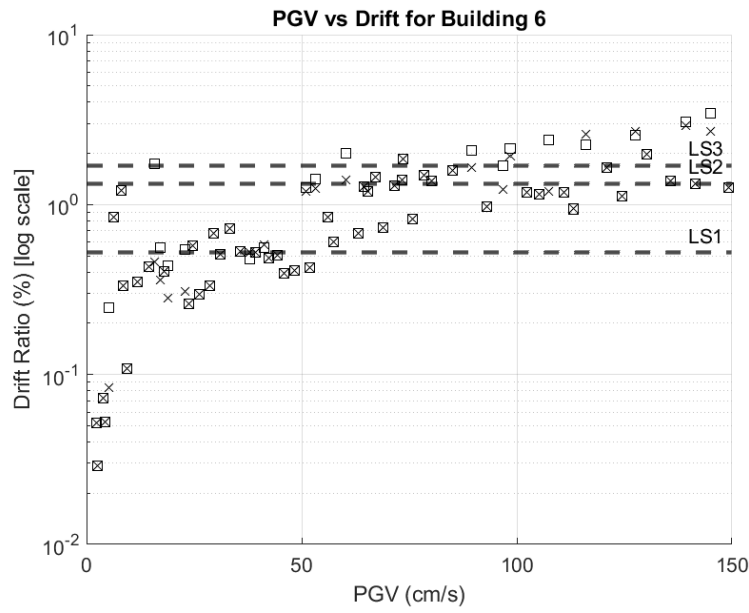


Figure 5.13. PGV vs drift ratio scatter plots for building simulation no. 6

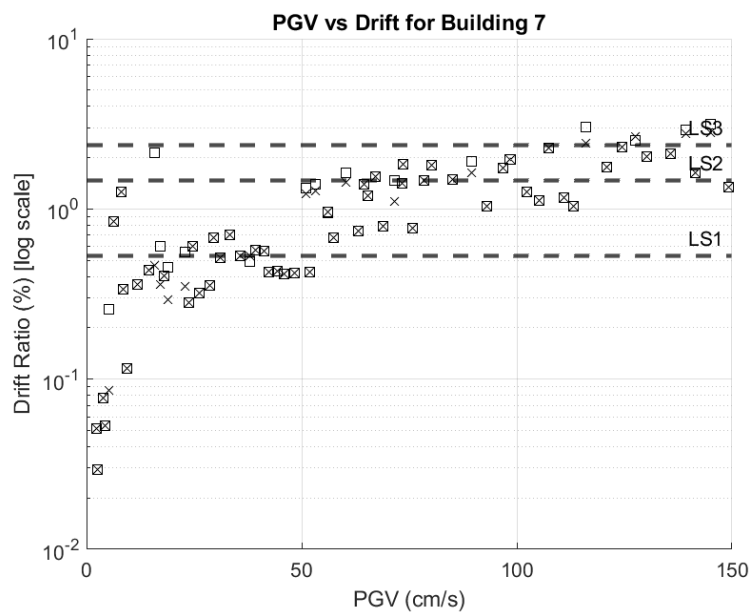


Figure 5.14. PGV vs drift ratio scatter plots for building simulation no. 7

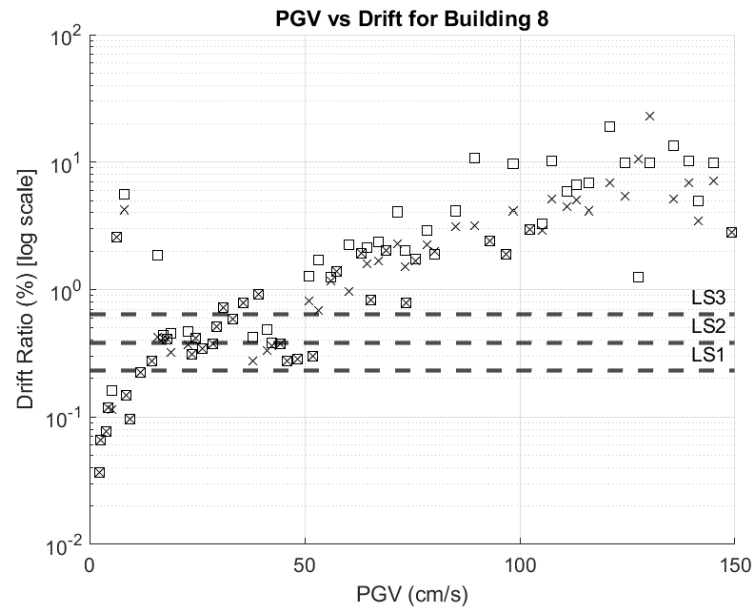


Figure 5.15. PGV vs drift ratio scatter plots for building simulation no. 8

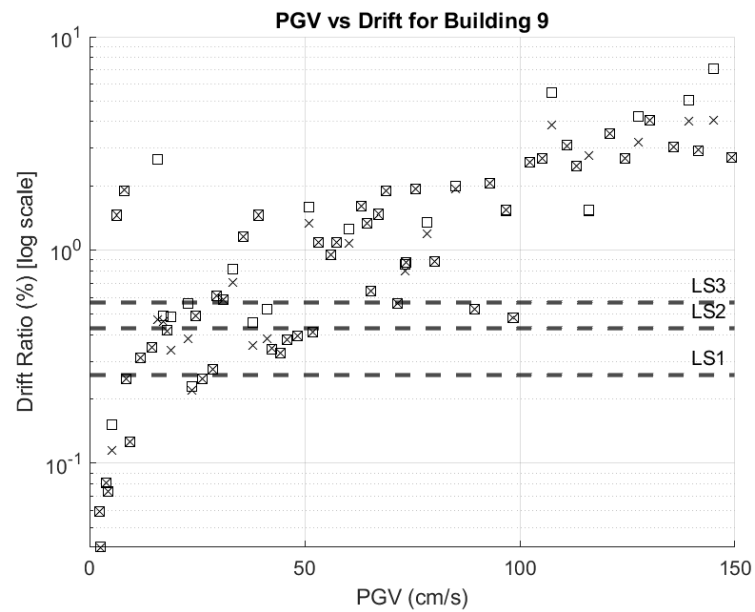


Figure 5.16. PGV vs drift ratio scatter plots for building simulation no. 9

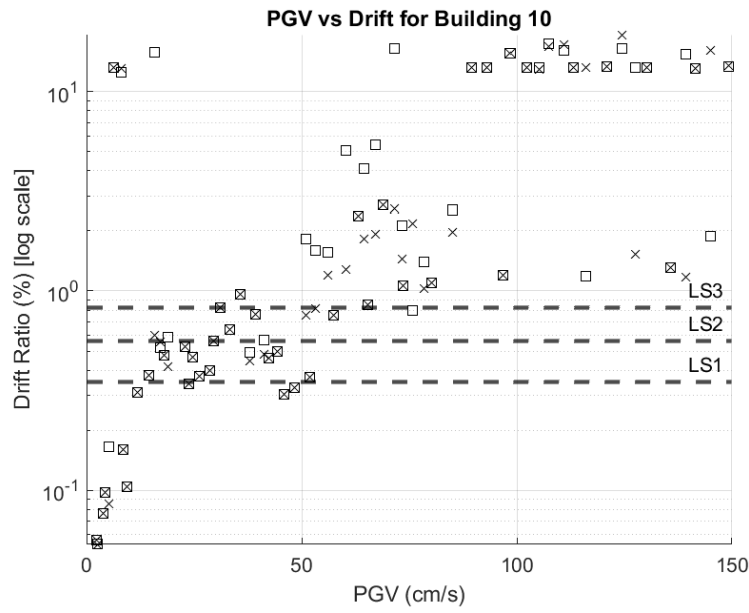


Figure 5.17. PGV vs drift ratio scatter plots for building simulation no. 10

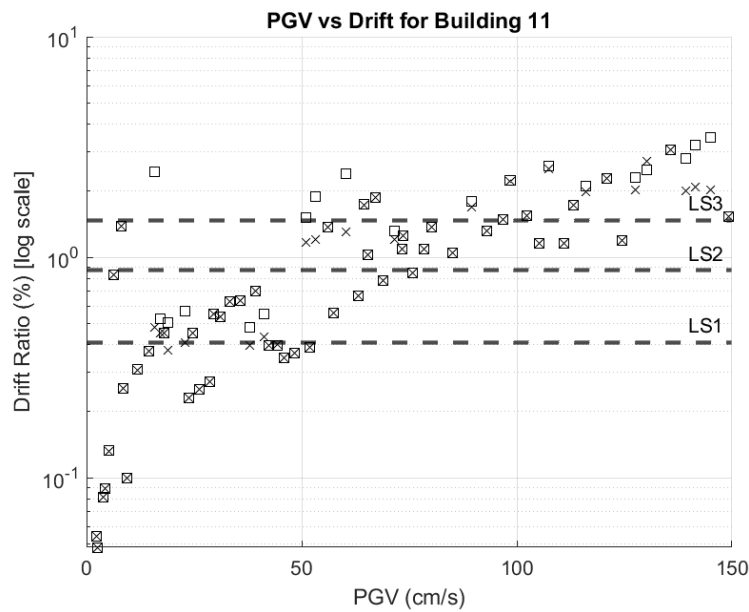


Figure 5.18. PGV vs drift ratio scatter plots for building simulation no. 11

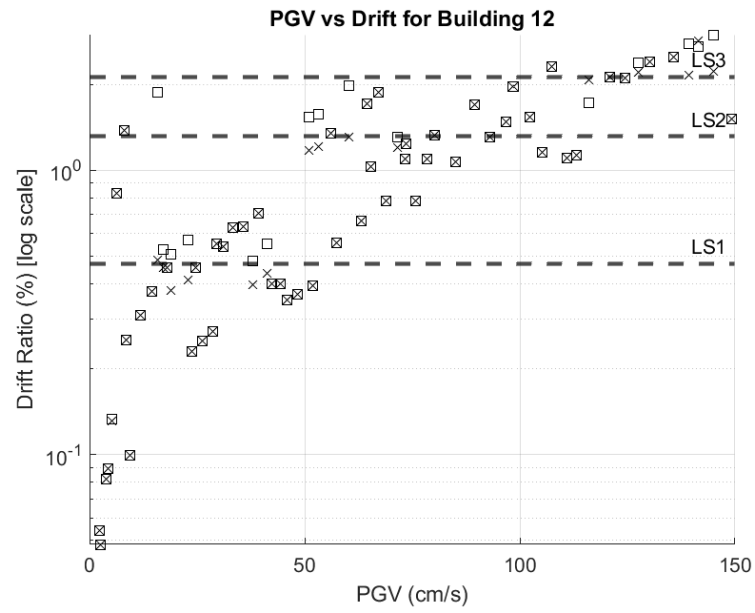


Figure 5.19. PGV vs drift ratio scatter plots for building simulation no. 12

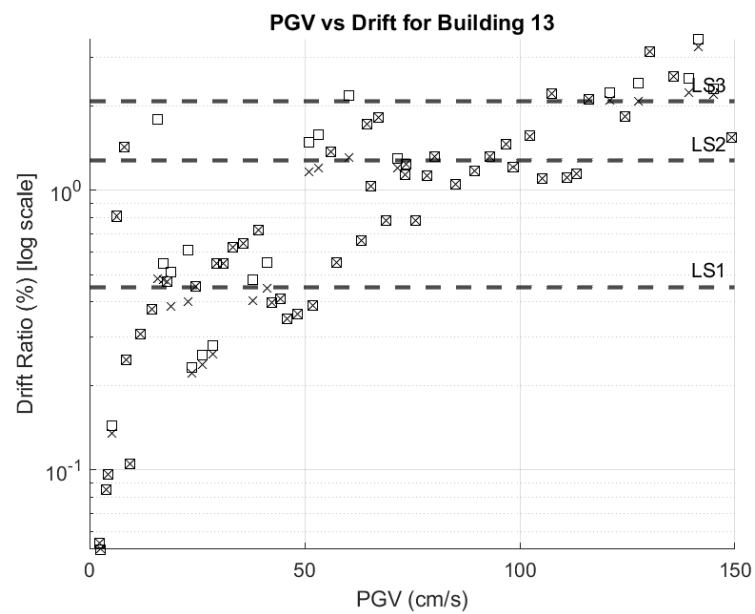


Figure 5.20. PGV vs drift ratio scatter plots for building simulation no. 13

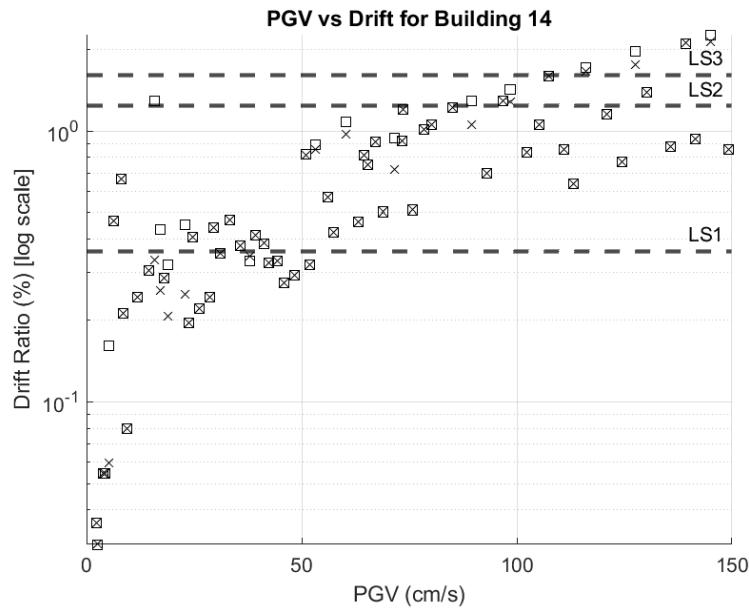


Figure 5.21. PGV vs drift ratio scatter plots for building simulation no. 14

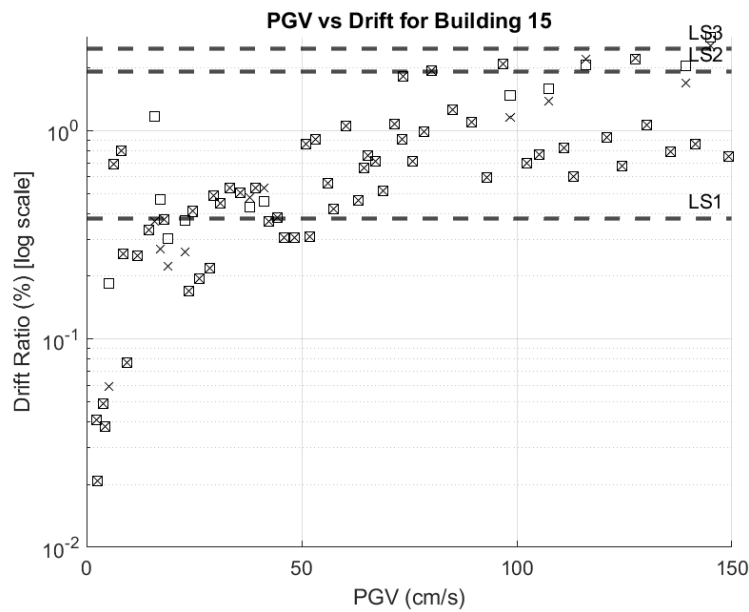


Figure 5.22. PGV vs drift ratio scatter plots for building simulation no. 15

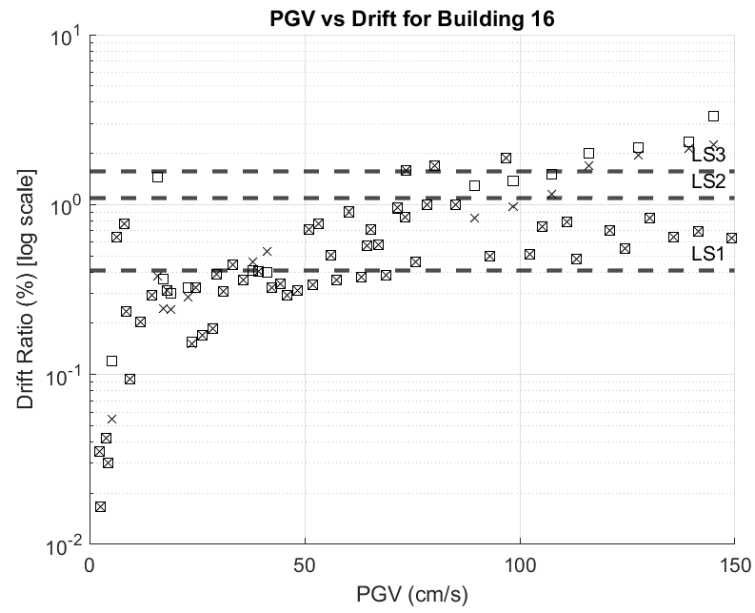


Figure 5.23. PGV vs drift ratio scatter plots for building simulation no. 16

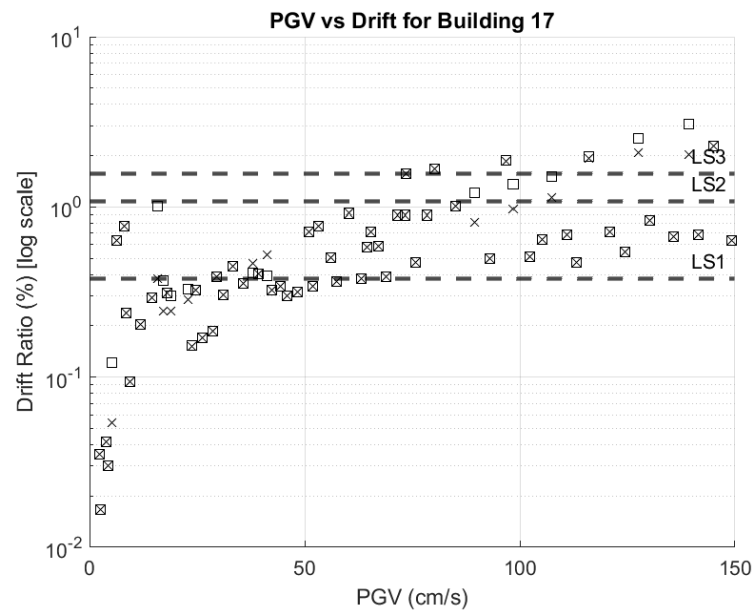


Figure 5.24. PGV vs drift ratio scatter plots for building simulation no. 17

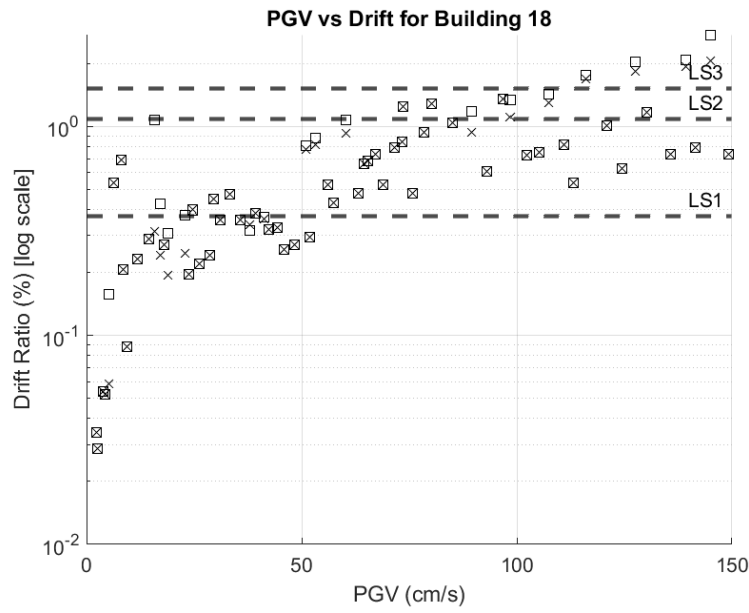


Figure 5.25. PGV vs drift ratio scatter plots for building simulation no. 18

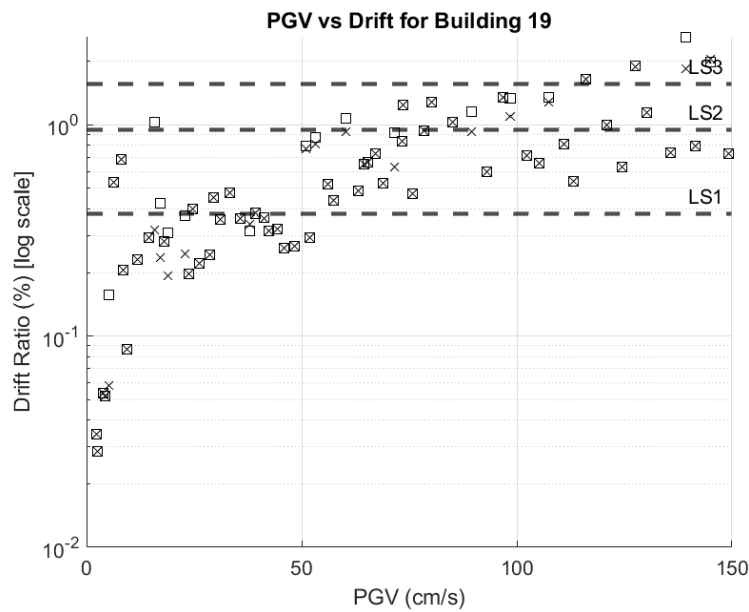


Figure 5.26. PGV vs drift ratio scatter plots for building simulation no. 19

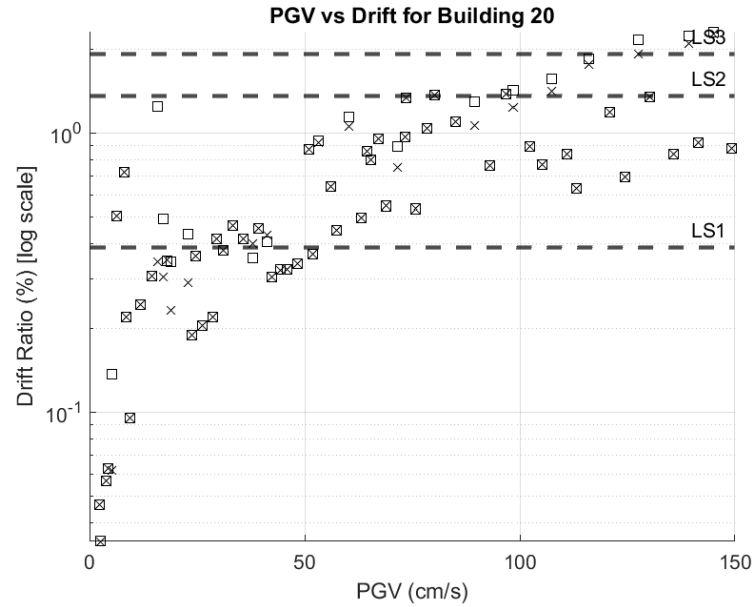


Figure 5.27. PGV vs drift ratio scatter plots for building simulation no. 20

In order to compare drift demands in terms of PGV with drift limit states (i.e., capacities), the exceedance or non-exceedance status of each data is recorded with the help of an indicator variable for each building simulation. In this binary coding approach, the indicator variable can take the values of “0” for the non-exceedance of the corresponding limit state and “1” for its exceedance. An example case is shown in Figure 5.28 for building simulation no.11 under the main event case. From the figures, it is revealed that among the 68 points of analysis data, the cases for exceedance of seismic capacity decrease going from lower to higher limit states, which is eventually just the opposite for cases of non-exceedance. This is an expected trend as in fragility analysis.

