Engineering Failure Analysis 153 (2023) 107521

Contents lists available at ScienceDirect

Engineering Failure Analysis

journal homepage: www.elsevier.com/locate/engfailanal

Effect on RC buildings of 6 February 2023 Turkey earthquake doublets and new doctrines for seismic design

Murat Ozturk [a], Musa Hakan Arslan [a,*], Hasan Hüsnü Korkmaz [b]

[a] *Konya Technical University, Faculty of Engineering and Natural Sciences, Department of Civil Engineering, 42250 Konya, Turkey*
[b] *Necmettin Erbakan University, Faculty Of Engineering And Architecture, Department of Civil Engineering, 42090 Konya, Turkey*

A R T I C L E I N F O

Keywords:
Earthquake
Reinforced concrete structure
Damage
Ductility
Reinforcement detail
Load-bearing system

A B S T R A C T

Two major catastrophic earthquakes, which occurred 9 h apart on the Eastern Anatolian Fault Zone and one of its branches on February 6, 2023 in Turkey, directly affected 11 provinces in the Eastern and South-Eastern Anatolia regions, where 14 million people live. These earthquakes are among the most destructive earthquakes in the history of Turkey, caused approximately 3 times more property damage and loss of life than the 1999 Marmara Earthquake. More than 24,000 aftershocks with magnitudes of up to 6.7 M_w occurred after these earthquakes. In this study, the causes of heavy destruction in some of the provinces and districts most affected by earthquakes (especially Hatay-Antakya, Kahramanmaraş, Gaziantep-Nurdagi, Gaziantep-Islahiye, Adıyaman-Gölbaşı, Malatya) were examined under sub-headings. The reasons for the damages were supported by the striking images obtained during the site investigation in the earthquake zone, and the issues that should be considered based on both seismic codes and implementations were also stated. In addition, in order to better understand the effects of earthquakes on reinforced concrete structures, the ratios of the loads acting on the structures in both earthquakes to the design load predicted by the code valid at the time the structures were built are presented in graphics. The results obtained show that the effects of design and application mistakes are quite high in the heavy destruction caused by earthquakes. But it is understood that the seismic code design criteria and requirements are insufficient in some regions. In addition, the establishment of residential areas in risky areas without taking any precautions has produced dramatic results.

1. Introduction

Turkey (Türkiye) is located in one of the most active fault zones in the world [1,2]. Catastrophic earthquakes (Erzincan 1983 and 1982, Marmara 1999, Duzce 1999, Van 2011 etc.) in Turkey have caused a lot of destruction, cusalties and injuries due to the fact that the building stock is vulnerable to seismic forces. In the literature, many post-earthquake damage analyzes have been made by reconnaise teams for almost all types of structural systems [3-15] In the afore-mentioned researches, the general consensus is that a building that is not built in comply with the earthquake codes will most likely suffer heavy damage or collapse, regardless of the type of building.

In Turkey, 80% or more of the building stock is reinforced concrete. The increase in migration, especially from villages to cities, brought along rapid, uncontrolled and multi-storey construction, and unfortunately, a significant part of the buildings could not get

* Corresponding author.
E-mail addresses: mozturk@ktun.edu.tr (M. Ozturk), mharslan@ktun.edu.tr (M.H. Arslan), hhkorkmaz@erbakan.edu.tr (H.H. Korkmaz).

https://doi.org/10.1016/j.engfailanal.2023.107521
Received 2 May 2023; Received in revised form 8 July 2023; Accepted 30 July 2023
Available online 6 August 2023
1350-6307/

enough of the earthquake codes in force as of the year they were built. In addition, there are problems in buildings built comply with earthquake codes due to some conceptual inadequacies of the old dated codes. For example, the fact that the earthquake codes that changed and developed in Turkey in parallel with the world were (normally) far from the ductile design and capacity design principle of the 1970′s version also caused the reinforced concrete structures built until the beginning of the 2000s to be insufficient in this context.

With the industrialization effort in developing Turkey, qualified ready-mixed concrete only started to be used in reinforced concrete buildings in the construction sector towards the end of the 1990s. Similarly, seismically ideal reinforced concrete steel started to be applied in the same years. Nevertheless, the vulnerabilities of reinforced concrete buildings, which were produced before the 2000s and constitute 35–40% of the building stock and are common in city centers, remain quite low under the influence of earthquakes.

6 February 2023 Turkey earthquake doublets was effective in 11 provinces in Turkey and caused an economic loss of approximately 110 billion dollars. After two major earthquakes that occurred on the same day, which is rare in the seismological literature, nearly 25% of the buildings in the cities close to the earthquake epicenter were severely damaged or collapsed [16]. In fact, a similar destruction rate was reported in the 1999 Marmara (7.6 M_w) and 1999 Düzce (7.2 M_w) Earthquakes in Turkey [17]. However, the fact that this ratio (25%) has not changed much despite the new structures built after TEC-1998 [18], TEC-2007 [19] and TBEC-2018 [20], which came into force with the 1999 earthquake and have very strict seismic design rules, is a result of heavy damage and destruction in the structures built in accordance with these codes. For this reason, after the 6 February 2023 earthquakes, the main question of the field researchers is whether there are design concept problems in the current earthquake codes. This question is valid for the reinforced concrete building stock in Turkey, as well as for Greece, Iran, Italy, Mexico, etc., which share a similar fate in terms of earthquakes. The collapse of a structure built in accordance with earthquake codes (this probability should be 10^{-6}) [21] can be based on two main reasons. The first is that the desired production cannot be carried out in buildings designed according to all the heavy conditions of the seismic codes, and the second is that the conditions of the codes have some deficits in terms of earthquake performance.

In this study, based on this motivation, a general evaluation was made for the damage to the reinforced concrete structures that occurred after the 6 February 2023 Earthquakes, and evaluations were presented in the context of construction and codes criteria. Considering the importance of the lessons to be learned in the great destruction caused by the 6 February 2023 Earthquakes for other earthquake prone regions and researchers as much as Turkey, it is thought that the evaluations to be made will play a critical role in terms of new codes and producing techniques.

2. Lessons learned from past earthquakes all over the world

Before moving on to the 6 February 2023 Kahramanmaraş Turkey earthquake examined in this article and its damage to reinforced concrete buildings, a brief overview of devastating earthquakes in world history would be helpful. Earthquake damage records, which provide healthy statistics, are becoming important, especially for other destructive earthquakes to give important clues to today's structural and earthquake engineers. In this way, it will be easier to produce earthquake resistant structures for future. Especially 1975 Mexico, 1990 Luzon (Philippines), 1992 Erzincan (Turkey), 1995 Hyogoken Nanbu (Kobe) and 1999 Marmara (Turkey), 1999 Düzce (Turkey) earthquakes are important in this context.

It is very important that in the Mexico Earthquake (6.5 M_w), while 94% of reinforced concrete buildings with six stories or lower were operational, the damage increased tragically in mid and high rise buildings and approximately 50% of these buildings were severely damaged. In particular, the softening in the local soil class in medium and high buildings caused an increase in the displacement demand and the buildings without sufficient ductility capacity were severely damaged or totally collapsed. In the 1990

Fig. 1. Plate movements around Turkey and epicenters of Feb. 6, 2023 earthquake doublets and Formation of the Anatolian plate [images from [26]] for 100, 50 and and 25 million years ago, respectively.

Luzon (Philippines) (7.0 M_w) Earthquake, the damage was concentrated in mid-high rise buildings (six-story and higher). Heavy damage was 18%, and the rate of totally collapsed buildings was 5%. In the 1992 Erzincan Earthquake (Turkey) (6.7 M_w) that took place in 1992, moderate and severe damage was not observed in buildings with three stories and lower, while the percentage of heavy damage in reinforced concrete buildings higher than four stories exceeded 30%. In the 1995 Hyogoken Nanbu (Kobe) Earthquake (6.9 M_w), the percentage of heavily damaged buildings was 6%, and the rate of collapsed buildings was 5.7%. In the two major earthquakes that occurred in Turkey in 1999 (Marmara (7.6 M_w) and Düzce (7.1 M_w)), the rate of heavily damaged or totally collapsed buildings is over 22%. In all of these earthquakes, it was observed that as the age of the structure and the number of stories increased, the vulnerability increased significantly. However, in most of the heavily damaged or totally collapsed buildings, shear walls were not used. The most important point that field researchers pointed out in these earthquakes was the negative effect of soft story formation on the performance of the building. For example, in the Kobe earthquake [22–24], it is seen that buildings with soft story collapse at a percentage of three times higher than regular buildings.

Similarly, in studies conducted after the 2003 Bam (Iran) (6.6 M_w) Earthquake, it was stated that there was a number of reinforced concrete structures with masonry infill walls collapsed due to the soft story.

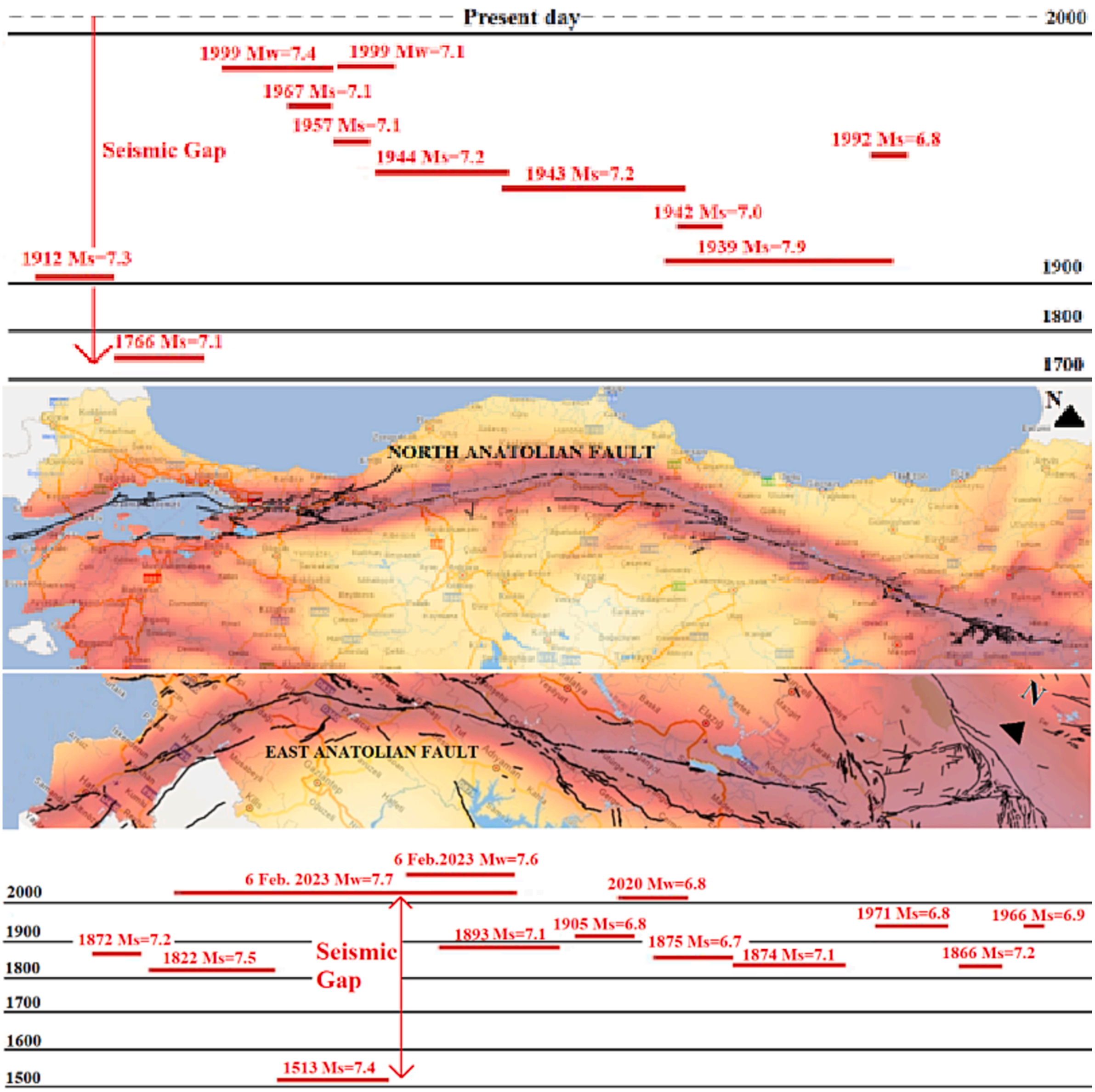


Fig. 2. Historical earthquakes along the NAF and EAF [33,39,41–43].

3. Seismotectonics of the region and characteristics of 6 February 2023 Kahramanmaraş Turkey earthquakes

The Anatolian transform fault system is probably the most active in the world [25]. As seen in Fig. 1, the eastern Mediterranean has a complex tectonic structure involving many plates such as Anatolian plate, African Plate, Eurasian plate, Aegean Microplate. Most of Turkey lies on the Anatolian plate and has been affected by the plate movements around it for millions of years. A mere 100 m years ago, this plate comprised part of the southern shore of sea called tethys, which separated from Africa from Eurasia [26]. Later, this sea closed up and northward-moving Anatolian plate was squeezed between the northward-moving Arabian plate, and the southward-moving Eurasian plate. The collision between the Eurasian and Arabian Plates, which started approximately 13 million years ago [27], after the subduction of the Tethys Ocean beneath Eurasia formed Eastern Anatolia and its present-day tectonics [28,29]. Eastern Anatolia, together with the Tibetan plateau, is regarded as one of the two largest regions in the world in which an active continent–continent collision is currently occurring [28]. The Arabian plate and the Eurasian plate still continue to converge and the Anatolian block is being extruded westward along the North and East Anatolian Fault zones at an average rate of about 20 mm / yr [29–32].

The North Anatolian Fault (NAF) zone, which separates the Eurasian plate from the Anatolian plate in northern Turkey, forms a narrow band between Karlıova and Mudurnu Valley, and splays into three strands in the eastern Marmara Sea region [30,33]. NAF is a right-lateral strike-slip fault and its length is about 1500 km.

NAF,where many destructive earthquakes occurred in Turkey, had a very active period, especially in the 20th century. Due to the stress transfer that started with the 7.9 magnitude (M_w:7.9) earthquake that occurred in Erzincan in 1939, 8 more destructive earthquakes occurred on the NAF in 60 years and a westward earthquake sequence was triggered (Fig. 2). Of these, the 1967 earthquake ended in the west of the Mudurnu valley where the fault bifurcated, and the 17 August 1999 İzmit (M_w: 7.4) and 12 November 1999 Düzce (M_w: 7.1) earthquakes, which occurred 3 months apart, occurred on the northern branch of the fault. There are researchers who claim that the probability of an earthquake $M_w \geq 7$ magnitude within 30 years, after the 1999 Golcuk and Duzce earthquakes is about 65% [34]. This part of the fault was last ruptured in 1766 and is considered one of the most hazardous seismic gaps in Turkey (Fig. 2).

The East Anatolian Fault Zone is a left lateral strike-slip fault line extending approximately 580 km between the Arabian and Anatolian plates in eastern Turkey [35,36]. The EAF system is located between Karlıova in the northeast and Karataş-Samandağ districts in the southwest. According to Duman and Emre [37], while the EAF is represented by a simple fault trace between Karlıova and Çelikhan, it is divided into two branches as north and south branches in the south of Çelikhan [38]. 7 segments, namely Karlıova, Ilıca, Palu, Pütürge, Erkenek, Pazarcık and Amanos, have been defined on the southern branch, which is the main line of the EAF with a length of approximately 580 km (Fig. 3). The northern branch extending between Çelikhan and Karataş districts and is expressed by the Sürgü, Çardak-Göksun, Savrun, Çokak, Toprakkale, Yumurtalık, Karataş, Düziçi-Osmaniye segments is called the Sürgü-Misis fault system and is approximately 380 km long [38] (Fig. 3). As can be seen from Fig. 2, the EAF had a relatively stagnant period in the 20th century.

On Feb. 6, 2023, at 04.17 (01.17 UTC) and 13.24 (10.24 UTC), two major earthquakes, M_w 7.7 and M_w 7.6, occurred on the southern and northern branches of the East Anatolian Fault (EAF), respectively. Especially since the last devastating earthquake that occurred in the Pazarcık segment in 1513, the energy accumulated in this part for about 500 years was released by the rupture of 300 km fault piece on 6 February 2023 01:17 UTC. When the rupture distribution in Fig. 2 is examined, it shows that the first earthquake with a magnitude of M_w 7.7 was formed by the broken of three separate earthquake forming parts in history at once [39]. Considering

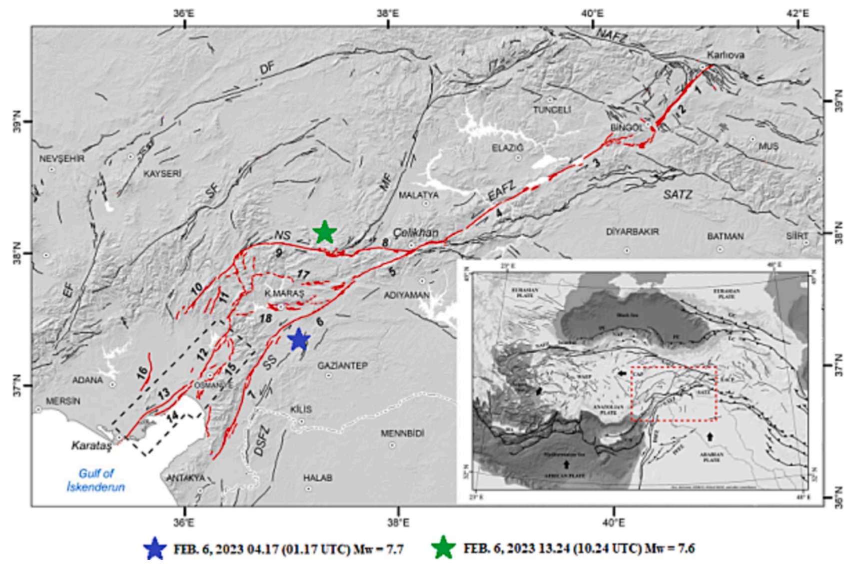


Fig. 3. East Anatolian fault [38].

that the Aug 17, 1999 Marmara Earthquake created a surface rupture of approximately 130 km [40], the extent of the catastrophic earthquake experienced can be better understood.

