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Failures of masonry and concrete buildings during the March 8, 2010 Kovancılar and Palu (Elazığ) Earthquakes in Turkey

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ABSTRACT

The March 8, 2010 earthquakes that hit Kovancılar and Palu districts of Elazığ province in Turkey and their impacts on masonry and concrete buildings are studied in this paper. According to United States Geological Survey (USGS), magnitudes of these earthquakes, which caused partial or total collapse in many buildings with life losses, were 6.1 and 5.5, respectively. This paper outlines the seismological aspects of the region, the characteristics of the strong ground motion, the geotechnical characteristics of the region and the structural damages based on site assessments. The structural damage level is observed to be directly proportional with the amount of the insufficient quality in the workmanship and usage of inadequate building materials. If a minimum amount of engineering attention had been paid during the construction stages, most of the existing buildings could have sustained the earthquakes without considerable damage.

1. Introduction

On March 8, 2010, an earthquake of moderate intensity shook Kovancılar district and the surrounding villages of Elazığ province at 04:32 (02:32 GMT) local time. Being one of the most seismically active zones in Turkey, Elazığ settles on the East Anatolian Fault (EAF), almost at the north-eastern end, where EAF meets the well known North Anatolian Fault (NAF) at the Karlıova triple junction. Magnitude and source characteristics of the earthquake are defined by various institutions as given in Table 1. In the table, h_{hypo} is the depth of hypocenter of the earthquake and M is the local magnitude (M_L) for the top two rows and the moment magnitude (M_w) for the other rows.

The earthquake struck a region of about 10,000 population and caused 42 human casualties and injuries of 137 individuals. According to the Governorship of Elazığ [1], approximately 3000 residential and 181 office buildings were heavily damaged. Stockbreeding sector is seriously affected with a loss of more than 3000 farm animals due to the collapse of barns. Most of the damages were observed in Okçular, Yukarı Demirci, Kayalık and Göçmezler villages within the political boundaries of Kovancılar district. Although Kovancılar and Palu city centers were the two closest districts to the epicenter, most buildings withstood against the shock without experiencing damages. Followed by the main event, however, an aftershock of $M_L = 5.6$ at 07:47GMT caused damages in Palu district (Table 2).

Few days after the earthquakes, our reconnaissance team arrived at the region (Fig. 1) to carry out site investigation and damage assessment. Kovancılar and Palu districts and the villages Kayalık, Yukarı Demirci, Okçular, Göçmezler, Taban Özü, Çakırkaş, Köklüce, Gökdere, Arındık and Beyhan, are taken into the scope of this survey.

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Table 1
08.03.2010 Elazığ–Kovancılar Earthquake characteristics.

Source	Time (GMT)	Latitude (N)	Longitude (E)	h_{hypo} (km)	M
DEMA[a]	02:32:30	38.7752	40.0295	5.01	5.8
KOERI[b]	02:32:31	38.807	40.100	5.00	6.0
USGS[c]	02:32:34	38.873	39.981	12.00	6.1
EMSC[d]	02:32:35	38.84	40.00	10.00	6.0

[a] Turkish Prime Ministry-Disaster and Emergency Management Agency, DEMA.
[b] Kandilli Observatory and Earthquake Research Institute.
[c] United States Geological Survey.
[d] European-Mediterranean Seismological Centre.

Table 2
08.03.2010 Elazığ-Palu Earthquake characteristics.

Source	Time (GMT)	Latitude (N)	Longitude (E)	h_{hypo} (km)	M
DEMA	07:47:37	38.7355	40.0090	5.00	5.6
KOERI	07:47:38	38.781	40.066	5.00	5.5
USGS	07:47:40	38.767	40.239	10.00	5.5
EMSC	07:47:40	38.73	40.20	2.00	5.5

In this paper observed structural damages are evaluated and discussed considering the local site conditions and the strong motion data provided by the national strong motion network of Turkish Prime Ministry-Disaster and Emergency Management Agency (DEMA).

2. Seismo-tectonic characteristics of the region

2.1. Seismic sources and historic earthquakes in the region

Each of the strong motions of March 8, 2010; namely the main Kovancılar earthquake and the major Palu aftershock, occurred nearly at the east end of the EAF zone. Starting from Bingöl-Karlıova at the north-east and towards Antakya at the south-west, EAF extends approximately 550 km long. With left-lateral strike slip moving segments, EAF constitutes the tectonic boundary between the Anatolian Plate and the northward-moving Arabian Plate [2]. Şaroğlu et al. [3] defined the active tectonic zones on EAF from east to west as Karlıova-Bingöl segment; Palu-Hazar Lake segment; Hazar Lake-Sincik segment; Çelikhan-Erkenek segment; Gölbaşı-Türkoğlu segment and Türkoğlu-Antakya segment. Historical studies indicate that Elazığ and its surroundings have experienced destructive earthquakes in the past [4–6]. Table 3 gives some of these events, while Fig. 2 illustrates the epicentral locations on the segments of EAF.

2.2. Seismic activity from March 8 to 18, 2010

Until March 18, a number of 275 aftershock motions are recorded pursuing the main event on March 8, 2010, including four of them with magnitudes $M_L \geqslant 5.0$. As emphasized previously, the major $M_L = 5.6$ aftershock hit Palu district causing partial or total collapse of some adobe and stone buildings. Fig. 3 exhibits the epicentral distribution of the aftershocks. Fig. 4 shows the magnitude distribution of the main earthquake and the following aftershocks on a time scale. As expected, the magnitudes as well as the frequency of the aftershock occurrence decrease steadily after the main event.

3. Evaluation of strong ground motion

3.1. Characteristics of strong motion records

Characteristics of the strong motion accelerograms provided by the National Strong Motion Recording Stations of DEMA are given in Table 4 for Kovancılar–Elazığ Earthquake (02:32GMT $M_L = 5.8$) at the epicentral coordinates of 38.7752N–40.0295E, where R_{epi} is the epicentral distance from the station and PGA is the peak ground acceleration. The strong motion accelerogram having $R_{epi} = 12.2$ km has the smallest epicentral distance. It was recorded at the Palu station located in the Directorate of Meteorology Building. However, the maximum PGA values of 0.063g, 0.068g and, 0.031g for NS, EW and UD directions, respectively, are found to be surprisingly low compared to the damage observed within the region.

Detailed information for the major aftershock of $M_L = 5.6$ occurred at 07:47GMT on the epicentral coordinates 38.7355N–40.0090E is given in Table 5. PGA for NS, EW and UD directions are measured as 0.078g, 0.049g and 0.056g, respectively, at the closest recording station with $R_{epi} = 8.0$ km.

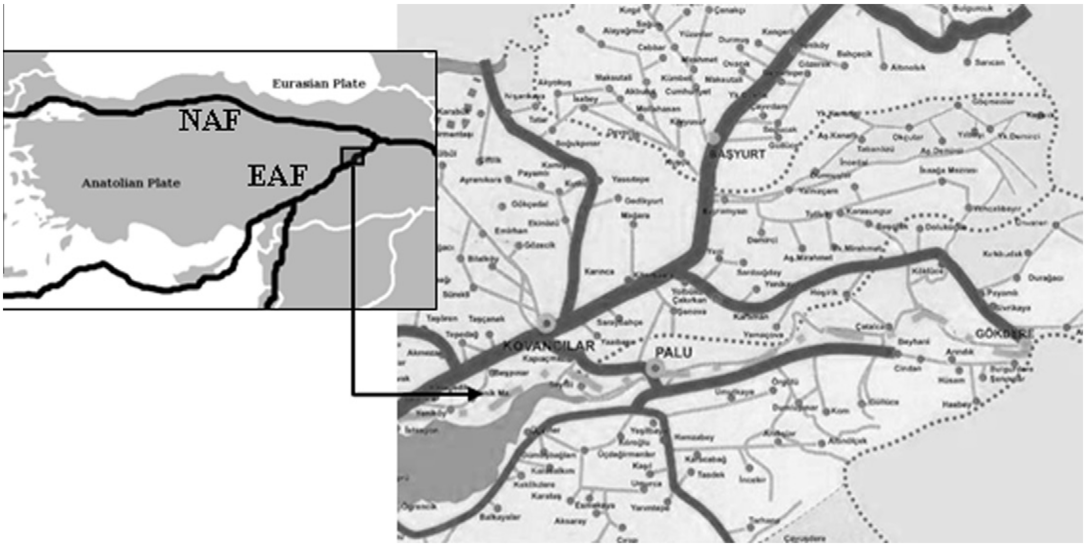

Fig. 1. Region affected by the main event and the aftershock.