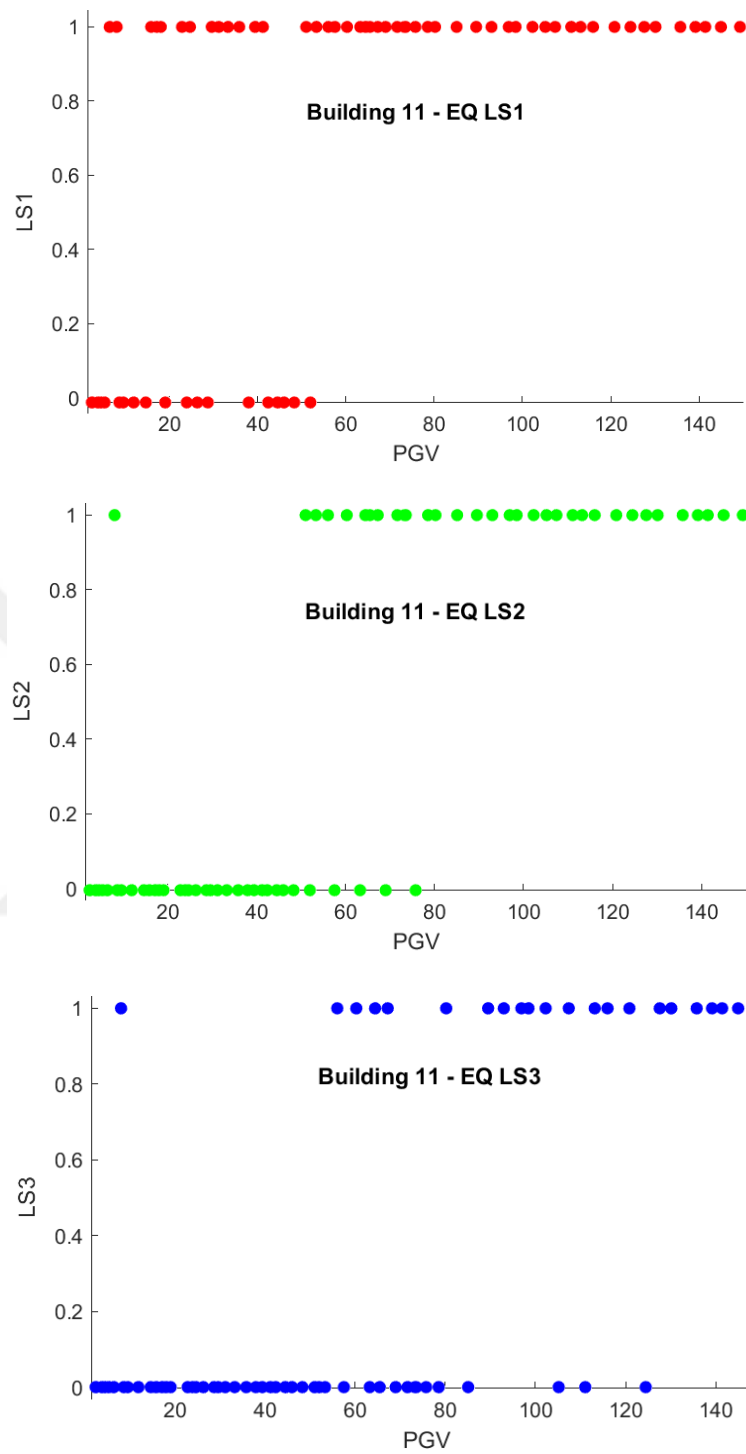


Figure 5.28. Exceedance or non-exceedance of limit states for building simulation no.11 under main event case

The last step is to obtain the fragility curves from the binary exceedance data. In order to achieve this task, Binary Logistic Regression has been employed. This approach is a type of regression model that can be used when the response variable is binary (Pregibon 1981, Cox and Snell 1989). By using this technique, fragility curves for all 20 building simulations and 3 limit states have been obtained in the case of the main event and sequence analysis. These plots are presented in Figures 5.29-5.48.