Approximately 9 h after the first earthquake, with its epicenter on the Pazarcık segment, a second earthquake with a magnitude of M_w 7.6, with its epicenter on the Sürgü-Çardak segment, probably triggered by the first earthquake, struck the region again. In the second earthquake, total rupture length is over 160 km with large surface displacements of the order of 2 – 8 m [44].

Over 570 aftershocks were recorded within 24 h after the first earthquake (M_w 7.7). Although the magnitude of the aftershocks gradually decreased, nearly 10.000 records were recorded in the three weeks after the earthquake. Interestingly, an aftershock of M_w 6.7 occurred approximately 11 min after the main shock [45]. The second earthquake (M_w 7.6) also triggered its own series of aftershocks, including two 6.0 magnitude aftershocks [46,47].

Two major catastrophic earthquakes (Table 1), which occurred 9 h apart on the Eastern Anatolian Fault Line and one of its branches on February 6, 2023 in Turkey, caused the death of about 51,000 people [48]. Thousands of people who have not yet been found are not included in this number. Over 15 million people living in the region were affected by these devastating earthquakes in 11 major cities and 17 districts. In these earthquakes, in which approximately 62,000 apartment buildings were destroyed, two million people had to migrate to the surrounding provinces. Most of the destroyed buildings were in the provinces of Hatay, Kahramanmaraş, Adıyaman, Gaziantep and Malatya.

4. General characteristics of building stock in epicenter

It is known that 47.4% of the general building stock in Turkey was built after 2000 (Fig. 4) [52]. There is a similar situation in the provinces in the earthquake epicenter. However, this rate was determined as 58.1% in Kahramanmaraş due to migration and commercial activities. It is known that structures built after TEC-1998 [18] are more advantageous than older structures. The main reason for this is that the country has experienced the 1999 Marmara (M_w 7.6) and 1999 (M_w 7.1) Düzce earthquakes, and in addition, TEC-1998 [18], which is quite comprehensive compared to the previous regulation, came into force in 1998. 80–85% of the buildings in this building stock are reinforced concrete [53]. The main reason why reinforced concrete building stock is more common in Turkey compared to other building types is that both engineering services and manufacturing are easier than other building types. However, due to the rapid population growth in cities and limited zoning areas, it is another reason to construct mid and hig-rise buildings easier as reinforced concrete. It is known that the slice, which was built between 1980 and 2000 in Turkey and constitutes approximately 30.9% of the building stock, is quite risky. The most important reason for this is that the code (TEC-1975 [54]), which was valid in the year these buildings were built, was not at a sufficient level in terms of capacity design and ductility, and the building inspection-control mechanism was not very effective in these years.

It is extremely important to find the answer (or answers) to the question; *"Why did 6 February Earthquakes cause such great destruction and casualties?"*. In addition to this, it is extremely important to better understand and learn from why the reinforced concrete buildings, which constitute a very important part of the building stock, have collapsed, what kind of deficiencies there are in the design of the buildings with the old dated codes, and unfortunately why the structures built according to the new codes are collapsed. Otherwise, unfortunately, the results of earthquakes that repeat at certain periods will always be similar.

After two major catastrophic earthquakes, it is necessary to evaluate the damage and total collapses especially in reinforced concrete buildings from a very broad perspective. Many sub-headings such as some constraints and changes in siesmic codes, mistakes on choies of residential location areas, structural and non-structural defects and insufficient knowledge of architects and engineers in designing a building to have sufficient earthquake performance are discussed below, respectively. Especially before mentioning the causes of damage, it would be useful to examine the development of earthquake codes and earthquake hazard maps that were in force in different periods in Turkey, which were cited in the study.

5. Historical development of earthquake codes and earthquake hazard maps in Turkey

Earthquake Hazard Maps 1945, 1947, 1963, 1972, 1996, 2018 and earthquake codes (1940, 1944, 1947, 1953, 1961, 1968, 1975, 1998, 2007, 2018) have been updated many times in Turkey depending on the developments in engineering seismology, the

Table 1
Earthquake parameters obtained from various institutions such as EMSC [49] AFAD [50], KOERI [51], USGS [45].

Parameter	06.02.2023 01:17 GMT Pazarcık Earthquake				06.02.2023 10:24 GMT Elbistan Earthquake			
		Sources Institutions						
	AFAD	KOERI	USGS	EMSC	AFAD	KOERI	USGS	EMSC
Location	37.04°E37.28°N	37.11°E 37.11°N	37.01°E 37.22°N	37.08°E 37.17°N	37.23°E38.08°N	37.20 °E38.07°N	37.19°E 38.01°N	37.24°E 38.11°N
Magnitude (M_w)	7.7	7.7	7.8	7.8	7.6	7.6	7.5	7.5
Hyp. Depth (km)	8.6	5	10	20	7.0	5	7.4	10

E Longitudinal, N Latitude, AFAD Turkish Ministry of Interior Disaster and Emergency Management Presidency, KOERI Kandilli Observatory and Earthquake Research Institute, USGS United States Geological Survey, EMSC European Mediterranean Seismological Centre.

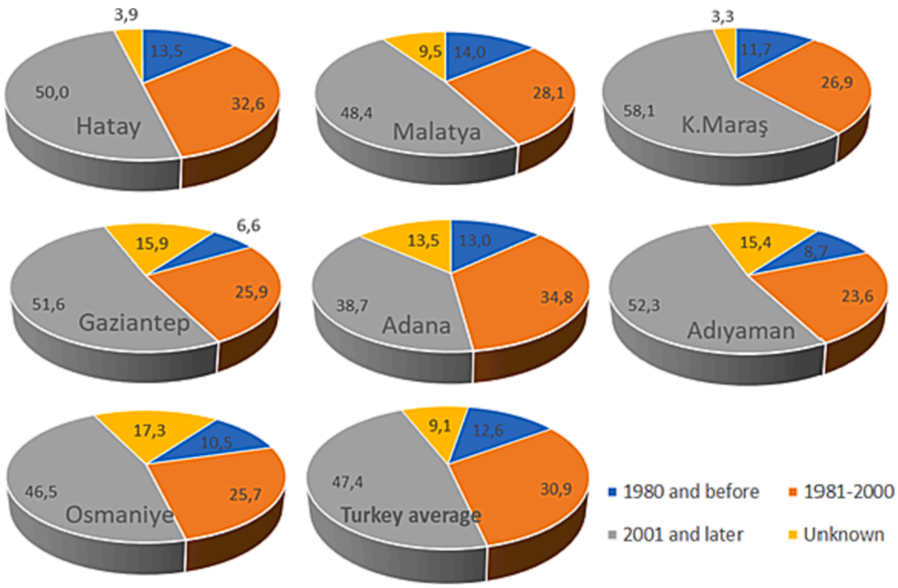


Fig. 4. Number and proportion of households by provinces and construction year of the residential building as of 2021, (adapted from statistics, Tuik [52]).

observations made after the earthquakes and the increase in earthquake records. When the earthquake codes from past to present are examined, it is seen that the total horizontal earthquake load affecting the structure is defined with a simple formulation in the first published codes. Table 2 summarizes how to calculate earthquake design loads according to the last four codes. In addition, the years in which the earthquake hazard map was changed are shown as dotted red line in the table. Since most of the existing building stock in Turkey was built after 1975, it will be sufficient to examine the TEC-1975 [54] and later codes. In Table 2, S represents spectrum coefficient, C_0 depends on earthquake zone and can be equal to 0.10, 0.08, 0.06 and 0.03. K and I represent type of structural system structure importance coefficient between 1 and 1.5, respectively. In addition, A_0 is the effective ground acceleration coefficient depending on the earthquake region, and S(T) is the spectrum coefficient. Since the earthquake hazard map, which entered into force in 1996 [55] and divides Turkey into 5 earthquake zones, remained in effect until 2018, the design load in the TEC- 2007 [19] code remained the same as in TEC-1998 [18]. In 2018, together with the earthquake code, the earthquake map was changed extensively and the "New Earthquake Hazard Map-2018″ [56] was implemented (Fig. 5), which was prepared for different earthquake levels and considering the distance to the active fault zones according to the geographical coordinates. In Fig. 5, earthquake hazard maps that have come into force in certain periods from 1972 to the present are shown. As can be seen, until the 1996 map, a significant part of the close vicinity of the EAF was defined as lower risk than the close vicinity of the NAF. Together with TEC-1998 [18], which came into force same year of the 1999 Marmara (7.6 M_w) and 1999 Düzce (7.2 M_w) earthquakes Turkey has achieved a modern earthquake code.

In TEC-1998 [18], for the first time, a horizontal elastic design response spectrum for 5% damping was defined based on soil classes, and the minimum design conditions were determined separately by defining high and normal ductility conditions. This change in TEC-1998 took place in parallel with the gradual inelastic behavior of seismic design in the world. Before 1990s, controls of member capacity according to section forces and controls of storey drift ratios, which were mostly conducted according to the linear elastic method, were replaced by deformation based controls with the end of the 1990s. After that, Priestley *et al.* [57] and Priestly [58] conducted pioneering studies in the definition of earthquake performance based on displacement and structural member's capacity. For this reason, it is clear that the buildings built according to the TEC-1975 [54] before 1998 were produced far from concepts such as capacity design and sufficient ductility concept.

In TEC-1998 [18], only one ground motion intensity level was considered, called design earthquake. This corresponds to an earthquake with a 10% probability of exceedance in 50 years and a return period of 475 years. With the TEC-1998 [18], the minimum design criteria were improved, the importance of shear walls increased, especially for displacement control, and the criteria for providing ductile behavior (the condition of the columns being stronger than the beams, the limitation of the axial load level during earthquakes in the columns, the longitudinal and transverse reinforcement details, etc.) took place in the code. While structural

Table 2
Earthquake design load calculation methods in different earthquake codes.

Code		TEC-1975		TEC-1998	TEC-2007	TBEC-2018	
Earthquake Design Load	**1972 Map**	$V_t = C.W$ $C = C_0.K.S.IS = \frac{1}{0.8 + T - T0}$	**1996 Map**	$V_t = C.W$ $C = \frac{A_0.I.S(T)}{R_a(T)}$	$V_t = C.W$ $C = \frac{A_0.I.S(T)}{R_a(T)}$	$V_t = C.W$ $C = \frac{S_{ae}(T)}{R_a(T)}$	**2018 Map**

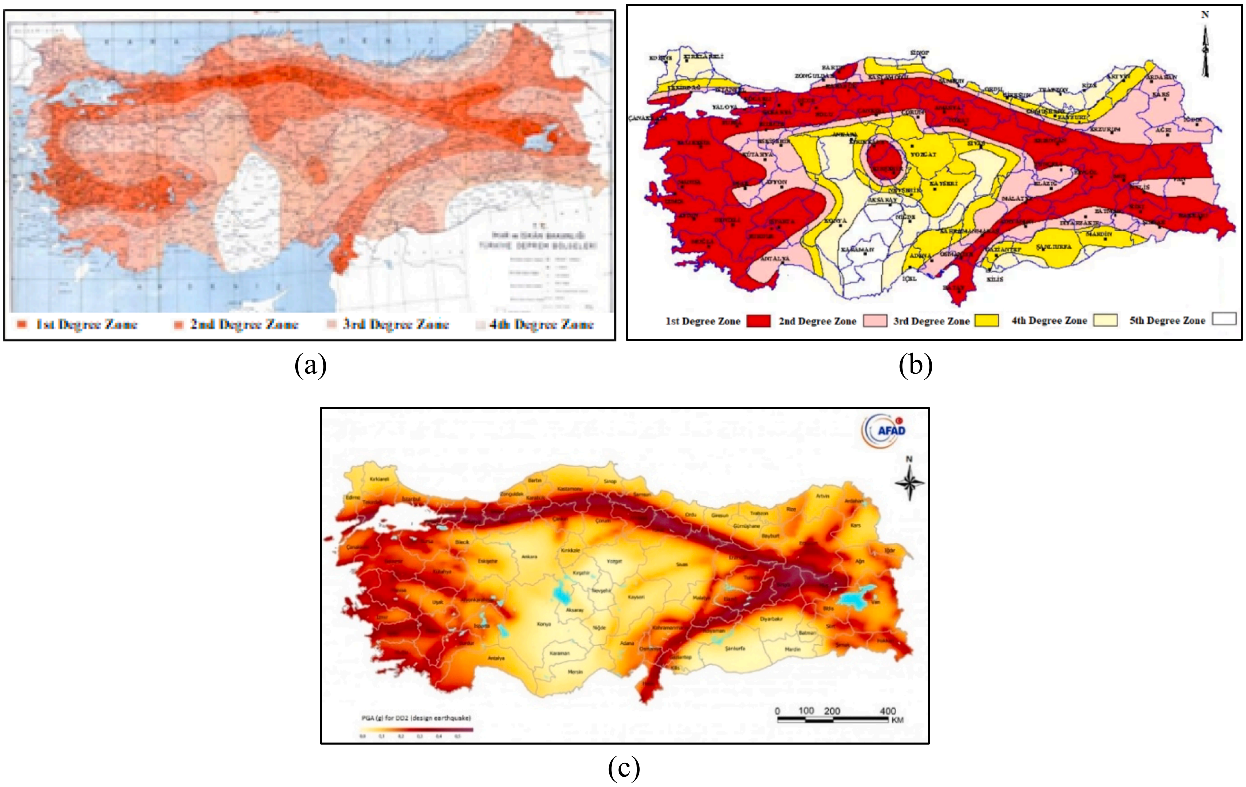


Fig. 5. Seismic zoning maps of Turkey (a) 1972 Map [59] (b) 1996 Map (b) [55] 2018 Map [56].

analyses were performed based on the single-mode equivalent earthquake load method in TEC-1975 [54] and before, more realistic methods such as modal analysis and time history analysis were made mandatory for some buildings with TEC-1998 [18]. The first seismic code in which the time history analysis was implemented was the seismic code adopted in the former Yugoslavia in 1981 [60].

Along with the developing material properties, the lower limit of concrete strength has been increased in each revision. The lower limit of concrete compressive strength, which was 14 MPa in reinforced concrete buildings to be built in earthquake zones until 1998, was increased to 16 MPa with TEC-1998 [18], 20 MPa with TEC-2007 [19] and 25 MPa with TBEC-2018 [20]. While S220 (plain bar, characteristic yield strength (f_{yk}) is 220 MPa) steel was used in columns and beams until 1998, the use of reinforcement under S420 class was prohibited with TEC-1998 [18]. With TBEC-2018 [20], the reinforcement steel grade has been changed to B420c (called as more ductile steel). However, the fact that the lower limits of concrete compressive strength were kept low in the previous codes and that concrete production and casting inspections were not carried out adequately, caused the most important problem in the existing building stock in Turkey to be insufficient concrete and reinforcement quality. For example, according to New Zealand codes, envisaged compressive strength of concrete as 20 MPa in the period of 1970–1981, 25 MPa in the period of 1982–1994, and 30 MPa after 1995.

The linear elastic method becomes disabled by the significant reduction in the rigidity of the structure under earthquake, together with the occurrence of the first considerable damage to the structure (eg, the first plastic hinge in the columns) [61]. For this reason, non-linear methods, which also determine the damage distribution and damage levels under earthquake effects, have started to come to the force with TEC-2007 [19]. The inelastic behavior of the structural elements is modeled with the methods called non-linear methods (single-mode static pushover analysis and multi-modal pushover analysis) and the damage to the structural elements is calculated for each loading stage within a dynamic stiffness matrix. In this way, damage distributions and possible collapse scenarios within the building have begun to be obtained. Strengthening of existing buildings and performance analysis are included in earthquake code with TEC-2007 [19]. With TEC-2007 [19], non-linear calculation procedure started to be used in the evaluation and reinforcement of existing buildings. With TBEC-2018 [20] which is still in force, like other modern seismic codes (ASCE 7-16 [62], EC-8 [63], NZSEE-2017 [64], NRC 2015 [65]), force-based design and deformation-based design in existing and new buildings started to be used together.

EC-8 [63], Brazilian [66], Portuguese [67], Greek [68] and Bulgarian [69] standards consider the earthquake effect which has a probability of 10% being exceeded in 50 years and a return period of 475 years for the condition of non-collapse. While Italian code defines two seismic earthquake levels for the design of conventional buildings (recurrence period of 50 years and 475 years), the Chilean Standard [70] does not clearly define the recurrence periods considered [71]. ASCE 7–16 [62] and NRC 2015 [65] define the most severe earthquake effect with a 2% probability of being exceeded in 50 years (corresponding to a return period of 2475 years), but in the design of ordinary structures, the seismic design forces obtained for the relevant earthquake are reduced by 2/3. In Turkey, with TBEC-2018 [20], different earthquake levels have been defined with a return period of 2475 years (DD1), 72 years (DD3) and 43 years (DD4), except for the design earthquake called DD2, with a 475-year return period. In addition, the multiple performance target approach, which varies according to the purpose of use of the buildings, has started to be applied for the new structures to be built. Depending on the purpose of use of the building and the earthquake risk in its location, different performance targets have been defined under different earthquake levels. In addition, until TBEC-2018 [20], only the horizontal effect of the earthquake in two perpendicular directions was considered, with TBEC-2018 [20] the vertical elastic design spectrum was also defined, and vertical earthquake loads began to be considered during the design phase.