Table 3
Previous earthquakes in and around Elazığ.

Date	Region	Intensity	Magnitude
29.05.1789	Palu	VII	7.0
12.05.1866	Karlıova	–	7.2
1866	Hazar Lake souths	–	5.5
03.05.1874	Palu-Hazar Lake	IX	7.1
1874	Maden-Diyarbakır	–	6.1
27.03.1875	Keban-Hazar-Sincik	VI	6.7
1875	Karlıova-Palu	–	6.1
1889	Palu	V	4.3
02.03.1905	Hazar Lake-Sincik	–	6.8
17.08.1949	Karlıova	IX	6.7
26.07.1967	Pülümür	VIII	5.9
22.05.1971	Bingöl	VIII	6.8 (7.1)
06.09.1975	Lice	VIII	6.6
25.03.1977	Palu	–	5.1
05.05.1986	Doğanşehir	VIII	5.9
06.06.1986	Doğanşehir	VIII	5.6
15.03.1992	Pülümür	VII	5.8
05.12.1995	Kığı	VI+	5.7
13.04.1998	Karlıova	VI	5.0
27.01.2003	Pülümür	VII	6.2
01.05.2003	Bingöl	VIII	6.4

The strong motion acceleration-time histories are processed by taking into account the characteristics of the previous earthquake motions recorded in the region [7] and the engineering intensities are determined. Fig. 5 shows the time histories for acceleration, velocity and displacement as well as for Fourier Amplitude Spectra (FAS).

In the process, initially, baseline correction is performed for the three components of the records of the main event for Palu station. Later, for removing the undesirable frequency (noise) contents in the records, acausal fourth order Butterworth band filter having corner frequencies found from FAS, is applied to the acceleration records. The processed time histories yielded the peak ground velocities of 6.0 cm/s, 5.9 cm/s and 3.1 cm/s, for the NS, EW and UD components, respectively. The maximum displacement is calculated as 2.3 cm for the NS component.

The effective durations t_{eff}, of the horizontal components are also computed assuming that t_{eff} is the time interval between the 95% and 5% values on the Arias Intensity (AI) curves. From the plots of AI (%) given in Fig. 6, the effective durations are calculated as 15.5 s and 13.8 s for the NS and EW components, respectively.

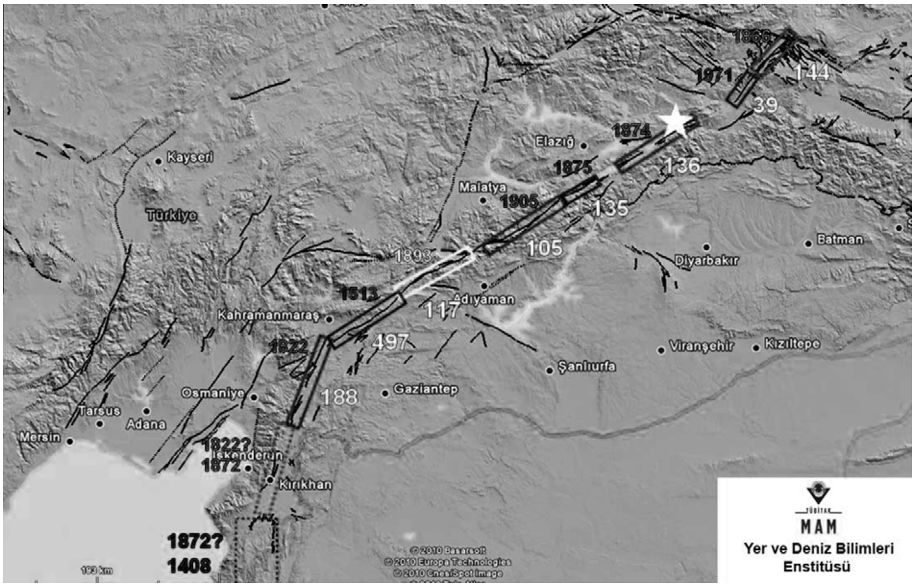

Fig. 2. Historic earthquakes in EAF zone [5].

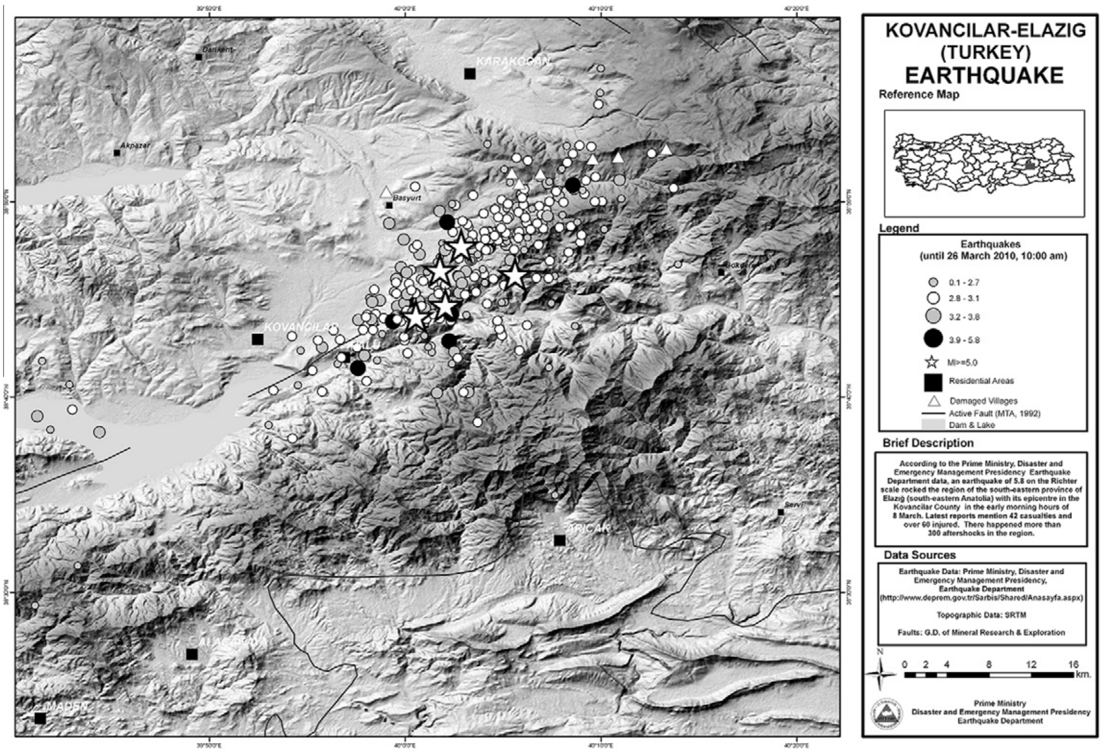

Fig. 3. Epicentral distribution of the aftershocks [4].

