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# Failures of masonry dwelling triggered by East Anatolian Fault earthquakes in Turkey

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ABSTRACT

Turkey is located on active fault zones such as the East Anatolian Fault (EAF), the North Anatolian Fault (NAF) and the Anatolian-Aegean Subduction Zone (AASZ). Ground seismicity activities of the NAF zone are relatively active compared with other faults. Eleven high-intensity earthquakes have been produced on this fault zone since 1939. Whereas the EAF zone was relatively quiescent in the last century, on the basis of historical records, destructive seismic activities occurred on the EAF zone in the last two decades. In this study, the rupture reason of faults, updated active fault data, and seismic maps are presented briefly. Moreover, failure reasons and failure mechanism of conventional masonry structures damaged from seismic ground motions on the EAF zone are evaluated in detail. Possible solutions are suggested on the basis of seismic codes. For this purpose, EAF-sourced earthquakes, i.e. 2003 Bingöl, 2010 Kovancılar (Elazığ), 2011 Maden (Elazığ), 2011 Tabanlı (Van) and Edremit (Van) hit in last two decades on this fault are investigated. Failures of conventional masonry buildings triggered from these earthquakes are assessed. Statistical evaluation, damage of earthquakes and failure pattern are deeply investigated and revealed. Eventually, one of the most significant reasons of severe damage or collapse to masonry structures due to this seismicity is the inability to construct the structures according to the requirements of seismic codes.

## 1. Introduction

The location of Turkey in the Eastern Mediterranean is one of the most active seismic zones in the world. There are 203 events of seismic ground motion records with Mw $\geq$ 6.0 recorded within Turkey and the vicinity territory between 1900 and 2012 [1]. There are 326 separate faults, fault zones or combined system on the updated active fault maps of Turkey. The longest traced fault zones were divided into sections based on their productivity of earthquakes individually. Finally, 485 separate fault sections considered to have the potential to produce seismic ground motion were identified across Turkey (Fig. 1). Four different neotectonic regions (Fig. 1) were proposed by Şengör [2,3] across Turkey: (1) North Anatolian region; (2) Eastern Anatolian contractional region; (3) Central Anatolian planar region; and (4) Western Anatolian extensional region. Each region has particular tectonic characteristics. The East Anatolian Fault (EAF), the North Anatolian Fault (NAF) and the Anatolian-Aegean Subduction Zone (AASZ) are widely known. The NAF has produced destructive earthquakes in the last century since 1939, continuing with the 1992 Erzincan earthquakes and the 1999 Kocaeli and Düzce earthquakes. The EAF zone, that joins the eastern end of the NAF zone to the Mediterranean Sea in the Gulf of Iskenderun, is a band of relatively active seismicity and tectonism [4,5]. The EAF, unlike the NAF, is relatively inactive according to historical records.

The Eastern Anatolian territory, between the Caucasus and the Bitlis–Zagros belt, is currently under a N–S convergent tectonic regime, whereas the western part of Anatolia is moving through N–S continental extension. The interior of Central Anatolia between these two regions is defined by rather complicated tectonic activity on the basis of recent events, presented along strike-slip, normal and reverse faults [7]. Thus, the described movement of the EAF has stored remarkable energy along the fault length [8].

## 2. Literature of field observations after strong seismicity and aim of the study

Researchers have carried out detailed field observations in the world on the damages of various types of structures affected by seismic

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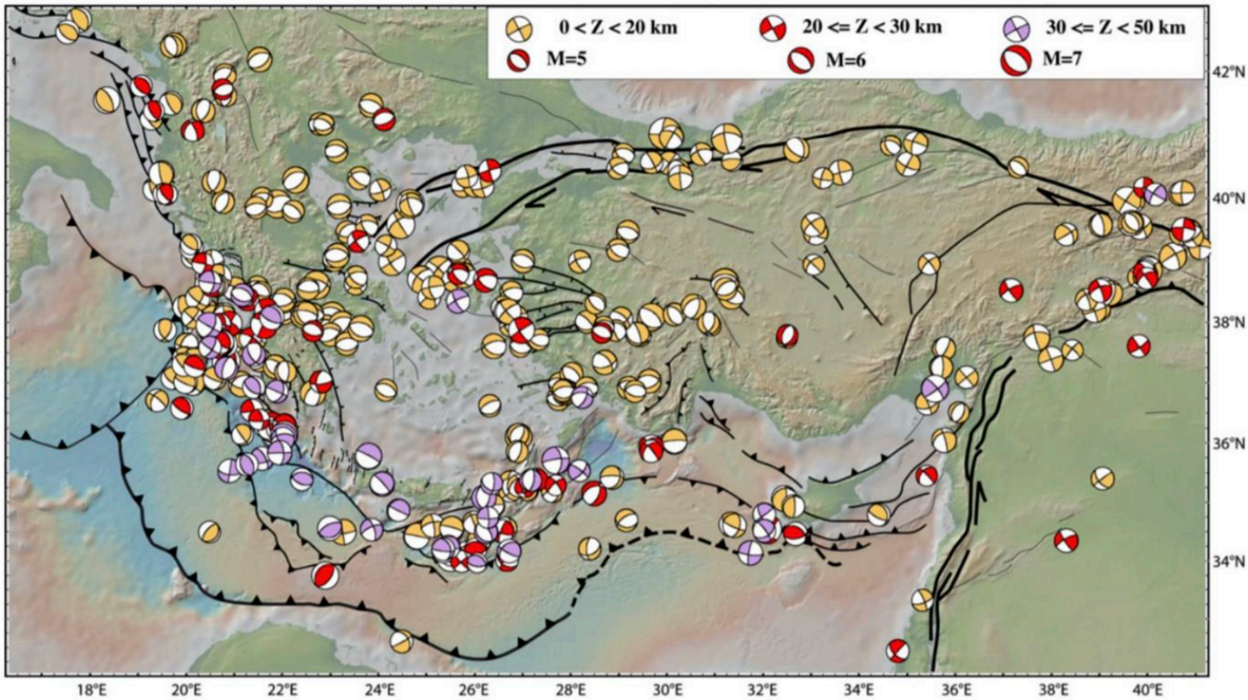