In Turkey, the elastic design acceleration spectrum, first defined by TEC-1998 [18], has largely accepted the definition in ASCE 7-16 [62] with TBEC-2018 [20]. Spectra defined in Turkish codes since 1998 have been defined for 5% damping ratio. Most earthquake codes around the world also define the design spectrum for a 5% damping ratio. Eurocode-8 [60], on the other hand, defines a correction factor for damping values different from 5%, denoted as η, when describing spectra. The Japanese code multiplies the spectrum by a value F_h a function based on the effective viscous and hysteretic damping [71].

It is known that local soil conditions significantly affect the response spectrum. In earthquake codes, acceleration, velocity and displacement spectral values are determined depending on the period of the building. Local soil classes also have a great effect on spectral values. In the 1975, 1998, 2007 earthquake codes, 4 different soil classes were defined (Z1, Z2, Z3 and Z4) dependent on the topmost soil layer thickness. In TBEC-2018 [20], the soil classes were expanded to 6 (ZA, ZB, ZC, ZD, ZE and ZF such as ASCE 7–16 [62] and EC-8 [63]), and site-specific analyzes and evaluations were requested for the weakest ground, ZF. The ground has been classifed in terms of average shear wave velocity $(V_s)_{30}$, average standard penetration blows $(N_{60})_{30}$ and average undrained shear strength $(c_u)_{30}$.

All earthquake codes recognize the necessity of classifying structures according to human density and importance. The purpose of this definition is to design important structures such as require immediate use after an earthquake, structures where the loss of life may be higher or strategic, within a greater margin of safety. This difference in safety is defined in the standards simply by applying a multiplying factor I to the seismic forces. As in TBEC-2018 [20], this coefficient is defined as 1 for residences in all regulations.

It is obvious that it would not be realistic to consider the behavior of the structure as elastic under a sudden and severe impact such as an earthquake. Therefore, it is aimed that the structures have sufficient deformation capacity under inelastic behavior and dissipate a large amount of energy. As long as the targeted ductility capacity is assured, it is possible to consider the transformation of the elastic spectra in design spectra, in which the considered ductility is implied [71]. For this purpose, all earthquake codes define reduction coefficients that consider the inelastic behavior, according to the construction material type and ductility capacity. In TBEC-2018, different ductility levels are defined for each type of structure under the name of high, mixed and limited ductility levels. In TBEC-2018 [20], the structural behavior coefficient for reinforced concrete building systems at 3 different ductility levels varies between 4 and 8. The reduction coefficients are calculated according to these behavior coefficients, whether the dominant vibration period of the structure is in the short or long period region, and the structure type. In other codes, ductility classifications are made under different names. ASCE 7-16 [62] classifies RC frame building systems as ordinary, intermediate and special and applies reduction coefficients ranging from 3 to 8. NZS divides structures into ductile, limited ductility and nominal ductility, and the reduction factors vary between 1.16 and 8.57 according to the short and long period region. EC-8 [60] classifies the building ductility as low, medium

Table 3
Ground motion characteristics of the earthquakes as recorded by some closest stations to the epicenter and PGA values for different exceedance probabilities in 50 years in TBEC-2018.

Province	District	R_{EPI} (km)	Measured peak ground acceleration PGA (g)			PGA according to TBEC-2018 (g)			
						Earthquake level in TBEC-2018 and probability of exceedance in 50 years (return period)			
						2% (2475 year)	10% (475 year)	50% (72 year)	68% (43 year)
			N–S	E–W	Vertical	DD1 (largest)	DD2 (standard)	DD3 (Frequent)	DD4 (service)
Pazarcık Earthquake (M_w = 7.7)									
K.Maraş	Dulkadiroğlu	28.40	0.46	**0.48**	0.36	0.870	0.470	0.158	0.105
Gaziantep	Nurdağı*	29.79	**0.56**	**0.60**	0.32	0.972	0.496	0.156	0.104
Gaziantep	Islahiye	48.30	**0.66**	**0.64**	0.59	1.06	0.563	0.166	0.107
Osmaniye	Center	72.18	0.14	0.19	0.14	0.60	0.31	0.11	0.08
Adıyaman	Tut	96.48	0.24	0.17	0.23	0.81	0.44	0.15	0.09
Malatya	Akçadağ	143.07	0.11	0.14	0.05	0.86	0.40	0.11	0.07
Hatay	Antakya	143.54	**1.21**	**1.02**	0.96	0.85	0.437	0.146	0.099
Adana	Çukurova	155.36	0.05	0.04	0.02	0.44	0.23	0.08	0.06
Elbistan Earthquake (M_w = 7.6)									
Adıyaman	Tut	68.73	0.12	0.13	0.07	0.81	0.44	0.15	0.09
Malatya	Akçadağ	70.17	**0.45**	0.39	0.29	0.86	0.40	0.11	0.07
Adana	Bahce	116.24	0.07	0.05	0.03	0.719	0.391	0.140	0.095
K.Maraş	Dulkadiroğlu	124.27	0.07	0.08	0.04	0.870	0.470	0.158	0.105
Gaziantep	Islahiye	131.79	0.04	0.05	0.02	1.06	0.563	0.166	0.107
Osmaniye	Center	140.65	0.05	0.07	0.03	0.60	0.31	0.11	0.08
Adana	Çukurova	205.81	0.03	0.02	0.03	0.44	0.23	0.08	0.06
Hatay	Antakya	229.36	0.02	0.02	0.02	0.85	0.437	0.146	0.099

and high. In medium and high ductility regions, EC-8 [60] considers lower reduction factors than ASCE 7-16 [62]. For structures with low ductility, EC-8 [60] recommend values close to each other, while ASCE 7-16 [62] and TBEC-2018 [20] recommend at least twice the values of these codes [72].

6. The reasons of heavy damages and collapses in the south and southeast of Turkey during the last earthquakes

As a result of the field investigations and analyzes, also considering that the earthquakes caused about 52.000 casualties and over 200.000 collapsed/severe damaged of buildings in 11 provinces covering an area of 108.812 square kilometers, it has been understood that severe demolution cannot be attributed to a single reason. The causes of damage observed as a result of field investigations

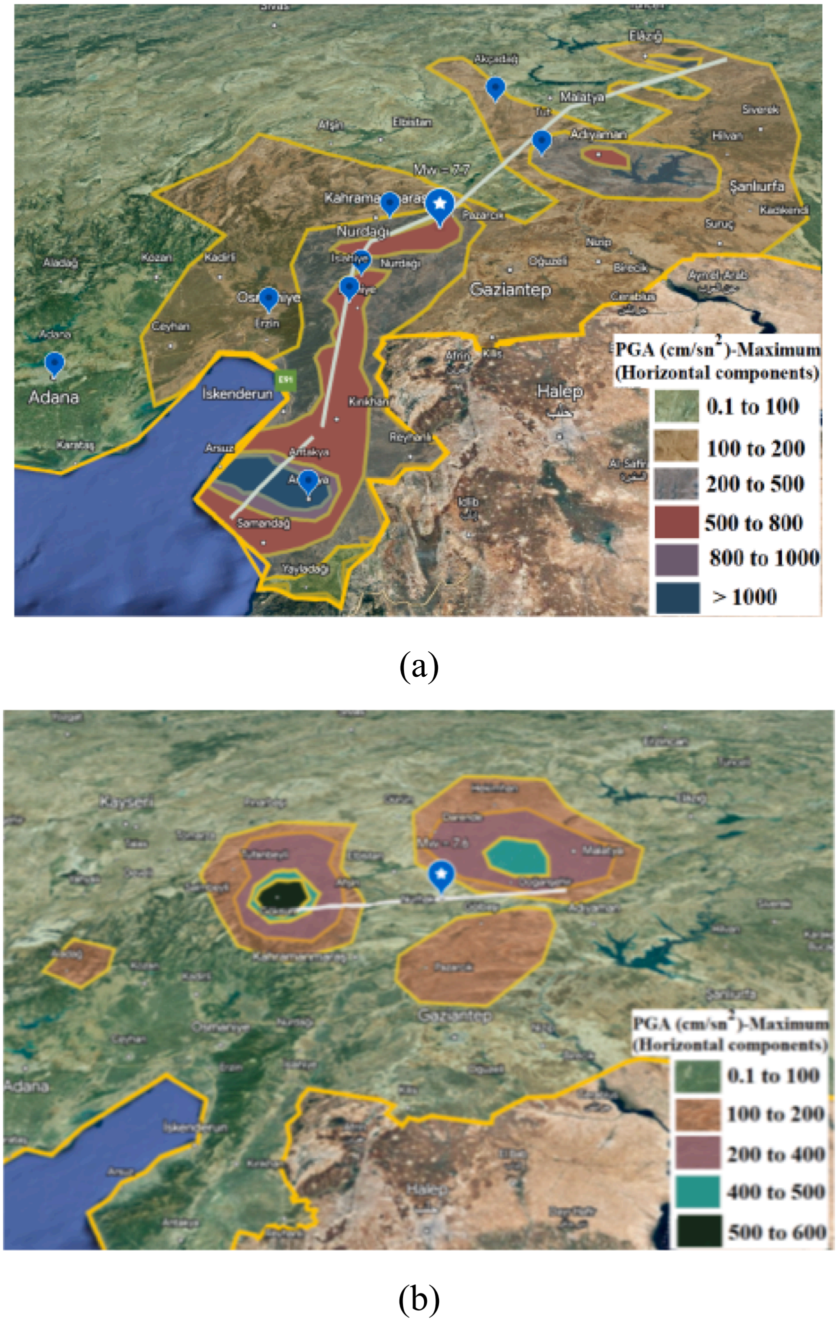


Fig. 6. Contour map of the peak ground acceleration (a) Magnitude-7.7 earthquake (b) Magnitude-7.6 earthquake (for maximum values of north–south and east–west horizontal components) (Prepared from Google Earth).

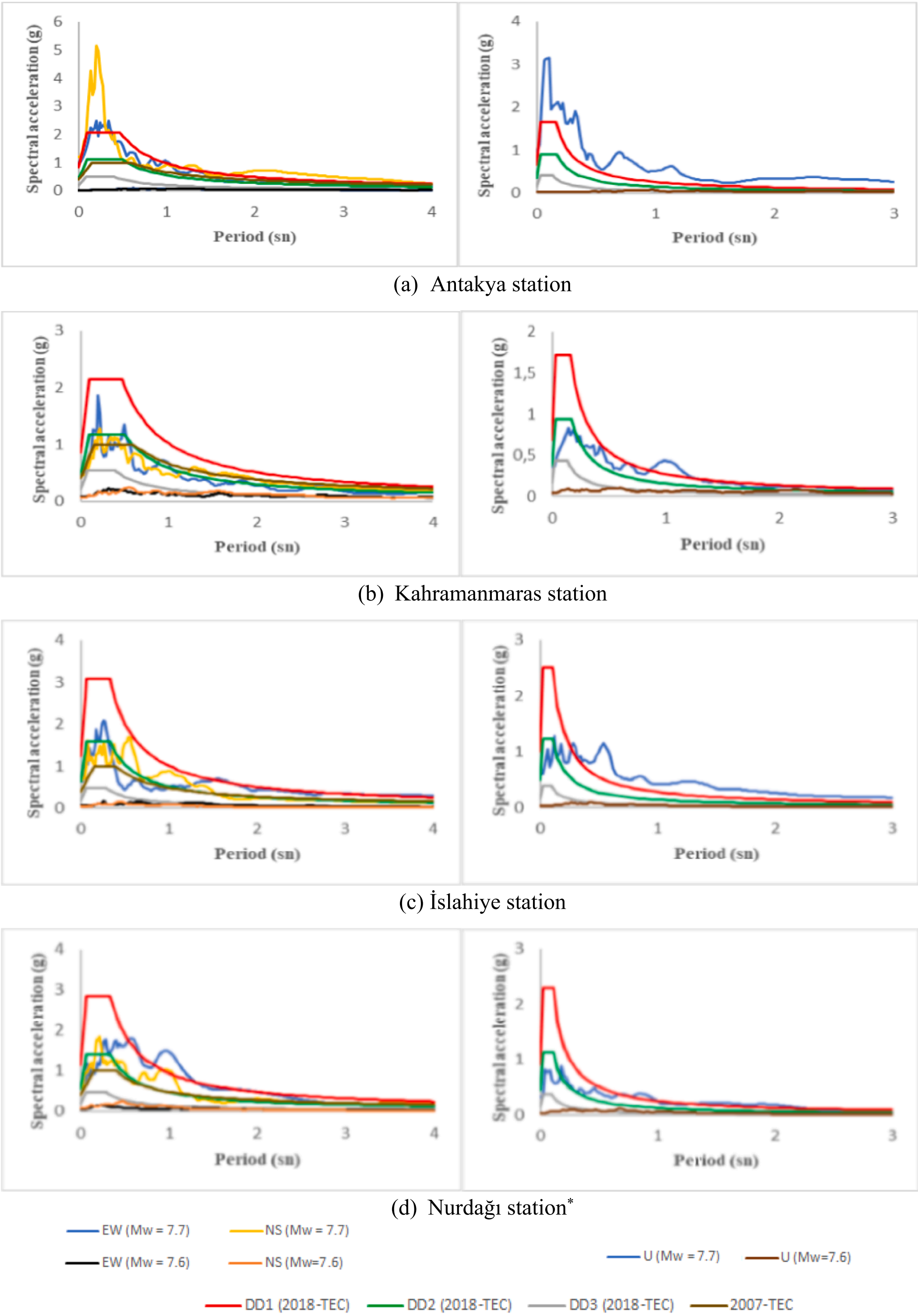


Fig. 7. Comparison of the horizontal elastic design spectra with response spectrum curves for different locations (damping ratio of 5 %) (* For the M_w 7.6 earthquake, the closest station to Nurdağı was considered.).

throughout the region are summarized different sub-headings.

6.1. Extraordinary effects of earthquakes and earthquake hazard map deficiencies

As stated in the previous section, TBEC-2018 [20] defines 4 different earthquake levels as the most severe earthquake with a return period of 2475 years (DD1), the design earthquake with a return period of 475 years (DD2), the frequent earthquake with a return period of 72 years (DD3) and the service earthquake with a return period of 43 years (DD4). It should be noted that residential buildings are designed considering the 475 year return period spectrum (DD-2). Therefore two major earthquakes struck the same area with an interval of 9 h, which is rarely seen in the world, has created extraordinary demands on the structures.

The magnitude-7.7 and magnitude-7.6 earthquakes (M_w 7.7 and M_w 7.6) were recorded by 290 and 267 stations, respectively by AFAD (Disaster and Emergency Management Presidency). Within the scope of this study, 8 locations where approximately 98% of the causalities occurred and where the greatest destruction was experienced during earthquakes and the recording station data in these regions were considered (Table 3).

Distribution of horizontal peak ground acceleration (PGA, in units of centimeters per second squared) in the south and southeast of Turkey, affected area by the earthquakes, and the locations of the recording stations considered in the study are shown in Fig. 6. The acceleration records were obtained from the AFAD [50] system as of 20.02.2023. The blue pins in Fig. 6.a. show the location of the recording stations. Also Table 3 shows the recorded PGA values for both earthquakes at these stations and also shows predicted PGA values for different earthquake levels (DD-1, DD-2, DD-3 and DD-4) in TBEC-2018 [20] according to local soil conditions.

As can be clearly seen in Fig. 6, the first earthquake with a magnitude of 7.7 (M_w 7.7) on the Eastern Anatolian Fault struck southern Turkey and peak ground acceleration values were recorded around the province of Hatay, especially around the Antakya district. The second earthquake with a magnitude of 7.6, which occurred on the Çardak - Sürgü fault 9 h after the first earthquake, struck the north of Kahramanmaraş and the surrounding of Malatya province. As stated before, according to TBEC-2018 [20], the design of residential buildings is made according to the DD2 design earthquake. In Table 3, the values at which the design earthquake predictions are exceeded in the considered locations are shown in bold and underlined. As can be seen, in the first earthquake with a magnitude of 7.7 (M_w 7.7), the design accelerations in both directions in Nurdağı, İslahiye and Antakya districts were exceeded, and the accelerations recorded in Antakya even exceeded the predictions made for the largest earthquake (DD1).

The comparison of the 5%-damped horizontal response spectra of the recorded motions at locations shown in Fig. 6.a during the earthquakes with the site-specific design spectra defined in the TBEC-2018 [20] are given in Fig. 7.

When the spectral acceleration values obtained from the records in the horizontal and vertical directions in the Antakya District of Hatay Province are compared with the horizontal elastic design spectrum values calculated for the DD-1, DD-2, DD-3 and DD-4 earthquake levels from the earthquake hazard map; it is seen that the spectral acceleration values of the earthquake that took place have acceleration values in the 0.10–0.50 sec period range, which are considerably higher than the values defined in the code. A similar situation is observed for the Dulkadiroğlu district of Kahramanmaraş between 0.25 and 0.70 s and the measured acceleration values exceed the DD-2 level.