3.2. Spectral properties and attenuation of PGA

Fig. 7 shows the acceleration, velocity and displacement spectra for the Palu-NS and -EW components of $M_L = 5.8$ Kovancılar earthquake for the damping ratios of $\xi = 0\%$; 2%; 5%; 10% and 20%. From the velocity spectra having a damping ratio 5%, the effective acceleration values a_{eff}, are computed as 40.0 cm/s^2 and 40.9 cm/s^2 for the NS and EW components, respectively [8]. On the other hand, Housner Intensity $SI_{0.20}$, which is accepted as another important indicator of the destructive-

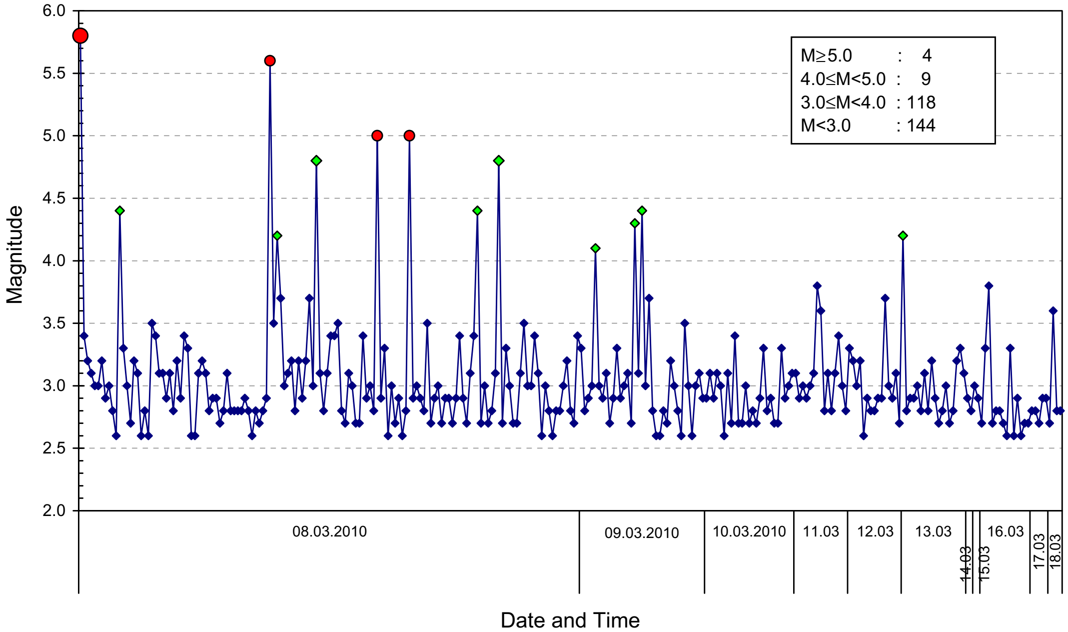

Fig. 4. Distribution of aftershock magnitudes along time scale.

Table 4
Characteristics of the strong motion records of M_L = 5.8 Kovancılar Earthquake.

Station information			R_{epi} (km)	PGA (cm/s^2)		
Name	Latitude (N)	Longitude (E)		NS	EW	UD
Palu Dir. Meteo. Bldg.	38.69577	39.9319	12.2	62.00	66.50	30.00
Bingöl Dir. Pub. Works Bldg.	38.89708	40.5032	43.3	55.31	34.27	25.50
Elazığ Dir. Pub. Works Bldg.	38.67043	39.19267	73.6	5.56	4.77	3.85
Diyarbakır Dir. Pub. Works Bldg.	37.93088	40.20278	95.2	3.44	5.10	2.59
Karlıova Municip. Garage	39.29345	41.00883	102.5	11.59	17.84	8.95
Batman Dir. Pub. Works Bldg.	37.873	41.15112	140.3	7.62	5.44	2.52
Mardin Dir. Pub. Works Bldg.	37.32632	40.72374	172.4	2.54	2.46	1.68
Adıyaman Dir. Pub. Works Bldg.	37.762	38.267	190.9	2.50	2.24	1.64

Table 5
Strong motion records of M_L = 5.6 Palu Earthquake (aftershock).

Station information			R_{epi} (km)	PGA (cm/s^2)		
Name	Latitude (N)	Longitude (E)		NS	EW	UD
Palu Dir. Meteo. Bldg.	38.69577	39.9319	8.0	76.5	48.00	55.00
Bingöl Dir. Pub. Works Bldg.	38.89708	40.5032	46.5	14.49	10.20	7.07
Elazığ Dir. Pub. Works Bldg.	38.67043	39.19267	71.3	3.54	4.21	3.33
Karlıova Municip. Garage	39.29345	41.00883	106.5	2.86	3.86	1.55

ness of a strong motion record, is computed for each horizontal component employing the 20% damped velocity spectra. As expected from the characteristics of the strong motion data, very low values of $SI_{0.20}$ are obtained as 14.2 cm and 15.0 cm for the NS and EW directions, respectively.

The acceleration response spectra with 5% damping are compared with the design spectrum defined in the Turkish Earthquake Resistant Design Code (TERDC) [9] for all soil classes, where $Z1$ represents the most stiff soil and $Z4$ the softest. As seen in Fig. 8, none of the records exceed the design spectra. In order to find the spectral amplification, the acceleration response spectra having the same damping ratio are normalized with respect to the corresponding PGA and are compared with the design spectra of the TERDC. The figure shows that the entire ensemble of the normalized spectra seems to be exceeding the design spectrum, even with an amplification factor up to 5, i.e., exceeding almost twice the design spectrum. This fact seems to be a significant result which has be considered in the possible modification of the design spectra of the TERDC.