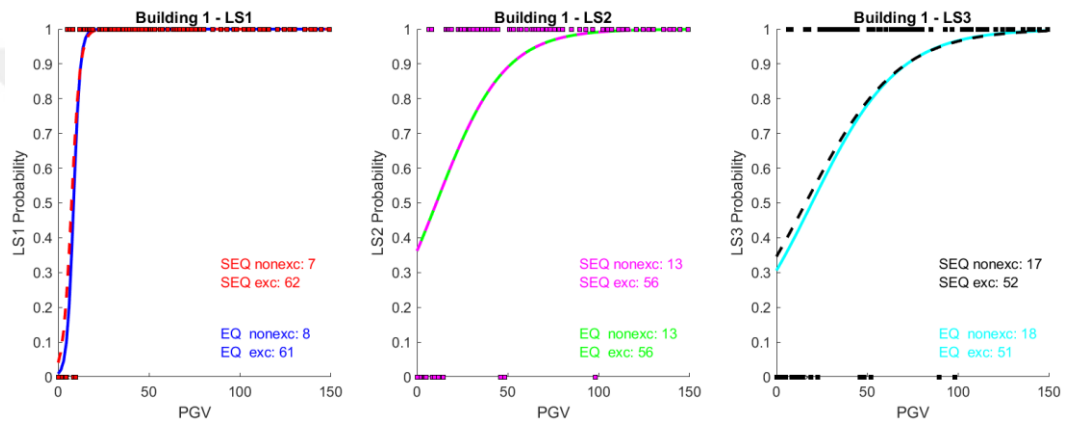


Figure 5.29. Fragility curves for building simulation no.1

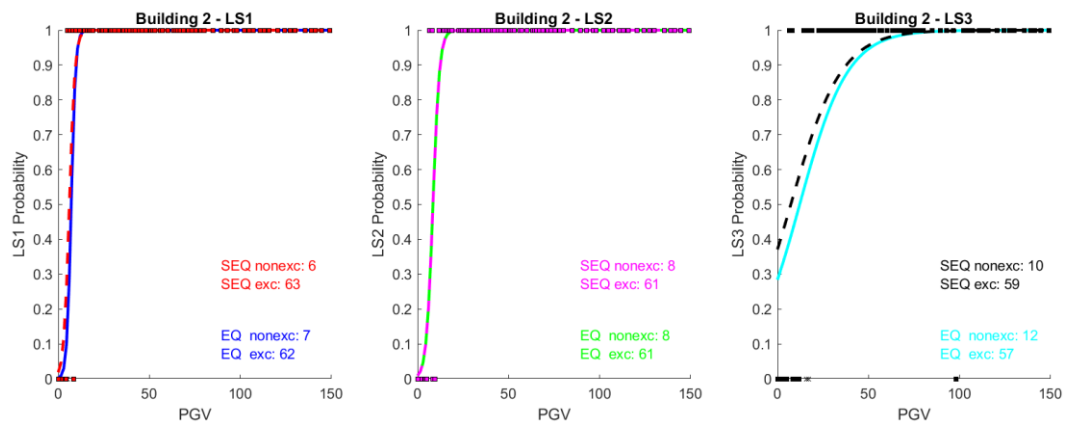


Figure 5.30. Fragility curves for building simulation no.2

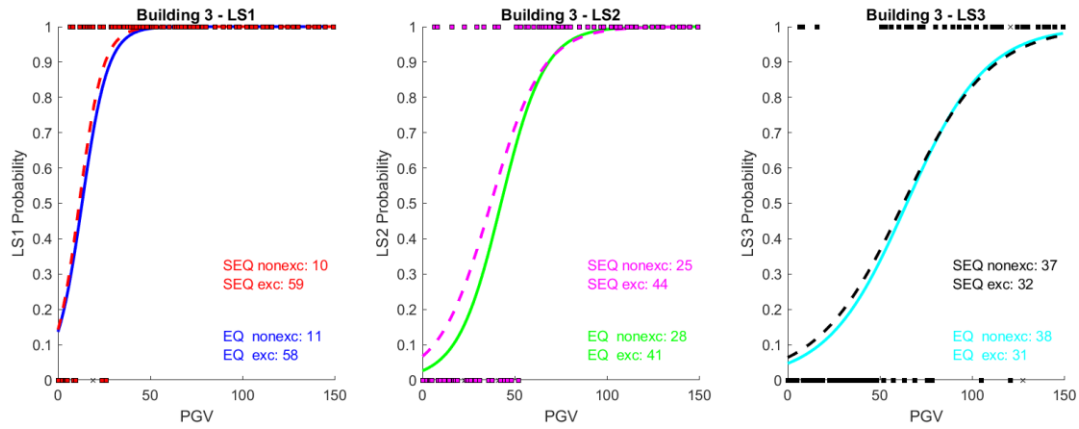


Figure 5.31. Fragility curves for building simulation no.3

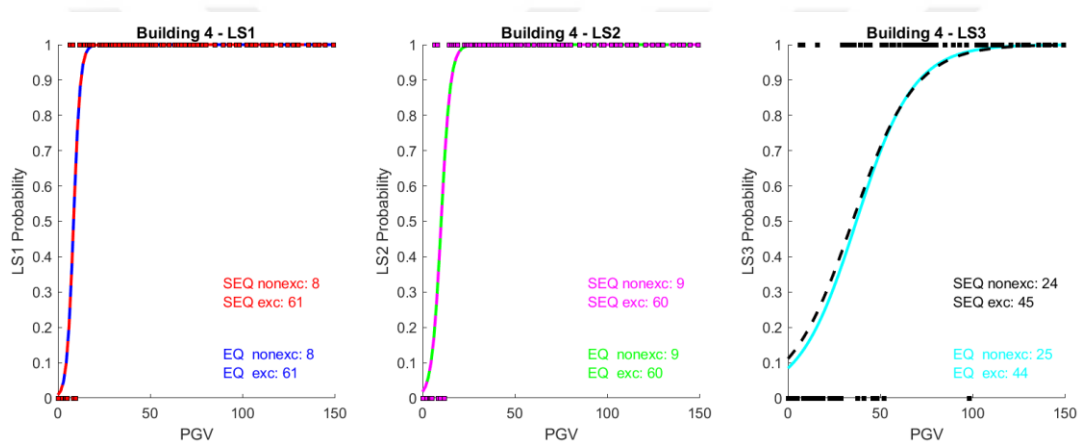


Figure 5.32. Fragility curves for building simulation no.4

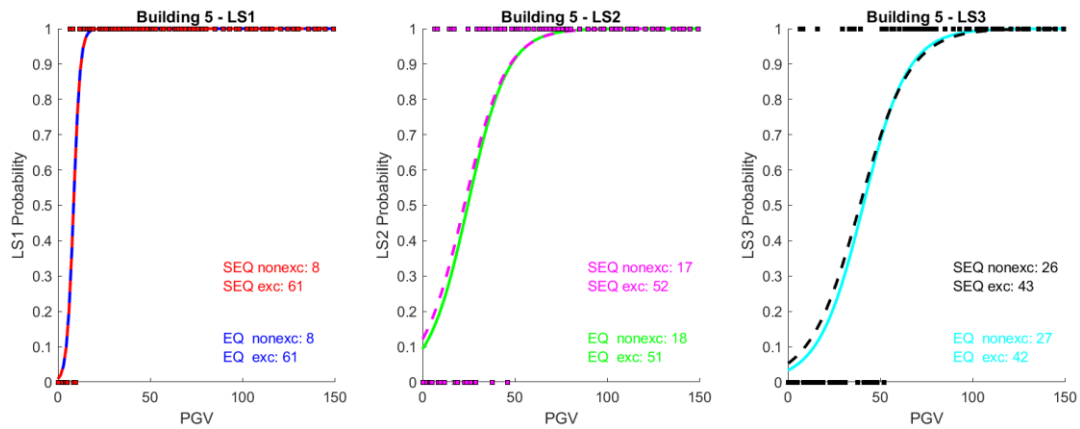


Figure 5.33. Fragility curves for building simulation no.5

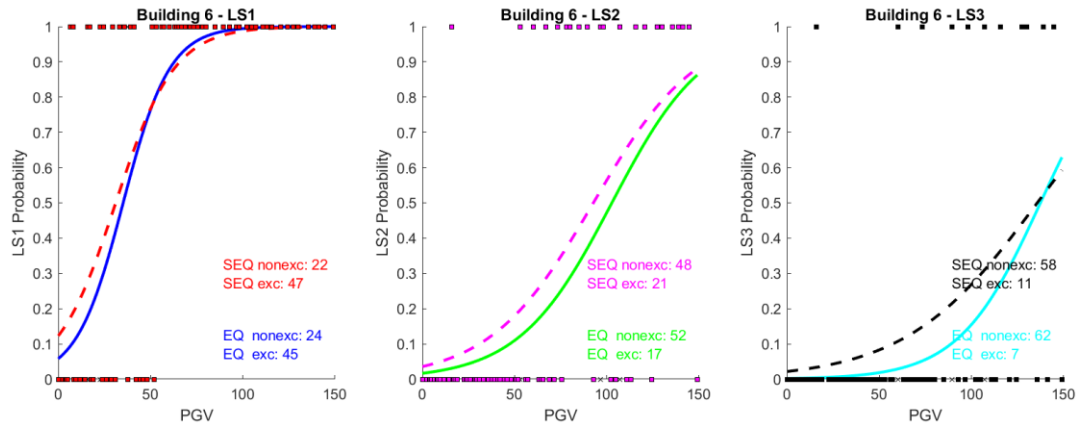


Figure 5.34. Fragility curves for building simulation no.6

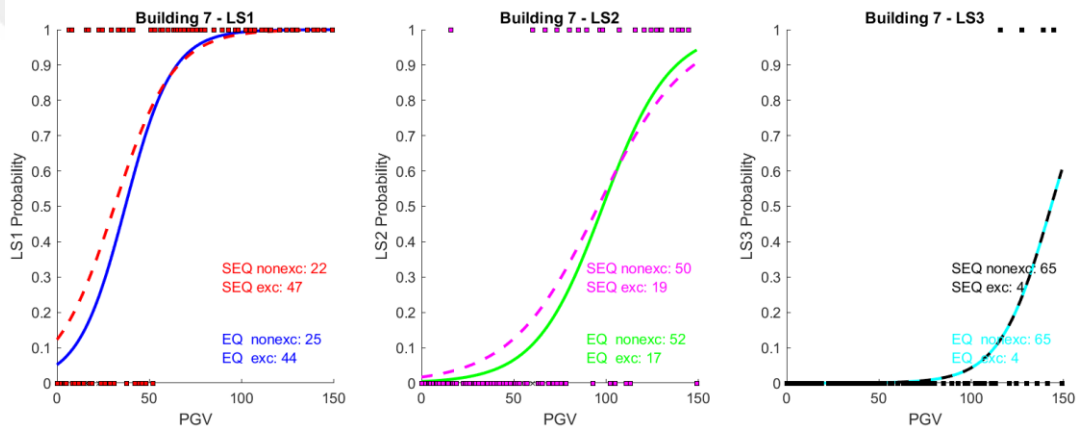


Figure 5.35. Fragility curves for building simulation no.7

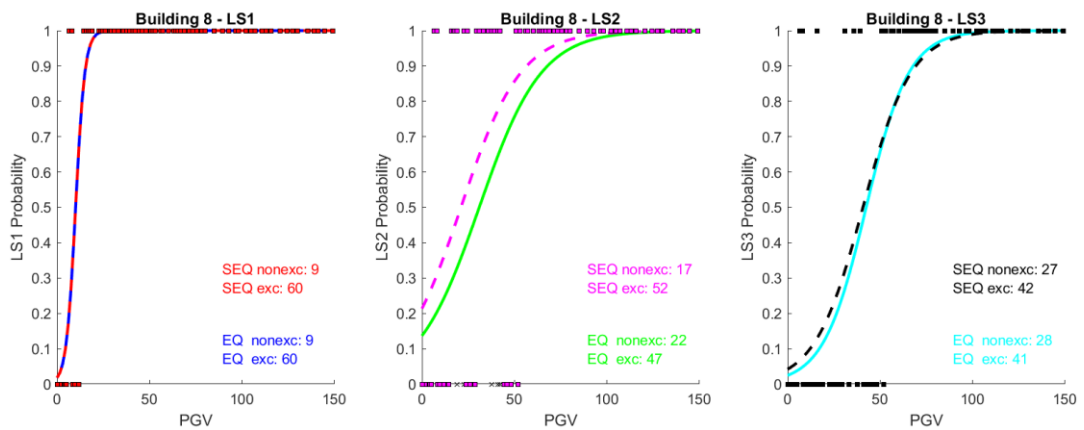


Figure 5.36. Fragility curves for building simulation no.8

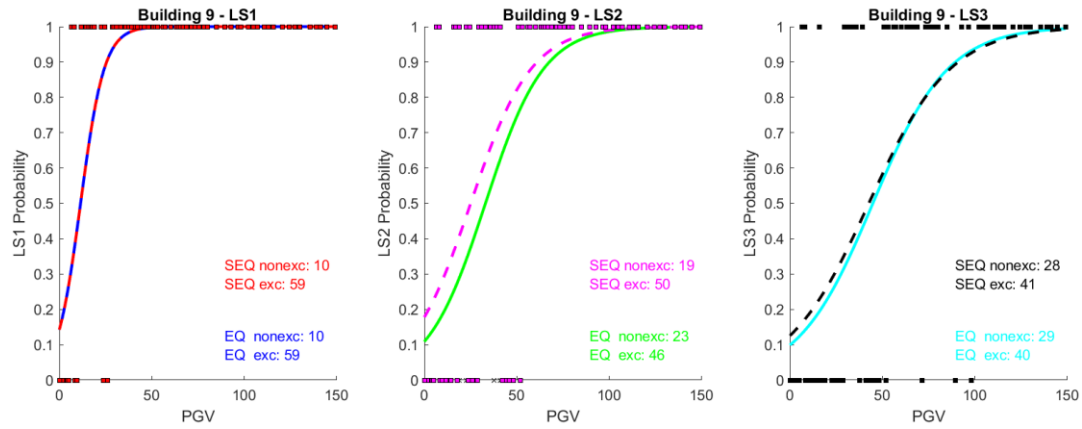


Figure 5.37. Fragility curves for building simulation no.9

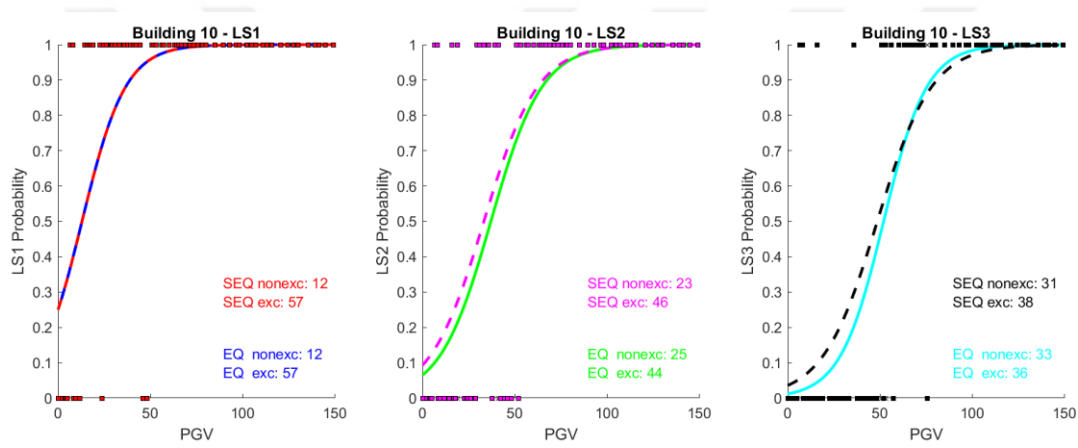


Figure 5.38. Fragility curves for building simulation no.10

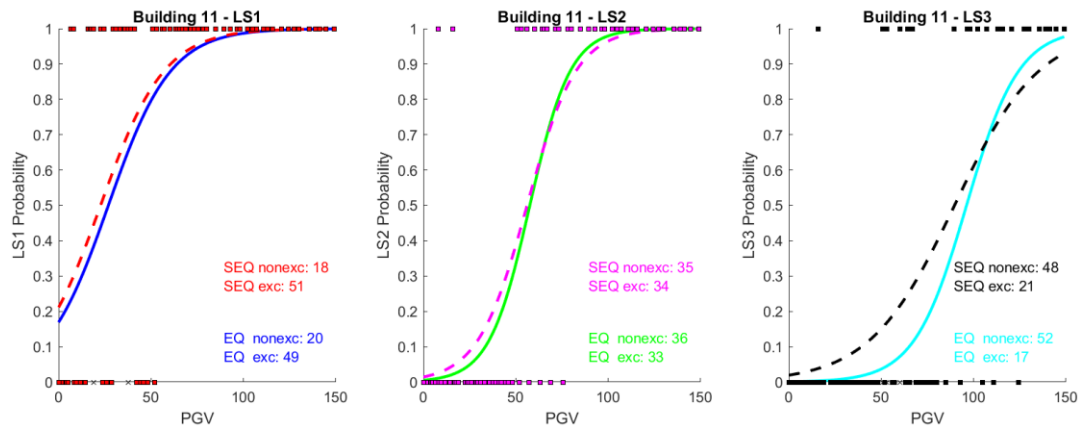


Figure 5.39. Fragility curves for building simulation no.11

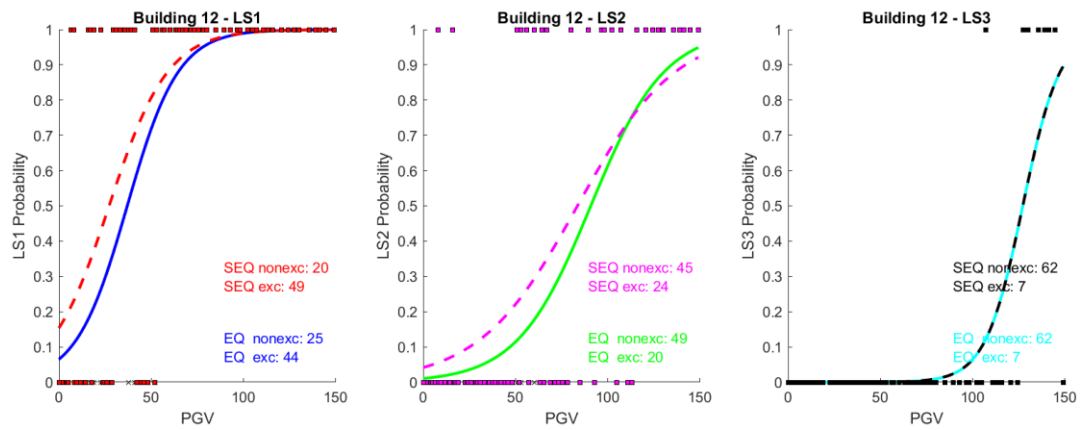


Figure 5.40. Fragility curves for building simulation no.12

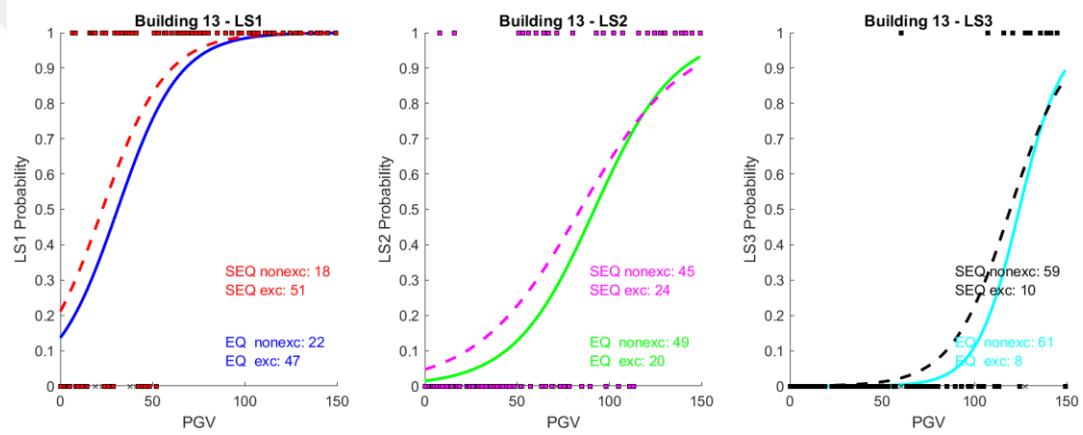


Figure 5.41. Fragility curves for building simulation no.13

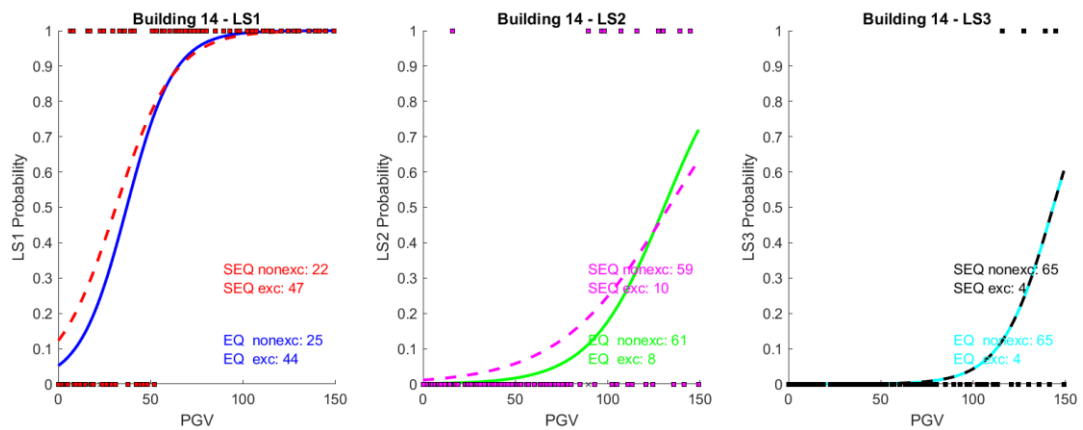


Figure 5.42. Fragility curves for building simulation no.14

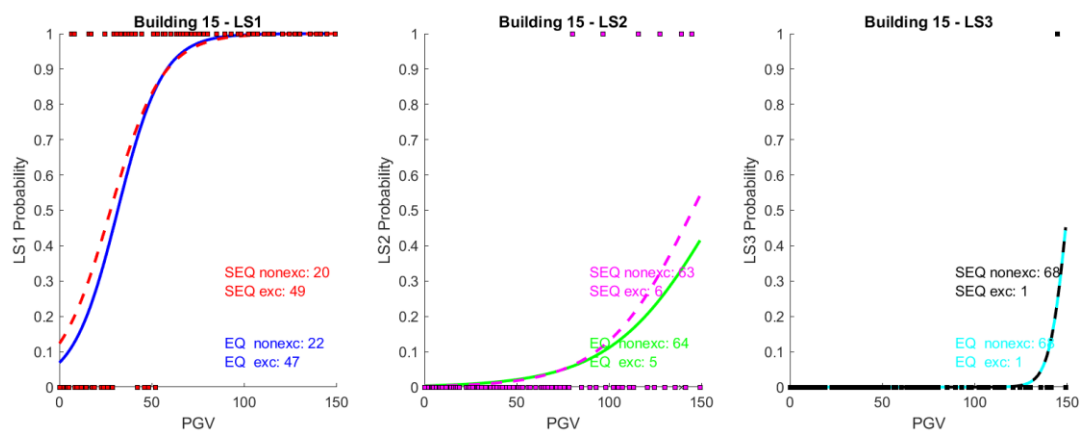


Figure 5.43. Fragility curves for building simulation no.15

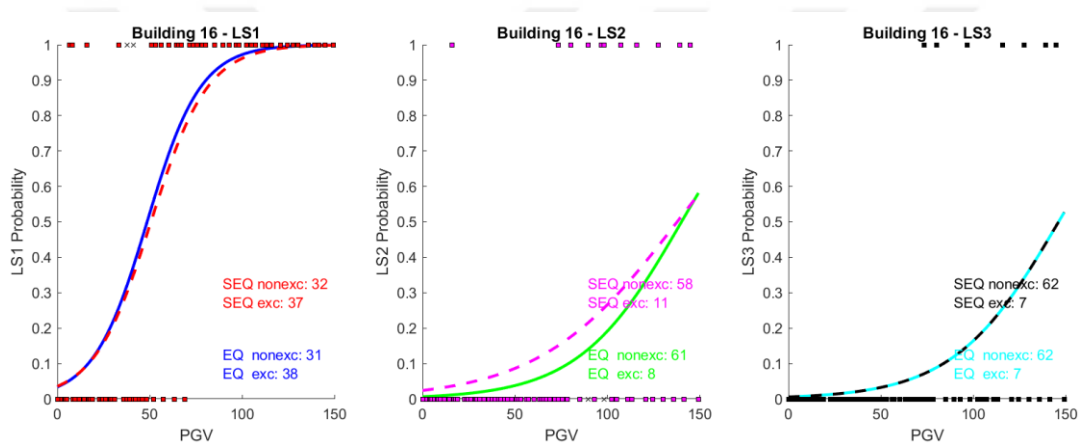


Figure 5.44. Fragility curves for building simulation no.16

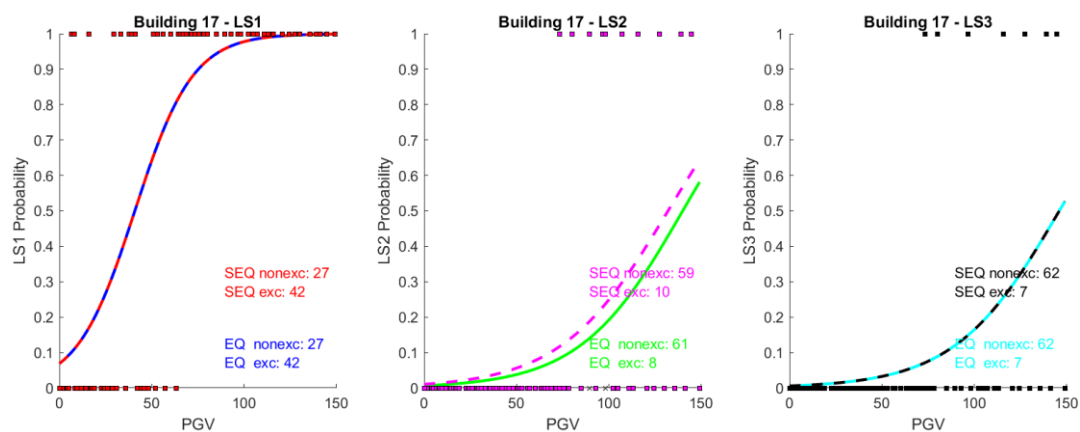


Figure 5.45. Fragility curves for building simulation no.17

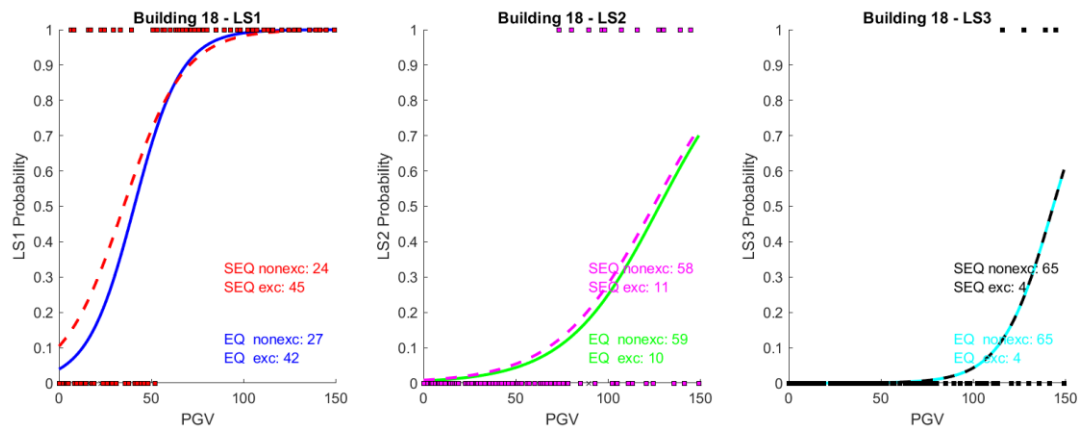


Figure 5.46. Fragility curves for building simulation no.18

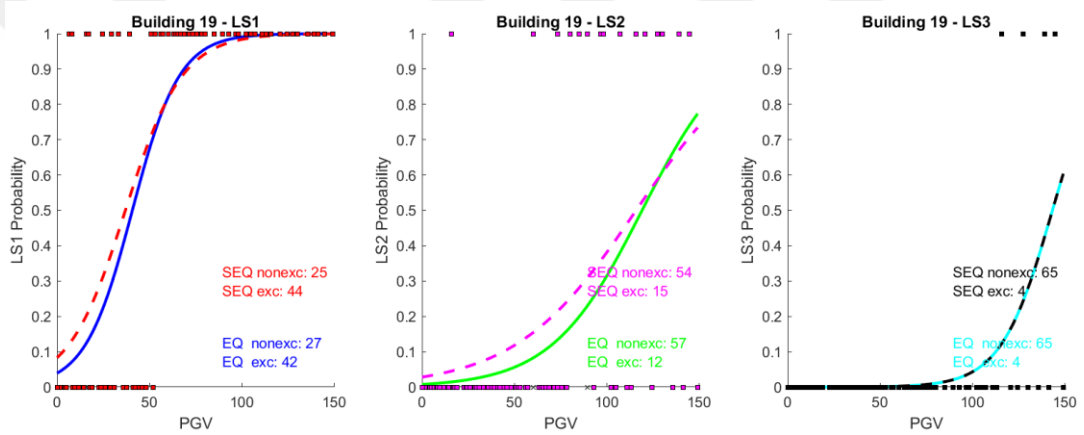


Figure 5.47. Fragility curves for building simulation no.19

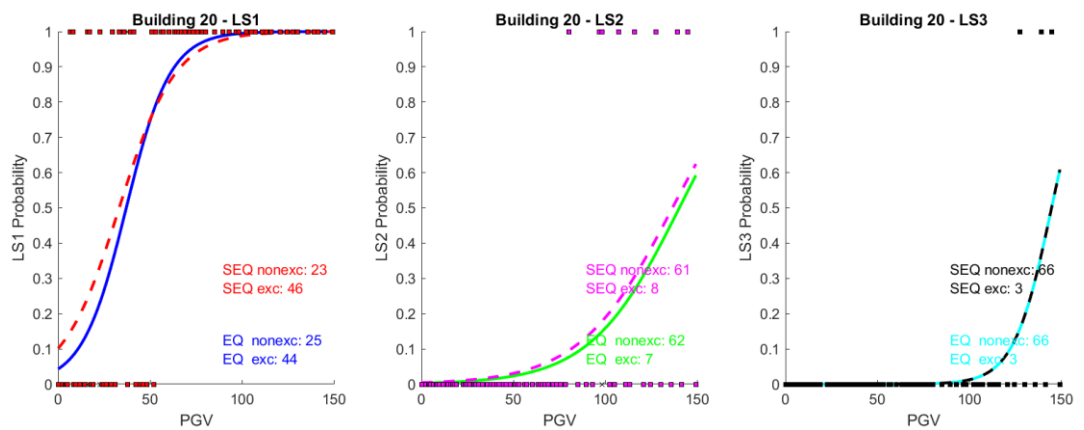


Figure 5.48. Fragility curves for building simulation no.20

A detailed comparison of individual building responses led to the following observations:

- **Impact of Soft Stories and Overhangs (Buildings 1-2 and 10-11):** Structures with soft stories and overhangs exhibited a significantly higher likelihood of exceeding the LS2 (moderate damage) and LS3 (extensive damage) thresholds. These imperfections make buildings more vulnerable to collapse, as they disrupt the uniform distribution of lateral forces and create weak points in the structural system. Additionally, the sequential effects of multiple earthquakes appeared to be less pronounced in buildings with these imperfections, possibly because such structures are more likely to fail early in the loading sequence rather than accumulate progressive damage.
- **Comparing Soft Stories and Overhangs (Buildings 15-16 and 19-20):** The presence of overhangs alone led to a slight increase in the probability of exceeding LS2 and LS3; however, soft-story configurations had a much more significant impact on structural performance. Soft stories introduce a discontinuity in stiffness and strength, making the lower levels particularly susceptible to severe damage or collapse. This finding aligns with field observations from the 2023 Kahramanmaraş earthquake, where buildings with soft stories exhibited substantial structural failures.
- **Influence of Seismic Code Evolution (Buildings 11-12 and Others):** A comparison between buildings designed under different seismic codes revealed that older design regulations correlate with a higher probability of exceeding all limit states. Structures, designed according to outdated codes, demonstrated greater vulnerability across all damage levels, highlighting the effectiveness of newer seismic provisions in improving building resilience. This underscores the critical role of modern seismic design standards in mitigating earthquake-induced damage and enhancing overall structural safety.

From these figures, it is observed that the sequential earthquakes cause additional damage when compared to the cases with the main event only. For Class C building models, damage evolution does not seem to be very significant. This trend is generally due to the fact that these buildings are very weak in strength and ductility, so they experience collapse (i.e. DS4) during the main event. In other words, there is no effect of multiple effects on the already failed class C building models. However, the trend is considerably different for class B building models which exhibit a variation in nearly all damage states in all building variations within the class. The evolution of damage is generally observed in LS2, but there are also shifts for the other limit states. When class A building models are considered, it is observed that shifts take place both for LS1 and LS2. This is due to the strong nature of these models in terms of strength and ductility. The aforementioned trends are not very different from the ones that had been encountered in Chapter 4 for SDOF analysis.

It should be mentioned that these trends are limited to structural models, analysis tools, and fragility curve techniques. First, it is important to state that the consideration of material and geometrical variability in structural models can increase the variability in response statistics, and in turn fragility analysis results. Second, the determination of limit states is very important in the quantification of fragility information and the calculation of damage state probabilities. Furthermore, the fragility curve generation technique can also play a vital role in obtaining different results during the comparison of seismic responses between main event and sequential event cases.

Finally, the buildings are grouped as class A, B, and C, and combined fragility curves are presented in Figures 5.49-5.51. These plots reveal that the differences in damage state probabilities are smoothed and yield lower values than the individual buildings. The shifts seem to take place in LS1 and LS2. For class B buildings, the sequential earthquakes cause a change in all limit states. This means, as in the case of the individual building cases, for group consideration, class B yields the most critical results for the impact of sequential earthquakes on RC frame buildings.

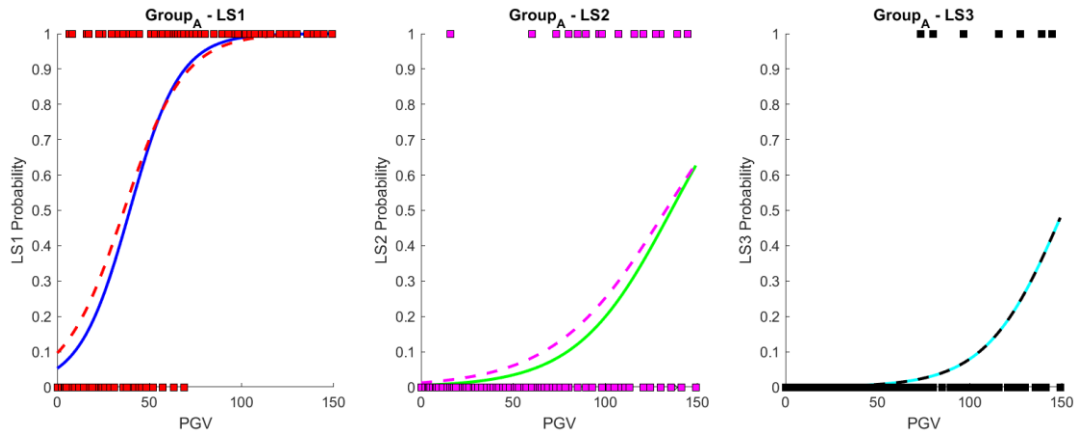


Figure 5.49. Combined fragility functions for class-A RC frame buildings (RF2A)

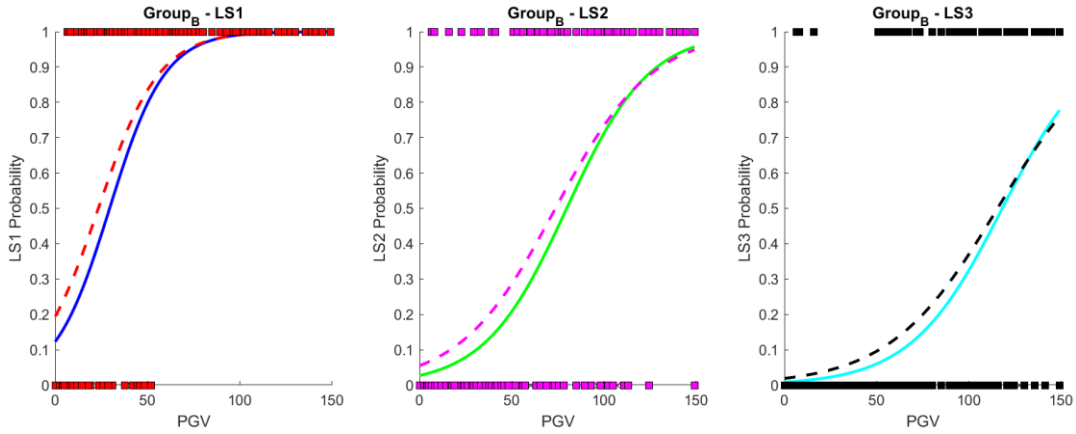


Figure 5.50. Combined fragility functions for class-B RC frame buildings (RF2B)

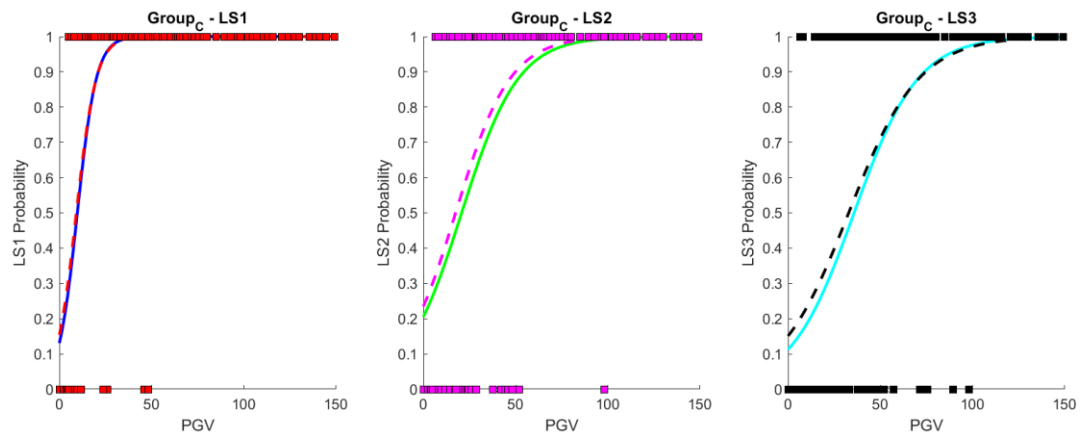


Figure 5.51. Combined fragility functions for class-C RC frame buildings (RF2C)

CHAPTER 6

CONCLUSION

6.1 Summary

This study examines the state-dependent seismic response and the related fragility properties of RC frame buildings subjected to sequential seismic events. Utilizing a comprehensive dataset derived from different ground motion databases, 105 ground motion records in 30 sequences are employed in this study for the analysis of RC frame buildings. The structural modeling is carried out at two different levels of idealization. The first level belongs to the single degree of freedom (SDOF) idealization of RC frame buildings whereas planar frame models have been utilized for the second level. In the first level, the SDOF parameters are obtained from the structural properties of actual buildings collected during field investigations in previous studies. The buildings are classified into different categories based on their height (1-3 stories and 4-8 stories) and design quality (A, B, C), with the parameters such as period (T), base shear coefficient (η), and ductility capacity (μ) being central to the analysis. The II-DAP program has been employed to conduct dynamic analyses on 126 buildings subjected to 105 sequence events, resulting in a substantial dataset of 13,230 individual analyses. The program facilitates the input of various SDOF system parameters, the selection of hysteretic models, and the specification of ground motion inputs, streamlining the analysis process through automated MATLAB scripts. Post-processing of results involves calculating the maximum displacement, input energy, and hysteretic energy for each analysis, enabling the determination of the building's damage state at the end of each seismic sequence. Results from these analyses are meticulously categorized into discrete damage states, ranging from "none/slight" to "collapse," based on predefined thresholds on the hysteresis model's skeleton curve. The study also introduces variability in structural

parameters through Latin Hypercube Sampling (LHS), generating a more realistic representation of building performance under seismic loads. The findings highlight the significant impact of sequential earthquake events on the damage evolution of RC buildings, with a particular emphasis on the most critical building classes.