In Gaziantep-İslahiye district, it was observed that the horizontal earthquake acceleration exceeded DD-2 for all period values above 0.25 s, and it was above the DD-1 level for periods over 1.0 s in vertical earthquakes. In Gaziantep Province Nurdağı district, it is seen that the horizontal acceleration values are above DD-1, especially after 1.0 s, and the vertical acceleration exceeds DD-2 after 0.35 s. Earthquakes have accelerations above the DD-2 level in structures with more than 2.0 periods in provinces such as Adana and Osmaniye. Similarly, acceleration values of buildings with a period of more than 1.20 s in Adıyaman province exceeded the design earthquake DD-2.

Most of the reinforced concrete buildings that collapsed and heavily damaged in the Feb. 6 earthquakes are 6–13 storey structures. TBEC-2018 [20] states that the dominant natural vibration period of the reinforced concrete building can be calculated approximately by the $C_T.H_N^{3/4}$ empirical relation. The period formula (and similar) given in TBEC-2018 is actually used in other earthquake codes around the world. Similar formulas are available in ATC 3-06 [73], EC-8 [63], ASCE 7-16 [62]. In the formulas given in the seismic codes, the building height is generally taken as the main parameter. This concept was developed by Goel and Chopra [74] from the measurements they made between the 1971 San Fernando earthquake and the 1994 Northridge earthquake. Similarly, many

Table 4
Approximate fundamental period formulas for reinforced concrete frame buildings.

Based on Building Heights (H)			Based on Number of Stories (N)		
Researchers	Formulation	Origin	Researchers	Formulation	Origin
Goel and Chopra [74]	$T_1 = 0{,}0466\ H^{0{,}9}$	1971 San Fernando EQ and 1994 Northridge EQ (USA)	Navarro et al. [83]	$T_1 = (0{,}049 \pm 0{,}001)N$	Adra Town (Spain).
Hong and Hwang [78]	$T_1 = 0{,}0294\ H^{0{,}804}$	Taiwan	Gallipoli et al. [84]	$T_1 = 0{,}016\ N$	Italy, Slovenia, Croatia and North Macedonia
Guler et al. [79]	$T_1 = 0{,}026\ H^{0{,}9}$	Turkey	Michel et al. [85]	$T_1 = 0{,}013\ N$	Grenoble City (France).
Pan et al. [80]	$T_1 = 0{,}02372\ H^{0{,}8325}$	Sumatran Earthquake (Singapore)	Ditommaso et al. [86]	$T_1 = 0{,}026\ N$	L'aquila Earthquake (2009) (İtaly)
Kaplan et al. [81]	$T_1 = 0.0195H$	Turkey			
Crowley and Pinho [82]	$T_1 = 0.038H$	Greece, Italy, Portugal, Romania, and ex-Yugoslavia			

researchers have suggested a period formula based on building total height. Other researchers and some codes suggested simplified equations based on the number of stories. For instance NEHRP-94, UBC, TEC-1998, EGC-93 [18,75–77] proposed the period formula as 0.1 N. Table 4 shows the suggested approaches. The formulas in Table 4 have been developed for reinforced concrete buildings without infill walls and shear walls. There are changes in the formulations due to the increased rigidity compared to shear walls and infill walls.

In the TBEC-2018 [20] equation, C_T is taken as 0.1 for buildings whose structural system consists only of reinforced concrete frames, while 0.07 is taken for buildings with shear walls and frames. Therefore, considering that the average storey height is 3 m, the collapsed buildings can be considered to be in the range of 0.5 s−1.5 s period. For this reason, the proportional comparison of the loads acting during earthquakes and the design loads for residential buildings with a period range of 0.1–1.5 s designed considering the different earthquake codes and earthquake hazard maps explained in section 3.1, is given in Figs. 8-11, for the 8 regions considered. The local soil classes of the recording stations were considered while making the comparisons. As stated in Section 3.1, the soil classification in TBEC-2018 [20] has been changed according to the previous codes. For this reason, while making the comparisons, the common features that define the soil classifications were analyzed and the soil class matching specified in Table 5 and also included in the literature was decided [87]. A value read from the vertical axis in the graphics shows the ratio of the load acting during the earthquake to the design load, on the structure having a certain period in that location. Also in the charts, a value of 1.0 is shown as a reference line. The points below this line indicate that the design load of the building having that period was not exceeded, while the points above it indicate that the design load was exceeded and the rate of exceedance. It should be remembered that in the codes before TBEC-2018 [20], the vertical elastic design spectrum was not defined and only the horizontal effect of the earthquake was considered. Therefore, the vertical load comparison was made only for the structures designed after 2018.

As can be understood from the examination of the graphics, during the first earthquake with a magnitude of 7.7 (M_w 7.7), the lateral load acting on all buildings with a period between 0.1 s–1.5 s and built after 1975 in the Antakya district of Hatay province, was above the design load at rates varying between 1.07 and 11 times. Similarly, in Nurdağı and İslahiye districts of Gaziantep province, where the highest loss of life occurred after Antakya, the design lateral loads were exceeded for all structures regardless of the year they were built and the vibration period. In these regions, the design load exceedance rate is between 1.10 and 4.33 times and 1.02 and 3.98 times, respectively.

For the M_w 7.7 earthquake, when the structures designed after 2018 are compared with the structures designed between 1998 and 2018, it is seen that the earthquake load/design load ratio decreases under 0.6 sec period in Antakya and increases in structures above 0.6 sec. In Nurdağı, on the other hand, the rates in buildings with a vibration period of over 0.9 sec and designed between 1998 and

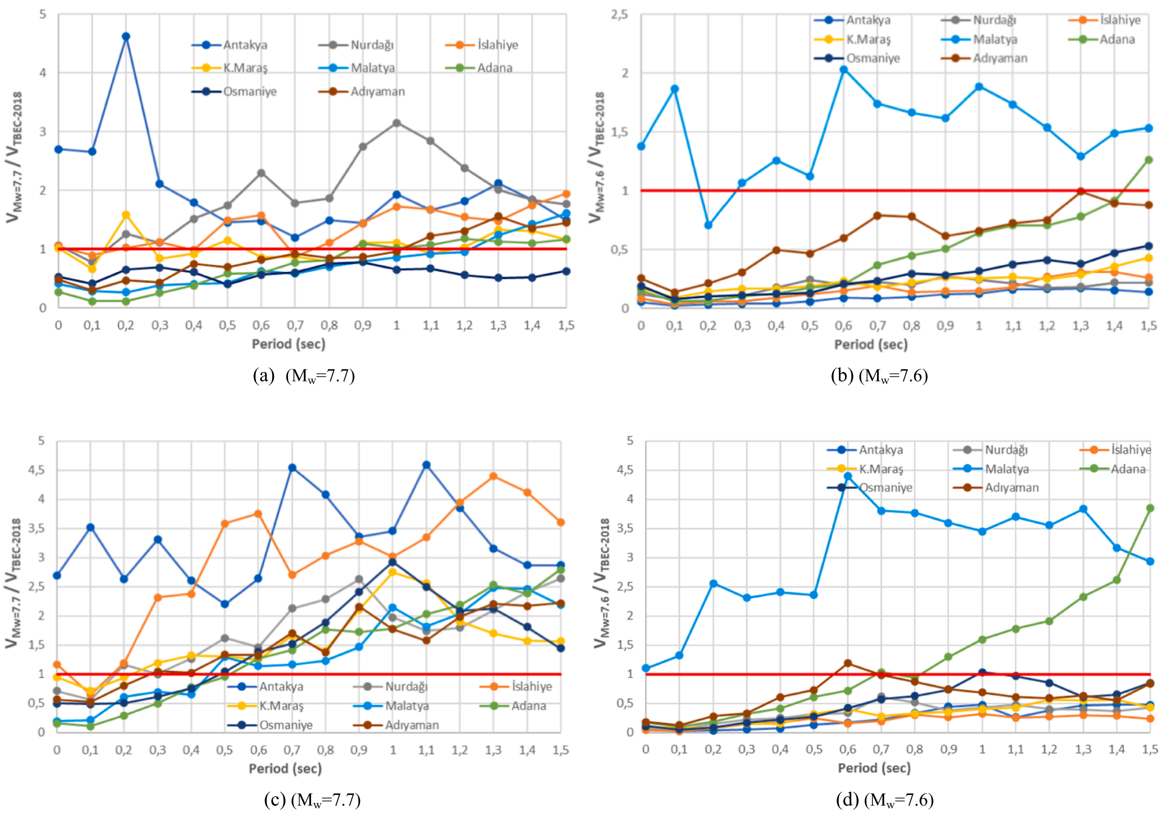
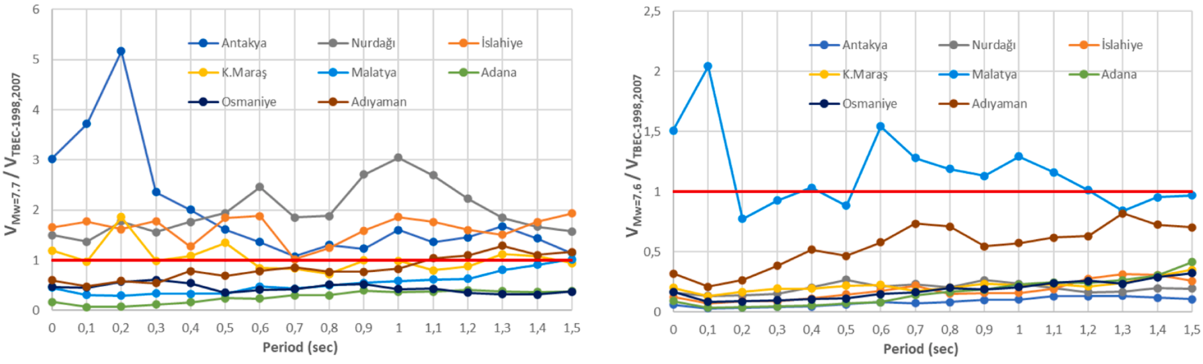
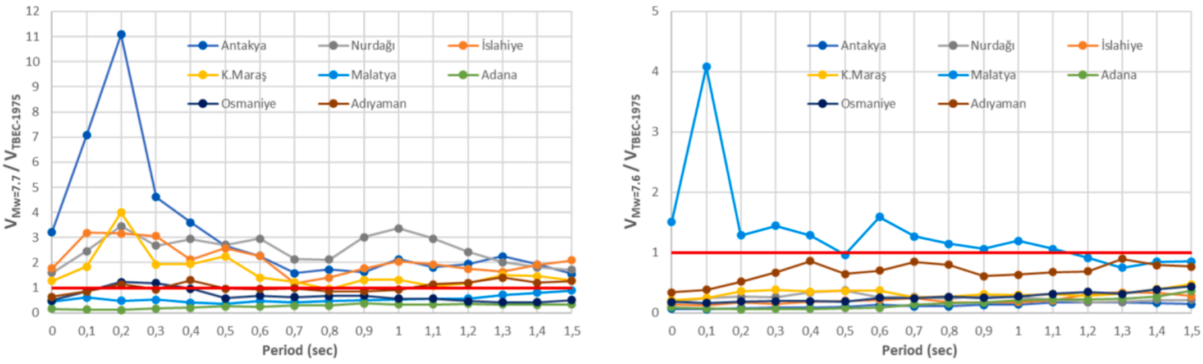


Fig. 8. (a-b). Comparison of Pazarcık (M_w = 7.7) and Elbistan (M_w = 7.6) earthquake load and horizontal design load for residential buildings designed after 2018. (c-d) Comparison of Pazarcık (M_w = 7.7) and Elbistan (M_w = 7.6) earthquake load and vertical design load for residential buildings designed after 2018.



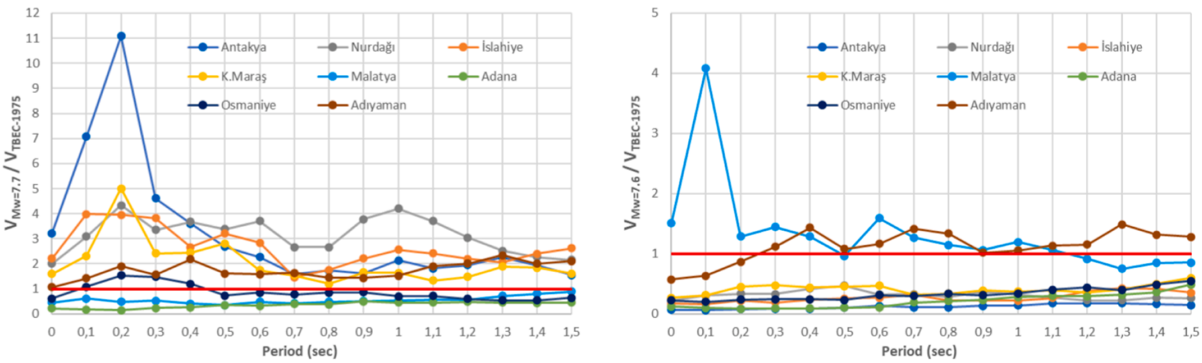
(a) (M_w=7.7) (b) (M_w=7.6)

Fig. 9. (a-b). Comparison of Pazarcık (M_w = 7.7) and Elbistan (M_w = 7.6) earthquake load and horizontal design load for residential buildings designed between 1998 and 2018.



(a) (M_w=7.7) (b) (M_w=7.6)

Fig. 10. (a-b). Comparison of Pazarcık (M_w = 7.7) and Elbistan (M_w = 7.6) earthquake load and horizontal design load for residential buildings designed between 1996 and 1998.



(a) (M_w=7.7) (b) (M_w=7.6)

Fig. 11. (a-b). Comparison of Pazarcık (M_w = 7.7) and Elbistan (M_w = 7.6) earthquake load and horizontal design load for residential buildings designed between 1975 and 1996.

2018 are lower than after 2018. The lower the ratios mean that the design loads considered are higher. Almost all buildings designed after 2018 in İslahiye remained on the safer side compared to other periods. At the location of the Dulkadiroğlu station in Kahramanmaraş, the lowest rates were obtained for the period between 1998 and 2018 in the structures with a vibration period of over 0.6 s. Under the 0.6 s vibration period, the design after 2018 remained on the safer side. For all the buildings designed between 1975 and 1998 in Kahramanmaraş, the design lateral loads were exceeded. Other provinces remained on the safer side compared to the

Table 5
Matching of soil classes.

Seismic Codes	Soil Classes			
TEC-1975,TEC- 1998,TEC-2007	Z1	Z2	Z3	Z4
TBEC-2018	ZA,ZB	ZC	ZD	ZE

above-mentioned regions. It is seen that the design lateral load of almost all the buildings in Osmaniye and Adana provinces has not been exceeded. For vertical loads in the M_w 7.7 earthquake, similarly, the design load was exceeded at rates varying between 2.20 and 4.59 times in all buildings in Antakya. In other regions, it is seen that the design vertical loads are exceeded in structures with a vibration period of more than 0.5 s. For the M_w 7.6 earthquake, it is seen that the most affected provinces are Malatya and Adıyaman. Design horizontal and vertical loads were generally exceeded in Malatya-Akçadağ location. On the other hand, it is seen that the buildings built between 1975 and 1996 in Adıyaman remained unsafe, while the design loads were not exceeded in the buildings in other provinces. However, it is possible that some of the structures that were heavily damaged in the first earthquake were collapsed in the second earthquake with a magnitude of 7.6 (M_w 7.6).

As stated before, the results of Figs. 8-11 for structures with 1 sec and 1.5 sec periods are also summarized in Fig. 12 and Fig. 13, since the buildings where casualties are experienced in earthquakes in all provinces are generally 6–13 storeys.

As it can be seen, while the design loads are exceeded at the highest rates in Gaziantep-Nurdağı district in a structure with 1 sec period, the most unfavourable rates are obtained in Gaziantep-İslahiye district for a structure with 1.5 sec period.

6.2. Fragility analysis

Seismic fragility analysis (FA) is very important to better define the response of a structure to earthquake. It is used for FAs for assessing potential effects and risks, disaster response planning, and retrofitting strategies [88–96]. In the FA, the probability of damage to structures at certain probability of exceedance (P_f) for a selected earthquake can be determined. In determining these, parameters such as horizontal spectral displacement (S_d), inter-story drift ratio (maximum lateral displacement between two successive floors, normalized to the floor height), global drift ratio (maximum roof deviation normalized to the height of a building) are used. In the literature, it is seen that vulnerability analyzes can generally be obtained by different methods such as expert-based, empirical, analytical and smart system based.

Figs. 14-16 shows simplified fragility curves (FC) for low-mide and high rise buildings, respectively. In the graphs, the vertical axis is P_f (probability of exceedance), and the horizontal axis is spectral displacement. Graphs are given for four different damage groups given in TBEC-2018 [20]. The graphics were obtained for ordinary reinforced concrete frame buildings, which were made especially before TEC-1998 [18] in Turkey and have a high probability of damage. These buildings usually do not have shear walls. Even if there is, it does not contribute significantly to the horizontal rigidity as it remains at a modest level. The gray line in the graphs shows 50% probability. FC graphics were obtained by modeling 2D frame type reinforced concrete buildings with different stories. Each building has its own specific FC. Simplified graphics given in this paper for regular reinforced concrete frames with no infill walls, no short columns, no soil problems. Obviously, FCs will change negatively according to these and similar parameters. The criteria in TBEC-2018 [20] are based on the damage groups given in the graphics.

6.3. Establishment of settlement on weak soil layers without taking necessary precautions

As can be seen in the Fig. 17 the $M_w = 7.7$ earthquake created the greatest ground accelerations in the Antakya district of Hatay province, which is approximately 140 km from the epicenter. Antakya is located in the Antakya-Samandağ graben, which extends to the northeast-southwest, formed by the Karasu Segment. A large part of the city is located on the fill areas at the bottom of the graben.