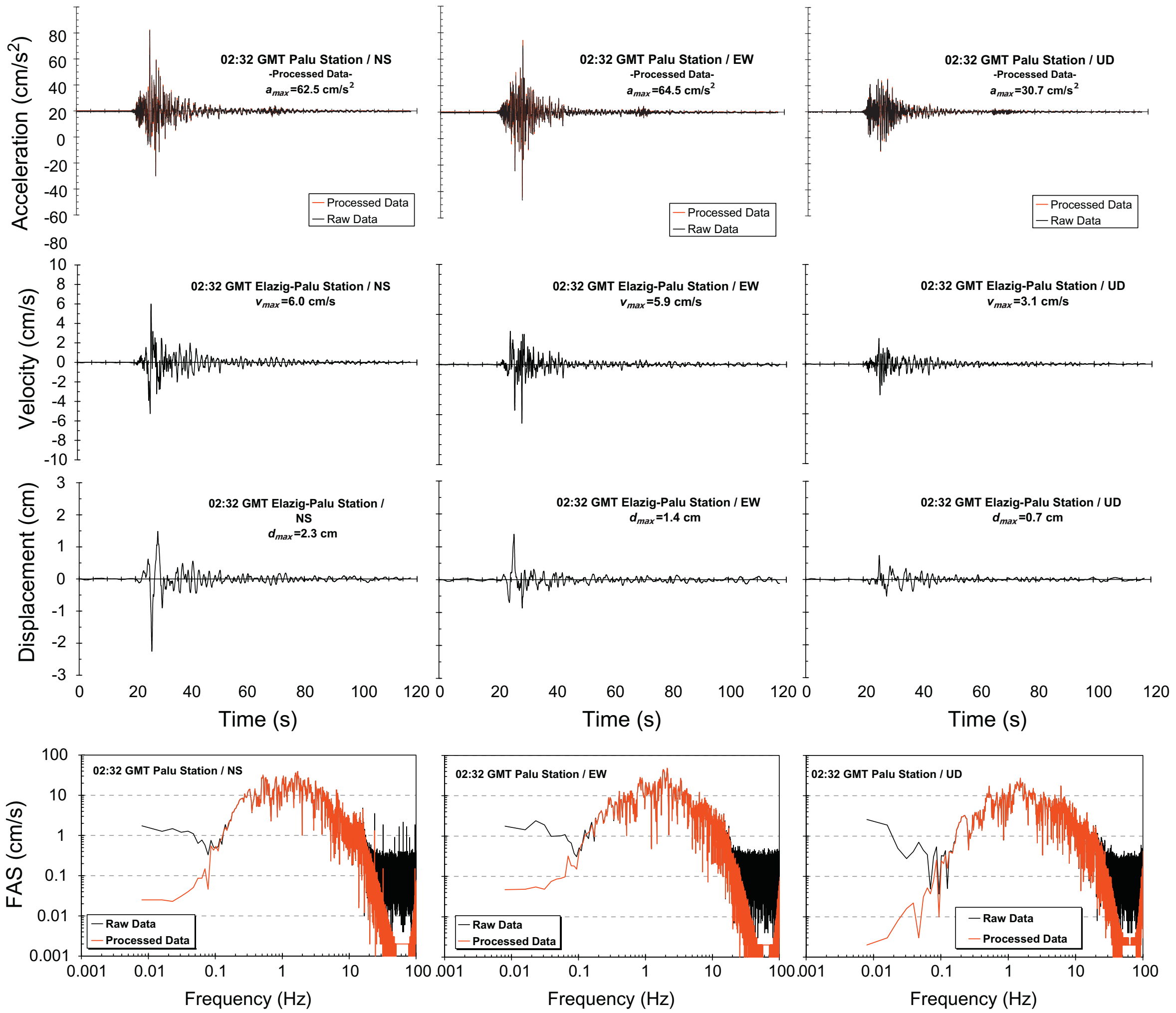

Fig. 5. Processed records: top three rows for acceleration, velocity and displacement histories and the bottom row for the comparison of FAS for raw and processed acceleration-time histories.

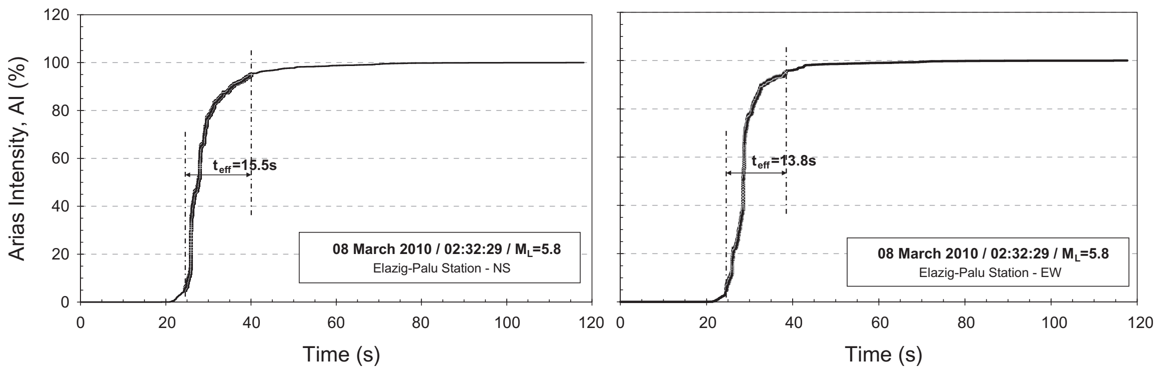

Fig. 6. AI curves for the horizontal components and the effective duration t_{eff}.

Detailed information for the amplification of each record is summarized in Table 6, in which the period ranges exceeding the design spectrum value are also given. The shortest duration for the maximum amplification of 2.5 is defined for the soil class *Z*1 in TERDC between the periods $T_A = 0.10$ s and $T_B = 0.30$ s and the longest duration is defined for soil class *Z*4 between the periods $T_A = 0.20$ s and $T_B = 0.90$ s.

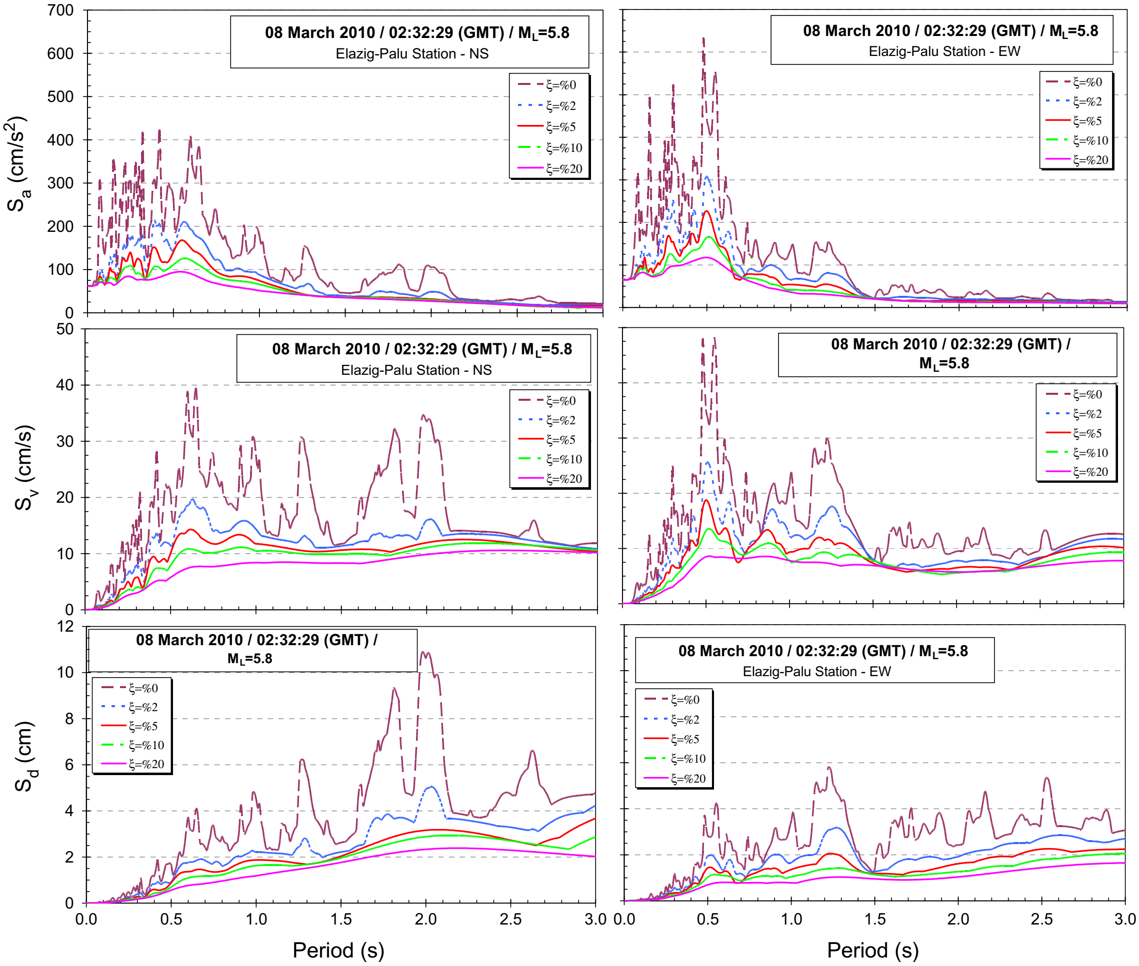

Fig. 7. Response spectra of the main event for various damping ratios.