**Fig. 1.** Fault zones and active faults in Turkey [6].

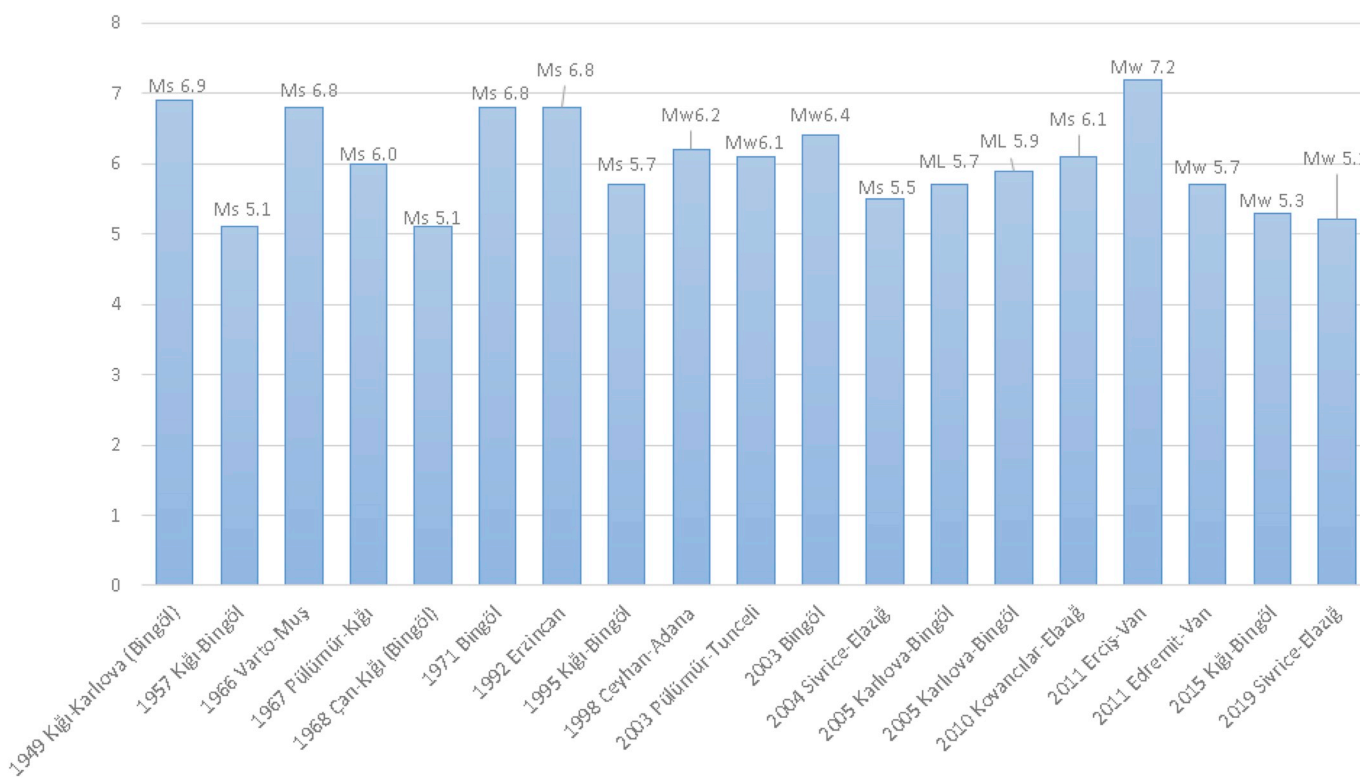



**Fig. 2.** M $\geq$ 5.0 earthquakes on the EAF [33].

shaking. Bayraktar et al. [9,10] carried out studies about the performance of masonry buildings after the 2004 Aşkale-Erzurum and Doğubayazıt-Ağrı earthquakes in Turkey. Celep et al. [11] investigated the seismological aspects with the geotechnical property of the region, the property of the ground motion, and the structural damages based on site assessments after the 2010 Kovancılar and Palu earthquakes in Turkey. Calayır et al. [12] carried out a field investigation about 2010 Kovancılar-Elazığ earthquake damages to reinforced concrete, masonry and different types of structures. Mahmood and Ingham [13] assessed the seismic vulnerability of unreinforced masonry buildings in Pakistan using three empirical (New Zealand, US and Indian) methods. Ural et al. [14] assessed the earthquake response of masonry buildings after the