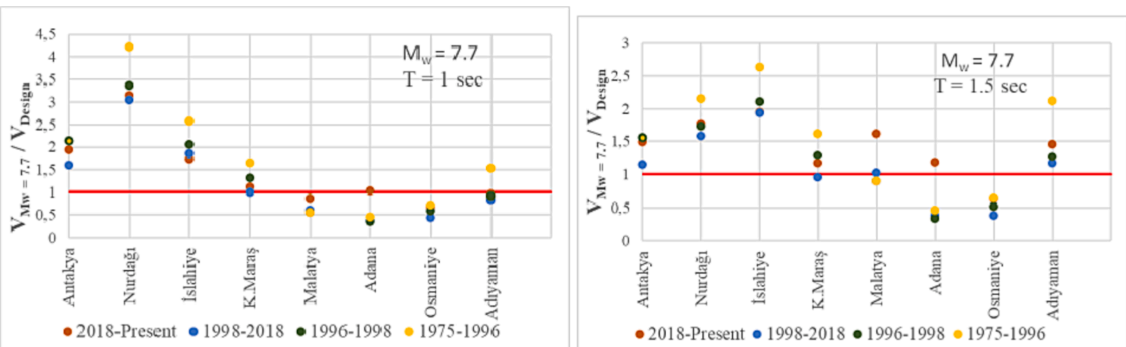


Fig. 12. Comparison of $M_w = 7.7$ earthquake load and design load for structures with 1 sec and 1.5 sec periods constructed at different time intervals (for the maximum horizontal component).

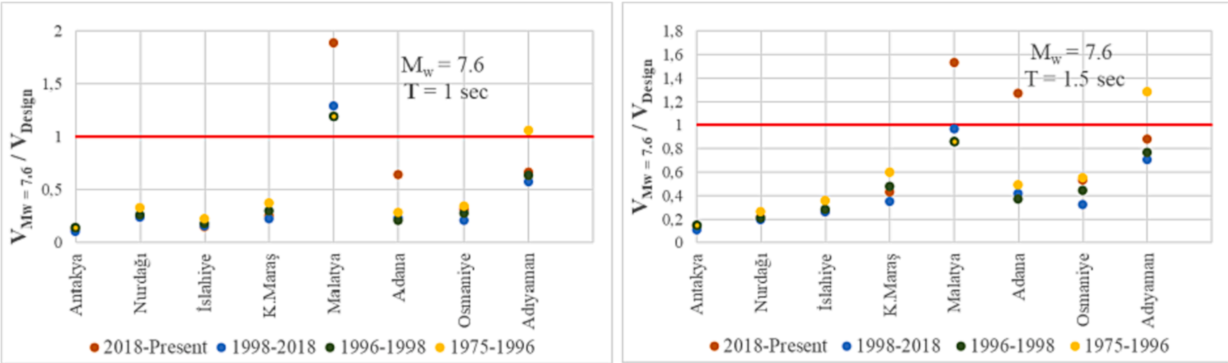


Fig. 13. Comparison of $M_w = 7.6$ earthquake load and design load for structures with 1 sec and 1.5 sec periods constructed at different time intervals (for the maximum horizontal component).

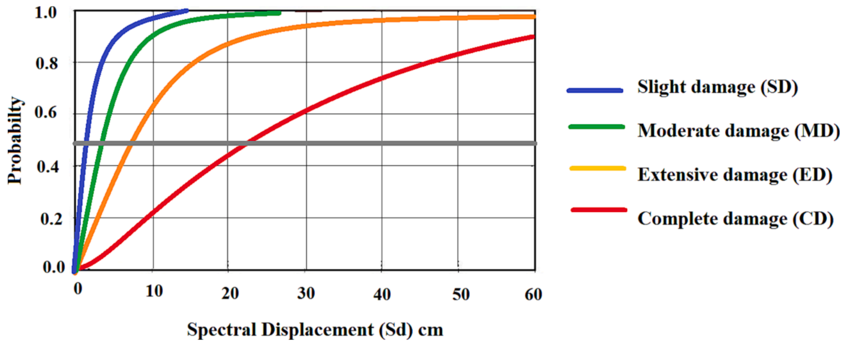


Fig. 14. Fragility curve for low-rise regular reinforced concrete frames.

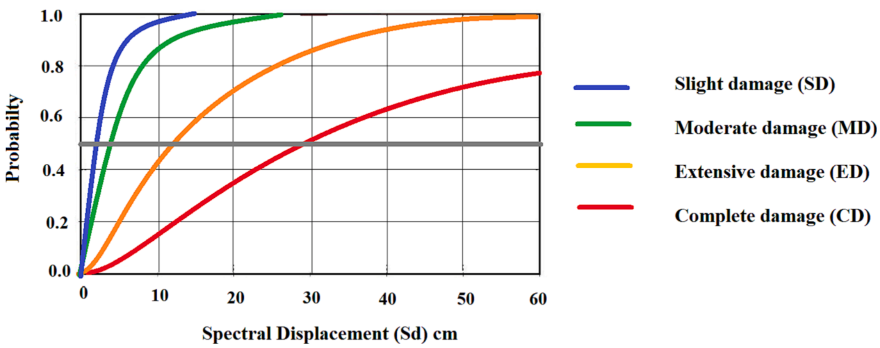


Fig. 15. Fragility curve for mid-rise regular reinforced concrete frames.

Alluviums cover a large area in the graben base [97]. It is known that during longer vibration periods, layers with soft soil properties increase the amplitude and duration of waves propagating from the underlying bedrock to the surface. Thus, structures built on the surface are exposed to a longer and more intense impact. Fig. 17 shows the heavy damage and collapsed buildings around the Asi river of Antakya. Also, in Fig. 18, the soil layers in the Antakya settlement area are shown according to their degree of weakness [97]. The areas in red in the figure show the weakest soils formed by alluvium, cones of deposits and colluvial soils. Weak soils (dark pink areas of figure) consist of quaternary river terraces east and west of the Asi River. It has gravel, sand and occasionally silt lithology. Yellow areas represent less firm soil covering a larger area to the west of the river, and gray areas indicate firm soil. In the figure showing the soil features, the location of the damage images is indicated with a black rectangle. The compatibility of damage distribution with soil characteristics in Antakya reveals the effect of basin amplification. Also it is known that the underground water level varies between 3 and 7.5 m in the boreholes drilled in the alluvium for the purpose of soil investigation [97]. These levels of silty and clayey sands caused the soil to lose its bearing capacity due to the liquefaction, especially in the Iskenderun district of Hatay province and on the banks of the Asi River in Antakya.

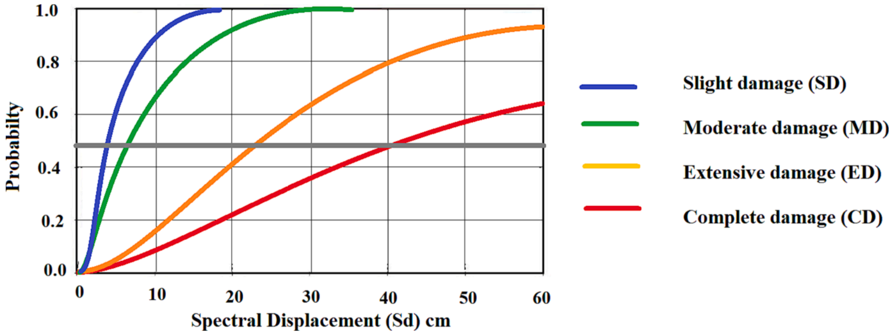


Fig. 16. Fragility curve for high-rise regular reinforced concrete frames.

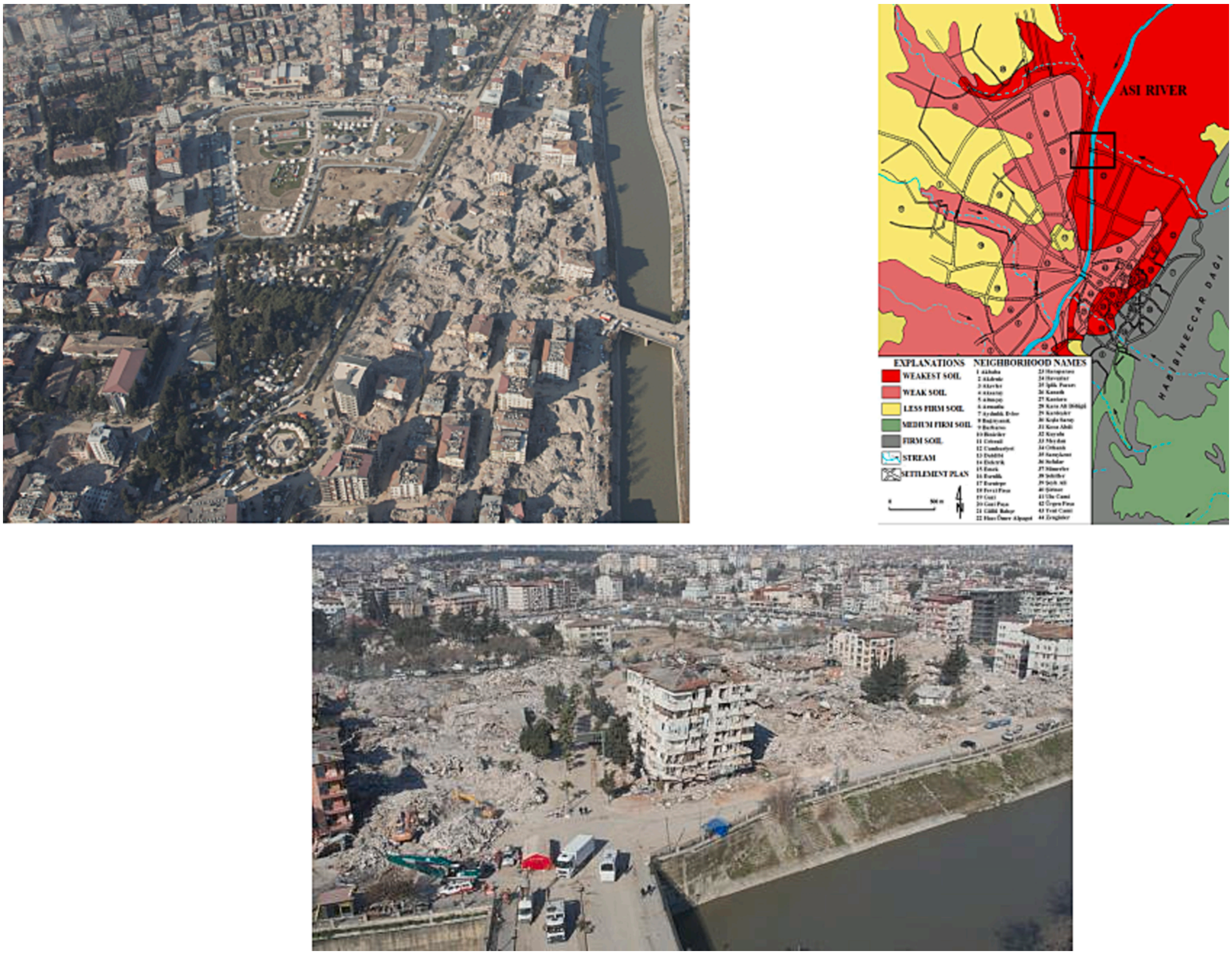


Fig. 17. Heavy destruction around the Asi river in Hatay-Antakya, and Antakya soil characteristics [97].

One of the geological effects of earthquakes is the ground behavior that develops at shallow depths due to the vibrations caused by the earthquake. During an earthquake while seismic waves, especially shear waves, propagate through saturated (below groundwater level) loose soils, causing an increase in pore water pressure. The increased pressure destroys the contact forces that hold the soil grains together, causing the grains to move away from each other and the soil to lose its strength. As the liquefied sand rises to the surface, it weakens the soil, so the soil that loses its strength becomes unable to bear the loads transferred by the structure. Therefore, structures above the ground either lean forward or backward or sink into the ground [98]. Such structural damages due to liquefaction occurred widely, especially in the Gölbaşı district of Adıyaman, which was established by the lake, except for the province of Hatay. Due to the soil liquefaction in Gölbaşı, some buildings were tilted to the side while most of them sank into the soil (Fig. 18).

Fig. 18. Effects of ground liquefaction in some regions during earthquakes.

Fig. 19. Sand deposits formed by soil liquefaction.

Liquefaction is observed at the surface alone or in the form of tandem sand volcanoes and deposits of sand along crevices (Fig. 19). Sand volcanoes occur as the liquefied soil rises to the surface, pushing the surface soil layer to the sides and forming a chimney.

6.4. Design defects and non-compliance with earthquake code conditions

6.4.1. RC moment frames with filled/un-filled ribbed slab

One of the most preferred floor systems in the building stock of the cities affected by the earthquake is ribbed slab due to the reasons such as providing heat and sound insulation with the filling materials used and being economical and easy workmanship due to flat formwork.

In ribbed slab systems where hidden beams whose height is the same as the slab height are used to create a flat ceiling surface, the lateral stiffness of the columns also decreases due to the low bending stiffness of the hidden beams. Therefore, since the relative storey drifts of the buildings increase under severe dynamic loads such as earthquakes, high rigidity elements such as shear walls are needed to prevent the disadvantage created by this situation. In the Earthquakes of Feb. 6, 2023, the ribbed slab structures formed by vertical load bearing elements that did not have sufficient shear walls, did not meet the code conditions, and had limited ductility capacities were destroyed in the earthquakes, causing great loss of life. In addition, due to architectural concerns, placing most of the columns perpendicular to the main road in the plan caused the structures with low lateral drift stiffness to collapse in the weak direction (Fig. 20).

There has never been a ban on the use of this type of slab system in earthquake codes since 1975. Only the codes states that in the

Fig. 20. Heavily damaged infilled ribbed slab RC buildings in Nurdağı and İslahiye.

case of using such slabs, if there is not enough shear walls in the load-carrier system, the ductility level should be taken as limited. In such slab systems, flat beams with very low h/b ratio ((height / width) usually 35/65 cm) are wider than columns and a design contrary to capacity design emerges. However, due to the lack of sufficient torsional rigidity of flat beams, the joint area cannot reach the capacity. In addition, as can be seen in Fig. 20, the narrower columns than the flat beams cause the beam reinforcements to lay outside the column. This situation also creates a bonding problem in connection region.

TBEC-2018 [20] allows buildings with RC moment frames with ribbed slabs without shear walls to be built only in areas where the earthquake risk is not high (areas away from the EAF and NAF faults), however, in this case, the structure is classified as limited ductility and is designed under a much higher seismic load.

6.4.2. Strong-beam weak-column effects

All seismic design codes have specific rules to ensure that seismic damage occurs mostly in ductile beams rather than columns that exhibit more brittle behaviour due to their axial load bearing. To that end, according to TBEC-2018 [20], in structural systems comprised of frames only or of a combination of frames and shear walls, the sum of the ultimate moment capacities of columns framing into a beam-column joint shall be at least 20% more than the sum of the ultimate moment resistances of beams framing into the same joint. In fact, it is seen that this rate is stated as 30% in some codes [99]. Thus, it is ensured that the plastic hinges are not formed in the columns, but in the beams where the ductile behaviour is more effective, and it is ensured that the energy transferred from the ground to the structure during the earthquake is consumed in these regions. Fig. 21 shows some buildings destroyed or heavily damaged by the formation of plastic hinges at the ends of the columns, whose deformation capacity is already insufficient due to the neglect of strong column design in the earthquake zone. In Turkey, it is common for the beams to be stronger than the columns, especially in the structures designed before the TEC-1998 [18] came into force under capacity design concept.

6.4.3. Soft and weak storey

One of the main reasons that cause collapse and damage in reinforced concrete structures during earthquakes, both in Turkey and around the world is the soft / weak storey behaviour of a critical storey [7,100–102]. In Turkey, a significant part of the buildings built on both sides of the main streets are likely to have soft storey behaviour in case of earthquakes. In these buildings, the ground stories are generally used for commercial purposes and glass walls are made for presentation instead of brick infill walls. Especially the buildings where the ground storey heights are higher than the other storeys are at much greater risk. In addition, parking garages can cause soft storey formations due to the need for large and free area.

Brick infill walls, in a way, are the first defence elements in the building against earthquakes. Depending on the strength of the

Fig. 21. Some strong-beam weak-column damages in Hatay-Antakya and K.Maraş.

materials forming the infill walls, they both increase the lateral load carrying capacity of the building and limit the lateral displacement of the system by acting as a pressure bar. As a result, the displacements in the soft storey increase, due to the removed infill walls and depending on the decrease in the lateral stiffness of the columns due to higher storey height. So, if there are not enough shear walls to limit the lateral displacement, either the building collapses on this storey or large permanent displacements occur depending on the deformation capacity of the columns. In TBEC-2018 [20], this irregularity is called rigidity irregularity between neighbouring floors and defined as the situation where the coefficient obtained by dividing the average relative floor displacement rate at any i^{th} floor by the average floor displacement rate at an upper or lower floor is greater than 2 for either of two perpendicular earthquake directions.