The second level of idealization extends the analysis from SDOF models to MDOF systems to examine the damage evolution of mid-rise RC frames under sequential earthquakes. 20 2D RC frame models, designed based on three different versions of the Turkish Earthquake Code (TEC-75, TEC-98, and TEC-07), are analyzed to assess the impact of design variations, material properties, and structural irregularities such as soft stories and cantilevered floors. The study follows a structured workflow. First, structural modeling details are presented, including material properties and section dimensions. Then, nonlinear pushover analyses are conducted to determine key limit states. Next, time history analyses are performed for both mainshock-only and sequential earthquake cases. Finally, the results are analyzed in terms of damage state probabilities and fragility curves, generated using binary logistic regression.

6.2 Conclusions

Within the scope of the employed sequential ground motion database and the simplified numerical models, the following conclusions can be stated:

- There are many factors to assess the damaging potential of sequential earthquakes. It depends on the strong ground motion characteristics of individual events within the sequence as well as their order in the series. If the most destructive event is the first one in a series followed by events with less damaging potential, the first event behaves like a single earthquake so that the damage state attained for a building during this event does not change even when it is exposed to other events. This case affects superior (high-code) building classes such that if they do not conceive any damage in the

first one, they remain in the elastic range after the sequence whereas inferior (low-code) buildings generally fail during the first earthquake. Therefore, the effect of remaining events in the sequence becomes irrelevant. Moreover, moderate-code buildings have complex responses to these multiple events depending on their structural variabilities, but the results show that sequential earthquakes cause the evolution of damage in such buildings at least by 15%.

- As observed from the response statistics, in cases of sequential events in which the most destructive earthquake is not the first one in a series, damage state evolution becomes more critical for most of the building classes investigated. In three of the first four most damaging sequences shown in Table 4.4, the most destructive event is the second one in the series. This may be due to the fact that the buildings (especially the vulnerable ones) conceive some damage during the first event with damaging potential. They are affected by the second but most destructive event more seriously as the damage state shifts to higher levels.
- As a generalization from the SDOF analyses, most destructive sequential earthquakes can be defined as the ones with more than one event having $M > 6.0$ and magnitudes close to each other (i.e., $MBRI > 0.2$) in addition to having closer distances to the epicenter of the considered event ($Repi < 70$ km). For this case, the best example is the sequential earthquakes of the year 2023 in Southeast Türkiye that caused devastation in 11 populated cities, leaving behind 300,000 severely damaged or collapsed buildings.
- The findings from 2D RC frame analysis indicate that buildings designed according to the older codes or possessing structural deficiencies experience more significant damage accumulation. Well-designed structures in accordance with the recent seismic codes generally exhibit better resistance, but sequential earthquakes still contribute to notable damage progression. The fragility curves reveal that for PGV values exceeding 50 cm/s, the probability of exceeding the first damage threshold is high across all models.

Sequential earthquakes further amplify the likelihood of exceeding higher damage states, emphasizing the need for resilience-focused design approaches.

- The most critical building class in both SDOF and MODF analysis seems to be moderate-code mid-rise RC frame buildings, i.e. RF2B. The reason is that response statistics exhibit a higher variance for this building class. Hence, it is sensitive to inherent structural properties as well as ground motion characteristics. This may also explain observations from post-earthquake field surveys following the 2023 Türkiye earthquakes, where structurally similar RC frame buildings, located adjacent to each other, exhibited contrasting outcomes — one collapsing while the other remained standing.
- The results also seem to be sensitive to the fragility curve generation technique, especially the attainment of limit states. In this study, binary logistic regression is used which only checks the exceedance or non-exceedance of a limit state without quantifying the difference between seismic demand and capacity (i.e. limit state).
- As also confirmed by other studies (Amadio *et al.*, 2003; Hatzigeorgiou & Beskos, 2009; Di Sarno, 2013; Mirrashid & Naderpour, 2021), structural safety issues may arise in today's conventional seismic design practices that depend on the effect of a single event. This can be verified from the analysis results as there are considerable damage state shifts for well-designed (i.e. A-class) building classes when they are subjected to sequential earthquakes. Therefore, research in this area should urgently be conveyed to the earthquake engineering design and practice.

6.3 Further Recommendations

The following recommendations can be made for future research:

- The analysis can be extended to different sub-classes of RC structures like dual (frame+shear wall) buildings and shear wall (tunnel form) buildings in order to assess the significance of sequential earthquakes on these construction typologies. Apart from RC structures, similar studies can be performed for masonry buildings, which also constitute a significant percentage of building stock in many earthquake-prone regions.
- The material variability has been considered in SDOF analysis but it was not realized during MDOF analysis due to the need for extensive computational power. In the future, the building simulations can be enhanced by the introduction of variability in reinforcing steel and concrete. This additional study may provide very valuable information, especially for moderate-code mid-rise RC frame buildings as also mentioned in Section 5.2.
- Different fragility curve generation techniques other than the logistic regression can be employed to see the differences in damage progress under sequential earthquakes.



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APPENDICES

A. Ground Motion Database

Table A.1 Ground Motion Database

No	Sequence	Location	Station	Site Class	Event	Date	Time	M	MBRI	MMI	Dist (km)	Depth (km)	Fault Mech.
S-1	Great-Major	Sumatra Indonesia	PSK1	Stiff Soil	E-001	12.09.2007	11:10:00	8.4	1.00	VI	394	34	Thrust Faulting
					E-002	12.09.2007	23:49:00	7.9	0.32	VII	168	30	
					E-003	13.09.2007	03:55:00	7.0	0.04	VII	144	22	
S-2	Great-Major	Tohoku Japan	MYG015	Stiff Soil	E-004	9.03.2011	02:45:20	7.3	0.02	V	179	32	Thrust Faulting
					E-005	11.03.2011	05:46:24	9.0	1.00	VIII	137	29	
					E-006	11.03.2011	06:15:40	7.9	0.08	VII	208	42	
					E-007	11.03.2011	06:25:50	7.7	0.05	IV	327	19	
S-3	Major-Major	Marmara Türkiye	8101	Stiff Soil	E-008	17.08.1999	00:01:40	7.6	1.00	IX	101	17	Strike-Slip
					E-009	12.11.1999	16:57:20	7.1	0.32	IX	5	10	
S-4	Major-Strong	Valparaiso Chile	4407	Soft Rock	E-010	3.03.1985	22:47:07	7.8	1.00	VIII	129	33	Thrust Faulting
					E-011	3.03.1985	23:38:31	6.3	0.03	VI	93	33	
S-5	Major-Strong	Cape Mendocino USA	89156	Stiff Soil	E-012	25.04.1992	18:06:04	7.0	1.00	VIII	16	10	Reverse Faulting
					E-013	26.04.1992	07:41:00	6.6	0.40	VIII	35	20	
					E-014	26.04.1992	11:18:00	6.6	0.40	VII	49	22	
S-6	Major-Strong	California USA	12149	Soft Rock	E-015	28.06.1992	11:57:34	7.3	1.00	IX	23	7	Strike-Slip
					E-016	28.06.1992	15:05:36	6.4	0.13	VIII	40	13	
S-7	Major-Strong	Kumamoto Japan	KMMH16	Soft Rock	E-017	14.04.2016	12:26:23	6.2	0.16	VII	6	11	Strike-Slip
					E-018	15.04.2016	16:25:06	7.0	1.00	IX	7	12	
S-8	Major-Strong	Ridgecrest USA	CLC	Soft Rock	E-019	4.07.2019	17:33:49	6.4	0.20	VIII	18	11	Strike-Slip
					E-020	6.07.2019	03:19:53	7.1	1.00	IX	10	8	
S-9	Major-Strong	Southeast Türkiye	4616	Soft Rock	E-021	6.02.2023	01:17:00	7.7	1.00	IX	21	8.6	Strike-Slip
					E-022	6.02.2023	01:28:00	6.6	0.08	VIII	11	6.2	
					E-023	6.02.2023	10:24:00	7.6	0.79	IX	87	7	
					E-024	20.02.2023	17:04:00	6.4	0.05	IX	155	21.73	
S-10	Major-Strong	Southeast Türkiye	NAR	Stiff Soil	E-025	6.02.2023	01:17:00	7.7	1.00	IX	15	8.6	Strike-Slip
					E-026	6.02.2023	01:28:00	6.6	0.08	VIII	23	6.2	
					E-027	6.02.2023	10:24:00	7.6	0.79	IX	78	7	
					E-028	20.02.2023	17:04:00	6.4	0.05	IX	171	21.73	
S-11	Major-Strong	Southeast Türkiye	2718	Stiff Soil	E-029	6.02.2023	01:17:00	7.7	1.00	IX	48	8.6	Strike-Slip
					E-030	6.02.2023	01:28:00	6.6	0.08	VIII	42	6.2	
					E-031	6.02.2023	10:24:00	7.6	0.79	IX	132	7	
					E-032	20.02.2023	17:04:00	6.4	0.05	IX	110	21.73	
S-12	Major-Strong	Southeast Türkiye	4612	Stiff Soil	E-033	6.02.2023	01:17:00	7.7	1.00	IX	96	8.6	Strike-Slip
					E-034	6.02.2023	01:28:00	6.6	0.08	VIII	89	6.2	
					E-035	6.02.2023	10:24:00	7.6	0.79	IX	67	7	
					E-036	20.02.2023	17:04:00	6.4	0.05	IX	215	21.73	
S-13	Major-Strong	Southeast Türkiye	3135	Soft Rock	E-037	6.02.2023	01:17:00	7.7	1.00	IX	142	8.6	Strike-Slip
					E-038	6.02.2023	01:28:00	6.6	0.08	VIII	136	6.2	
					E-039	6.02.2023	10:24:00	7.6	0.79	IX	222	7	
					E-040	20.02.2023	17:04:00	6.4	0.05	IX	36	21.73	
S-14	Major-Strong-Moderate	Chi-Chi Taiwan	TCU068	Soft Rock	E-041	20.09.1999	17:47:16	7.6	1.00	IX	3	7	Reverse Oblique
					E-042	20.09.1999	17:57:00	5.9	0.02	V	41	8	
					E-043	20.09.1999	18:03:00	6.2	0.04	VI	48	8	
					E-044	20.09.1999	21:46:00	6.2	0.04	VI	68	18	
					E-045	22.09.1999	00:14:00	6.2	0.04	VII	52	10	
					E-046	25.09.1999	23:52:00	6.3	0.05	VII	36	16	
S-15	Major-Strong-Moderate	Fiordland New Zealand	MANS	Rock	E-047	21.08.2003	12:12:49	7.1	1.00	VII	51	24	Reverse Faulting
					E-048	21.08.2003	12:45:40	5.9	0.06	-	45	25	
					E-049	21.08.2003	14:12:27	6.1	0.10	V	40	24	

Table A.2 (cont'd) Ground Motion Database

S-16	Major-Strong-Moderate	Van Türkiye	1302	Soft Rock	E-050	23.10.2011	10:41:00	7.3	1.00	IX	115.7	19.02	Thrust Faulting
					E-051	23.10.2011	10:56:00	5.7	0.03	VII	107.2	18.8	
					E-052	23.10.2011	11:32:00	6.2	0.08	VII	115.2	17.3	
					E-053	23.10.2011	20:45:00	5.9	0.04	VII	84.4	13.96	
					E-054	25.10.2011	14:55:00	5.3	0.01	VII	132.7	18.28	
S-17	Major-Moderate	Kocaeli Türkiye	5401	Soft Rock	E-055	9.11.2011	19:23:00	5.6	0.02	VII	103	6.09	Strike-Slip
					E-056	17.08.1999	00:01:40	7.6	1.00	IX	35.96	15.0	
					E-057	31.08.1999	08:10:52	5.1	0.00	VI	37.65	17.7	
					E-058	13.09.1999	11:55:29	5.8	0.02	VIII	25.47	19.6	
S-18	Major-Moderate	Duzce Türkiye	1401	Stiff Soil	E-059	12.11.1999	16:57:20	7.1	1.00	IX	11.21	10.0	Strike-Slip
					E-060	12.11.1999	17:18:00	5.6	0.03	VII	68.56	10.0	
					E-061	12.11.1999	17:22:00	5.3	0.02	-	65.93	10.0	
S-19	Major-Moderate	Canterbury New Zealand	CACS	Stiff Soil	E-062	3.09.2010	16:35:00	7.2	1.00	VIII	30	11	Strike-Slip
					E-063	3.09.2010	16:52:00	5.5	0.02	-	16	7	
					E-064	3.09.2010	17:18:00	5	0.01	-	30	6	
S-20	Strong-Strong	Corinth Greece	KORA	Soft Rock	E-065	24.02.1981	20:53:37	6.6	1.00	IX	32	17.6	Normal Faulting
					E-066	25.02.1981	02:35:53	6.3	0.50	IX	30.4	30	
S-21	Strong-Strong	Romania	VRI	Soft Rock	E-067	30.05.1990	10:40:06	6.9	1.00	VIII	5.5	89	Reverse
					E-068	31.05.1990	00:17:48	6.6	0.50	V	6.9	90.1	
S-22	Strong-Moderate	Chalfant Valley USA	54171	Stiff Soil	E-069	20.07.1986	14:29:46	5.9	0.32	VIII	24.7	6.7	Strike-Slip
					E-070	21.07.1986	14:42:26	6.4	1.00	VIII	23.7	10	
					E-071	21.07.1986	14:51:09	5.6	0.16	VI	23.4	5	
					E-072	31.07.1986	07:22:40	5.8	0.25	VII	15.5	9.2	
S-23	Strong-Moderate	Umbro Italy	CLF	Soft Soil	E-073	26.09.1997	00:33:11	5.7	0.50	VIII	2.8	5.7	Normal Faulting
					E-074	26.09.1997	09:40:24	6.0	1.00	VIII	4.8	5.7	
					E-075	3.10.1997	08:55:20	5.2	0.16	VI	5.9	4.8	
					E-076	6.10.1997	23:24:51	5.4	0.25	VII	6	5.5	
					E-077	12.10.1997	11:08:35	5.2	0.16	-	12.8	4.8	
					E-078	14.10.1997	15:23:09	5.6	0.40	VII	12	6	
S-24	Strong-Moderate	L'Aquila Italy	AQK	Soft Rock	E-079	6.04.2009	01:32:40	6.1	1.00	VIII	1.8	8.3	Normal Faulting
					E-080	6.04.2009	02:37:04	5.1	0.10	-	6.2	8.7	
					E-081	7.04.2009	17:47:37	5.5	0.25	VI	8.4	17.1	
					E-082	9.04.2009	00:52:09	5.4	0.20	-	16.5	11	
					E-083	9.04.2009	19:38:16	5.2	0.13	-	18.2	9.3	
S-25	Strong-Moderate	Christchurch New Zealand	CACS	Stiff Soil	E-084	21.02.2011	23:51:00	6.2	1.00	IX	16	5	Oblique
					E-085	22.02.2011	00:04:00	5.5	0.20	VII	16	6	
					E-086	22.02.2011	01:50:00	5.6	0.25	VII	15	7	
					E-087	13.06.2011	01:01:00	5.3	0.13	-	21	9	
					E-088	13.06.2011	02:20:00	6	0.63	VII	19	7	
S-26	Strong-Moderate	Emilia Italy	MRN	Stiff Soil	E-089	20.05.2012	02:03:50	6.1	1.00	VI	16.1	9.5	Thrust Faulting
					E-090	20.05.2012	03:02:47	5.1	0.10	-	7.4	9.1	
					E-091	29.05.2012	07:00:02	6	0.79	VII	4.1	8	
					E-092	29.05.2012	10:55:56	5.5	0.25	VII	6.7	4.3	
					E-093	29.05.2012	11:00:22	5.5	0.25	-	6.9	7.2	
S-27	Strong-Moderate	Central Italy	AMT	Soft Rock	E-094	24.08.2016	01:36:32	6	0.25	VII	8.5	8.1	Normal Faulting
					E-095	24.08.2016	02:33:29	5.3	0.05	VI	20.9	8	
					E-096	26.10.2016	17:10:36	5.5	0.08	VII	30	8.1	
					E-097	26.10.2016	19:18:06	5.9	0.20	VII	2.5	7.5	
					E-098	30.10.2016	06:40:18	6.6	1.00	VIII	7.7	9.2	
S-28	Moderate-Moderate	Lazio Italy	ATN	Rock	E-099	7.05.1984	17:49:43	5.9	1.00	VII	10.1	20.5	Normal Faulting
					E-100	11.05.1984	10:41:48	5.5	0.40	VI	19.2	12.1	
S-29	Moderate-Moderate	Christchurch New Zealand	CACS	Stiff Soil	E-101	23.12.2011	00:58:00	5.8	0.79	VII	22	10	Oblique
					E-102	23.12.2011	01:06:00	5.4	0.32	-	23	7	
					E-103	23.12.2011	02:18:00	5.9	1.00	VII	18	7	
S-30	Moderate-Moderate	Khoy Iran	6508	Soft Rock	E-104	23.02.2020	05:56:00	5.9	1.00	VII	50.89	14.9	Strike-Slip
					E-105	23.02.2020	16:00:00	5.9	1.00	VII	51.16	8.1	