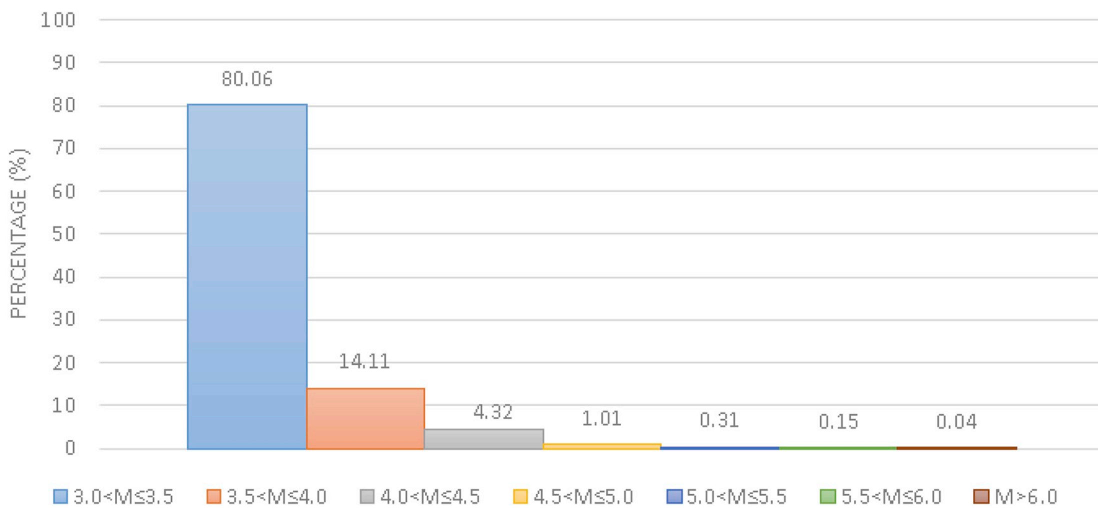



**Fig. 3.** Percentage of earthquakes according to magnitude [33].

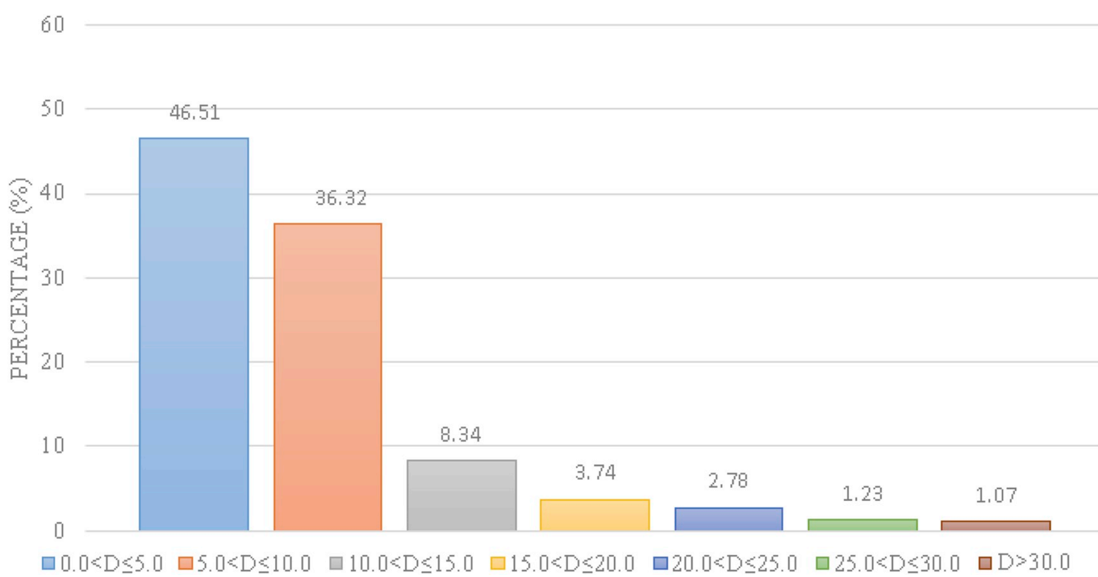



**Fig. 4.** Percentage of earthquakes according to depth [33].

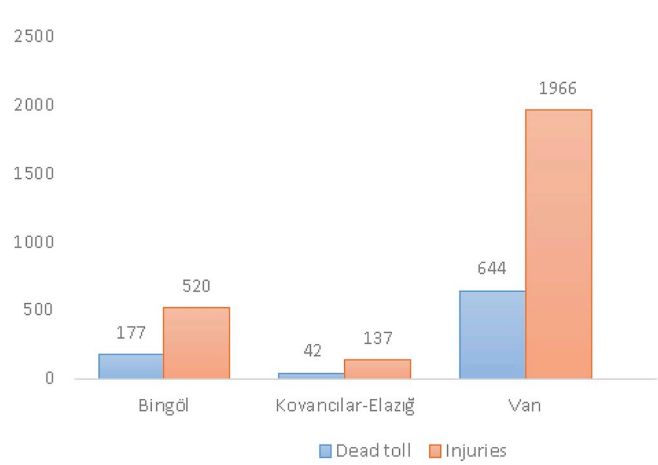



**Fig. 5.** Death and injury statistics of the three major devastating earthquakes on the EAF [12,16,21,24,34].

2007 Bala earthquake. Augenti and Parisi [15] observed structural failures of reinforced concrete and unreinforced masonry buildings during the L'Aquila earthquake that struck Italy in 2009. Sayın et al. [16] observed the damages on masonry and adobe structures after the 2011 Maden-Elazığ earthquake in Turkey. Yön et al. [17] investigated the seismic response of buildings to the 2011 Simav earthquake in Turkey. Ramao et al. [18] addressed the performance of constructions after the 2011 Lorca, Spain earthquake, observing damages in both historical and recent constructions. Sorrentino et al. [19] studied the behavior of vernacular buildings in the 2012 Emilia earthquakes in Italy. They gave some suggestions to improve the earthquake response of these type of structures. Penna et al. [20] evaluated the performance of seismically designed modern masonry buildings in comparison to older ones after the May 20, 2012 Emilia (Italy) earthquake. Sayın et al. [21] assessed failures of masonry structures after the 2011 Van earthquakes. Yön et al. [22] classified the damage and failure of structural and nonstructural elements of urban/rural dwellings arising from all seismic activity in Turkey. Moreover, they presented widespread construction practice interconnected with the misuse of current code. Yön and Onat [23] evaluated the results of one of the EAF-sourced earthquakes that occurred on December 3, 2015, Kığı-Bingöl (Mw = 5.5). Yön et al. [24]