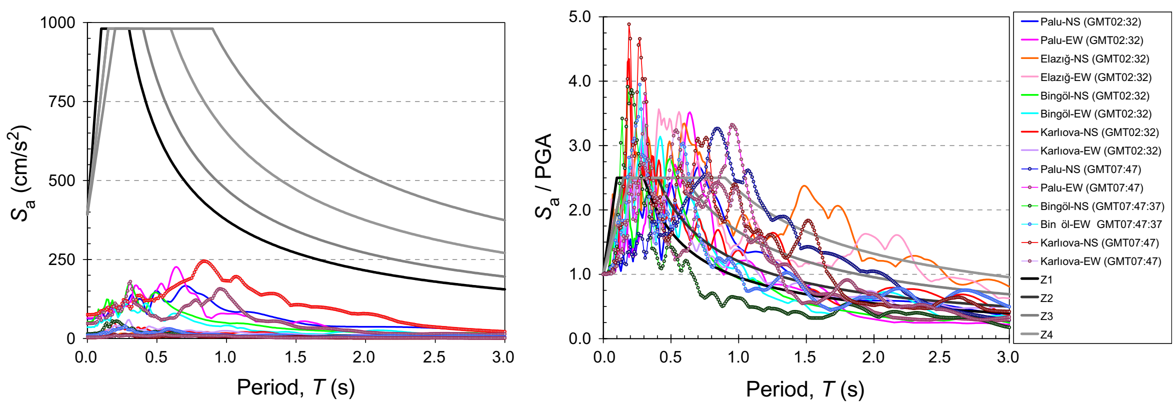

Fig. 8. Comparisons of acceleration response spectra with the design spectra (left) and amplification distribution along period (right).

Taking into account the observed structural damages, attenuation of PGA is also investigated to figure out the maximum ground motion parameters at the epicenter. Hence, the actual attenuation of the PGA of the processed records as a function of the epicentral distances given in Tables 4 and 5, are presented in Fig. 9. In the same figure, the predictions of different attenuation relationships can also be seen [10–17]. They display a reasonable agreement with each other.

Table 6
Amplification of spectral acceleration exceeding design spectrum defined in the TERDC.

Event	Station	Comp	Max. S_a/PGA	Period range	
				T_A (s)	T_B (s)
02:32 GMT	Palu	NS	2.71	0.67	0.75
M_L = 5.8 – Kovancılar		EW	3.50	0.51	0.71
	Elazığ	NS	3.34	0.42	0.77
		EW	3.57	0.22	0.77
	Bingöl	NS	3.33	0.28	0.53
		EW	3.14	0.19	0.45
	Karlıova	NS	4.35	0.17	0.42
		EW	3.43	0.28	0.31
07:47 GMT	Palu	NS	3.30	0.74	1.10
M_L = 5.6 – Palu		EW	3.86	0.77	1.04
	Bingöl	NS	3.91	0.13	0.28
		EW	3.95	0.18	0.63
	Karlıova	NS	4.88	0.16	0.80
		EW	3.24	0.20	0.59

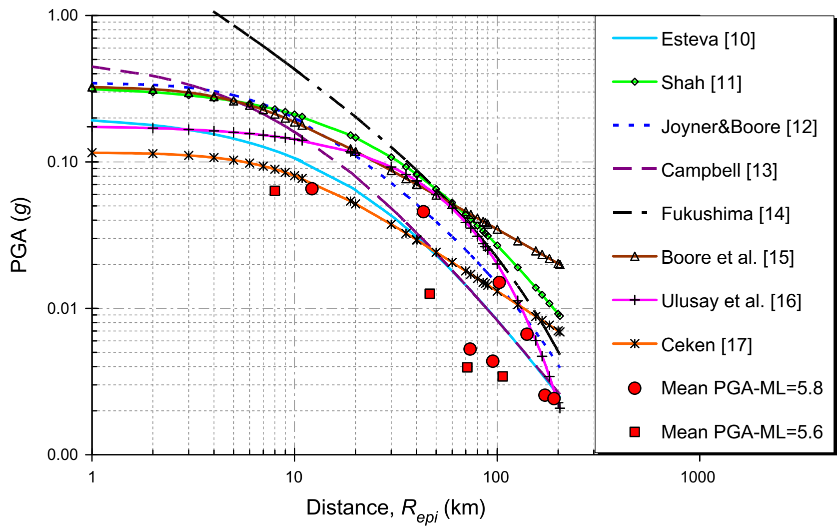

Fig. 9. Various attenuation relations and the calculated values of PGA depending on the distance to the epicenter for the present seismic activity.

In Fig. 9, the mean values of the horizontal components from the main event and the major aftershock are taken into account. As seen, most attenuation relations display a close agreement with the distant measurements; however, the records of the closest station are only in good agreement with the relationships of Esteva [10] and Çeken [11] for the main and aftershock motions. Considering these two attenuation relationships, the epicentral PGA is estimated as 0.20g for M_L = 5.8 Kovancılar and 0.12g M_L = 5.6 Palu earthquakes.

4. Geological and geotechnical characteristics of the site

Geology of the area is quite complex. However, in the earthquake area, mainly metamorphic, magmatic and sedimentary rocks are found. Murat River, which runs southwards in a valley is the reason of the sedimentary layers in the area. Limestone, Pliocene deposits, sandstone, conglomerate and volcanic rock units outcrop at the mountain of Murat and the Valley of Murat River. Pliocene deposits, which consist of clay–sand–gravel, appear at the other parts of the site. Limestone is observed at the north-east part of Kovancılar and its villages. Failures related to geological and geotechnical characteristics of the site and the corresponding parameters of the motion are summarized below.

4.1. Slope failures in the area

During the Kovancılar (M_L = 5.8) and the Palu Earthquakes (M_L = 5.6), numerous slope failures are observed in the earthquake region. These failures were observed particularly on the embankments along Palu-Köklüce road within the valley of

Fig. 10. Location of Yukarı Demirci Village.

Fig. 11. Tension cracks on the side of the road in the region.

the Murat River and on the natural slopes along the southern parts of the Murat Mountain, where Okçular, Yukarı Demirci and Kayalık villages are located (Figs. 10 and 11). The villages, where heavy damages occurred, are located on soil layers consisting of clay–sand–gravel mixture. Rubbles also cover the slope of the hills [18]. Okçular, Yukarı Demirci and Kayalık Villages are settled on such slopes. Fig. 10 shows the Yukarı Demirci Village located on the slopes of the neighboring mountain.

Fig. 12a. Failure surface.

Fig. 12b. Slope failures around the stream in the region.

Inclinations of the natural slopes vary from 10° up to 35°. Clays with silt, sand and gravel are dominated in the site. The thickness of the clayey layer decreases up to the hill. Weathered limestone, sandstone and volcanic units are outcrop. The groundwater table lies just below the interface between soil layer and rock units. The thickness of the partially saturated soil layers increases through the basin. Tension cracks with various widths from 0.5 cm to 50 cm are observed on the ground surface at the site. Fig. 11 presents the tension cracks developed at the side of the road due to the slope failure around the village of Yukarı Demirci. Rubbles consisting of silty, clayey sandy stones have fallen down from the steep slopes around the road (Fig. 12a). Fig. 12b shows the slope failures along a stream in the region. Similar slope failures are observed on the slopes of the Murat Mountain along the Murat River Valley, some of which caused damages especially in poor quality structures with inadequate foundations. Fig. 13a exhibits a building influenced by the sliding of the foundation soil with the inclination angle of approximately 35°. The building has only a ground story and a partial basement. During the earthquake, settlement induced shear cracks were developed on the walls of the structure due to slope failure. Tension cracks also occurred on the soil around the structure, which clearly indicate the motion under the foundation of the building (Fig. 13b). From the failure and the observations on the visible part of the sliding surface, the entire shape of the failure surface seems to be planar or curved having a very large curvature radius.