In addition, sudden changes in the strength and stiffness of the storeys adversely affect the performance of the buildings during the earthquake. This behaviour, which is called as weak storey in the literature, is called rigidity irregularity between neighbouring floors in TBEC-2018 [20]. In the regulation, it is stated that this irregularity occurs if the ratio (η_{ci}) of the total effective cutting area calculated for one floor to the cutting area calculated for the upper floor is less than 0.8. TBEC-2018 [20] does not allow this coefficient to be less than 0.6 at any floor, and in this case, it obliges to increase the stiffness of that floor by increasing the dimensions of the load bearing members. If this coefficient is between 0.6 and 0.8, the earthquake load reduction coefficient used to reduce elastic earthquake loads is multiplied by $1.25.\eta_{ci}$ and the building is designed under a larger earthquake load. While calculating the effective cutting area, the infill wall areas parallel to the earthquake direction are also included at the rate of 15%. Major removal of infill walls on a storey may cause weak storey behaviour as well as soft storey formation. In addition, although it is prohibited with TEC-2007 [19], weak storey may occur if the shear wall on the upper floors is seated on the columns on the lower floor, which can be observed in previously built structures. In Fig. 22 and Fig. 23 damages and collapses due to soft and weak storey formations in some structures in K.Maraş (Elbistan and Türkoğlu), Gaziantep (Nurdağı) and Hatay (Antakya) are shown. As seen in Fig. 20, one of the common features of most of the buildings that were heavily damaged or totally collapsed in the region is that the ground storey is higher than the other storeys and is used for commercial purposes. Although TEC-1998 [18] (and later other codes) includes some sanctions on soft-storied buildings, it did not impose a limit on the construction of these buildings. Therefore, it can be expected that this irregularity section will change with more severe sanctions in the future TEC versions. Especially in regions with DTS1 and DTS2, if the building has four or more storeys, the ground storey of the buildings should be no higher than the other storeys and the use of commercial purposes should be limited.

6.4.4. *Short column*

Short column occurs when the lateral displacement of a column is partially restricted due to the load-bearing system (mezzanine floors or stairway slabs) or due to the gaps provided between the columns in the infill walls. In this case, the free part of the column tries to deform like other columns in the same storey, while the restricted part remains rigid and undergoes insignificant deformation [103].

Therefore, the shear force demanded in the free part of the reduced span increases significantly due to increased lateral stiffness. Therefore, the column is subjected to a higher shear force than envisaged at the design stage and shear damage is inevitable (Fig. 24). As it is an important cause of damage in all severe earthquakes that occurred in Turkey, it was frequently reported among the causes of earthquake damage in the world [3–5,104–106]. TBEC-2018 [20] requests that in cases where short column formation is unavoidable, the shear force acting on the column should be determined by the capacity design approach and the flexural capacity at the ends of the column should be increased by 40%. In addition, the code has set strict rules such as the continuation of the transverse reinforcements with reduced spacing as at the ends of the columns, for the entire storey height, in the columns that have turned into short column. The authors did not detect a building totally collapsed due to the formation of short columns, and they also think that this damage acts as a kind of insurance just like the shear damaged beams between shear walls in tunnel formwork buildings.

However, it is obvious that the ratio of the number of possible short columns to the total number of columns on the ground floor will be an important indicator in terms of structural performance. In future code revisions, structural penalty provisions may be introduced depending on the short column ratio in the critical storey.

6.4.5. *Pounding effect*

Pounding damage was observed during the 1985 Mexico earthquake, the 1988 Sequenay earthquake, the 1992 Cairo earthquake, the 1994 Northridge earthquake, the 1995 Kobe earthquake, 1999 Kocaeli earthquake and 2011 Van-Turkey earthquake [3,107].

Fig. 22. Collapses due to weak and/or soft storey in reinforced concrete buildings.

Fig. 23. Damages and collapses observed in some structures due to soft storey behavior.

Especially in cities with high population density, there are many structures that are in contact with each other, despite having different dynamic characteristics, due to economic concerns such as maximum land usage requirements. TEC-1998 [18] seismic codes did not give precise guidelines to preclude pounding. For this reason, structures with different vibration characteristics during earthquakes may cause heavy damage and fractures in the columns, especially if the floor levels are not aligned (Fig. 25).

According to TBEC-2018 [20], the gap between adjacent buildings should not less than the value obtained by multiplying a coefficient α by the square root of the sum of the squares of the displacements from neighbouring buildings for each floor. The coefficient α doubles if the floors in adjacent buildings are at different levels, even if they are on some floors. In addition, the minimum gap between each building should be at least 30 mm for buildings up to 6 m and should be increased at least 10 mm for each 3 m in height after 6 m.

In structural modelling, analysis is made by ignoring the rotations and displacements in the foundations. According to this situation, there are quite big differences between the horizontal displacements demanded by the earthquake and the horizontal displacements that will occur in real. For this reason, in future earthquake codes, the structure-ground interaction should be examined more carefully in adjacent structures. Otherwise, despite the gaps mentioned, pounding (and hammering) will be inevitable.

6.4.6. Poor concrete strength

In field surveys carried out after each earthquake in Turkey, it has been reported many times that one of the most important reasons for collapse is poor concrete quality [3,4,6,7,108]. Hand-made concrete produced uncontrollably, non-standard size of aggregates, use of water or aggregate from rivers to prepare the mortar, lack of compaction during concrete placement are among the factors that lead to low compressive strength of concrete. Investigations on structures damaged or collapsed after earthquakes show that the average concrete compressive strength is around 10 MPa, and even much lower compressive strengths can be obtained.

Fig. 24. Damages caused by the formation of short columns in some buildings.

Fig. 25. Pounding Effect / Damage and collapses in adjacent buildings.

For example, a total of 1679 randomly selected core samples extracted from 167 public buildings showed that, 33 buildings (20%) have concrete strength of less than 8 MPa while 56 buildings (33%) have concrete strength of between 8 and 10 MPa [109]. In the study, when the results are examined according to the construction years of the buildings, it is seen that the worst concrete strengths are obtained from the structures built between 1975 and 1998. In line with the legal investigations carried out for the buildings where loss of life was experienced in the Feb. 6 earthquakes, it was observed that the average compressive strength obtained from nearly 300

Fig. 26. Building wreckage resembling a sand pile.

concrete core samples taken from the buildings was below 10 MPa [110]. Low concrete compressive and tensile strength increased shear damage in columns and combined with factors such as the use of plain rebar and insufficient splice length, led to adherence decomposition. Fig. 26 shows, the wreckage of some buildings in Kahramanmaraş city center and Antakya that collapsed and caused the death of many people. As can be seen, the wreckage resembles a pile of sand, symbolizing poor concrete quality. The undamaged Kahramanmaraş Chamber of Civil Engineers building, shown with an arrow in the figure, gives a stunning message. Also in Fig. 26, shear damage in 3 columns due to low concrete strength and insufficient shear reinforcement is shown. The average value of the 25 MPa characteristic strength required in TBEC-2018 [20] can be expected to be in the order of 30 MPa, from f_{ck} + 1.18.ss (Here s_s is standard deviation). However, on-site measurements show that the existing concrete strength only provides one third of this value. While this situation does not cause significant moment-bearing problems in slabs and beams that are forced under the bending effect, low concrete strength in columns where axial force is dominant, causes compression failure well above the balanced failure level.

6.4.7. Reinforcement detail errors and deficiencies

a. Inadequate transverse reinforcement

As detailed in Section 3, the Feb. 6 earthquakes created demands in many provinces and districts in southern and eastern Turkey well above the design codes. Therefore, the importance of transverse reinforcement has increased even more. According to Turkish Earthquake Codes (TEC-1998, TEC-2007, TBEC-2018 [18–20]), the design shear force should be determined by the capacity design approach at the column, beam, shear wall and column-beam joints, considering this possible situation. However, the lack of capacity design approach caused the greatest destruction and damage, especially in buildings built before 1998. The tensile stress, which occurs in the concrete of the load bearing element in the diagonal direction due to the shear force, is transferred to the reinforcement after the concrete cracks. However, if the member does not have sufficient transverse reinforcement, brittle shear failure occurs in the diagonal tension mode [4] (Fig. 27).

While increasing the shear strength of the transverse reinforcement in concrete columns is the main task, it also has important functions such as limiting the longitudinal reinforcement against buckling and improving the deformability by confining the concrete core. According to TBEC-2018 [20], the transverse reinforcement spacing in the confinement zones of the columns cannot exceed 150 mm and 1/3 of the small dimension of the column, and 200 mm in the middle zone. It has been observed that the stirrup spacing in most of the heavily damaged buildings is well above the maximum value allowed by the design codes. For this reason, heavy shear damages occurred on the columns and the reinforcements were buckled (Fig. 28). As seen in the Fig. 28 while the distance between the stirrups in the column is about 280 mm, no stirrup densification was made in the plastic hinge regions of any column.

In addition to the high vertical accelerations of the earthquakes and the low concrete strength, serious compression damages were also observed due to insufficient and un-ribbed transverse reinforcement and inappropriate hook detailing. According to TBEC-2018 [20] all stirrups must have 135° seismic hooks at both ends and length of the hooks should not be less than 6Φ, where Φ is the rebar diameter, and 80 mm for ribbed bars. An example of compression damage is shown with arrow, in the upper right of the Fig. 28.

Although there are heavy design rules on this subject in TEC-1998 [18] and later, in the field survey made by the authors after the earthquakes in question, the double-sided 135-degree hook and crossties applied to prevent the buckling of the longitudinal reinforcement, especially used in order not to open the stirrup arms, were not encountered in the relatively new reinforced concrete buildings. It is extremely interesting. This shows that this section of the code are only a book source and that they are not fully implemented / or not applied. TBEC-2018 [20] defines shear walls with H_w / L_w ≥ 2.0 as bending shear wall and requires special reinforcement detail requirements. Here, H_w is the height of the shear wall and L_w is the length of the shear wall in plan. The code

Fig. 27. Column shear failures due to low concrete quality and insufficient shear reinforcement.

Fig. 28. Some examples of damage caused by insufficient transverse reinforcement and incorrect end hook angles.

Fig. 29. Damages caused by insufficient shear capacity in shear walls.

calculates the shear capacity of the shear walls as in Eq. (1). In the equation, ρ_{sh} represents the volumetric ratio of the horizontal web reinforcements.

$$V_r = A_{ch}\left(0.65f_{ctd} + \rho_{sh}f_{ywd}\right) \tag{1}$$

Due to the inadequacy of the horizontal web reinforcements and the lower concrete tensile strength than the design value (f_{ctd}), the shear walls of some buildings also caused heavy shear damage (Fig. 29).

In the TBEC-2018 [20] and other previous versions, no shear wall area condition is given depending on the building height and ground floor area (except for tunnel formwork buildings).

b. Inadequate lap splice lengths and anchorage lengths

According to TBEC-2018 [20], lap splices of column longitudinal reinforcements should be made in the middle third of the column's height. The splice length should not be less than l_b given in TBC-2000 [111] (Eq. (2)). In almost all structures built before 2018, lap splices in columns were typically made just above floor level or foundation. That means that the lap splices in the column are in the plastic hinge region, which is the most critical region of the RC elements during an earthquake.

$$l_b = 0.12\frac{f_{yd}}{f_{ctd}}\varnothing \geq 20\varnothing \tag{2}$$

TBEC-2018 [20] also specifies that, in cases where beams framing into columns are not extended to the other side of columns,

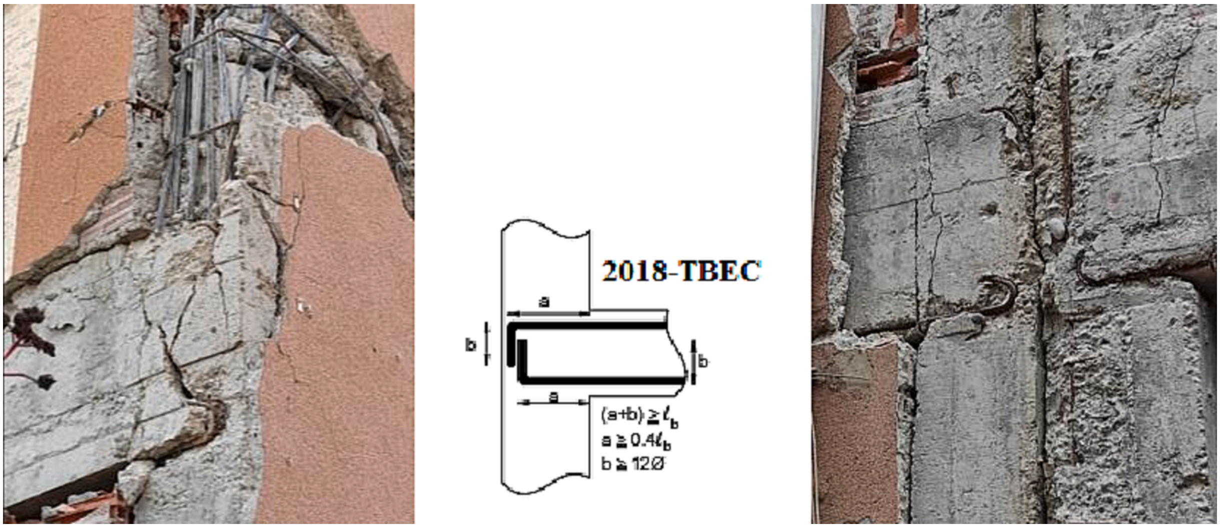


Fig.30. Damaged beams due to insufficient anchorage length.

bottom and top beam reinforcement shall be extended up to the face of the other side of the confined core of the column and then shall be bent 90° from inside the stirrups. In this case, the total length of the horizontal part of the beam reinforcement in the column and the vertical part bent at 90° should not be less than l_b. In addition, the horizontal part (a) should not be less than 0.4 l_b and vertical part (b) than 12ϕ. In Fig. 30 the code requirements, the damages due to insufficient anchorage lengths that do not provide with this condition and the stripped reinforcements are shown. It is very difficult to meet these requirements, especially in beam supports embedded in narrow corner columns.

c. Beam-Column joints with insufficient shear capacity

According to TBEC-2018 [20] the design shear force should be determined by the capacity approach at the beam-column joints and the transverse reinforcement spacing calculated according to this value should not exceed 150 mm in confined joints and 100 mm in unconfined joints. TBEC-2018 [20] categorizes beam-column joints as confined and unconfined in frame systems comprised of columns and beams of high ductility level. Where beams are supported on all four sides of a column and the width of each beam is not less than 3/4 the width of the column to which it joins, such a beam-column joint is called a confined joint. However, in the earthquake-affected areas, it was observed that the beam-column joints were heavily damaged in the buildings constructed before the modern earthquake regulations, since transverse reinforcement was not used and the concrete had low strength (Fig. 31). Similar to ACI 318 [73], while the maximum shear capacity limit is given for the column-beam junction in TBEC-2018 [20], there is no clue as in EC-8 [63] on how to calculate the capacity at the column-beam junction.

6.5. Insufficient strengthening applications

In Turkey, especially after TEC-2007 [19], there was a very innovative section on performance analysis and retrofit/sterengthening analyses of existing RC buildings. In 2007 and later, strengthening applications were made in many public buildings (such as schools, hospitals, military facilities, etc.) to increase the structural performance. The government has allocated a lot of resources for the rehabilitation of public buildings in this regard [112–114]. However, just as in the existing buildings, unfortunately, many damages have been recorded in the reinforced buildings due to both project and producing mistakes.

Although it is not a common cause of damage and collapse, in the surveys made by the authors in the earthquake zone, it has been determined that, especially some of the school buildings were destroyed or severely damaged despite the strengthening, in the earthquake affected area.

For example, Fig. 32 shows a high school building located in Kahramanmaraş-Türkoğlu, 10 km from Nurdağı, and the damage caused by the earthquakes. As can be seen from the response spectra given for Nurdağı and Kahramanmaraş in Fig. 7, it is understood that the damages occurred in the first earthquake of magnitude 7.7 (M_w 7.7). Shear walls that comply with the code requirements are placed on the interior and exterior axes of the building. However, due to the lack of anchorage reinforcements and insufficient anchorage lengths, which will enable the existing columns and beams to work together with the newly added shear walls, the shear walls were stripped from the columns and beams, shear force could not be transferred to the shear walls, so the existing structure with insufficient strength and ductility came to the verge of collapse. As can be seen from Fig. 32, only 1 or 2 anchor reinforcements were placed in the upper part between the columns and the shear walls on the outer axes, but no anchorage was made afterwards. Also, it is seen that these anchors were placed carelessly too close to the outer surface of the shear walls. It is seen that the anchorage lengths are insufficient at the shear wall-column interfaces on the interior axes. A striking point in the building is that some of the columns have

Fig. 31. Failure of beam-column joints.

Fig. 32. Damage caused by earthquakes in a strengthened high school building in Kahramanmaraş.

very heavy torsional shear damages. It is understood that during the magnitude 7.7 earthquake (M_w 7.7), the shear walls were stripped from the existing structural elements at different times, therefore, due to the change in the centre of rigidity and also depending on the magnitude of the earthquake a large torsional moment occurred in the structure. As a result heavy torsional shear damage occurred in 2 columns near the entrance door. This damage to the column can be seen in the the bottom middle picture.

A school in the Ören Neighborhood of Malatya's Akçadağ district was damaged and collapsed due to similar reasons (Fig. 33). From the response spectra for Malatya-Akçadağ in the Fig. 8, it is seen that the effect of the first earthquake in the region was low, while the second earthquake with a magnitude of 7.6 (M_w 7.6) caused great accelerations. Eyewitnesses also confirmed that the collapse occurred in the second earthquake. It is understood that shear walls were added to the frame openings of the school, which was strengthened in 2020, and some of the columns were wrapped with Carbon Fiber Reinforced Polymer (CFRP).