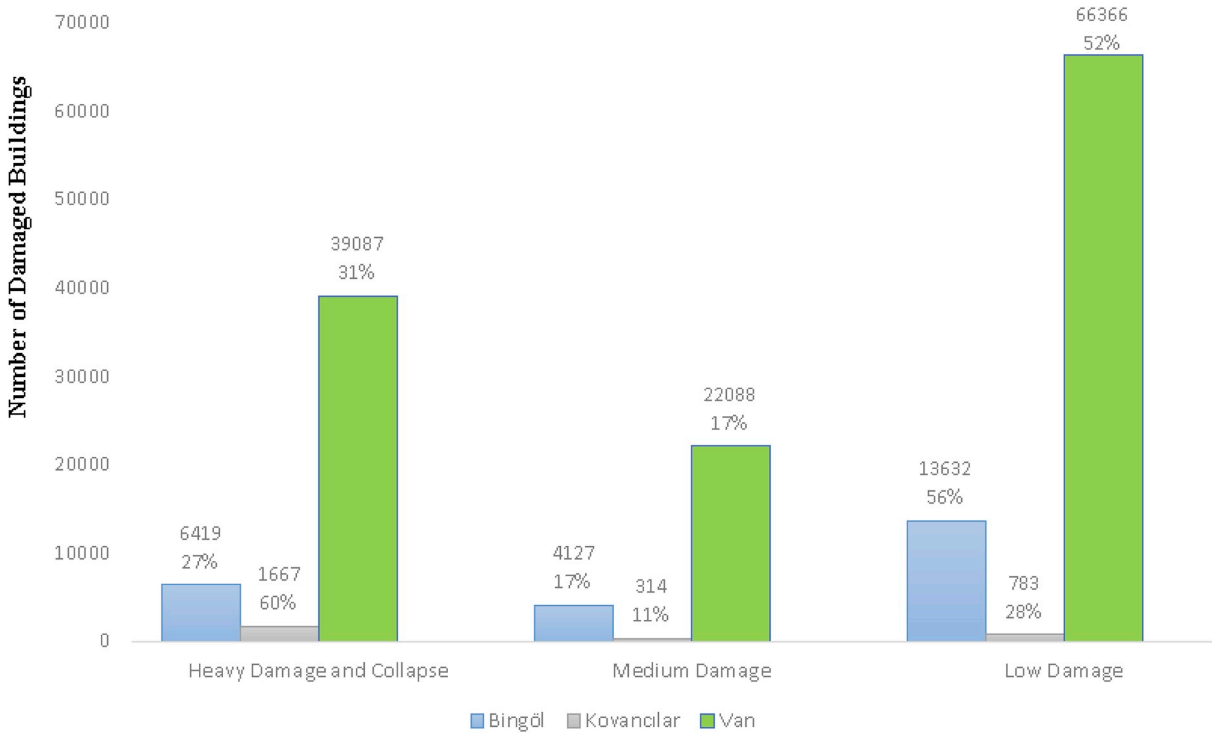



**Fig. 6.** Damage level and number of buildings effected by 2003 Bingöl earthquake, 2010 Kovancılar earthquake and 2011 Van seismic sequence [24,34–36].

**Table 1**
Characteristic parameters of the earthquakes [38].

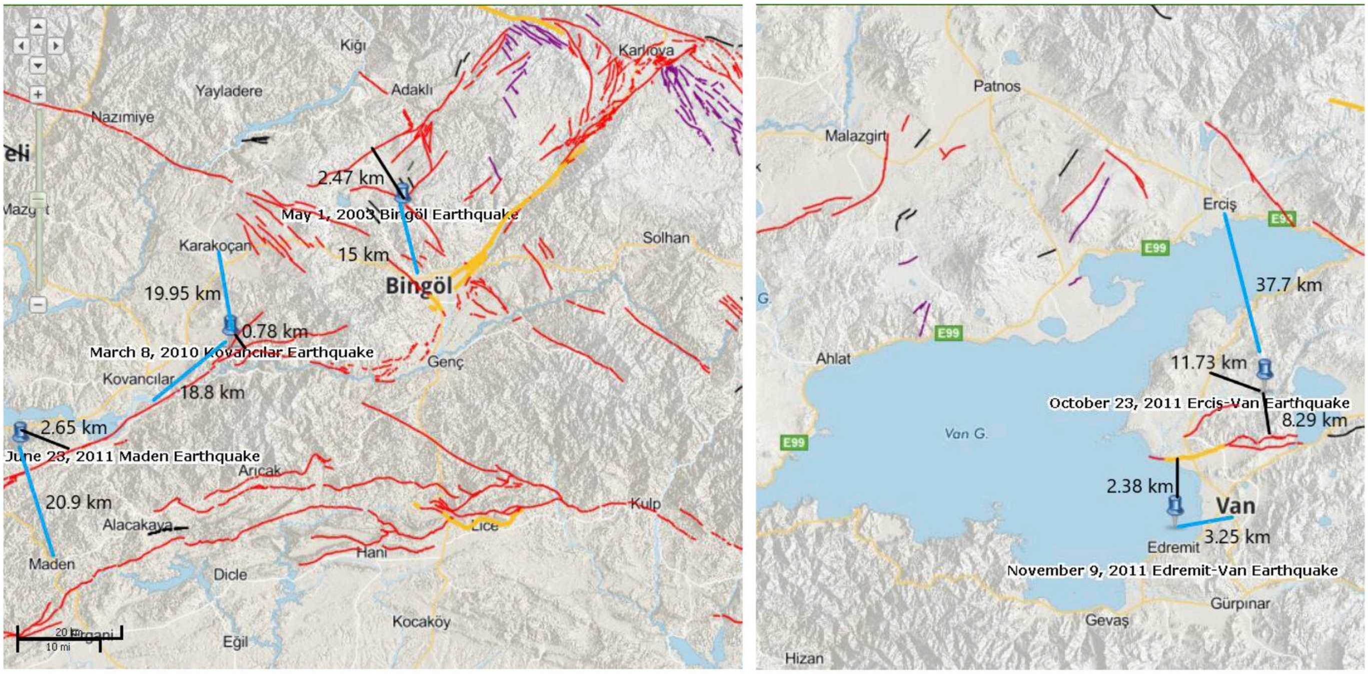
| Earthquake | Date | Station | Station Distance to Epicenter (km) | Lat. | Long. | Depth (km) | $M_w$ | $M_L$ | PGA (cm/s$^2$) | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | N–S | E-W | U-D |
| Bingöl | May 1, 2003 | Bingöl | 11.49 | 38.998 | 40.463 | 10.00 | 6.3 | 6.6 | 545.53 | 276.83 | 472.26 |
| Kovancılar-Elazığ | March 8, 2010 | Palu | 14.72 | 38.7665 | 40.0712 | 5.00 | – | 5.8 | 62.00 | 66.50 | 30.00 |
| Maden-Elazığ | June 23, 2011 | Maden | 21.01 | 38.576 | 39.596 | 13.42 | – | 5.3 | 30.89 | 38.62 | 15.99 |
| Erciş-Van | Oct. 23, 2011 | Muradiye | 45.77 | 38.689 | 43.465 | 19.02 | 7.0 | 6.7 | 178.50 | 169.50 | 79.50 |
| Edremit-Van | Nov. 9, 2011 | Van | 3.37 | 38.447 | 43.263 | 6.09 | – | 5.6 | 148.08 | 245.90 | 150.54 |