4.2. Effect of the yield acceleration on slope displacements

Two of the main earthquake damages are slope failures and landslides. Newmark [19] developed a method for prediction of the permanent displacement of a slope subjected to a ground motion. The method considers a rigid block to be in a stable static equilibrium on an inclined plane assuming the soil resistance against sliding to be purely frictional. Effect of the vertical acceleration is neglected for simplicity. For the present case, the yield acceleration a_y and the upper bound of the permanent displacement d_{max} occurred in a slope failure are calculated utilizing the approach proposed by Newmark. Making use of the yield acceleration a yield coefficient of acceleration $k_y = a_y/g$ can be defined as well. When the maximum acceleration of an earthquake does not exceed the yield acceleration, it is expected that slope failure does not occur. Displacement of a slope having a low yield acceleration will be greater than that of a slope with higher yield acceleration. An upper bound to the permanent displacement produced by an earthquake motion is defined by Newmark as

$$d_{max} = 0.5\,\frac{v_{max}^2 a_{max}}{a_y^2} \quad (1)$$

where v_{max} and a_{max} (or PGA) correspond to the maximum velocity and acceleration of the earthquake motion. For the Kovancılar Earthquake, the numerical evaluation is carried out for the two records, namely, for the Palu-EW component of the Kovancılar Earthquake (M_L = 5.8), and the Palu-NS component of the Palu Earthquake (M_L = 5.6) (Table 7).

The regions, where slope failures are observed, generally lay on the slopes of hills consisting of clay, sand and gravel mixture. For the evaluation of the yield coefficient of the acceleration, only the internal friction of soil is required and cohesion is neglected for the simplicity of the calculations. Inclinations of the slopes of hills generally vary between 10° and 35° in the areas of settlement. Therefore, it is assumed that the internal friction angles of the layers are between 20° and 35°, which correspond to loose to the medium dense soils.

Fig. 14 illustrates the relationship between the coefficient of the yield acceleration k_y and the maximum displacement d_{max}, whereas Fig. 15 shows the variation of d_{max} with respect to a_{max}/a_y ratio. As it is seen, the maximum displacement decreases as the yield acceleration increases. As the inclinations of slopes increase, development of large displacements is observed. This kind of failures are observed in the Murat River valley on the high slopes of the Murat Mountain and on the high

Fig. 13a. A rural building on slope at Yukarı Demirci Village.

Fig. 13b. A view of the building from backyard.

Table 7
Characteristic parameters of Kovancılar and Palu Earthquakes.

Place	Magnitude	Max. acceleration (cm/s^2)	Max. velocity (cm/s)	Max. displacement (cm)
Kovancılar-EW	5.8	64.5	5.9	1.17
Palu-NS	5.6	74.9	10.8	1.49

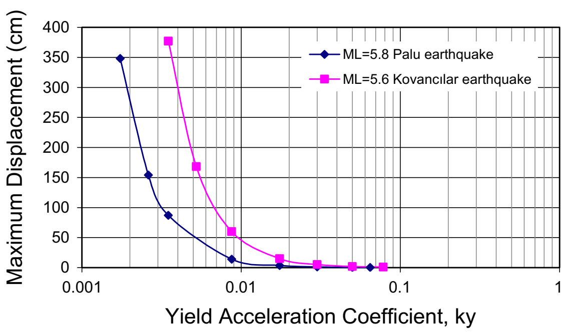

Fig. 14. The relationship between the yield acceleration coefficient and the expected maximum displacement.

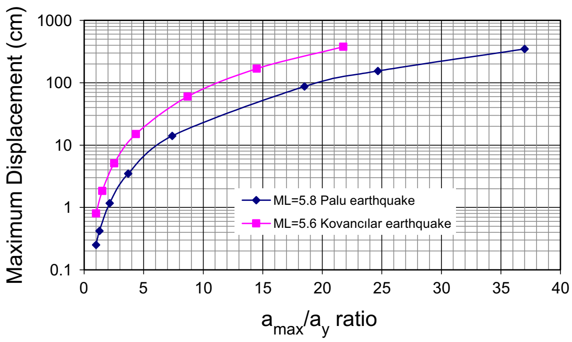

Fig. 15. The relationship between the maximum displacement and the a_{max}/a_y ratio.

slopes of streams next to Yukarı Demirci Village as shown in Fig. 12b. The maximum displacements on the ground surface at foundation levels in the villages of Yukarı Demirci, Okçular and Kayalık were observed to be up to 20 cm, which is comparable to the calculated displacement values within a few centimeters.

5. Damages to building structures

5.1. General characteristics of the buildings in the region

Buildings in Turkey can be divided into two main categories, namely, engineered and non-engineered buildings, having percentages displaying a great variation depending from area to area. However, most of the low-rise residential buildings constructed in villages and small towns are built according to traditional rules adopted in the area by the use of local materials available within the region. These non-engineered buildings are generally masonry; however; often they do not comply with the simple rules of the masonry construction. On the other hand, the relatively new medium rise buildings are constructed by proper civil engineering attention. The design and construction of these engineered buildings generally comply with the regulations of the corresponding codes and standards. Therefore, these buildings have a certain level of structural safety as foreseen in the related codes. Nevertheless, sometimes this safety level can not be maintained due to lack of sufficient attention in design and construction phases. On the other hand, the non-engineered low rise traditional masonry houses have also displayed satisfactory performances during past earthquakes, provided that some simple rules were adopted in their construction. In rural areas of Elazığ, traditional masonry buildings can be found very widely. However, due to the limited economic resources, most of such masonry houses are built without considering the requirements for proper masonry construction. Parallel to the poor workmanship and poor quality of construction, quality of the masonry is deteriorated year by year due to harsh environmental conditions. Past destructive earthquakes showed that most of the damages occurred in non-engineered buildings.

The types and qualities of buildings in the region are strongly dependant on the level of economic development. In relatively rich areas, like the Elazığ city or the district centers, the most common construction type is reinforced concrete, whereas in the poorest mountain villages, the most common construction type is stone masonry with mud mortar binder.

5.1.1. Buildings in the Elazığ city center

In the city center of Elazığ, the structural systems of the most buildings are reinforced concrete frames. The number of stories of these reinforced concrete structures are generally between four and six, while there are 8–9 storey reinforced concrete frame buildings as well, particularly in relatively new residential areas. Only few of these mid-rise buildings have shear walls in their structural systems around the stair and elevators. The buildings generally do not have a basement. Currently, in the construction of new reinforced concrete buildings in the city center of Elazığ, mostly ready-mixed concrete and deformed bars are used. There are also many low-rise (1–2 storey) buildings in the Elazığ city center, some of which have historical significance. Most of these low-rise buildings are stone masonry, while the masonry units are clay bricks in the relatively new buildings. Since the epicenter of the earthquake is quite far from the city center (approximately 75 km) of Elazığ and the focal depth is limited (5–10 km), no considerable damage is reported in the buildings in the city center of Elazığ irrespective of the quality of the construction.

5.1.2. Buildings in the Kovancılar and Palu districts

While Palu is a remarkably older district and has been one of the main city centers in the region for centuries, Kovancılar district has recently been developed much more rapidly due to its geographical location, which is on a main road connection between Elazığ and Bingöl cities. In Kovancılar district center, the structural systems of most of the buildings consist of reinforced concrete frames. These buildings generally do not include a basement and are 3–6 storeys high. Exceptionally, there are 7 and 8 storey reinforced concrete frame buildings as well (Fig. 16). As seen in this figure, generally infill walls are built

Fig. 16. Typical reinforced concrete frame buildings in Kovancılar.