B. Response Spectra of the Events in the Database

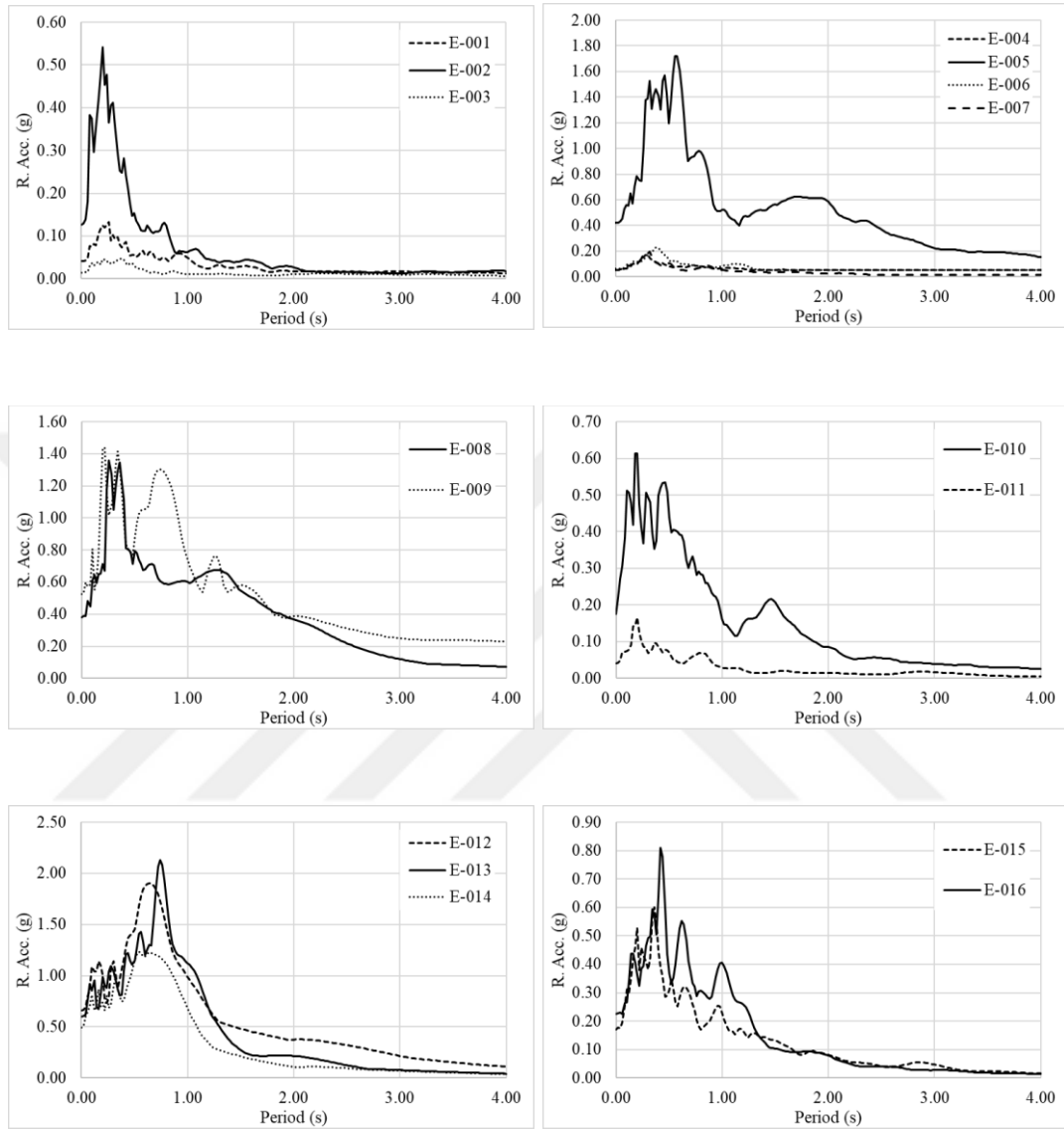


Figure B.1. Response Spectra of Sequences S1-6

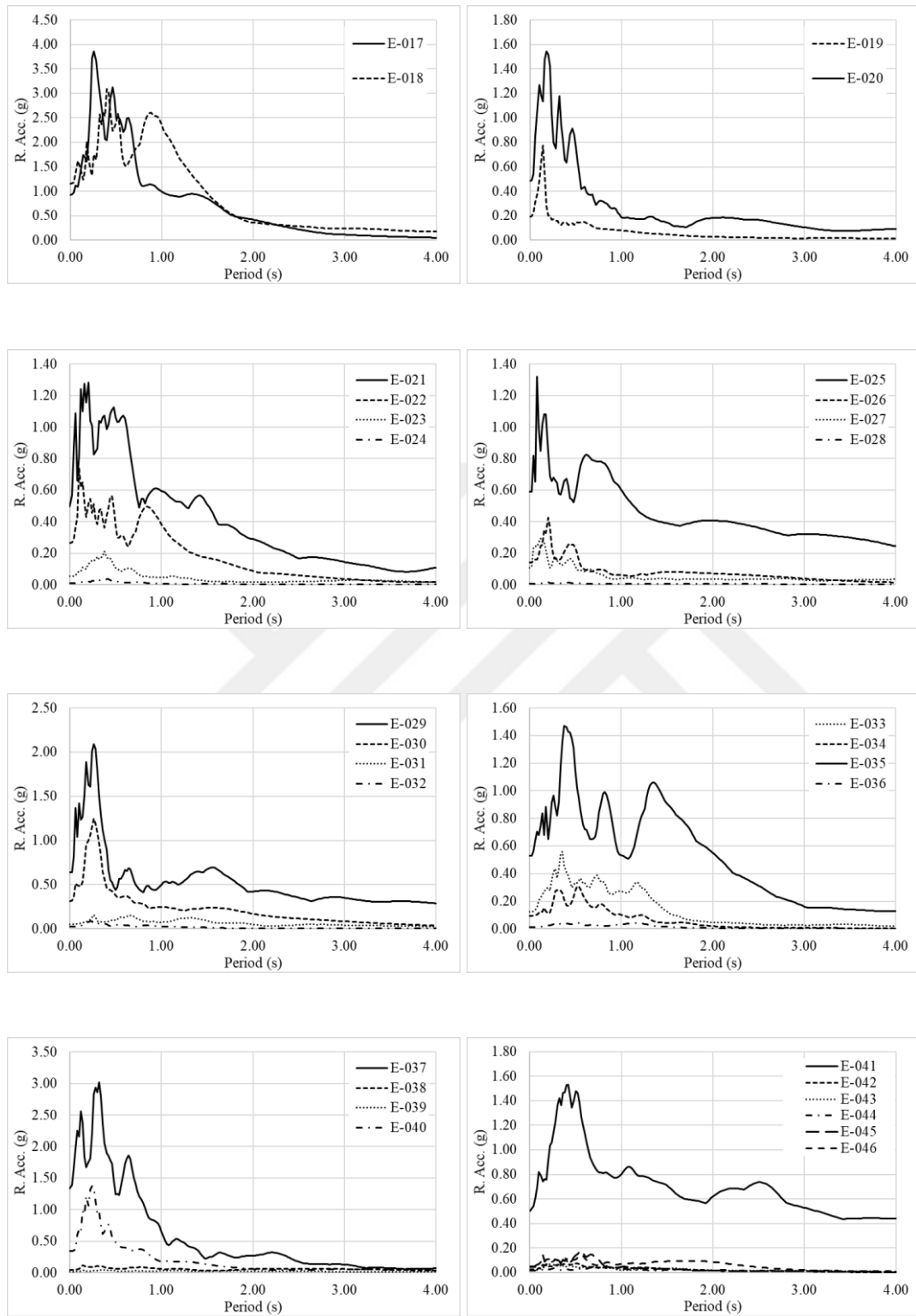


Figure B.2. Response Spectra of Sequences S7-14

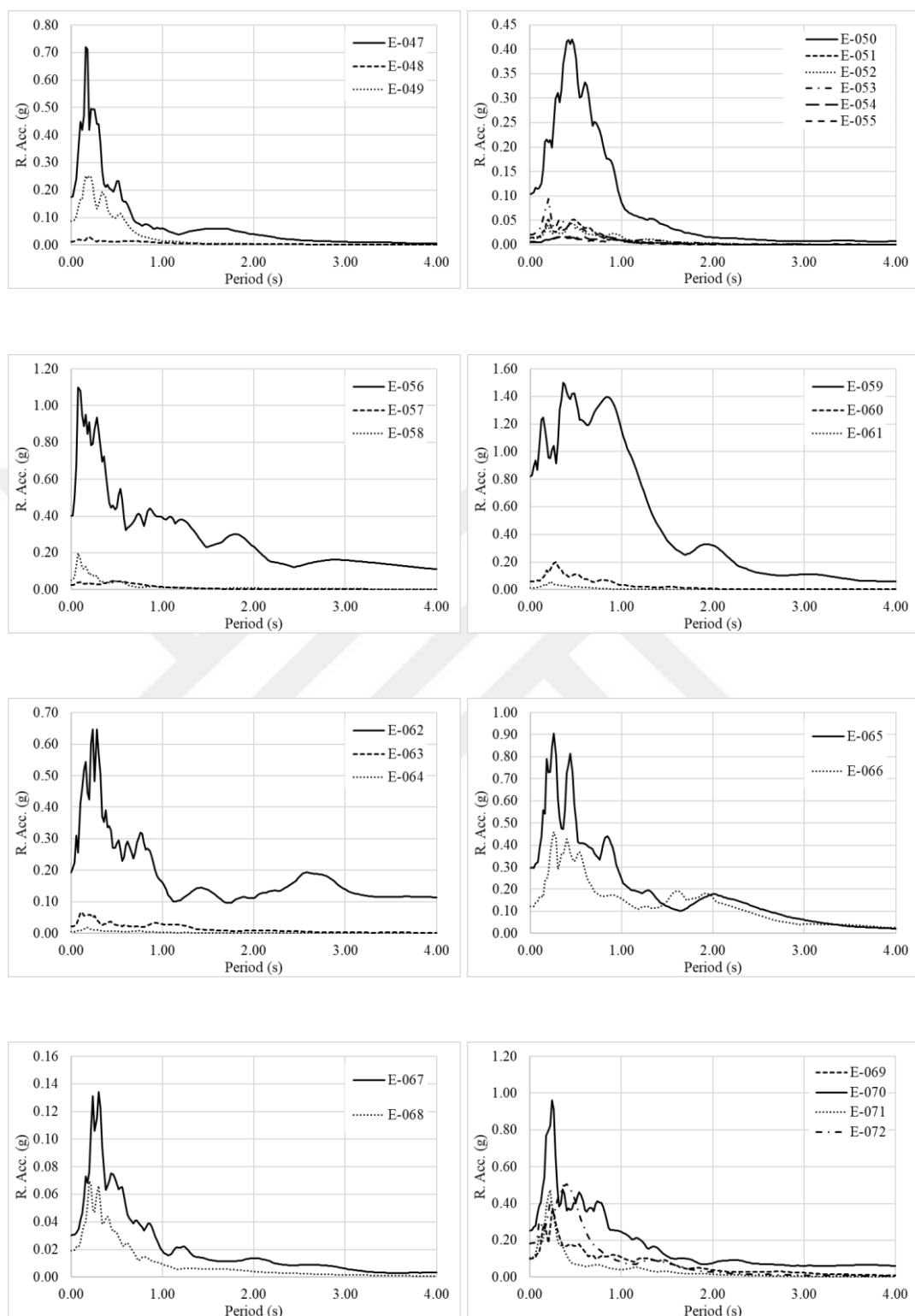


Figure B.3. Response Spectra of Sequences S15-22

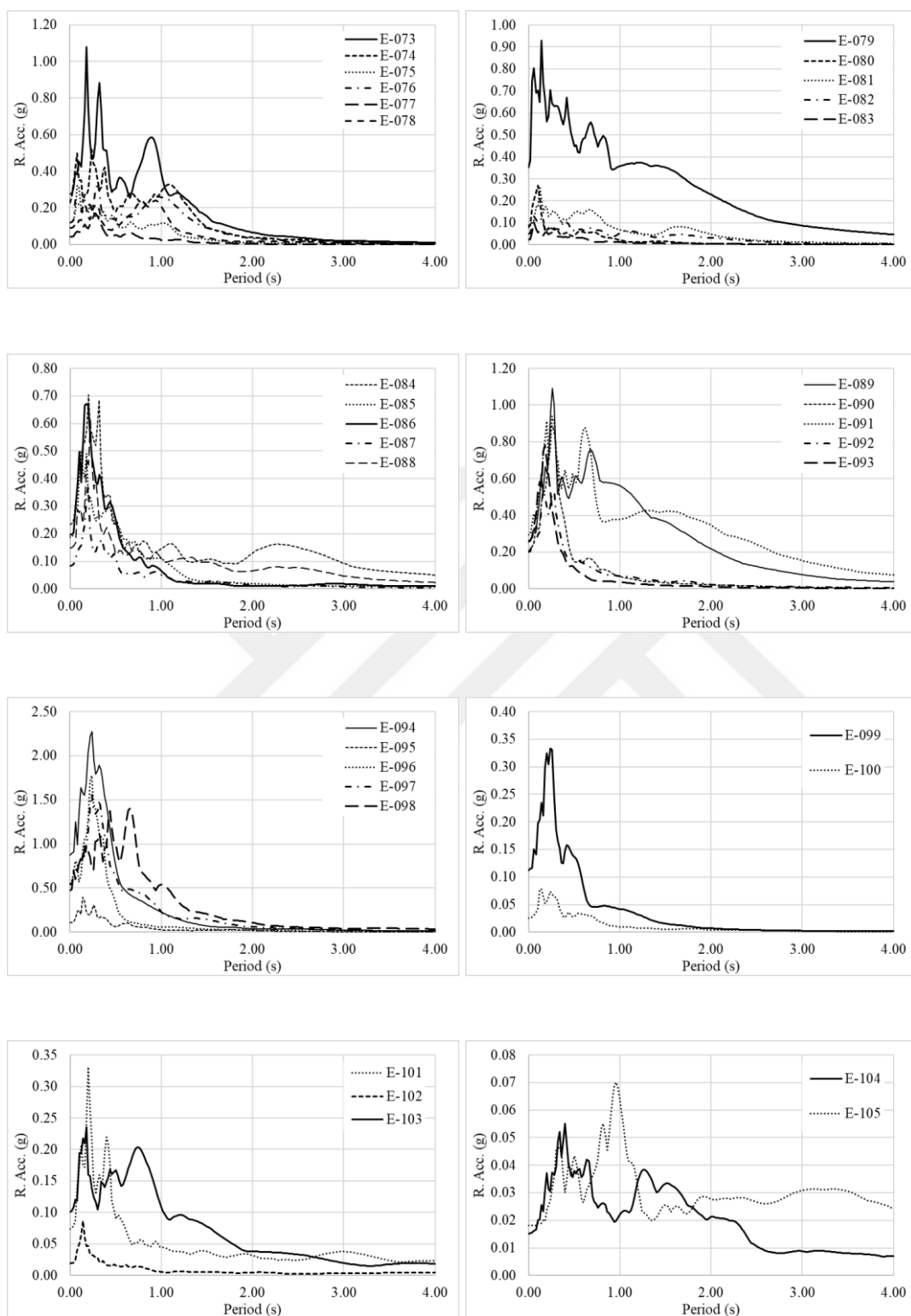


Figure B.4. Response Spectra of Sequences S23-30

C. Intensity Measures of the Events in the Database

Table C.1 Intensity Measures of the Events in the Database

No	Event	PGA (g)	PGV (m/s)	V/A (s)	AI (m/s)	a_{rms} (10^{-1} g)	CAV (g.sec)	t_{eff} (sec)	I_H ($g \cdot s^2$)	I_E (m/s)
S-1	E-001	0.04	0.04	0.105	0.07	0.06	0.447	44.9	0.014	0.152
	E-002	0.13	0.08	0.061	0.75	0.17	1.547	50.3	0.024	0.224
	E-003	0.02	0.02	0.120	0.01	0.02	0.115	32.2	0.006	0.059
S-2	E-004	0.05	0.05	0.093	0.10	0.05	0.571	44.5	0.013	0.051
	E-005	0.42	0.08	0.019	14.20	0.55	9.420	115.8	0.295	1.475
	E-006	0.06	0.04	0.066	0.16	0.06	1.007	201.0	0.018	0.074
S-3	E-007	0.05	0.04	0.067	0.16	0.06	0.979	112.7	0.014	0.053
	E-008	0.36	0.56	0.159	1.39	0.58	8.263	11.1	0.243	0.838
	E-009	0.51	0.80	0.160	3.02	0.87	13.809	10.9	0.270	1.901
S-4	E-010	0.18	0.15	0.085	1.13	0.30	1.534	39.0	0.071	0.295
	E-011	0.04	0.03	0.068	0.03	0.09	0.185	18.9	0.011	0.067
S-5	E-012	0.66	0.90	0.138	3.83	0.64	1.522	16.1	0.293	0.873
	E-013	0.60	0.61	0.104	3.16	0.72	1.035	4.4	0.226	0.324
	E-014	0.49	0.43	0.089	1.44	0.48	0.683	4.1	0.152	0.312
S-6	E-015	0.17	0.19	0.114	0.72	0.24	1.180	34.1	0.061	0.217
	E-016	0.23	0.19	0.086	0.62	0.26	0.711	10.0	0.074	0.150
S-7	E-017	0.94	0.93	0.100	12.58	0.52	2.912	9.4	0.364	0.525
	E-018	1.18	1.16	0.100	11.95	0.51	3.624	10.9	0.444	1.431
S-8	E-019	0.19	0.10	0.055	0.24	0.07	0.382	11.5	0.025	0.110
	E-020	0.48	0.42	0.089	3.03	0.26	1.489	15.0	0.105	0.719
S-9	E-021	0.51	0.84	0.167	3.77	0.48	2.500	41.0	0.214	0.949
	E-022	0.26	0.22	0.085	0.44	0.17	0.520	8.2	0.103	0.154
	E-023	0.05	0.13	0.244	0.09	0.07	0.430	35.7	0.019	0.175
	E-024	0.01	0.01	0.101	0.00	0.01	0.070	32.0	0.004	0.017
S-10	E-025	0.59	0.93	0.161	3.33	0.45	2.300	42.6	0.214	2.042
	E-026	0.14	0.10	0.076	0.10	0.10	0.330	16.4	0.041	0.149
	E-027	0.11	0.25	0.230	0.10	0.08	0.380	32.3	0.020	0.216
	E-028	0.01	0.01	0.182	0.00	0.01	0.040	47.7	0.003	0.020
S-11	E-029	0.66	1.13	0.175	4.25	0.61	2.240	20.8	0.254	2.343
	E-030	0.31	0.33	0.108	0.62	0.20	0.520	5.2	0.115	0.325
	E-031	0.05	0.14	0.281	0.06	0.06	0.350	30.8	0.034	0.287
	E-032	0.03	0.02	0.067	0.01	0.02	0.100	22.7	0.008	0.031
S-12	E-033	0.12	0.14	0.118	1.23	0.27	1.850	50.7	0.074	0.262
	E-034	0.09	0.07	0.078	0.14	0.09	0.490	28.4	0.034	0.065
	E-035	0.53	0.73	0.139	3.19	0.44	1.950	25.9	0.336	1.035
	E-036	0.01	0.02	0.136	0.01	0.02	0.120	41.0	0.008	0.016
S-13	E-037	1.40	0.65	0.048	6.77	0.59	2.870	22.6	0.289	0.633
	E-038	0.05	0.09	0.192	0.04	0.05	0.290	39.9	0.028	0.324
	E-039	0.02	0.09	0.576	0.02	0.03	0.290	87.7	0.014	0.214
	E-040	0.35	0.25	0.073	1.31	0.28	0.980	8.4	0.092	0.353

Table C.1 (cont'd) Intensity Measures of the Events in the Database

S-14	E-041	0.50	2.13	0.431	3.25	0.38	1.856	12.5	0.362	2.846
	E-042	0.03	0.02	0.077	0.01	0.04	0.101	23.5	0.013	0.030
	E-043	0.02	0.03	0.159	0.01	0.03	0.158	59.0	0.013	0.086
	E-044	0.01	0.02	0.168	0.01	0.03	0.099	28.5	0.011	0.051
	E-045	0.05	0.04	0.086	0.04	0.06	0.211	22.2	0.015	0.039
	E-046	0.04	0.09	0.197	0.04	0.07	0.221	21.6	0.039	0.075
S-15	E-047	0.17	0.10	0.056	0.45	0.12	0.730	14.0	0.029	0.076
	E-048	0.01	0.01	0.067	0.00	0.01	0.034	22.1	0.003	0.006
	E-049	0.09	0.03	0.036	0.04	0.06	0.139	5.4	0.008	0.016
S-16	E-050	0.10	0.09	0.084	0.15	0.07	0.419	15.8	0.030	0.119
	E-051	0.01	0.01	0.075	0.00	0.01	0.047	9.3	0.003	0.006
	E-052	0.01	0.01	0.087	0.00	0.01	0.041	12.7	0.004	0.008
	E-053	0.02	0.01	0.052	0.01	0.02	0.073	11.4	0.004	0.010
	E-054	0.00	0.01	0.119	0.00	0.01	0.028	25.2	0.002	0.004
	E-055	0.01	0.00	0.067	0.00	0.01	0.028	34.4	0.001	0.008
S-17	E-056	0.40	0.57	0.147	1.79	0.35	1.296	15.4	0.134	0.788
	E-057	0.02	0.02	0.062	0.00	0.01	0.034	9.5	0.004	0.007
	E-058	0.05	0.02	0.048	0.02	0.04	0.124	10.5	0.005	0.019
S-18	E-059	0.82	0.67	0.083	2.51	0.54	1.211	9.0	0.246	0.532
	E-060	0.06	0.04	0.067	0.02	0.08	0.105	8.6	0.011	0.020
	E-061	0.01	0.01	0.060	0.00	0.03	0.020	4.6	0.002	0.003
S-19	E-062	0.19	0.46	0.245	0.86	0.12	1.546	36.4	0.076	0.950
	E-063	0.02	0.02	0.068	0.01	0.02	0.094	23.3	0.007	0.019
	E-064	0.00	0.00	0.058	0.00	0.01	0.024	38.2	0.001	0.003
S-20	E-065	0.30	0.25	0.085	0.84	0.33	0.830	13.8	0.097	0.209
	E-066	0.12	0.14	0.120	0.23	0.19	0.417	15.0	0.075	0.218
S-21	E-067	0.03	0.02	0.079	0.02	0.04	0.100	12.2	0.009	0.026
	E-068	0.02	0.01	0.069	0.00	0.03	0.042	13.0	0.004	0.007
S-22	E-069	0.13	0.10	0.078	0.09	0.12	0.304	21.1	0.033	0.102
	E-070	0.25	0.18	0.072	0.50	0.29	0.604	12.8	0.075	0.500
	E-071	0.11	0.05	0.043	0.07	0.11	0.199	13.9	0.016	0.027
	E-072	0.18	0.12	0.065	0.16	0.16	0.285	14.2	0.036	0.058
S-23	E-073	0.28	0.23	0.086	0.61	0.30	0.519	5.5	0.082	0.131
	E-074	0.23	0.17	0.077	0.36	0.22	0.513	11.4	0.056	0.106
	E-075	0.12	0.09	0.074	0.06	0.14	0.151	6.1	0.018	0.030
	E-076	0.12	0.11	0.096	0.11	0.14	0.239	8.5	0.044	0.064
	E-077	0.04	0.02	0.052	0.01	0.07	0.069	7.4	0.007	0.012
	E-078	0.09	0.07	0.080	0.08	0.11	0.234	8.3	0.028	0.055
S-24	E-079	0.35	0.36	0.103	1.22	0.28	0.999	11.3	0.145	0.312
	E-080	0.08	0.02	0.031	0.03	0.04	0.131	5.7	0.007	0.012
	E-081	0.09	0.07	0.079	0.06	0.06	0.217	13.6	0.030	0.056
	E-082	0.05	0.04	0.085	0.02	0.04	0.151	14.6	0.019	0.038
	E-083	0.02	0.01	0.041	0.01	0.02	0.073	11.2	0.004	0.010
S-25	E-084	0.23	0.24	0.104	0.48	0.15	0.656	11.2	0.071	0.336
	E-085	0.19	0.08	0.046	0.13	0.08	0.274	7.3	0.025	0.065
	E-086	0.20	0.10	0.050	0.23	0.09	0.371	6.7	0.020	0.073
	E-087	0.08	0.05	0.063	0.06	0.08	0.196	9.6	0.014	0.032
	E-088	0.15	0.11	0.076	0.18	0.09	0.439	13.8	0.046	0.219

Table C.1 (cont'd) Intensity Measures of the Events in the Database

S-26	E-089	0.26	0.46	0.179	0.87	0.21	0.647	6.0	0.161	0.235
	E-090	0.21	0.11	0.053	0.21	0.13	0.221	2.7	0.027	0.037
	E-091	0.29	0.58	0.199	1.33	0.35	0.847	7.0	0.189	0.512
	E-092	0.20	0.12	0.058	0.17	0.10	0.286	6.5	0.025	0.042
	E-093	0.26	0.13	0.051	0.15	0.11	0.187	2.8	0.015	0.027
S-27	E-094	0.87	0.44	0.051	1.89	0.66	0.648	3.7	0.076	0.132
	E-095	0.11	0.05	0.048	0.06	0.08	0.162	5.0	0.014	0.027
	E-096	0.09	0.04	0.043	0.04	0.09	0.114	3.4	0.008	0.019
	E-097	0.09	0.05	0.060	0.06	0.09	0.187	9.6	0.019	0.045
	E-098	0.53	0.38	0.073	1.56	0.50	0.788	5.4	0.092	0.367
S-28	E-099	0.11	0.04	0.034	0.05	0.11	0.163	10.0	0.012	0.021
	E-100	0.03	0.01	0.048	0.01	0.04	0.055	11.4	0.004	0.009
S-29	E-101	0.07	0.07	0.101	0.07	0.06	0.304	27.3	0.020	0.221
	E-102	0.02	0.01	0.056	0.00	0.02	0.081	29.9	0.003	0.051
	E-103	0.10	0.10	0.099	0.09	0.08	0.287	12.6	0.035	0.182
S-30	E-104	0.02	0.02	0.164	0.01	0.03	0.105	24.1	0.012	0.063
	E-105	0.02	0.04	0.249	0.01	0.03	0.136	23.9	0.015	0.190



D. Building Samples

Table D.1 Building Samples Properties used in SDOF Analysis

No			Mass (N)	T (s)	η	μ	α_s (%)	λ	α_c (%)	γ
1	RF1A	1	5400000	0.257	0.340	11.800	4	0.2	-20	800
2		2	5400000	0.090	0.368	9.047	4	0.2	-20	800
3		3	5400000	0.191	0.320	7.179	4	0.2	-20	800
4		4	5400000	0.408	0.482	14.852	4	0.2	-20	800
5		5	5400000	0.368	0.497	10.528	4	0.2	-20	800
6		6	5400000	0.474	0.464	12.585	4	0.2	-20	800
7		7	5400000	0.261	0.272	8.267	4	0.2	-20	800
8		8	5400000	0.078	0.405	5.728	4	0.2	-20	800
9		9	5400000	0.350	0.450	10.778	4	0.2	-20	800
10		10	5400000	0.535	0.421	9.691	4	0.2	-20	800
11		11	5400000	0.448	0.557	11.557	4	0.2	-20	800
12		12	5400000	0.628	0.515	8.169	4	0.2	-20	800
13		13	5400000	0.220	0.395	7.553	4	0.2	-20	800
14		14	5400000	0.306	0.350	5.954	4	0.2	-20	800
15		15	5400000	0.510	0.388	6.610	4	0.2	-20	800
16		16	5400000	0.592	0.435	4.920	4	0.2	-20	800
17		17	5400000	0.394	0.307	10.101	4	0.2	-20	800
18		18	5400000	0.725	0.374	8.751	4	0.2	-20	800
19		19	5400000	0.488	0.242	0.269	4	0.2	-20	800
20		20	5400000	0.321	0.414	13.277	4	0.2	-20	800
21	RF1B	1	5400000	0.390	0.193	8.924	4	0.2	-25	400
22		2	5400000	0.413	0.184	9.753	4	0.2	-25	400
23		3	5400000	0.514	0.138	3.216	4	0.2	-25	400
24		4	5400000	0.261	0.277	6.828	4	0.2	-25	400
25		5	5400000	0.179	0.430	7.929	4	0.2	-25	400
26		6	5400000	0.196	0.217	5.858	4	0.2	-25	400
27		7	5400000	0.464	0.266	11.987	4	0.2	-25	400
28		8	5400000	0.363	0.328	7.151	4	0.2	-25	400
29		9	5400000	0.286	0.332	9.173	4	0.2	-25	400
30		10	5400000	0.125	0.385	5.549	4	0.2	-25	400
31		11	5400000	0.435	0.500	6.227	4	0.2	-25	400
32		12	5400000	0.826	0.355	7.686	4	0.2	-25	400
33		13	5400000	0.612	0.475	4.256	4	0.2	-25	400
34		14	5400000	0.534	0.413	6.502	4	0.2	-25	400
35		15	5400000	0.610	0.234	4.738	4	0.2	-25	400
36		16	5400000	0.005	0.111	8.279	4	0.2	-25	400
37		17	5400000	0.475	0.309	7.377	4	0.2	-25	400
38		18	5400000	0.232	0.360	8.618	4	0.2	-25	400