performed a study related to the damage and failure reason of nonstructural elements due to the Van seismic sequence in the year 2011. They emphasized that two of the EAF-sourced sequential earthquakes, Erciş-Van and Edremit-Van, especially damaged bearing and non-bearing structural elements, and the majority of the fatalities resulted for this reason. In addition to the literature listed above, it is possible to find scientific papers both focused on field observation and supported by numerical analysis. For instance, Brandonisio et al. [25] investigated damage and performance evaluation of masonry churches in the 2009 L'Aquila earthquake. Milani and Valente [26] studied failures of seven masonry churches damaged during the 2012 Emilia-Romagna (Italy) earthquake. They performed nonlinear dynamic analyses and conventional static approaches of the structures. Clementi et al. [27] assessed the seismic behavior of heritage masonry structures using numerical modeling of "San Francesco" located in Cagli, Marche Region, Italy. Endo et al. [28] assessed the seismic response of a historical masonry church struck by the 2009 L'Aquila earthquake with both field observation and nonlinear analysis (static and dynamic). Indirli et al. [29] studied the technical characteristics of unreinforced masonry (URM) buildings and assessed their seismic behavior during the Abruzzo 2009 seismic event in Italy with both physical and numerical analysis. These studies proved that numerical analysis results support observations.

In this paper, EAF-sourced earthquakes are evaluated in terms of performance of conventional masonry structures damaged from seismic ground motions on the EAF zone. For this purpose, EAF-sourced earthquakes, i.e. 2003 Bingöl, 2010 Kovancılar (Elazığ), 2011 Maden (Elazığ), 2011 Erciş (Van) and Edremit (Van), hit in last two decades on this fault are investigated. Failures of conventional masonry buildings triggered from these earthquakes are assessed. Statistical evaluation, damage of earthquakes and failure pattern are deeply investigated and revealed. Consequently, one of the significant severe failure reasons of masonry dwellings due to this seismicity is the inability to construct the structures according to the requirements of current seismic codes.

## 3. Brief overview of seismological activity of EAF zone

A knowledge of the seismic events on the EAF Zone is crucial to get deep knowledge of the current shaking of the eastern Mediterranean. East of the intersection between the NAF and EAF zones, near Karlıova, is a region of mixed strike-slip and thrust faulting that extends from the Turkey-Iraq border north into the Caucasus [5]. The EAF is composed of six main fault segments, and the traces of the faults vary between 45 km (31 km, [30]) and 145 km (112 km, [30]). These are Karlıova-Bingöl, Palu-Hazar Lake, Hazar Lake-Sincik, Çelikhan-Erkenek, Gölbaşı-Türkoğlu and Türkoğlu-Antakya segments, respectively, from




a).Bingöl, Maden and Kovancılar earthquakes

b). Van earthquake sequences

**Fig. 7.** Earthquake epicenter, fault surface rupture and relative to the locations of city centers (Adopted from Ref. [38]).

the NE to SW direction [31]. Moreover, Emre et al. [30] modified the segment number to seven by adding one more segment, Sincik-Pütürge. Major faults are presented in Fig. 2 with fault zone, thrust zone and Karlıova triple junction. Remarkable energy was discharged by the 1998 Adana-Ceyhan (Mw = 6.2) earthquake, 2003 Tunceli-Pülümür (Mw = 6.0) and Bingöl earthquakes (Mw = 6.3), 2004 Elazığ-Sivrice earthquake (Mw = 5.6), 2005 Bingöl-Karlova earthquakes (Mw = 5.6-5.8-5.6), 2010 Elazığ-Kovancılar and Palu earthquakes (ML = 5.8–5.6), 2011 Elazığ-Maden earthquake (ML = 5.8–5.6) occurring on the EAF and 2011 Van earthquakes (Mw = 7.0 and ML = 5.6) occurring on the Bitlis suture zone situated east of the North and East Anatolian Faults. The last seismic activity on the EAF occurred on April 4, 2019 Sivrice-Elazığ (Mw = 5.2). In the last twenty-five years, between 37.0–40.8° latitude and 38.0–45.0° longitude, 9404 earthquakes with magnitude ranging from 3.0 to 7.5 have been recorded in the east of Turkey. These seismic ground motions caused thousands of human fatalities and billions of dollars of property losses.