Fig. 17. Typical mixed-type stone-adobe buildings in Palu.

using hollow clay bricks. In Kovancılar, there are many low-rise masonry buildings as well. It should be noted that there are also many mixed type buildings, which are partly reinforced concrete and partly masonry. Combinations of these different structural systems are sometimes present in different storeys and sometimes within a storey. Kovancılar is about 15 km far from the epicenter of the earthquake and no damage is reported in the buildings irrespective of the quality of the construction.

In Palu district, which is the closest settlement to the epicenter of the major aftershock, most of the buildings are two storey stone or adobe masonry houses. Many buildings are constructed using stone and adobe blocks together (Fig. 17). Foundations of all masonry buildings are constructed using stone only. Some of these buildings were severely damaged. Relatively newer masonry buildings in Palu district are constructed with brick masonry walls having concrete tie beams and slabs. Some of the one-storey brick masonry buildings have experienced only slight damage depending on the level of construction quality (Fig. 18). It should be noted that while there were many damaged buildings in the center of Palu and nearby villages, the damages were often limited and were not to the extent of total collapse. No casualties were reported from Palu district and its villages.

5.1.3. Buildings in villages of Kovancılar and Palu

The entire reported death toll was in the villages of Kovancılar, particularly in Okçular, Yukarı Demirci, Kayalık and Göçmezler. In these mountain villages, the buildings are mostly 1–2 storey stone masonry with mud mortar binder. It should be noted that these stone masonry buildings have generally been constructed more than 30 years ago. The relatively newer buildings are built with brick walls, which are sometimes constructed with solid bricks and sometimes hollow bricks with brittle characteristics. More recently, buildings with weak reinforced concrete frame structural systems have been constructed. However, since these buildings are also non-engineered, they can be regarded as confined masonry buildings with

Fig. 18. Slightly damaged brick masonry buildings in Palu.

Fig. 19. Typical failures of stone masonry buildings (*Göçmezler and Yukarı Demirci villages*).

weak tie beams and columns. Mostly, the buildings in the villages are constructed by the owners, whereas small-scale individual constructors generally carry out the construction works in the districts.

5.2. Structural damages and causes of failures in the villages

5.2.1. Stone masonry buildings with wooden roof beams and clay roofs

Most of the failures causing casualties occurred in the stone masonry buildings constructed with mud mortar binder and heavy clay roofs formed on the irregular wooden beams supported by two main walls of the buildings. Often roofs having very weak support connections or very small support lengths tend to separate from the walls very easily (Fig. 19). The thicknesses of multi-leaf stone walls are generally about 50 cm. These thick stone walls and clay roofs are preferred by the local people due to their heat insulation characteristics. They provide cool environment in summer and relatively warm environment in winter. In some buildings, animals such as cattle are kept as well. As seen in Fig. 19, the remarkably poor performance of this type of buildings can mainly be attributed to the irregular wall construction with stones of small dimensions insufficiently tied with larger dimension stones or wooden ties. Very poor quality of the mud mortar binder and its deterioration by time are also among main reasons of poor performance. The configurations of typical stone walls with relatively larger stones at the outer surface insufficiently tied to inner irregular wall parts can be seen in Fig. 20.

Another main reason of the poor performance of the typical masonry buildings in the region is the very heavy compacted clay roofs constructed for resisting the harsh climate conditions in terms of insulation against cold and sometimes rain (Fig. 21).

As it is well known and formulized by Mohr–Columb failure criterion, the shear strength of the masonry walls is a function of the adhesion between the masonry units provided by the binder and the friction between the units. In the case of these typical stone masonry buildings in the region, the adhesion is very low due to usage of mud binder prepared without

Fig. 20. Typical configurations of stone masonry walls (*Göçmezler and Yukarı Demirci villages*).

Fig. 21. Heavy compacted clay roofs (*Göçmezler village*).

proper attention. Furthermore, the friction forces between the stone surfaces are marginal, since the stones are of irregular shapes, not prismatic, and the contact surfaces are not sufficient to cause adequate amount of friction. Additionally, wooden beams of roofs are aligned in one direction and the beams are supported by two walls opposite to each other. Therefore, the weight of the heavy roof and corresponding horizontal earthquake loads are transferred to these walls. Consequently, while the walls supporting the wooden beams are subjected to excessive vertical loads due to heavy clay roofs, the other walls are vulnerable against shear, since the friction between the stones is very low in the absence of vertical stresses other than those caused by the self weight of the walls. The walls, which do not support the slabs or roofs, are also more vulnerable against

Fig. 22. Inadequate and adequate connections of walls at corners (*Arındık and Köklüce villages*).

Fig. 23. Excessive wall lengths without any lateral support or restraints and large openings (*Yukarı Demirci village*).

out-of-plane flexural failure, because they behave as a cantilever member supported only at the base due to their weak connections to the slab or to the roof system.

The other main reasons of the failure of these stone masonry buildings are: (i) inadequate corner connections of the walls (Fig. 22); (ii) insufficiency of the wooden beams in one direction to provide the diaphragm action; (iii) insufficient wooden ties in walls in longitudinal and transverse direction to provide sufficient redundancy and continuity to the structural system; (iv) large window openings, which are normally not permitted in masonry constructions (Fig. 23) and (v) excessive wall lengths without any lateral support or restraints (Fig. 23).

5.2.2. Stone masonry buildings with concrete tie beams and slabs

Stone masonry buildings having concrete tie beams, slabs and roofs generally performed better than the relatively older stone masonry buildings having wooden tie beams and clay roofs. The thickness of the stone walls of this type of buildings is also generally about 50 cm. While cement mortar is used as binder in some of this type of buildings, in some cases, mud mortar is used. Relatively better performance of this type of buildings is basically due to three factors: (i) usage of cement mortar binder between stone units (in some cases); (ii) the diaphragm action provided by the reinforced concrete slabs together with tie beams and (iii) relatively lower weight of the roof. However, some of this type of buildings also suffered remarkable damage and some of them collapsed totally. Major reasons of failures of this type of buildings are observed to be: (i) poor workmanship in wall construction including usage of improper stones and low quality and insufficient amount of binder (Fig. 24); (ii) poor workmanship of wall foundations built with stones (Fig. 25); (iii) insufficient connections of multi-leaf walls in transverse direction (Fig. 24) and (iv) excessive wall lengths without any support or restraints (Fig. 26).

5.2.3. Masonry buildings with timber skeleton and solid brick blocks

This type of buildings is called as hımış and they have a wooden skeleton with infill units in between the wooden members. The hımış buildings y are generally constructed in Western and Northern Turkey, where wood is relatively easier to

Fig. 24. Poor workmanship in wall construction, poor binder, insufficient connection in transverse direction (*Okçular village*).

Fig. 25. Poor workmanship in the foundation (*Kayalık village*).

Fig. 26. Excessive wall lengths without any support or restraints and large openings (*Göçmezler village*).

find. Although the common building materials in the region are stone and clay, a limited number of hımış buildings can be found in the earthquake affected area. Fig. 27 shows a damaged hımış building. As seen, its seismic performance has been better than many heavy stone masonry buildings. Generally hımış buildings experienced some damage in different extents, but did not collapse totally due to the flexible behavior of the wooden skeleton. While foundations of hımış buildings are

Fig. 27. Typical damage of hımış buildings in earthquake affected region (*Okçular village*).

Fig. 28. Failure of a confined brick masonry building (*Kayalık village*).

built with stones, the thickness of solid brick walls is approximately 10 cm. Relatively better seismic performance of this type of buildings is attributed to (i) timber skeleton, which behaves as bracing; (ii) adequate tensile characteristics of timber and (iii) remarkably lower weight of this type of structures. However, it is clear that the insulation of this type of buildings is significantly poor, with respect to stone masonry buildings having an average wall width of 50 cm.