Table D.1 (cont'd) Building Samples Properties used in SDOF Analysis

39	RF1C	19	5400000	0.316	0.298	6.539	4	0.2	-25	400
40		20	5400000	0.348	0.255	10.056	4	0.2	-25	400
41		1	5400000	0.483	0.177	3.566	4	0.2	-30	200
42		2	5400000	0.322	0.180	3.887	4	0.2	-30	200
43		3	5400000	0.426	0.386	3.986	4	0.2	-30	200
44		4	5400000	0.352	0.121	5.307	4	0.2	-30	200
45		5	5400000	0.576	0.324	4.687	4	0.2	-30	200
46		6	5400000	0.388	0.218	6.458	4	0.2	-30	200
47		7	5400000	0.565	0.142	5.565	4	0.2	-30	200
48		8	5400000	1.029	0.259	5.953	4	0.2	-30	200
49		9	5400000	0.072	0.166	3.217	4	0.2	-30	200
50		10	5400000	0.300	0.201	4.240	4	0.2	-30	200
51		11	5400000	0.150	0.236	6.152	4	0.2	-30	200
52		12	5400000	0.505	0.292	5.855	4	0.2	-30	200
53		13	5400000	0.117	0.306	4.953	4	0.2	-30	200
54		14	5400000	0.268	0.223	1.958	4	0.2	-30	200
55		15	5400000	0.215	0.267	4.854	4	0.2	-30	200
56		16	5400000	0.254	0.275	5.127	4	0.2	-30	200
57		17	5400000	0.623	0.215	7.834	4	0.2	-30	200
58		18	5400000	0.409	0.245	2.902	4	0.2	-30	200
59		19	5400000	0.469	0.251	4.336	4	0.2	-30	200
60		20	5400000	0.370	0.190	6.907	4	0.2	-30	200
61	RF2A	1	13500000	0.401	0.423	4.674	4	0.2	-20	800
62		2	13500000	0.683	0.275	8.076	4	0.2	-20	800
63		3	13500000	0.527	0.249	2.573	4	0.2	-20	800
64		4	13500000	0.235	0.226	6.457	4	0.2	-20	800
65		5	13500000	0.646	0.403	5.848	4	0.2	-20	800
66		6	13500000	0.725	0.359	9.551	4	0.2	-20	800
67		7	13500000	0.801	0.378	8.991	4	0.2	-20	800
68		8	13500000	0.495	0.336	4.124	4	0.2	-20	800
69		9	13500000	0.607	0.194	9.414	4	0.2	-20	800
70		10	13500000	0.901	0.303	7.766	4	0.2	-20	800
71		11	13500000	0.309	0.525	6.740	4	0.2	-20	800
72		12	13500000	1.024	0.231	4.933	4	0.2	-20	800
73		13	13500000	0.954	0.293	5.521	4	0.2	-20	800
74		14	13500000	0.429	0.350	7.591	4	0.2	-20	800
75		15	13500000	0.737	0.508	7.093	4	0.2	-20	800
76		16	13500000	1.324	0.127	8.312	4	0.2	-20	800
77		17	13500000	0.817	0.468	10.724	4	0.2	-20	800
78		18	13500000	1.115	0.315	7.357	4	0.2	-20	800
79		19	13500000	0.856	0.390	11.493	4	0.2	-20	800
80		20	13500000	0.562	0.442	6.184	4	0.2	-20	800

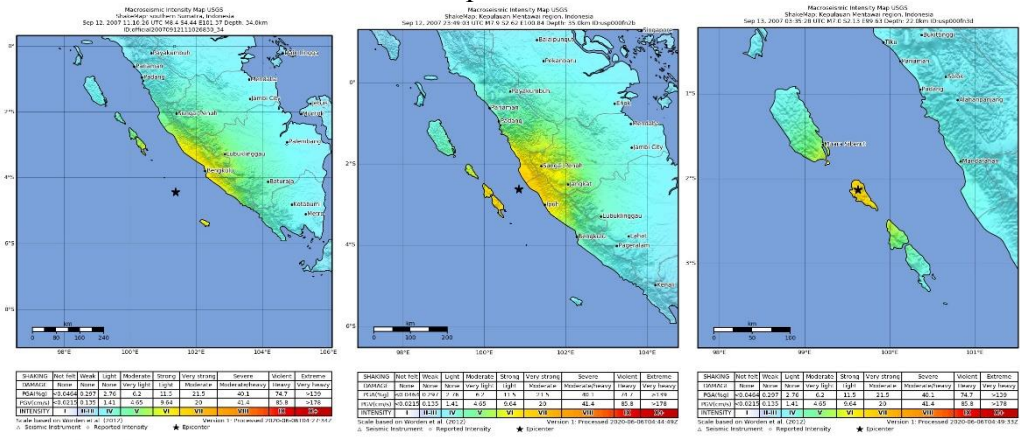
Table D.1 (cont'd) Building Samples Properties used in SDOF Analysis

81	RF2B	1	13500000	0.676	0.237	6.939	4	0.2	-25	400
82		2	13500000	0.632	0.191	4.461	4	0.2	-25	400
83		3	13500000	0.511	0.297	6.087	4	0.2	-25	400
84		4	13500000	0.100	0.324	7.387	4	0.2	-25	400
85		5	13500000	0.728	0.069	5.598	4	0.2	-25	400
86		6	13500000	1.112	0.212	9.869	4	0.2	-25	400
87		7	13500000	0.548	0.372	5.860	4	0.2	-25	400
88		8	13500000	0.978	0.177	8.274	4	0.2	-25	400
89		9	13500000	0.618	0.339	8.858	4	0.2	-25	400
90		10	13500000	0.419	0.137	6.251	4	0.2	-25	400
91		11	13500000	1.270	0.452	3.925	4	0.2	-25	400
92		12	13500000	0.822	0.261	7.685	4	0.2	-25	400
93		13	13500000	0.902	0.314	4.701	4	0.2	-25	400
94		14	13500000	0.581	0.258	6.745	4	0.2	-25	400
95		15	13500000	0.876	0.284	3.404	4	0.2	-25	400
96		16	13500000	0.274	0.276	5.299	4	0.2	-25	400
97		17	13500000	0.428	0.220	7.055	4	0.2	-25	400
98		18	13500000	0.767	0.226	5.013	4	0.2	-25	400
99		19	13500000	0.770	0.377	2.772	4	0.2	-25	400
100		20	13500000	1.019	0.146	6.367	4	0.2	-25	400
101	RF2C	1	13500000	1.498	0.163	3.845	4	0.2	-30	200
102		2	13500000	0.446	0.132	3.418	4	0.2	-30	200
103		3	13500000	0.737	0.139	4.884	4	0.2	-30	200
104		4	13500000	0.403	0.250	4.416	4	0.2	-30	200
105		5	13500000	1.053	0.191	6.314	4	0.2	-30	200
106		6	13500000	0.857	0.128	6.150	4	0.2	-30	200
107		7	13500000	0.238	0.276	3.964	4	0.2	-30	200
108		8	13500000	0.803	0.233	4.680	4	0.2	-30	200
109		9	13500000	0.995	0.071	6.887	4	0.2	-30	200
110		10	13500000	0.594	0.178	6.789	4	0.2	-30	200
111		11	13500000	0.916	0.156	2.715	4	0.2	-30	200
112		12	13500000	0.835	0.104	4.980	4	0.2	-30	200
113		13	13500000	0.614	0.174	4.335	4	0.2	-30	200
114		14	13500000	0.641	0.225	5.455	4	0.2	-30	200
115		15	13500000	0.494	0.153	7.660	4	0.2	-30	200
116		16	13500000	0.335	0.198	5.982	4	0.2	-30	200
117		17	13500000	0.724	0.113	5.228	4	0.2	-30	200
118		18	13500000	0.555	0.084	5.439	4	0.2	-30	200
119		19	13500000	0.669	0.206	5.813	4	0.2	-30	200
120		20	13500000	0.958	0.212	3.328	4	0.2	-30	200



E. Shake Maps of the Events

Sequence – 1



Sequence – 2

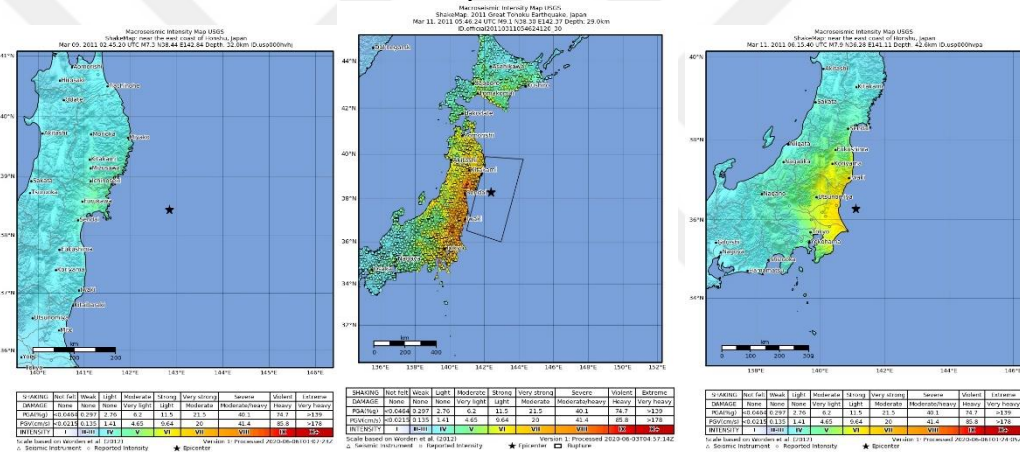
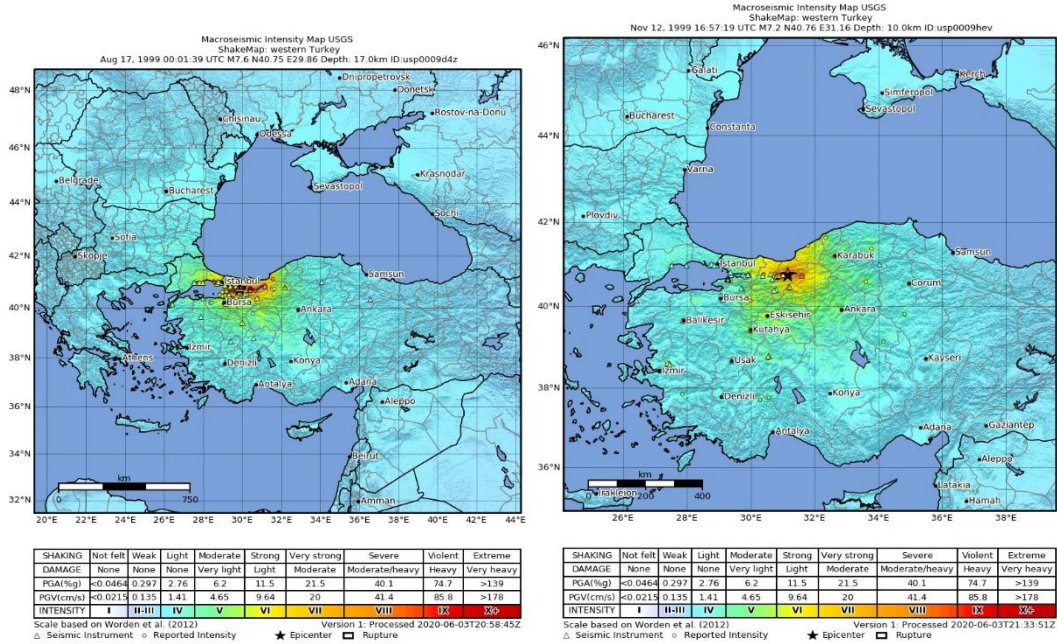


Figure E.1. Shake maps of the Sequence 1-2 (USGS)

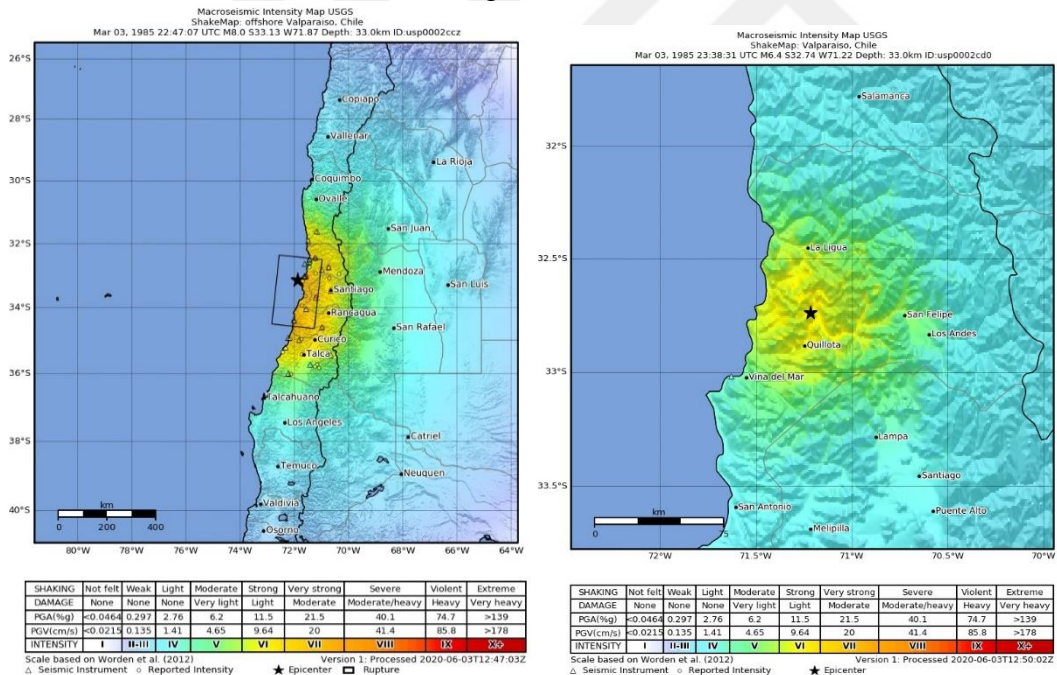
Sequence – 3



E-008

E-009

Sequence – 4

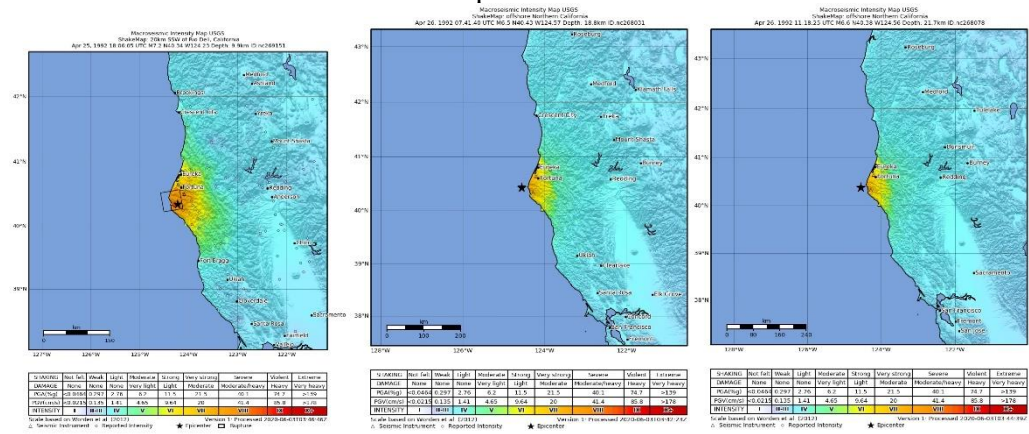


E-010

E-011

Figure E.2. Shake maps of the Sequence 3-4 (USGS)

Sequence – 5



Sequence – 6

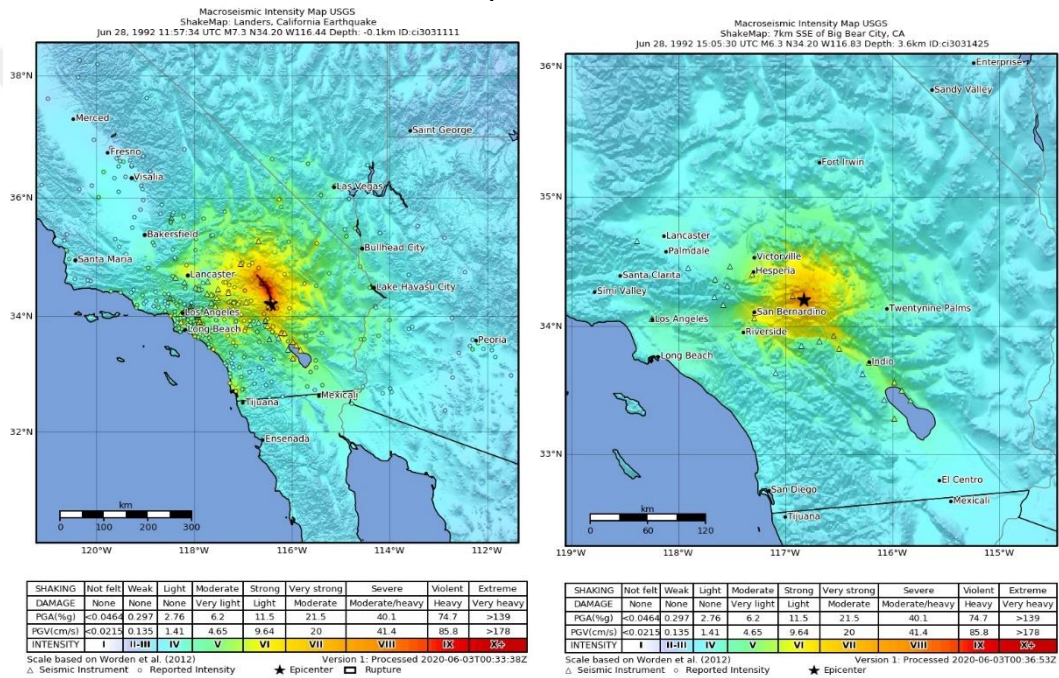
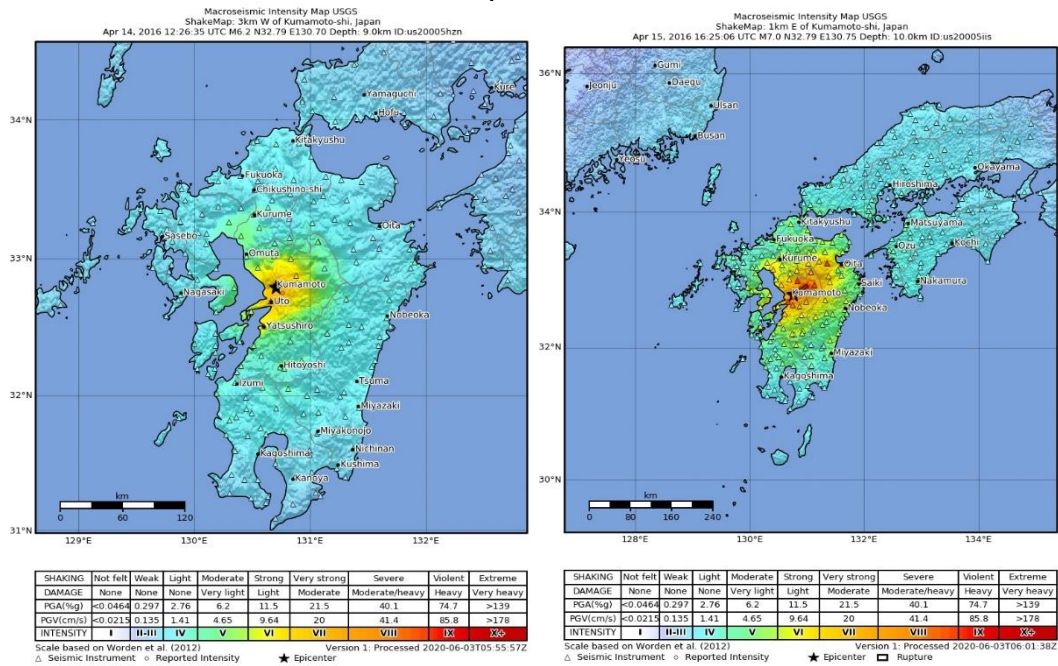


Figure E.3. Shake maps of the Sequence 5-6 (USGS)

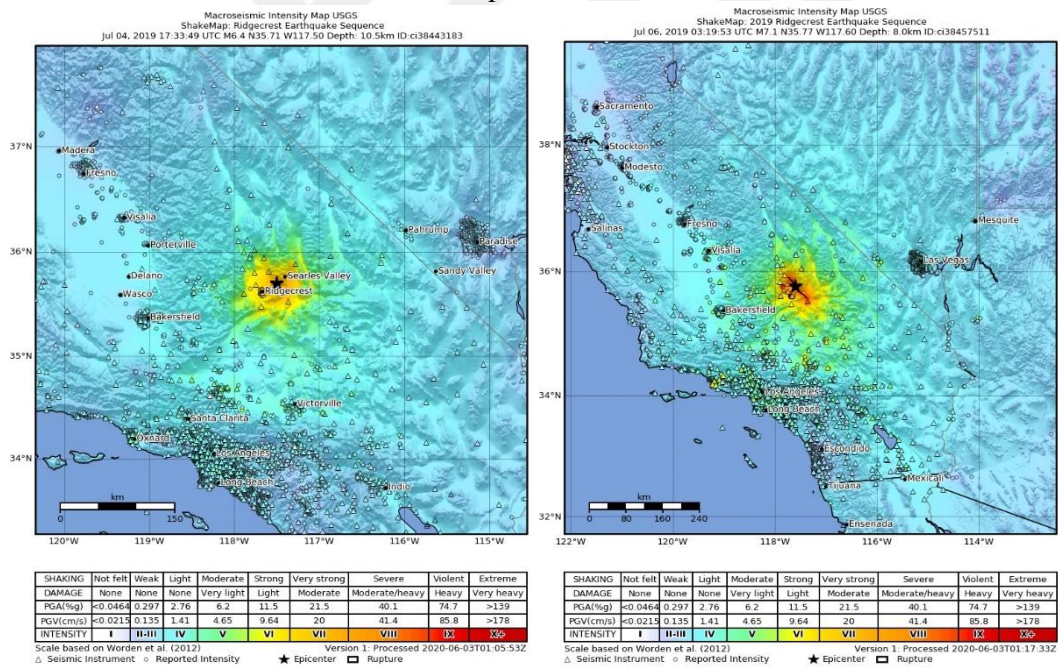
Sequence – 7



E-017

E-018

Sequence – 8



E-019

E-020

Figure E.4. Shake maps of the Sequence 7-8 (USGS)