Earthquakes records were obtained from Kandilli Observatory and Earthquake Research Institute (KOERI) [32]. These seismic sequential variations are presented in Fig. 2.

The percentage distribution of earthquakes according to magnitude are shown in Fig. 3. This figure presents that more than 90% of the earthquakes that occurred had magnitude less than 4.0, and only 0.5% of the earthquakes had magnitude larger than 5.0. Although the magnitude of earthquakes is quite low, the damage level of the structures causes remarkable loss of life and property. This reality evidently reveals the structural deficiencies of the structure stock in these regions.

In addition to this, the depth of the earthquakes and the percentage distribution according to depth are shown in Fig. 4. In Fig. 4, the horizontal axis shows depth in km. As seen from Fig. 4, more than 80% of the earthquake hypocenters are less than 10 km deep. This graph is strong evidence of shallow earthquakes. It can be considered that shallow earthquakes are the one of the main reasons of major damages.

## 4. Results of strong seismicity on EAF

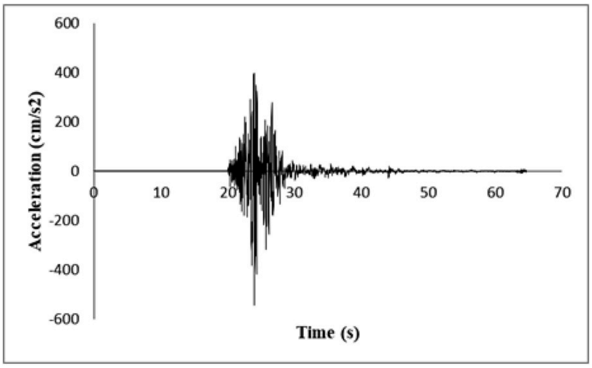
The 2003 Bingöl, 2010 Kovancılar-Elazığ, 2011 Maden-Elazığ, 2011 Tabanlı-Van and 2011 Edremit-Van earthquakes were the major earthquakes. The 2003 Bingöl, 2010 Kovancılar (Elazığ) and 2011 Van earthquake sequences were devastating and resulted in human injuries and fatalities. Moreover, these earthquakes resulted in damage and collapse of residences. After each earthquake, the inspection process was started by loss adjusters. Then, death toll, damage and collapse statistics were revealed. Fig. 5 represents death and injury statistics of the three devastating earthquakes on the EAF.

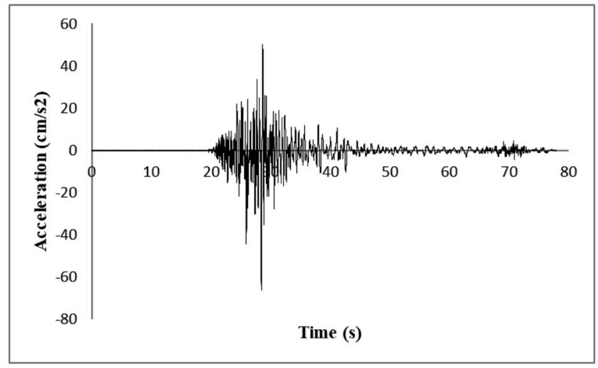
As seen from Figs. 5, 863 people were killed, and 2623 people were injured from these listed earthquakes. Damage of the dwellings and commercial buildings were evaluated, and results are demonstrated in Fig. 6 for the 2003 Bingöl earthquake, 2010 Kovancılar earthquake and 2011 Van seismic sequence, respectively.

After the 2003 Bingöl earthquake, 6208 (28%) dwellings were heavily damaged, 3246 (48%) buildings were moderately damaged, and 12888 (58%) dwellings were slightly damaged, 11% of the commercial buildings were heavily damaged, 48% of the commercial buildings were moderately damaged, and 41% of the commercial buildings were slightly damaged. Rather less loss occurred at the 2010 Kovancılar earthquake. Specifically, 1580 (62%) dwellings were heavily damaged, 269 (22%) dwellings were moderately damaged, and 709 buildings were slightly damaged, 87 (42%) commercial buildings were heavily damaged, 45 (11%) commerical buildings were moderately damaged, and 74 (28%) buildings were slightly damaged. After the 2011 Van earthquake sequence, 36203 (32%) dwellings were heavily damaged, 18181 (26%) dwellings were moderately damaged, and 58374 (54%) dwellings were slightly damaged, 2884 (20%) commercial buildings were heavily damaged, 3907 (16%) commercial buildings were moderately damaged, and 7992 (52%) commerical buildings were slightly damaged.

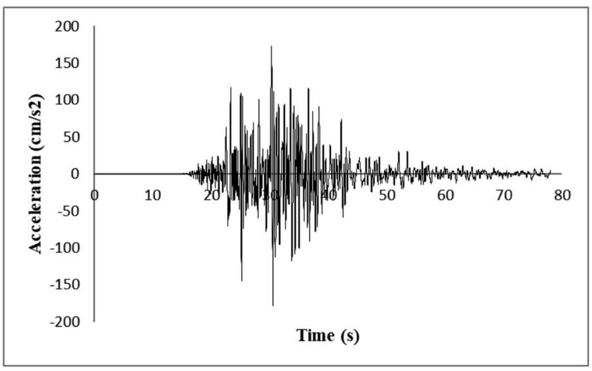
## 5. Characteristics of major earthquakes on EAF