5.2.4. Brick masonry buildings

All brick masonry buildings in the region are unreinforced and most of them do not have any vertical and horizontal tie beam. In other words, they are generally unconfined and unreinforced masonry. Mostly solid clay bricks or clay bricks with limited holes are used in the construction of walls. In case of few confined masonry buildings, i.e., in buildings having vertical and horizontal tie beams, generally hollow bricks are used. These hollow bricks are normally used for non-structural walls. They have a large percentage of holes and they display non-ductile behavior. Furthermore, confined masonry system is generally used for two storey buildings, while unconfined masonry buildings in the region are generally single storey. Since the horizontal and vertical concrete tie members are very weak in the confined brick masonry buildings, expectedly, their seismic performances were relatively poor, as shown in Fig. 28. It should be noted that the confined brick masonry building shown in Fig. 28 was constructed only 3 years ago. On the other hand, the unconfined brick masonry buildings were observed to behave slightly better due to usage of better quality bricks (Fig. 29) and relatively lower height.

5.2.5. Adobe and adobe-stone buildings

In the villages, there are few buildings, which are constructed with adobe blocks and wooden ties. Slabs and roofs of these buildings are formed with wooden beams and clay fills. Besides the villages, there are many adobe buildings in Palu district as well. While the adobe buildings are generally only one storey high in the villages, there are one and two storey adobe

Fig. 29. Performance of an unconfined unreinforced brick masonry building (*Okçular village*).

Fig. 30. An example of adobe buildings in Palu.

buildings in Palu district (Fig. 30). There are also some two storey buildings, with a combination of structural walls built with adobe blocks and stones (Fig. 17). The foundations of the adobe buildings in villages and Palu district are always built with stones. While the adobe and the adobe-stone buildings also suffered damages of various extents, the performances of these buildings were not as poor as improperly constructed stone masonry buildings.

5.2.6. Buildings with reinforced concrete frames

In the villages, there are only a few two-storey reinforced concrete frame buildings. While the buildings with reinforced concrete frame structural systems did not collapse totally, some of them experienced severe damage (Fig. 31). It should be noted that the reinforced concrete frame buildings are relatively new. Damages of the reinforced concrete frame buildings can mainly be attributed to low quality of concrete and improper detailing of reinforcing bars.

6. Performance of lifelines

In the region, there was no widespread damage in the main transportation system. Other than some slope failure or rock fall problems in the mountainous areas, which were solved quickly in the aftermath of the earthquake, all roads in the area remained operational. No considerable damage was observed in the other lifelines, including power transmission lines. Some examples of these lifelines remained operational in the earthquake affected area are shown in Fig. 32. Actually, since the most affected areas are generally remote mountain villages, in these areas there are not widespread established lifelines, such as water piping and sewage systems, gas pipes, and telecommunication services.

Fig. 31. A reinforced concrete frame building in Yukarı Demirci.

Fig. 32. Examples of good performance of lifelines.

7. General evaluation and recommendations

7.1. General evaluation

General evaluation of the observations in the earthquake area can be summarized as follows:

a. It is well known that the magnitude of earthquake is somehow proportional to the damage level in buildings. Although the magnitude of the earthquake is reported to be 6.1 (or 5.8) in Richter scale, the structural damages occurred in a limited area. This is probably due to the lower focal depth of the seismic activity. As known, the damage assessment is one of the crucial steps of response and recovery phases of the disaster management. It is observed that the central and the local administrations, including the Ministry of Public Works and the Elazığ Governorship, paid the required attention and assessed the building damages officially in a relatively short time.
b. Life losses were due to collapse of buildings, which are of stone masonry type having mud binder. They often have roofs consisting of wooden beams in one direction having a thick clay cover. For the other building types, damages were limited and they caused less life losses. Most of the structural failures were associated with deficiencies in the structure caused by lack of supervision and/or with improper construction practices (poor materials, poor workmanship).
c. Observations reveal that masonry buildings, which conform to the main rules of construction, did not experience significant damage. By considering these observations together with the previous experiences, one can deduct that masonry buildings constructed according to the requirements of the TERDC have adequate seismic safety to resist to moderate earthquakes with controlled damages. The same statement can be made for the reinforced concrete buildings as well.
d. The maximum acceleration of the records is found to be around 0.10-0.20g. In fact the acceleration value is low compared to the value 0.40*g*, the effective ground acceleration defined in the TERDC for the earthquake affected area. This fact exhibits that the damaged buildings, whether they are masonry or concrete, are those which have very low seismic load capacity. This information indicates another important fact related with the seismic capacity of the buildings which did not experience any damage after these earthquakes. Satisfactory performances of the undamaged buildings do not confirm that these buildings necessarily have adequate seismic safety.
e. Failures of natural slopes, which were associated with earthquake induced forces, have affected poorly constructed structures and caused damages.

7.2. Recommendations

Recommendations for risk reduction in future are summarized below:

a. The decision of the Turkish government after the earthquakes indicates that an intense construction activity will take place in the area for the replacement of the damaged buildings. This activity will involve design of various typical buildings, selection of the construction areas and the actual construction. Considering the conditions in Turkey, generally, the weak link of these phases is the construction stage. It is of prime importance to pay attention to the construction stage to attain the construction quality foreseen in the design phase.
b. Construction of masonry buildings in rural areas of Turkey is unavoidable. Furthermore, when properly designed and constructed, masonry construction is a good option for buildings of two stories in rural areas. While there is an excellent printed information document prepared by the Ministry of Public Works and Settlement showing the construction steps of a masonry building, it seems that the dissemination of the information has not been done adequately. It is recommended that this type of information documents should be distributed properly. Village elders and teachers are the potential people to be informed in detail. Furthermore, the village people can be informed, when they are in military for compulsory military service.
c. The requirements for the earthquake resistant masonry construction are already given in TERDC. These requirements can be grouped into two categories. The requirements in the first category may only involve some simple construction rules (in terms of geometry and dimensions) to be applied for the masonry buildings having only one or two stories. The requirements in the second category may be the additional rules to be applied for masonry buildings, when the requirements of the first category can not be satisfied. In this way construction of simple masonry buildings by the local people conforming to TERDC can be made possible.
d. As it is well known, in the earthquake area, construction of adobe or stone buildings is preferred due to local availability and their insulation properties. It is worth to state one well known important fact that any type of structure (concrete, masonry, timber and adobe) having adequate seismic safety can be constructed by following the requirements of the seismic codes within the certain limits described by the relevant codes.

8. Conclusions

Conclusions deducted from the observations in the earthquake area can be summarized as follows:

a. There were failures of natural slopes in some localities in the region during M_L = 5.8 Kovancılar and M_L = 5.6 Palu earthquakes. These failures, which were associated with the triggering of the earthquake induced forces, have affected poorly constructed structures.
b. Although the peak accelerations were around 0.1–0.2g, there were many collapsed and damaged buildings. However, it is clearly seen that all collapsed and damaged buildings were non-engineered buildings constructed improperly. The buildings, which were constructed with a minimum amount of care withstood against the earthquakes without any damage.
c. It was clearly observed that if a minimum set of practical rules, outlined above, have been obeyed in construction, the collapses and casualties could have been avoided.
d. In many parts of Turkey and many other developing countries, which are prone to earthquakes, there are many non-engineered houses like the ones collapsed in Elazığ. To avoid further significant losses because of this kind of small scale seismic events, proper construction of new buildings should be encouraged and existing non-engineered buildings should be evaluated and strengthened at least to avoid total collapse.

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