Sequence – 9 (10,11,12,13)

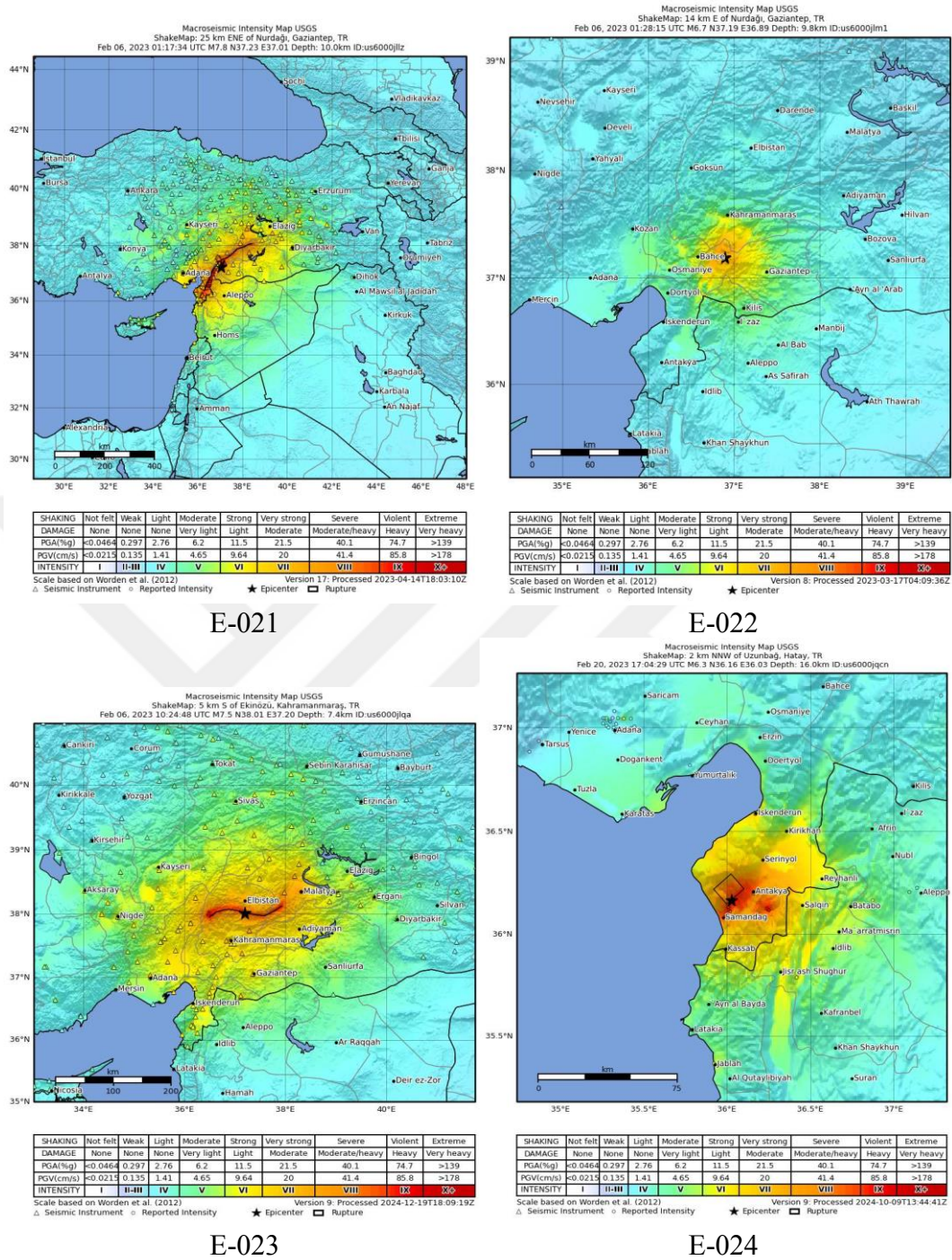
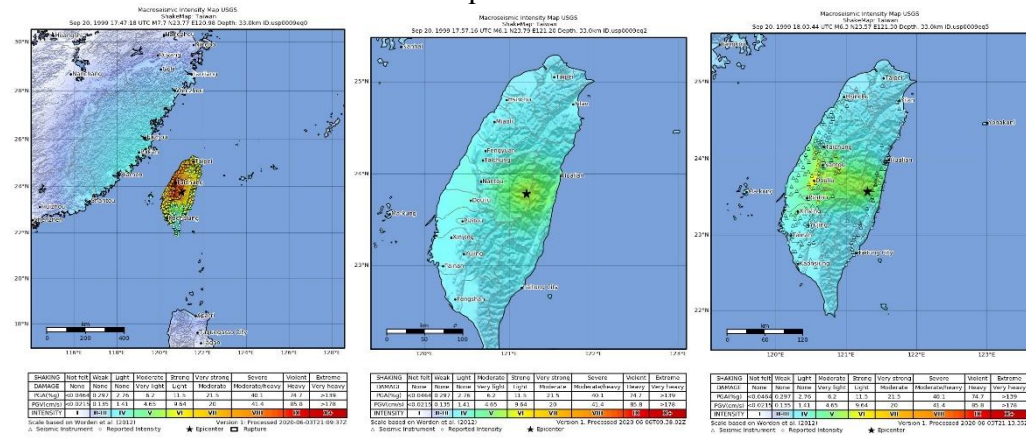


Figure E.5. Shake maps of the Sequence 9-13 (USGS)

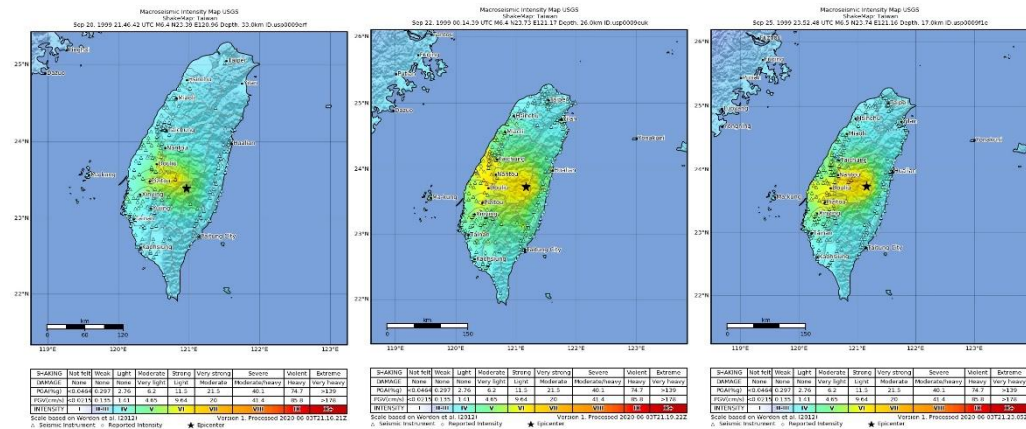
Sequence – 14



E-041

E-042

E-043

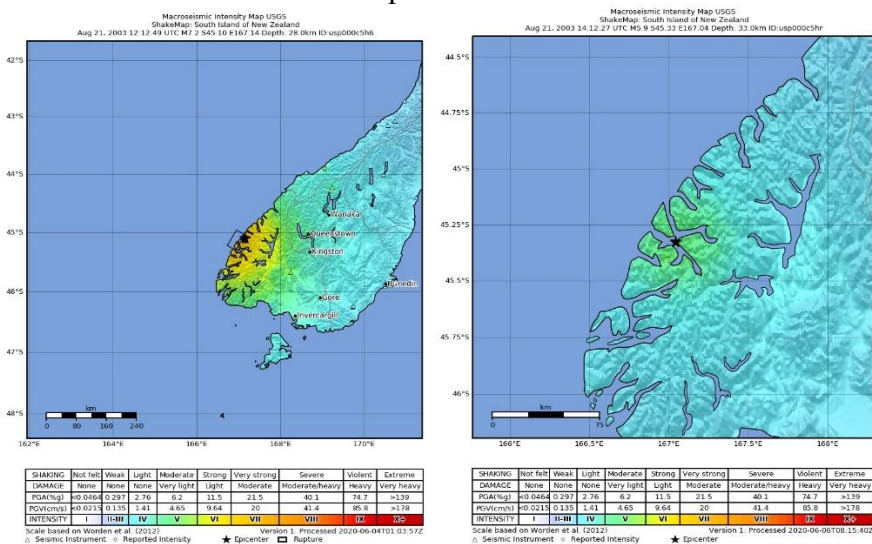


E-044

E-045

E-046

Sequence – 15

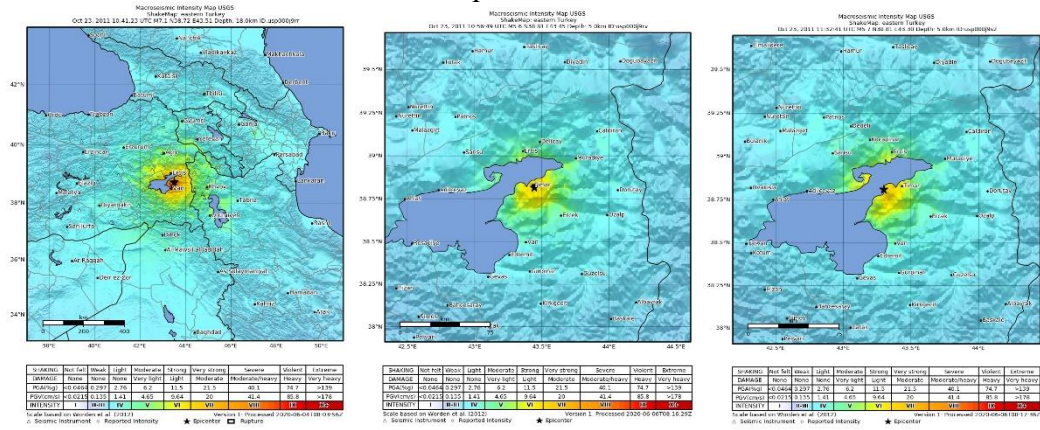


E-047

E-049

Figure E.6. Shake maps of the Sequence 14-15 (USGS)

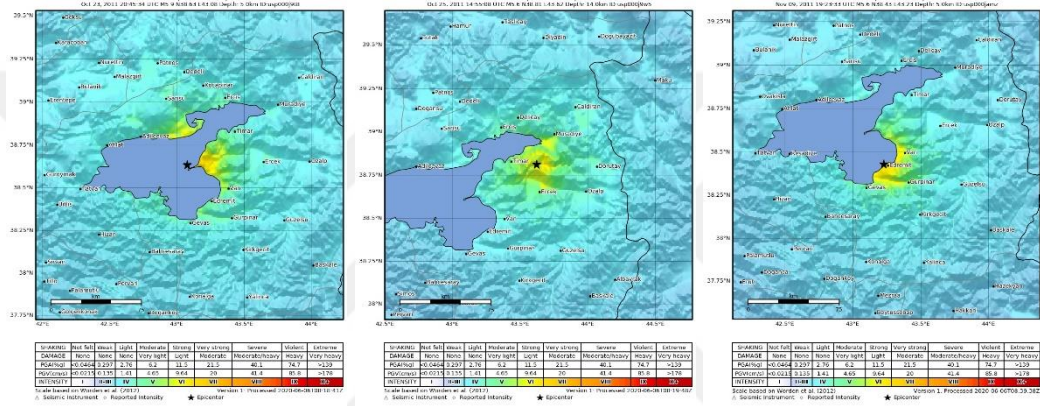
Sequence – 16



E-050

E-051

E-052

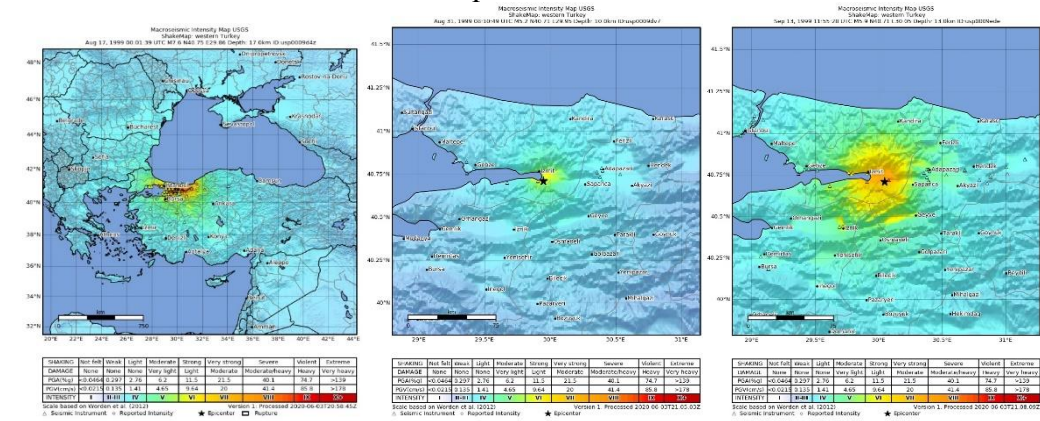


E-053

E-054

E-055

Sequence – 17



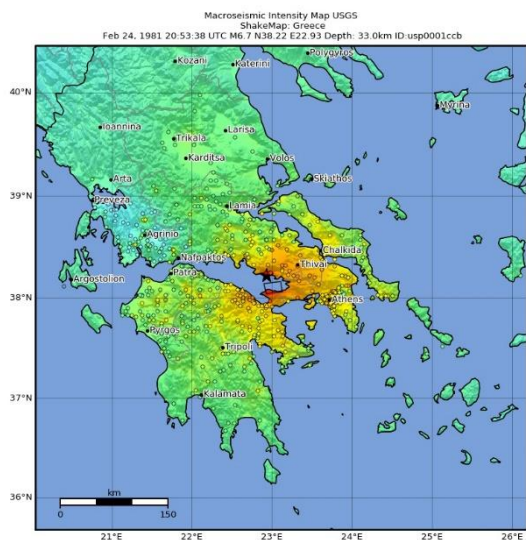
E-056

E-057

E-058

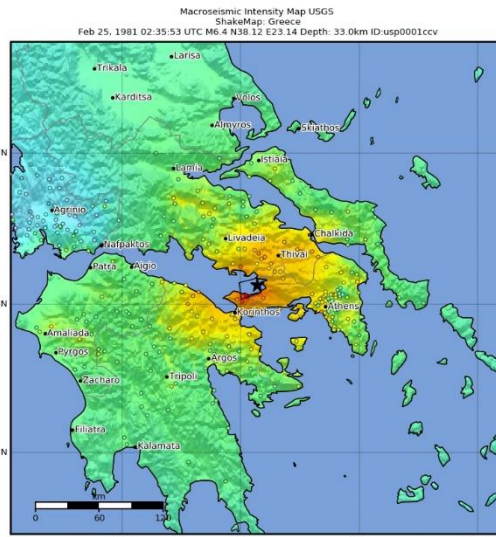
Figure E.7. Shake maps of the Sequence 16-17 (USGS)

Sequence – 20



SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(%)	<0.0464	0.297	2.76	6.2	11.5	21.5	40.1	74.7	>139
PGV(cm/s)	<0.0215	0.135	1.41	4.65	9.64	20	41.4	85.8	>178
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X

Scale based on Worden et al. (2012)
Version 1: Processed 2020-06-03T11:34:29Z
△ Seismic Instrument ○ Reported Intensity ★ Epicenter □ Rupture



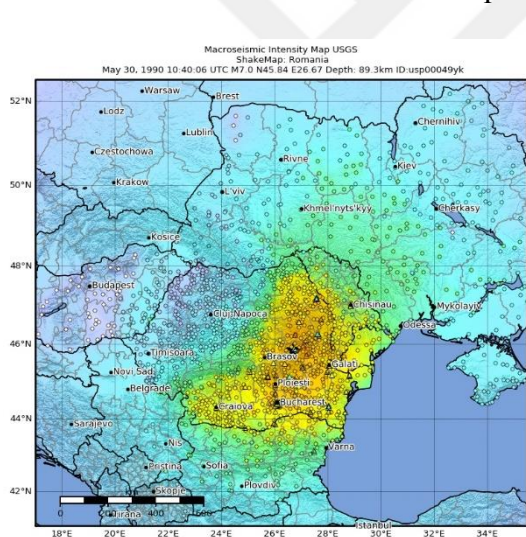
SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(%)	<0.0464	0.297	2.76	6.2	11.5	21.5	40.1	74.7	>139
PGV(cm/s)	<0.0215	0.135	1.41	4.65	9.64	20	41.4	85.8	>178
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X

Scale based on Worden et al. (2012)
Version 1: Processed 2020-06-03T11:37:51Z
△ Seismic Instrument ○ Reported Intensity ★ Epicenter □ Rupture

E-065

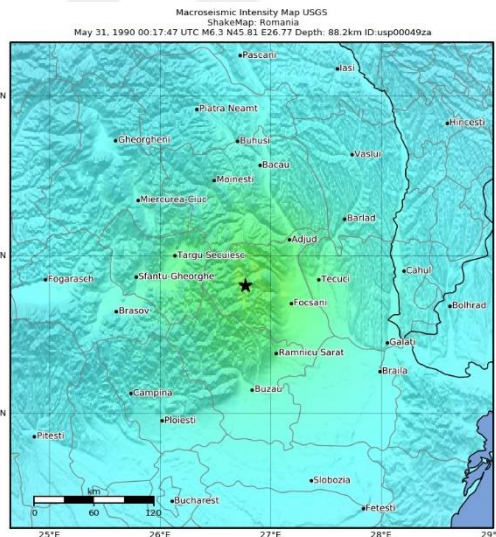
Sequence – 21

E-066



SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(%)	<0.0464	0.297	2.76	6.2	11.5	21.5	40.1	74.7	>139
PGV(cm/s)	<0.0215	0.135	1.41	4.65	9.64	20	41.4	85.8	>178
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X

Scale based on Worden et al. (2012)
Version 1: Processed 2020-06-03T14:44:56Z
△ Seismic Instrument ○ Reported Intensity ★ Epicenter □ Rupture



SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(%)	<0.0464	0.297	2.76	6.2	11.5	21.5	40.1	74.7	>139
PGV(cm/s)	<0.0215	0.135	1.41	4.65	9.64	20	41.4	85.8	>178
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X

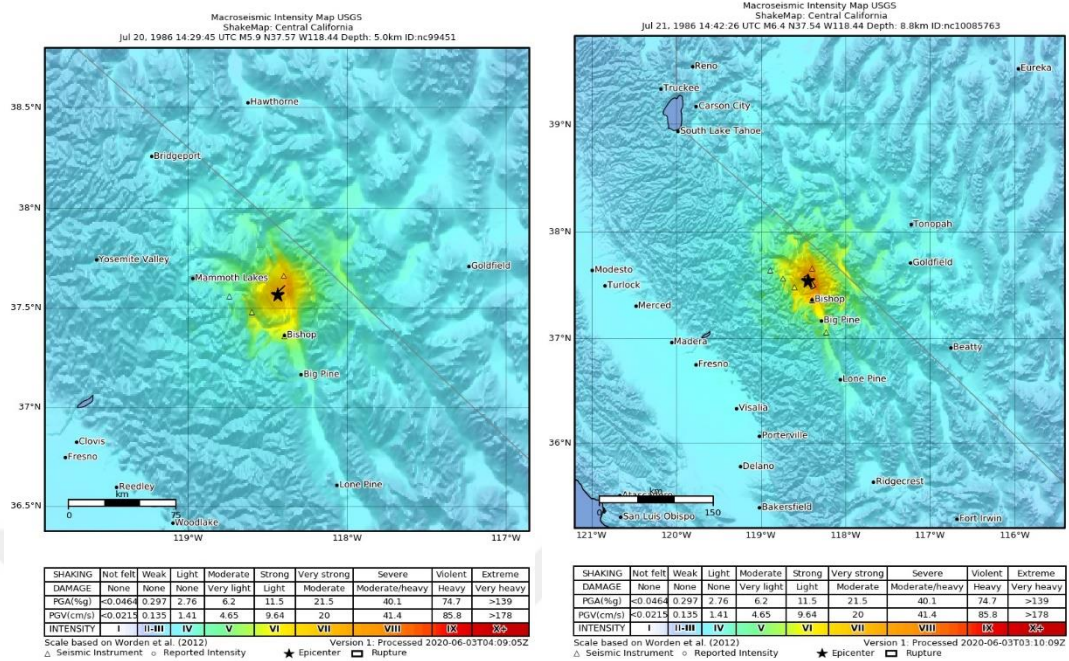
Scale based on Worden et al. (2012)
Version 1: Processed 2020-06-06T07:09:58Z
△ Seismic Instrument ○ Reported Intensity ★ Epicenter □ Rupture