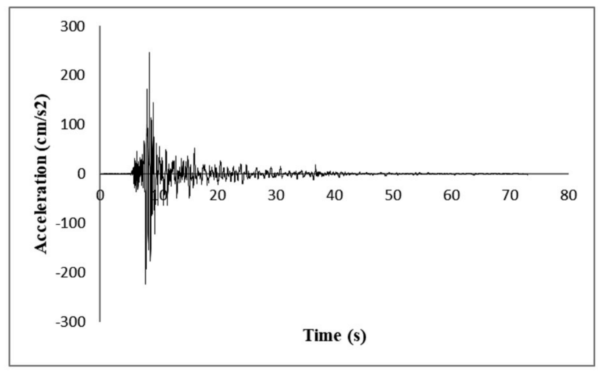
The acceleration records obtained from Republic of Turkey, Interior Ministry, Disaster and Emergency Management Agency (DEMA) [37] and characteristic parameters are presented in Table 1. This table also represents the distance of the accelerometer location relative to the



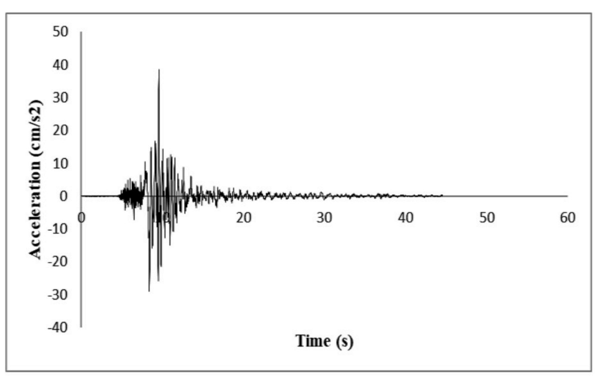




a) N-S component of May 1, 2003 Bingöl earthquake b) E-W component of March 8, 2010 Elazığ-Kovancılar- earthquake




c) N-S component of October 23, 2011 Erciş-Van earthquake d) E-W component of November 9, 2011 -Edremit-Van earthquake




e) E-W component of June 23, 2011 Elazığ-Maden

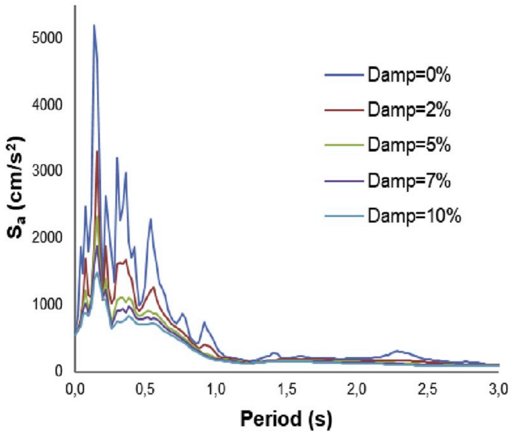
**Fig. 8.** Peak Ground Accelerations of various components of earthquakes.

earthquake epicenter. In addition, a map to show the fault surface rupture, relative to the locations of damaged buildings are provided in Fig. 7.

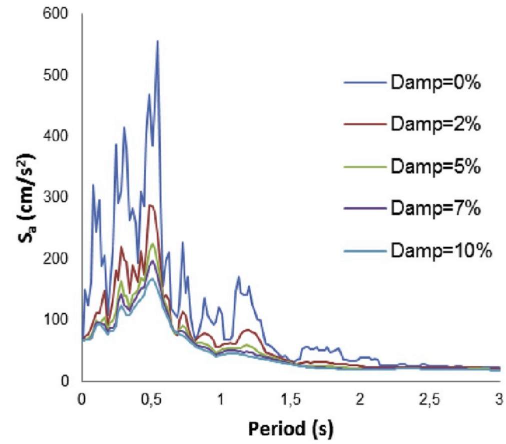
In Fig. 7, the black solid line shows the distance between fault and epicenter, while the cyan color represents the distance between city center and epicenter.

Fig. 8 shows the maximum peak ground acceleration (PGA) values of various components of these records. The station records were reproduced to obtain the acceleration response spectra of the PGA components for $\xi = 0$, 2, 5, 7 and 10% damping ratios, which are presented in Fig. 9. These values show the effect of damping.

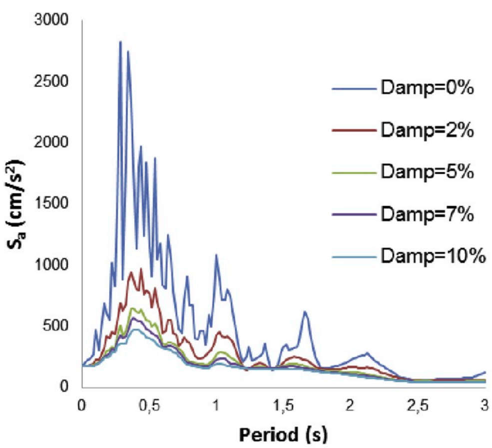
The acceleration response spectra of various components of investigated earthquakes according to the damping ratio of 5% are depicted in Fig. 10a together with the design spectra of four soil classes defined in the Turkish Seismic Code (TSC-2007) [39]. In addition to this, normalized spectra according to the maximum accelerations are presented in Fig. 10b. According to the TSC-2007, the stiffness of the soil class decreases from Z1 to Z4. The design spectral curves, which are calculated for the first seismic zone according to all soil classes, are larger (except for N–S components of the Bingöl earthquake) than the response spectra