According to the observations, it was seen that there was no damage to the added shear walls and that they were stripped rigidly from the frame elements, similar to the school in Kahramanmaraş. It is thought that the anchor lengths are insufficient depending on the existing structure concrete strength and the epoxy properties used. It is also known that the cleanliness of the anchor holes significantly affects the bond strength at the concrete-binder interface. Failure to clean the holes from concrete debris and using low quality epoxy can explain this damage.

In addition, it is understood that only the column is wrapped without increasing the strength and rigidity of the beam-column joints, therefore, the joint areas with insufficient shear and ductility capacity could not respond to the demands of strong earthquakes after the shear walls lose their effect. Also, with the loss of the effect of the shear walls, the formation of short columns due to architectural factors was effective in the collapse.

Fig. 33. Collapse caused by earthquakes in a strengthened secondary school in Malatya-Akçadağ.

6.6. The building owner obligations and errors

In the field post-earthquake observations, it is seen that the structures with especially weak earthquake performance are mostly old buildings. The main reason for this was stated in the previous section that the old structures were not designed to provide sufficient

Fig. 34. Damages caused by users to the load-bearing elements during the use of the building.

earthquake performance and did not receive an adequate engineering service. However, another important issue is that the owner of the building does not pay the necessary attention to its structure. For example, not taking adequate precautions for corrosion in a building exposed to corrosion, loading the building randomly and out of use, damaging the structural system of the building for owner errors, etc. Otani (1997) [22] stated that “the owner is responsible for maintaining his building to the existing code level”.

Various changes are made by the users in the load-carrier systems of reinforced concrete buildings in Turkey, over time. The most common of these is the removal of infill walls on the basement or ground stories. In fact, the positive effect of the walls, which are considered only as loads in structural modeling, on the earthquake performance of the building is well-known [115,116] If the infill walls are built adjacent to the column-beam system, they provide a significant increase in the lateral load carrying capacity during the seismic event. This increase in stiffness reduces the displacement and natural period of the structure. However, with the removal of the walls to the especially in critical storey (such as ground storey), stiffness loss and structural irregularities occur. Rarely, in some buildings, cutting (or drilling) of the ground floor columns can be encountered by the authors (Fig. 34).

Another common user intervention is the passage of plumbing pipes through columns and beams in basements and ground storeys [117,118] Especially with these changes made in the support sections of the beams, the integrity of the frame is deteriorated. Most of the time, unfortunately longitudinal reinforcements bars in the beams may be cut. In addition to these, the authors observed examples where rainwater pipes were passed through the reinforced concrete columns in the earthquake epicenter regions. Some examples of this are given in Fig. 31.

7. Result and conclusions

Within the scope of this study, a general evaluation was made for the damage to the reinforced concrete structures that occurred after the 6 February 2023 earthquakes. Evaluations were made in the context of reinforced concrete design, member & connection reinforcement detailing, material properties, soil conditions and earthquake codes. The acceleration spectra obtained from the earthquake acceleration records were also compared with the elastic acceleration spectra given in the TBEC-2018 [20]. The results of the examined earthquake pairs, which are effective in a very large region and perhaps have a unique place in the history of earthquakes, are summarized below;

1) The fact that the reinforced concrete structure stock is quite large in Turkey has caused the majority of damaged and totally collapsed buildings to be reinforced concrete buildings.
2) The fact that the spectral values are well above the acceleration values specified in the Turkey Earthquake Hazard Map also shows that the recurrence period of the earthquakes is equal to the most severe earthquakes that occur once in 2475 years. The fact that these earthquakes occur on the same day and at this intensity constitutes an important example in terms of earthquake engineering history.
3) Most of the totally collapsed buildings were built before TEC-1998 [18] and are far from desired the ductile design criteria. However, the number of reinforced concrete structures built according to TEC-1998 [18] and most updated codes and unfortunately heavily damaged or collapsed (it is around 2% of the total buildings).
4) It is seen that the production problems of buildings before or after TEC-1998 [18] are generally similar. The general weakness of the structures is insufficient lateral rigidity. In particular, the very modest selection of column sections and the inadequacy of the shear walls in respect of the height of the building caused the rigidity of the buildings to be insufficient.
5) Totally collapsed occurred due to the inability of having insufficient rigidity to respond to the earthquake, which causes more displacement demands, and the insufficient global ductility of the structures. Insufficient transverse reinforcement in the column and column-beam connection regions, crossties reinforcement that is almost never used, too large distances between the stirrup arms, non-used 135° stirrup hooks have been an important factor in the rapid buckling of the longitudinal reinforcements. The fact that the vertical accelerations of the earthquakes were above the norms of the codes caused compression failures due to the already insufficient concrete strength and pan-cake style collapses occurred in the buildings.
6) The fact that the ground storey in buildings was designed both higher and without infill walls for commercial purposes has triggered soft and weak storey irregulariteis, and totally collapses are very common, especially in multi-storey buildings facing the main street.
7) It has been revealed once again that slab type preferences are extremely important. Beam widthes in the ribbed slabs were too large compared to the columns dimensions, causing the damage to be more severe in these buildings. In addition, the very thin slab thickness (around 7 cm) in these slabs deteriorated the rigid diaphragm feature of the slabs under earthquake loading.
8) Strong beams that did not comply with the capacity design concept triggered the unwilling hinge mechanism in the columns.
9) Insufficient embedtmenth length and level of development length in the column-beam connections have caused adherence dissolution.
10) The fact that stirrups are not confinned in the plastic hinge regions has emerged as a very common manufacturing deficiency. This resulted in shear failure of the column-beam connections.
11) In city centers such as Hatay and Adıyaman, leaning (forward or backward) or sinking into the soil, occurred in a significant part of the buildings with the effect of liquifection. Even though this type of collapse seems advantageous in terms of casualties compared to totally collapse, it has shown that the selection of the soil on which the structures are built is extremely important. A similar situation occurred in Gölcük and Adapazarı in the 1999 Marmara earthquake.
12) User errors in the structures affected the structural performance negatively. The most common one is the habit of passing basement and ground floor installation pipes through beams and columns.

13) From the field observations, it is seen that there are manufacturing problems in the strengthening of existing reinforced concrete buildings. Particularly, it was observed that the shear walls used for strengthening were debonded from the connection anchors to the columns and beams. The fact that the columns were not jacketed at the column region of the strengthened shear walls triggered this situation.
14) Although there is a subsection of strengthening in the scope of TBEC-2018 [20], the lack of comprehensive information for strengthening applications also causes some differences in practice. For example, it is seen that there is a need for a strengthening code with critical calculations and details that will affect the behavior of the reinforced building, such as CFRP application, CFRP anchor details, shear wall-column connection details, shear wall -foundation connection details, etc.
15) The most important factor among the causes of earthquake damage is the wrong selection of architectural projects and reinforced concrete projects in terms of earthquake engineering. Especially in the analysis of reinforced concrete buildings, with the commercial package programs widely used in Turkey, civil engineers perform result-oriented analysis. Although this situation seems appropriate in theory, in practice it causes buildings that are difficult to implement and whose behavior will be contrary to expectations.

When the causes of damage given in this study are examined, the general conclusion is that; for earthquake resistant building design, first of all, the right and sutiable location must be selected. Then, adequate architectural and reinforced concrete project services should be obtained. After that, the sufficient control of the produced project should be done. In addition, the structure should not be damaged by the users over time and should be protected against external influences (eg corrosion, etc.). In addition to these, earthquake codes in Turkey need to introduce strict rules (underlined in this paper) regarding the conditions affecting the earthquake performance negatively and, if necessary, further clarify the application limits in earthquake-risk areas.

Novelty Statement

6 February 2023 Turkey earthquake doublets was effective in 11 provinces in Turkey and caused an economic loss of approximately 110 billion dollars. After two major earthquakes that occurred on the same day, which is rare in the seismological literature, nearly 25% of the buildings in the cities close to the earthquake epicenter were severely damaged or collapsed. In fact, a similar destruction rate was reported in the 1999 Marmara (7.6 M_w) and 1999 Düzce (7.2 M_w) Earthquakes in Turkey. However, the fact that this ratio (25%) has not changed much despite the new structures built after TEC-1998, TEC-2007 and TBEC-2018, which came into force with the 1999 earthquake and have very strict seismic design rules, is a result of heavy damage and destruction in the structures built in accordance with these codes. For this reason, after the 6 February 2023 earthquakes, the main question of the field researchers is whether there are design concept problems in the current earthquake codes. This question is valid for the reinforced concrete building stock in Turkey, as well as for Greece, Iran, Italy, Mexico, etc., which share a similar fate in terms of earthquakes. In this study, a general evaluation was made for the damage to the reinforced concrete structures that occurred after the 6 February 2023 Earthquakes, and evaluations were presented in the context of construction and codes criteria. Considering the importance of the lessons to be learned in the great destruction caused by the 6 February 2023 Earthquakes for other earthquake prone regions and researchers as much as Turkey, it is thought that the evaluations to be made will play a critical role in terms of new codes and producing techniques. **The earthquake pair examined in the study, analyzes based on seismological data, very comprehensive structural damage examples and code-based recommendations constitute the novel part of the study.**

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

Data availability

No data was used for the research described in the article.

References

[1] T. Taymaz, O. Tan, S. Yolsal, Active tectonics of Turkey and surroundings and seismic risk in the Marmara sea region, in: Proceedings of the IWAM04, Mizunami, Japan, 31 March 2004.
[2] D. Kalafat, C. Zulfikar, S.O. Akcan, Seismicity of turkey and real-time seismology applications in determining earthquake hazard, Acad. Platform J. Nat. Haz. Disaster Manage. 2 (2) (2021) 96–111, https://doi.org/10.52114/apjhad.1039670.
[3] M. Ozturk, Field reconnaissance of the October 23, 2011, Van, Turkey, Earthquake: lessons from structural damages, ASCE J. Perform. Constr. Facil. 29 (5) (2015) 04014125, https://doi.org/10.1061/(ASCE)CF.1943-5509.0000532.
[4] M.H. Arslan, H.H. Korkmaz, What is to be learned from damage and failure of reinforced concrete structures during recent earthquakes in Turkey? Eng. Fail. Anal. 14 (1) (2007) 1–22, https://doi.org/10.1016/j.engfailanal.2006.01.003.
[5] H. Sezen, A.S. Whittaker, K.J. Elwood, K.M. Mosalam, Performance of reinforced concrete buildings during the august 17, 1999 Kocaeli, Turkey earthquake, and seismic design and construction practise in Turkey, Eng. Struct. 25 (1) (2003) 103–114, https://doi.org/10.1016/S0141-0296(02)00121-9.
[6] A. Doğangün, Performance of reinforced concrete buildings during the May 1, 2003 Bingöl Earthquake in Turkey, Eng. Struct. 26 (6) (2004) 841–856, https://doi.org/10.1016/j.engstruct.2004.02.005.
[7] A. Nuhoğlu, et al., A reconnaissance study in Izmir (Bornova Plain) affected by October 30, 2020 Samos earthquake, Int. J. Disaster Risk Reduct. 63 (2021), 102465, https://doi.org/10.1016/j.ijdrr.2021.102465.
[8] M. Inel, H.B. Ozmen, H. Bilgin, Re-evaluation of building damage during recent earthquakes in Turkey, Eng. Struct. 30 (2) (2008) 412–427, https://doi.org/10.1016/j.engstruct.2007.04.012.

[9] M.H. Arslan, H.H. Korkmaz, F.G. Gulay, Damage and failure pattern of prefabricated structures after major earthquakes in Turkey and shortfalls of the Turkish Earthquake code, Eng. Fail. Anal. 13 (4) (2006) 537–557, https://doi.org/10.1016/j.engfailanal.2005.02.006.
[10] B.S. Bakır, M.T. Yılmaz, A. Yakut, P. Gülkan, Re-examination of damage distribution in Adapazarı: geotechnical considerations, Eng. Struct. 27 (7) (2005) 1002–1013, https://doi.org/10.1016/j.engstruct.2005.02.002.
[11] E. Işık, F. Avcil, A. Büyüksaraç, R. İzol, M.H. Arslan, C. Aksoylu, E. Harirchian, O. Eyisüren, E. Arkan, M.Şakir Güngür, M. Günay, H. Ulutaş, Structural damages in masonry buildings in Adıyaman during the Kahramanmaraş (Turkiye) earthquakes (Mw 7.7 and Mw 7.6) on 06 February 2023, Engineering Failure Analysis Volume 151 (2023).
[12] M. Bruneau, Building damage from the Marmara, Turkey earthquake of August 17, 1999, J. Seismol. 6 (2002) 357–377, https://doi.org/10.1023/A:1020035425531.
[13] Z. Celep, A. Erken, B. Taskin, A. Ilki, Failures of masonry and concrete buildings during the March 8, 2010, Kovancılar and Palu (Elazıg) Earthquakes in Turkey, Eng. Fail. Anal. 18 (2011) 868–889, https://doi.org/10.1016/j.engfailanal.2010.11.001.
[14] E. Isik, M.C. Aydin, A. Buyuksarac, 24 January 2020 Sivrice (Elazig) earthquake damages and determination of earthquake parameters in the region, Earthq. Struct. 19 (2) (2020) 145–156, https://doi.org/10.12989/eas.2020.19.2.145.
[15] G. Dogan, A.S. Ecemis, S.Z. Korkmaz, M.H. Arslan, Korkmaz H.H., Buildings damages after Elazığ Turkey earthquake on January 24, 2020, Nat. Hazards 109 (2021) 161–200, https://doi.org/10.1007/s11069-021-04831-5.
[16] H. Sucuoglu, U. Yazgan, A. Yakut, A screening procedure for seismic risk assessment in urban building stocks, Earthq. Spectra 23 (2) (2007) 441–458.
[17] I.E. Bal, Gulay, F.G., S.S. Tezcan, A new approach for the preliminary seismic assessment of RC buildings: P25 scoring method, in: Proceedings of the 14th World Conference on Earthquake Engineering, Beijing, China, 2008.
[18] TEC, Ministry of Public Works and Settlement, Specification for structures to be built in disaster areas; Part III-earthquake disaster prevention (ABYYHY-98), Government of Republic of Turkey, 1998.
[19] TEC, Turkish Earthquake Design Code, Ministry of Public Works and Settlement, Ankara, 2007.
[20] TBEC, Turkey Building Earthquake Code, Republic of Turkey Ministry of Interior Disaster and Emergency Management Authority, Ankara, 2018.
[21] U. Ersoy, G. Özcebe, E. Canbay, Reinforced Concrete I, Ankara, Turkey, 2018.
[22] Shunsuke Otani, Lessons learned from past earthquakes, in: Proceedings of Fourth Turkish National Conference on Earthquake Engineering, Ankara, Turkey, September 17-19, 1997.
[23] M. Dolsek, P. Fajfar, Soft storey effects in uniformly infilled reinforced concrete frame, J. Earthq. Eng. 5 (2001) 1.
[24] A. Mehrabian, H. Achintya, Some lessons learned from post-earthquake damage survey of structures in Bam, Iran earthquake of 2003, Struct. Surv. 23 (3) (2005) 180–192.
[25] American Museum of Natural History "Anatolian Fault, Turkey, https://www.amnh.org.
[26] The Economist, Turkey sits at the crossroads of tectonic plates as well as civilisations, https://www.economist.com/.
[27] J.F. Dewey, M.R. Hempton, W.S.F. Kidd, F. Şaroğlu, A.M.C. Şengör, Shortening of continental lithosphere: the neotectonics of eastern Anatolia—a young collision zone, Geol. Soc. Lond. Spec. Publ. 19 (1986) 1–36, https://doi.org/10.1144/GSL.SP.1986.019.01.0.
[28] Ö. Hacıoğlu, A.T. Başokur, E.T. Çiftçi, Crustal structure of a young collision zone: the Arabia-Eurasia collision in northeastern Turkey investigated by magneto telluric data, Earth Planets Space 70 (2018) 161, https://doi.org/10.1186/s40623-018-0932-3.
[29] A.M.C. Şengör, Y. Yılmaz, Tethyan evolution of Turkey: a plate tectonic approach, Tectonophysics 75 (3–4) (1981) 181–241, https://doi.org/10.1016/0040-1951(81)90275-4.
[30] B. Alpar, C. Yaltırak, Characteristic features of the North Anatolian Fault in the eastern Marmara region and its tectonic evolution, Mar. Geol. 190 (1–2) (2002) 329–350, https://doi.org/10.1016/S0025-3227(02)00353-5.
[31] McClusky, et al., Global positioning system constraints on plate kinematics and dynamics in the Eastern Mediterranean and Caucasus, J. Geophys. Res. 105 (B3) (2000) 5695–5719, https://doi.org/10.1029/1999JB900351.
[32] D.P. McKenzie, Active tectonics of the Mediterranean region, Geophys. J. Int. 30 (2) (1972) 109–185, https://doi.org/10.1111/j.1365-246X.1972.tb02351.x.
[33] E. Aydindag, Fractal Analysis of Active Fault Data in The San Andreas and the North Anatolian Fault Zones, M.Sc. Thesis, Istanbul University Institute of Graduate Studies in Science and Engineering, Istanbul, Turkey, 2015.
[34] T. Parsons, S. Toda, R.S. Stein, A. Barka, J.H. Dieterich, Heightened odds of large earthquakes near Istanbul: an interaction-based probability calculation, Science New Series 288 (5466) (2000) 661–665, https://doi.org/10.1126/science.288.5466.661.
[35] M.Ş. İmamoğlu, E. Cetin, The seismicity of Southeast Anatolian and Vicinity, J. Dicle Univ. Ziya Gokalp Faculty ofEducation 9 (2007) 93–103.
[36] F. Bulut, M. Bohnhoff, T. Eken, C. Janssen, T. Kılıç, G. Dresen, The East Anatolian Fault Zone: seismotectonic setting and spatiotemporal characteristics of seismicity based on precise earthquake locations, J. Geophys. Res. 117 (B7) (2012) B07304, https://doi.org/10.1029/2011JB008966.
[37] T.Y. Duman, Ö. Emre, The East Anatolian Fault: geometry, segmentation and jog characteristics, Geol. Soc. Lond. Spec. Publ. 372 (2013) 495–529, https://doi.org/10.1144/SP372.14.
[38] T.Y. Duman, et al., Paleoseismology of the western Sürgü-Misis fault system: East Anatolian Fault, Turkey, Mediterranean Geosci. Rev. 2 (2020) 411–437, https://doi.org/10.1007/s42990-020-00041-6.
[39] The 2023 Pazarcık (Mw=7.8) and Elbistan (Mw=7.6), Kahramanmaraş Earthquakes in the Southeast Turkey, Sakarya University Disaster Management Application and Research Center and Department of Geophysics, Sakarya, Turkey, 2023.
[40] C. Cetin, M. Utkucu, O. Alptekin, A Study of Spatial Distribution of Aftershock Seismicity Parameters (B And P Values) Along the Fault Rupture Zone of 17 August İzmit Earthquake, Istanbul Univ. Eng. Faculty J. Earth Sci. 18 (2) (2005) 123–138.
[41] 24 January 2020 Elazığ-Sivrice Earthquake Preliminary Investigation Report, Istanbul University, Istanbul, Turkey, 2020.
[42] M. Palutoglu, A. Sasmaz, 29 November 1795 Kahramanmaraş Earthquake, Southern Turkey, Bulletin of the Mineral, Res. Explor. 155 (2017) 187–202, https://doi.org/10.19111/bulletinofmre.314211.
[43] 24 January 2020 Sivrice (Elazığ) Earthquake (M_W=6.8) Field Observations and Evaluation Report, The İnstitute of Mineral Research and Exploration, Ankara, Turkey, 2020.
[44] METU Technical Report, Preliminary Reconnaissance Report on February 6, 2023, Pazarcık Mw=7.7 and Elbistan Mw=7.6, Kahramanmaraş-Türkiye Earthquakes, Earthquake Engineering Research Center, Middle East Technical University, Ankara, Turkey, 2023.
[45] USGS, US Geological Survey, National Earthquake Information Center, World Data Center for Seismology.
[46] USGS US Geological Survey, National Earthquake Information Center, M 6.0 – 5 km NE of Göksun, Turkey.
[47] USGS US Geological Survey, National Earthquake Information Center, M 6.0 – 10 km SE of Doğanşehir, Turkey.
[48] F.K. Oz, 50,783 people confirmed dead in Turkey earthquakes, Anadolu Agency, www.aa.com.tr, 22 April 2023.
[49] EMSC, European Mediterranean Seismological Centre, 2023.
[50] AFAD, Turkish Ministry of Interior Disaster and Emergency Management Presidency, 2023.
[51] KOERI B, Bogazici University Kandilli observatory and earthquake research institute regional earthquake-Tsunami Monitoring Center, 2023.
[52] TUIK Turkish Statistical Institute, Survey on Building and Dwelling Characteristics, 2021.
[53] B.Ö. Ay, T.E. Azak, A Comparative Investigation of Changing Building Characteristics in Turkey, Cukurova Univ. J. Faculty Eng. 36 (4) (2021) 1111–1126, https://doi.org/10.21605/cukurovaumfd.1048380.
[54] TEC, Ministry of Public Works and Settlement. Specification for structures to be built in disaster areas (ABYYHY-75), Government of Republic of Turkey, 1975.
[55] 1996 Earthquake Zone Map, Republic of Turkey Ministry of Public Works and Settlement, Ankara, Turkey, 1996.
[56] AFAD, 2018 Earthquake Hazard Map, Turkish Ministry of Interior Disaster and Emergency Management Presidency, Ankara, Turkey, 2018.
[57] M.J.N. Priestley, M.C. Calvi, M.J. Kowalsky, Displacement-Based Seismic Design of Structures, IUSS Press, Pavia, 2007, p. 670 pp..
[58] M.J.N. Priestley, "Performance-Based Seismic Design" Keynote Address, in: Proc 12th World Conference on Earthquake Engineering" Auckland, 2000, p. 22..