E-065

E-066

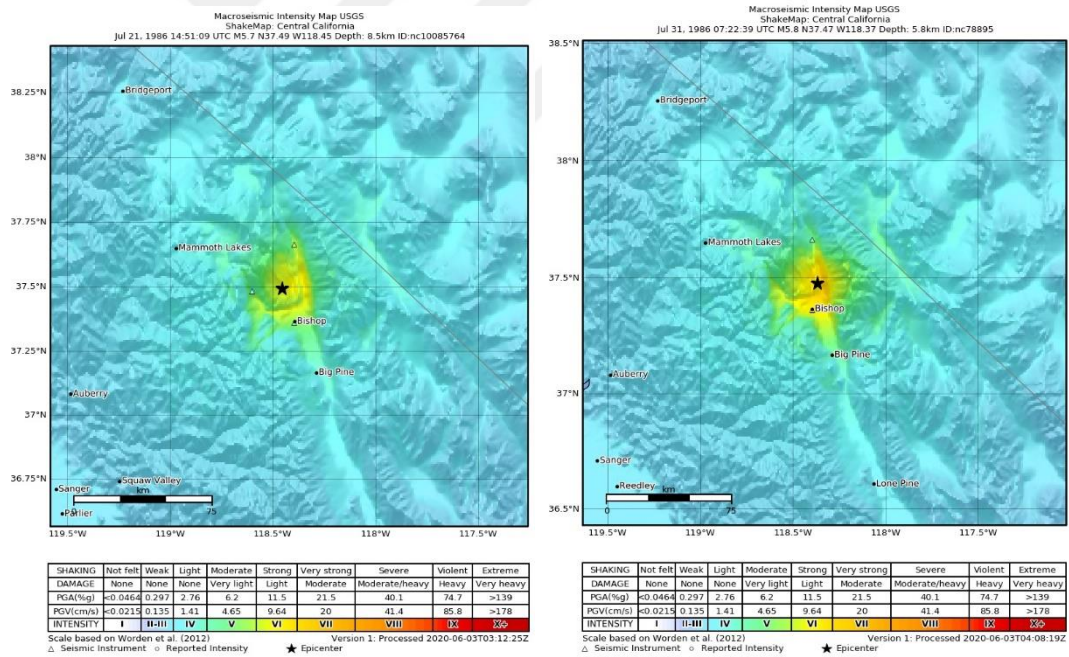
Figure E.8. Shake maps of the Sequence 20-21 (USGS)

Sequence – 22



E-069

E-070

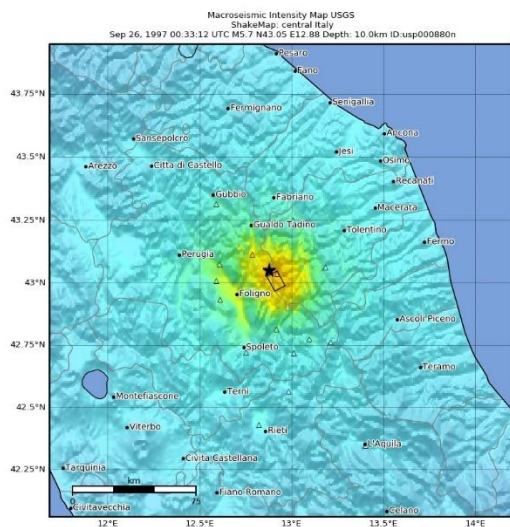


E-071

E-072

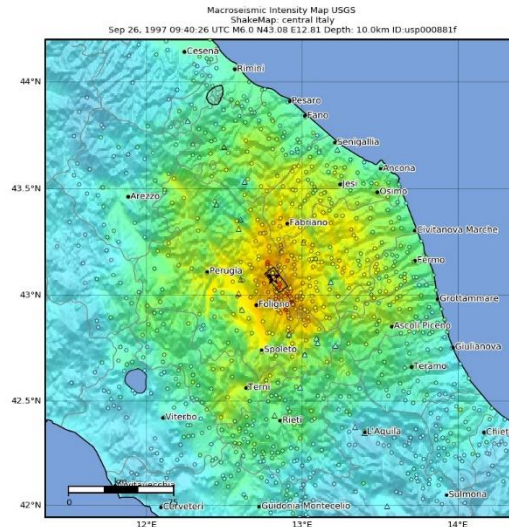
Figure E.9. Shake maps of the Sequence 22 (USGS)

Sequence – 23



SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(g)	<0.0464	0.297	2.76	6.2	11.5	21.5	40.1	74.7	>139
PGV(cm/s)	<0.0215	0.135	1.41	4.65	9.64	20	41.4	85.8	>178
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X

Scale based on Worden et al. (2012)
Version 1: Processed 2020-06-03T19:46:20Z
△ Seismic Instrument ○ Reported Intensity ★ Epicenter □ Rupture

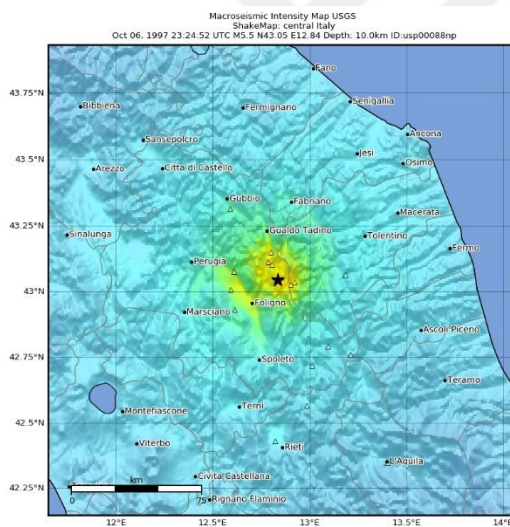


SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(g)	<0.0464	0.297	2.76	6.2	11.5	21.5	40.1	74.7	>139
PGV(cm/s)	<0.0215	0.135	1.41	4.65	9.64	20	41.4	85.8	>178
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X

Scale based on Worden et al. (2012)
Version 1: Processed 2020-06-03T19:47:52Z
△ Seismic Instrument ○ Reported Intensity ★ Epicenter □ Rupture

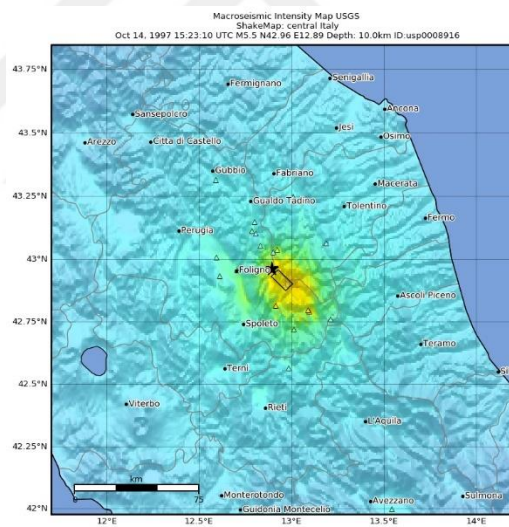
E-073

E-074



SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(g)	<0.0464	0.297	2.76	6.2	11.5	21.5	40.1	74.7	>139
PGV(cm/s)	<0.0215	0.135	1.41	4.65	9.64	20	41.4	85.8	>178
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X

Scale based on Worden et al. (2012)
Version 1: Processed 2020-06-03T19:51:30Z
△ Seismic Instrument ○ Reported Intensity ★ Epicenter □ Rupture



SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
DAMAGE	None	None	None	Very light	Light	Moderate	Moderate/heavy	Heavy	Very heavy
PGA(g)	<0.0464	0.297	2.76	6.2	11.5	21.5	40.1	74.7	>139
PGV(cm/s)	<0.0215	0.135	1.41	4.65	9.64	20	41.4	85.8	>178
INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X

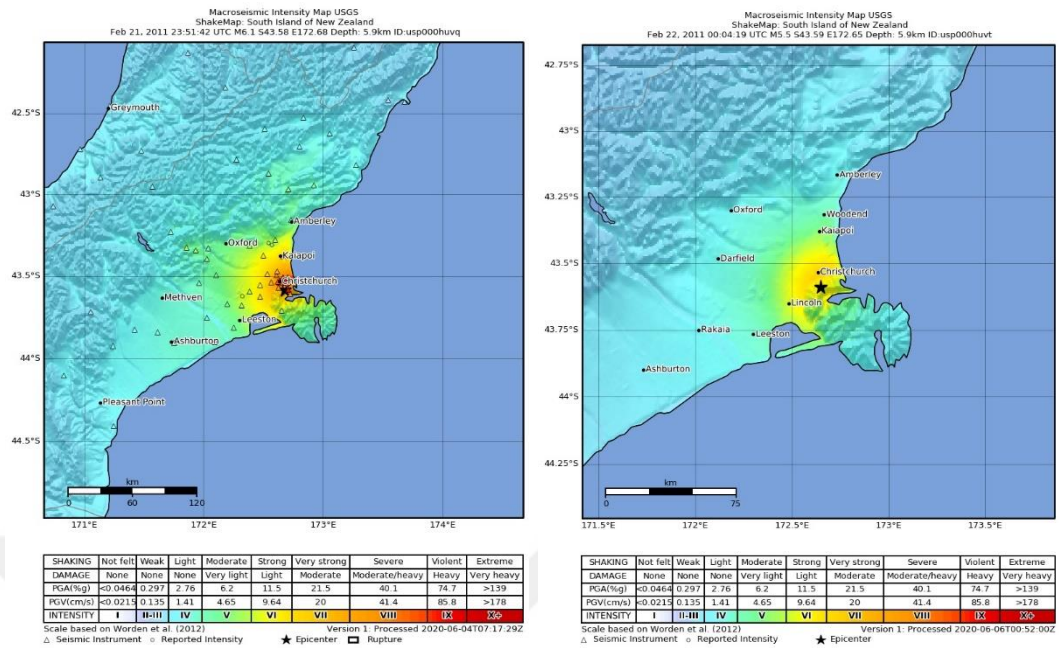
Scale based on Worden et al. (2012)
Version 1: Processed 2020-06-03T19:54:00Z
△ Seismic Instrument ○ Reported Intensity ★ Epicenter □ Rupture

E-076

E-078

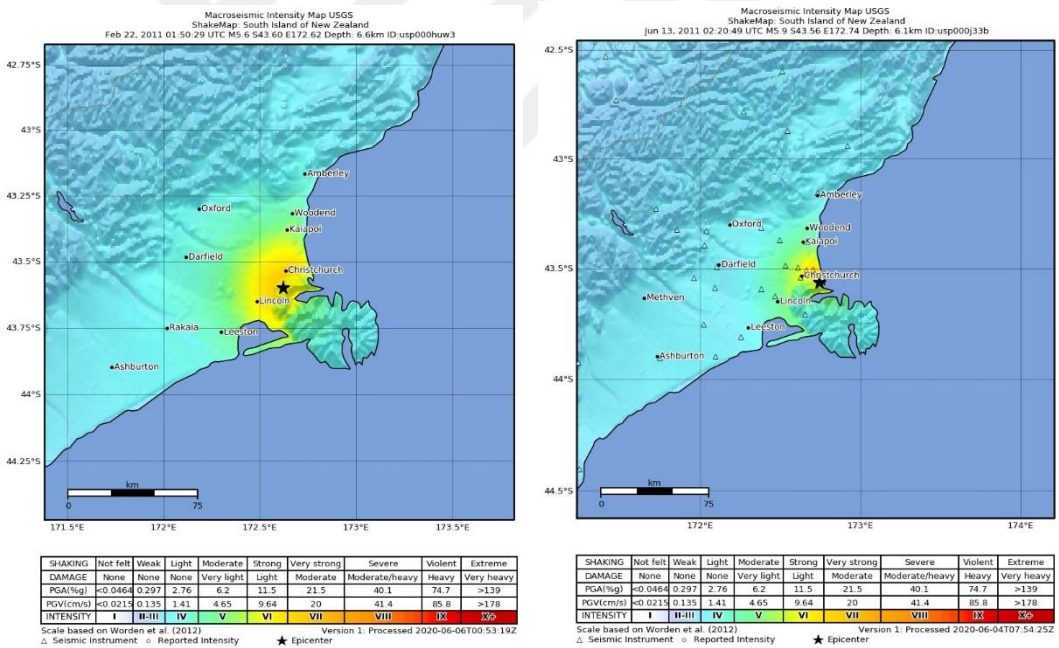
Figure E.10. Shake maps of the Sequence 23 (USGS)

Sequence – 25



E-084

E-085

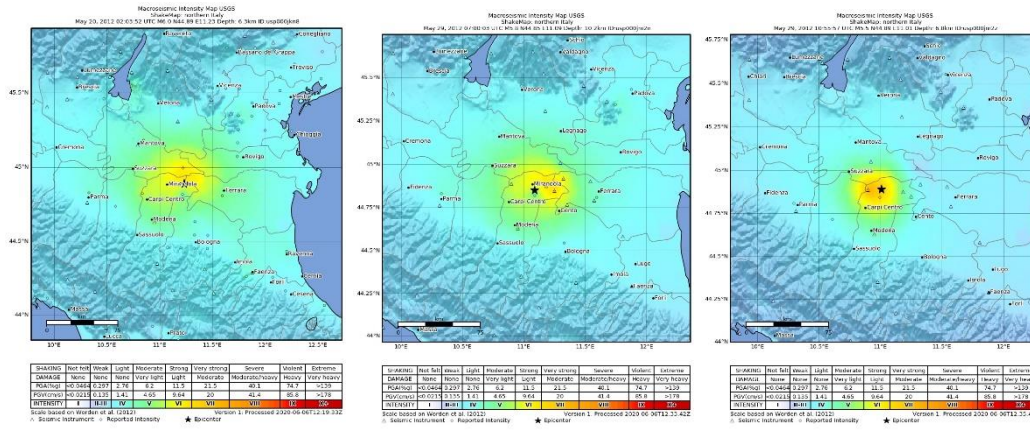


E-086

E-088

Figure E.11. Shake maps of the Sequence 25 (USGS)

Sequence – 26



Sequence – 27

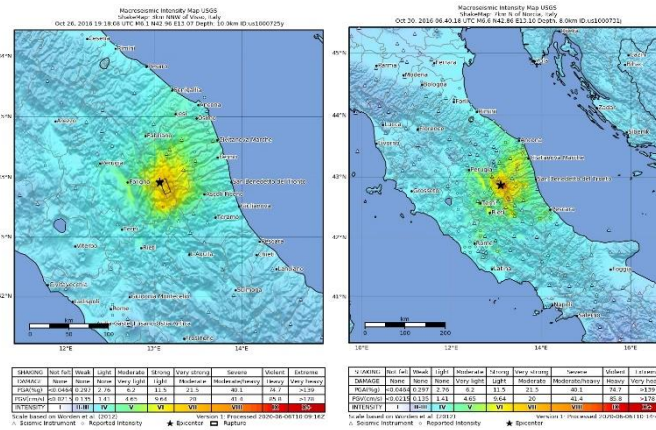
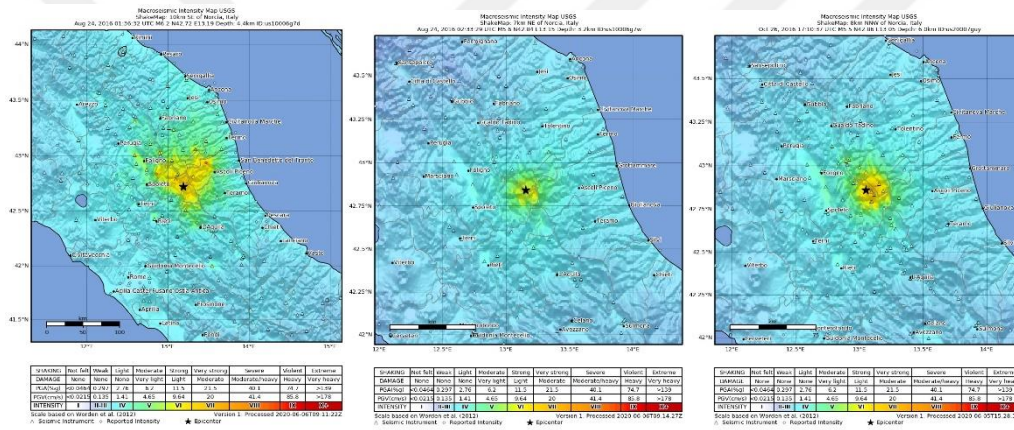
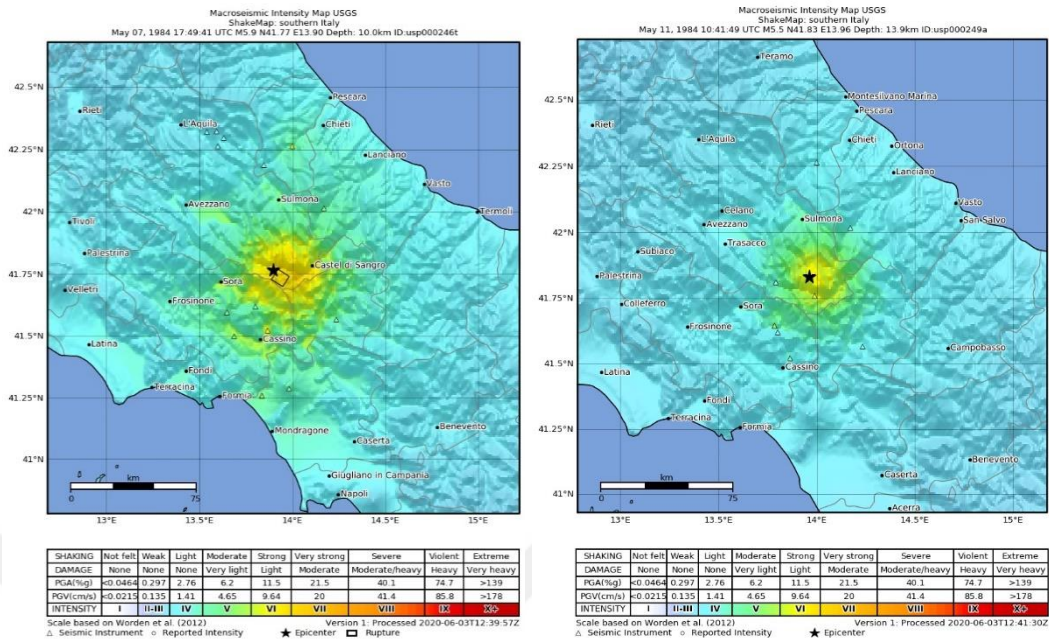


Figure E.12. Shake maps of the Sequence 26-27 (USGS)

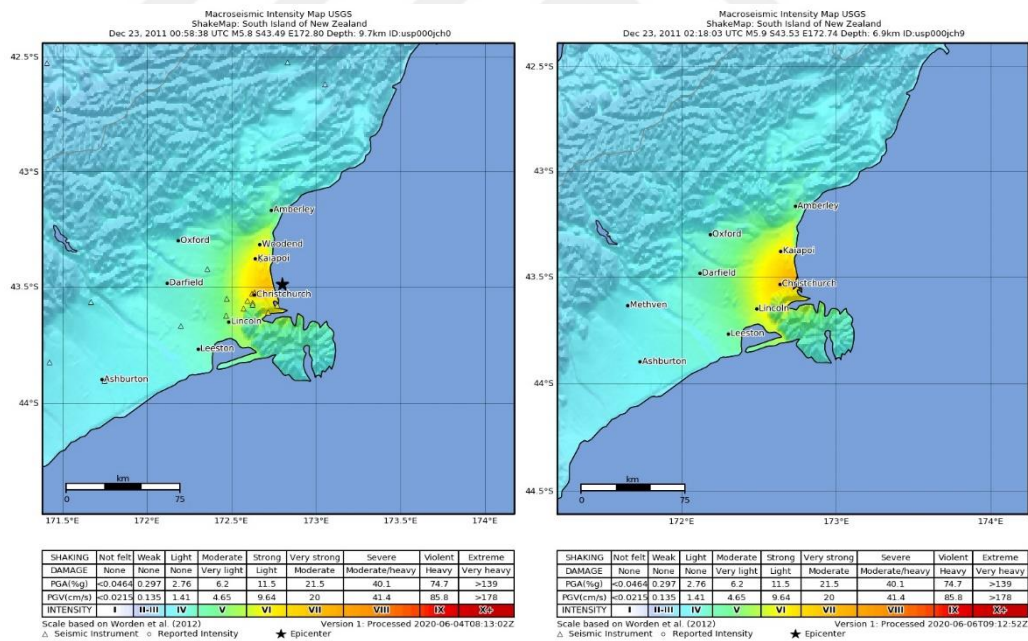
Sequence – 28



E-099

E-100

Sequence – 29

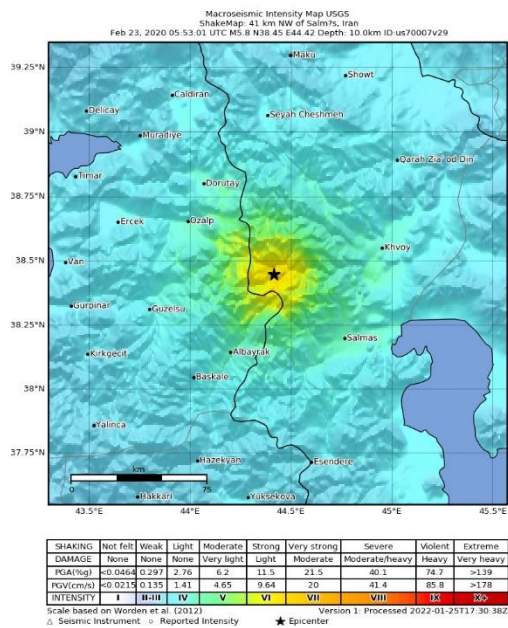


E-101

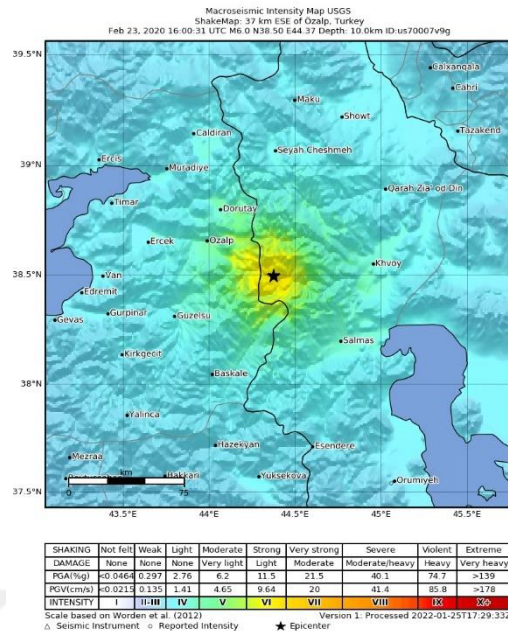
E-103

Figure E.13. Shake maps of the Sequence 28-29 (USGS)

Sequence – 30



E-104



E-105

Figure E.14. Shake maps of the Sequence 30 (USGS)

CURRICULUM VITAE

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EDUCATION

Degree	Institution	Year of Graduation
MS	METU Civil Engineering	2017
BS	METU Civil Engineering	2014
High School	Ankara Atatürk High School	2009

FOREIGN LANGUAGES

Advanced English, Elementary German, Beginner Italian

PUBLICATIONS

1. Gökmen F., Binici B., Aldemir A., Taghipour A., Canbay E. "Seismic behavior of autoclaved aerated concrete low rise buildings with reinforced wall panels", Bulletin of Earthquake Engineering, 17(7), 3933-3957 (2019)
2. Binici B., Gokmen F., Canbay E., Aldemir A., Ogdu M. K., Uzgan U., et al. "In situ seismic testing of a reinforced autoclaved aerated concrete building", Mauerwerk, 22(5), 305-313 (2018)

HOBBIES

Orienteering, Cooking, Movies, F1, Video Games