[59] B. Ozmen, S. Pampal, The Evolution of Earthquake Zoning Maps in Turkey, 4th International Conference on Earthquake Engineering and Seismology, Anadolu University, Eskişehir, Turkey, 2017.
[60] Jus, Code of technical regulations for the design and construction of buildings in seismic regions, Official Gazette of SFR Yugoslavia 31/81 (1981).
[61] I. Yüksel, Z. Polat, Yield state investigation of reinforced concrete frames from a new point of view, Eng. Struct. 27 (1) (2005) 119–127.
[62] American Society of Civil Engineers, Minimum Design Loads and Associated Criteria for Buildings and Other Structures: Asce. Published by American Society of Civil Engineers, 2017.
[63] Eurocode 8 (EC8), Design of Structures for Earthquake Resistance, Part 1: General Rules, Seismic Actions, and Rules for Buildings, European Committee for Standardization (CEN), 2018.
[64] NZSEE, The seismic assessment of existing buildings. New Zealand Society for Earthquake 867 Engineering (NZSEE), Wellington, New Zealand, 2017.
[65] NRC, Structural Commentaries (User's Guide – NBC 2015: Part 4 of Division B), National 862 Research Council Canada (NRC), Ontario, Canada, Ottawa, 2015.
[66] Associação Brasileira de Normas Técnicas, Ações e Segurança nas Estruturas – Procedimento (NBR 8681), ABNT, Rio de Janeiro, Brazil, 2003.
[67] A. Carvalho, M.L. Sousa, Análise estatística do catálogo sísmico de Portugal Continental. TechnicalReport n°2/2001 – G3ES. LNEC, Lisbon. Portugal (16) (PDF) Seismic zonation for Portuguese National Annex of Eurocode 8, 2001.
[68] Greek Code for Seismic Resistant Structures (EAK 2000) Greece, 2000.
[69] Bulgarian National Annex to Eurocode 8 – BDS EN 1998-1 (EC 8-1), 1998.
[70] NCh433.Of96 – Earthquake Resistant Design of Buildings, Official Chilean Code, Chile, 1996.
[71] S.H. Santos, et al., Comparative Study of International Major Codes for the Seismic Design of Buildings, in: IABSE Symposium- Synergy of Culture and Civil Engineering – History and Challenge, Wrowlav, Poland, 2020.
[72] M.H.S. Elawady, Ductility Considerations in Seismic Design of Reinforced Concrete Building, Master Thesis, School of Technology and Management of the Polytechnic Institute of Leiria, Portugal, 2007.
[73] Applied Technology Council. Tentative provisions for the development of seismic regulations for buildings. (ATC3-06), Applied Technology Council, Palo Alto, CA, 1978.
[74] R.K. Goel, A.K. Chopra, Period formulas for moment-resisting frame buildings, J. Struct. Eng. 123 (11) (1997) 1454–1461.
[75] NEHRP, Recommended Provisions for the development of Seismic Regulations for New Buildings, Building Seismic Safety, Council, Washington, D.C., 1994.
[76] Uniform Building Code (UBC-1997), International Conference of Building Officials, Whittier, Calif.
[77] Egyptian Code for Computation of Loads and Forces in Structural and Building Work (EGC-1993). Housing and Building Research Center, Cairo, Egypt.
[78] L.-L. Hong, W.-L. Hwang, Empirical formula for fundamental vibration periods of reinforced concrete buildings in Taiwan, Earthq. Eng. Struct. Dyn. 29 (3) (2000) 327–337.
[79] K. Guler, E. Yuksel, A. Kocak, Estimation of the fundamental vibration period of existing RC buildings in Turkey utilizing ambient vibration records, J. Earthq. Eng. 12 (S2) (2008) 140–150.
[80] T.-C. Pan, K.S. Goh, K. Megawati, Empirical relationships between natural vibration period and height of buildings in Singapore, Earthq. Eng. Struct. Dyn. 43 (3) (2014) 449–465.
[81] O. Kaplan, Y. Guney, A. Dogangun, A period-height relationship for newly constructed mid-rise reinforced concrete buildings in Turkey, Eng. Struct. 232 (2021), 111807.
[82] H. Crowley, R. Pinho, Simplified Equations for Estimating the Period of Vibration of Existing Buildings, in: 1st European Conference on Earthquake Engineering and Seismology, Geneva, 2006.
[83] M. Navarro, F. Vidal, T. Enomoto, F.J. Alcalá, A. García-Jerez, F.J. Sánchez, et al., Analysis of the weightiness of site effects on reinforced concrete (RC) building seismic behaviour: the Adra town example (SE Spain), Earthq. Eng. Struct. Dyn. 36 (10) (2007) 1363–1383.
[84] M. Gallipoli, M. Mucciarelli, B. Šket-Motnikar, P. Zupanćić, A. Gosar, S. Prevolnik, et al., Empirical estimates of dynamic parameters on a large set of European buildings, Bull. Earthq. Eng. 8 (3) (2010) 593–607.
[85] C. Michel, P. Guéguen, P. Lestuzzi, P.-Y. Bard, Comparison between seismic vulnerability models and experimental dynamic properties of existing buildings in France, Bull. Earthq. Eng. 8 (6) (2010) 1295–1307.
[86] R. Ditommaso, M. Vona, M. Gallipoli, M. Mucciarelli, Evaluation and considerations about fundamental periods of damaged reinforced concrete buildings, Nat. Hazards Earth Syst. Sci. 13 (7) (2013).
[87] V.H. Akansel, B.F. Soysal, K. Kadas, H.P. Gulkan, An evaluation of the 2018 seismic hazard map of turkey on the basis of spectrum intensity, Turkish J. Earthq. Res. 2 (2) (2020) 115–137, https://doi.org/10.46464/tdad.737433.
[88] M. Villar-Vega, V. Silva, H. Crowley, C. Yepes, N. Tarque, A.B. Acevedo, et al., Development of a fragility model for the residential building stock in South America, Earthq. Spectra 33 (2) (2017) 581–604.
[89] Y. Lee, D. Moon, A new methodology of the development of seismic fragility curves, Smart Struct. Syst. 14 (5) (2014) 847–867.
[90] A. Singhal, A.S. Kiremidjian, Method for probabilistic evaluation of seismic structural damage, J. Struct. Eng.-ASCE 122 (1996) 1459–1467.
[91] S. Akkar, H. Sucuoğlu, A. Yakut, Displacement-based fragility functions for low-and mid-rise ordinary concrete buildings, Earthq. Spectra 21 (4) (2005) 901–927.
[92] S. Uma, H. Ryu, N. Luco, A. Liel, M. Raghunandan, Comparison of Main-Shock and Aftershock Fragility Curves Developed for New Zealand and Us Buildings, in: Proceedings of the ninth pacific conference on earthquake engineering structure building and Earthquake-Resilient Society, Auckland, New Zealand, 14–16, 2011.
[93] A. Modica, P.J. Stafford, Vector fragility surfaces for reinforced concrete frames in Europe, Bull. Earthq. Eng. (2014) 1–29.
[94] V. Silva, H. Crowley, H. Varum, R. Pinho, L. Sousa, Development of a Fragility Model for Moment-frame RC buildings in Portugal, in: 2nd ICVRAM, Liverpool, UK, 2014a.
[95] U. Hancilar, E. Çaktı, M. Erdik, G.E. Franco, G. Deodatis, Earthquake vulnerability of school buildings: probabilistic structural fragility analyses, Soil Dyn. Earthq. Eng. 67 (2014) 169–178.
[96] A. Billah, M. Alam, Seismic fragility assessment of highway bridges: a state-of-the-art review, Struct. Infrastruct. Eng. (2014) 1–29.
[97] H. Korkmaz, The relationship between ground conditions and earthquake effect in Antakya, J. Geog. Sci. 4 (2) (2006) 49–66, https://doi.org/10.1501/Cogbil_0000000066.
[98] J. Bothara, at al., General observations of effects of the 30th September 2009 Padang earthquake, Indonesia, Bull. New Zealand Soc. Earthq. Eng. 43 (3) (2010) 143–173, https://doi.org/10.5459/bnzsee.43.3.143-173.
[99] Eurocode 8, Design of structures for earthquake resistance, Part 1: general rules, seismic actions and rules for buildings, EN 1998–1:2004, European Committee for Standardization, Brussels, 2004.
[100] Q. Zhe, Z. Baijie, C. Yuteng, F. Haoran, Rapid report of seismic damage to buildings in the 2022 M 6.8 Luding earthquake, China, Earthq. Res. Adv. 3 (1) (2023) 100180, https://doi.org/10.1016/j.eqrea.2022.100180.
[101] A. Bayraktar, A.C. Altunişik, M. Pehlivan, Performance and damages of reinforced concrete buildings during the October 23 and November 9, 2011 Van, Turkey, earthquakes, Soil Dynam. Earthq. Eng. 53 (2013) 49–72, https://doi.org/10.1016/j.soildyn.2013.06.004.
[102] J.M. Humar, D. Lau, J.R. Pierre, Performance of buildings during the 2001 Bhuj earthquake, Can. J. Civ. Eng. 28 (2001) 979–991, https://doi.org/10.1139/l01-070.
[103] S.C. Alih, M. Vafaei, Performance of reinforced concrete buildings and wooden structures during the 2015 M_w 6.0 Sabah earthquake in Malaysia, Eng. Fail. Anal. 102 (2019) 351–368, https://doi.org/10.1016/j.engfailanal.2019.04.056.
[104] P. Ricci, F. De Luca, G.M. Verderame, 6th April 2009 L'Aquila earthquake, Italy: reinforced concrete building performance, Bull. Earthq. Eng. 9 (2011) 285–305, https://doi.org/10.1007/s10518-010-9204-8.

[105] L. Halder, S.C. Dutta, R.P. Sharma, S. Bhattacharya, Lessons learnt from post-earthquake damage study of Northeast India and Nepal during last ten years: 2021 Assam earthquake, 2020 Mizoram earthquake, 2017 Ambasa earthquake, 2016 Manipur earthquake, 2015 Nepal earthquake, and 2011 Sikkim earthquake, Soil Dyn. Earthq. Eng. 151 (2021) 106990, https://doi.org/10.1016/j.soildyn.2021.106990.

[106] W.Y. Kam, S. Pampanin, K. Elwood, Seismic performance of reinforced concrete buildings in the 22 February Christchurch (Lyttleton) earthquake, Special Issue, Bull. New Zealand Soc. Earthq. Eng. 44 (4) (2011) 239–279, hdl.handle.net/10092/17651.

[107] I.J. Sharma, Seismic Pounding Effects in Buildings, B.S. Thesis, Dept. of Civil Engineering National Institute of Technology, Rourkela, India, 2008.

[108] K. Adalier, O. Aydingun, Structural engineering aspects of the June 27, 1998 Adana-Ceyhan (Turkey) earthquake, Eng. Struct. 23 (4) (2001) 343–355, https://doi.org/10.1016/S0141-0296(00)00046-8.

[109] M. Inel, S.M. Senel, H. Un, Experimental evaluation of concrete strength in existing buildings, Mag. Concr. Res. 60 (4) (2008) 279–289, https://doi.org/10.1680/macr.2007.00091.

[110] URL.https://www.haberturk.com/video/haber/izle/urkuten-karot-sonucu-beton-kalitesi-olmasi-gerekenin-yarisi/803305.

[111] TBC-2000, Turkish Standards Institution, TS 500–2000, Requirements for Design and Construction of Reinforced Concrete Structures, Ankara, Turkey, 2000.

[112] M.Y. Kaltakci, M.H. Arslan, U.S. Yilmaz, H.D. Arslan, A new approach on the strengthening of primary school buildings in Turkey: an application of external shear wall, Build. Environ. 43 (6) (2008) 983–990, https://doi.org/10.1016/j.buildenv.2007.02.009.

[113] H.D. Arslan, B. Köken, Evaluation of the space syntax analysis in post-strengthening hospital buildings, Archit. Res. 6 (4) (2016) 88–97, https://doi.org/10.5923/j.arch.20160604.02.

[114] H.D. Arslan, B. Köken, From the architectural point of view: statistical evaluation of the existing and strengthening hospital buildings, Int. J. Sci. Eng. Res. 7 (6) (2016) 670–674.

[115] Y. Kaltakcı, M. Öztürk, M.H. Arslan, R. Sezer, H.D. Arslan, Performance Assesment of Strengthened Reinforced Concrete Buildings in Terms of Carrying System and Architecture, EACEF - International Conference of Civil Engineering[S.l.] 1 (2011) 302–309.

[116] M.H. Arslan, An evaluation of effective design parameters on earthquake performance of RC buildings using neural networks, Engineering Structures 32 (7) (2010) 1888–1898.

[117] C. Aksoylu, S. Yazman, Y.O. Ozkılıç, L. Gemi, M.H. Arslan, Experimental analysis of reinforced concrete shear deficient beams with circular web openings strengthened by CFRP composite, Compos. Struct. 249 (2020), https://doi.org/10.1016/j.compstruct.2020.112561.

[118] M.A. Mansur, Effect of openings on the behavior and strength of R/C beams in shear, Cem. Concr. Compos. 20 (6) (1998) 477–486, https://doi.org/10.1016/S0958-9465(98)00030-4.