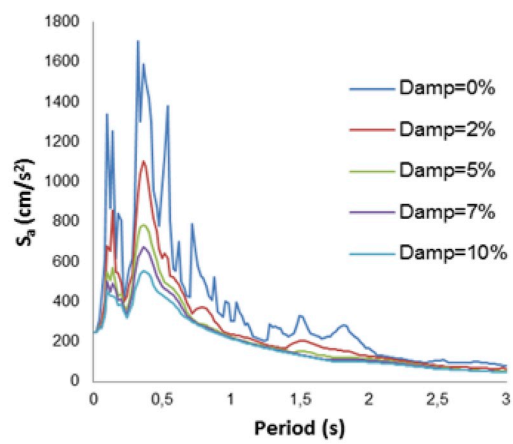

**a)** N-S component of May 1, 2003 Bingöl-earthquake



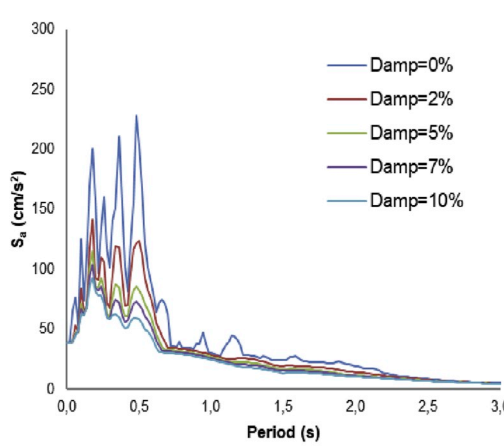

**b)** E-W component of March 8, 2010 Elazığ- Kovancılar earthquake




c) N-S component of October 23, 2011 Erciş-Van earthquake

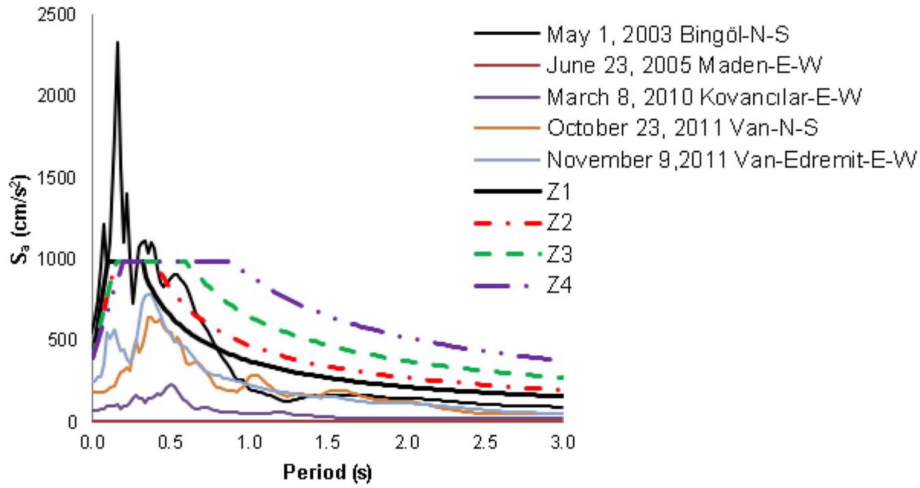



d) E-W component of November 9, 2011 Edremit - Van earthquake

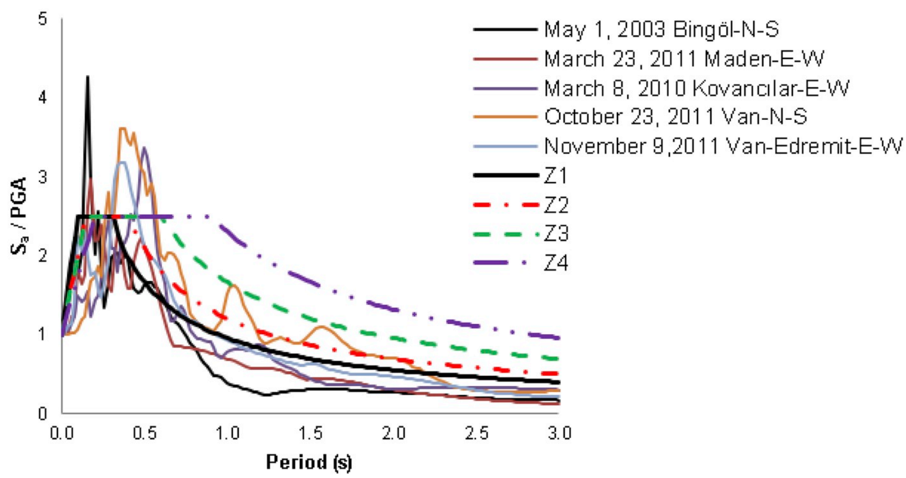



e) E-W component of June 23, 2011 Maden

**Fig. 9.** Response spectra for various components of the earthquake's acceleration records.




a) Spectral curves




b) Normalized spectral curves

**Fig. 10.** Comparison of response and design spectra.

of the earthquake records as seen in Fig. 10a. In addition to this, the amplification factors of these normalized earthquake acceleration records exceed the limit (2.5) of the design code. This situation can be seen in Fig. 10b. Although the response spectra of the ground motions are quite low (except N–S component of Bingöl earthquake) compared to the design spectra, the damage and loss of life and property that occurred evidently reveal the structural deficiencies of the structure stock in the affected regions.

## 6. Classification of masonry structures and proposed solutions

Most of the masonry structures of the affected regions which are one- and two-storey buildings were built by local people without any engineering service. These buildings consist of adobe, stone, and briquette masonry with cement or mud mortar. Failure and damage classification of masonry structures due to the earthquakes are presented below.

### *6.1. Earthen roofs*

Masonry structures, used for housing and barns, in the east of Turkey are built using local materials and conditions. These available materials are not always proper material for construction. The earthquake resistance of these type of structures is weak because they are designed to resist only vertical loads. Heavy earthen roofs increase the mass of the structures and cause large inertial forces during the ground motions. Field observations of the authors present that heavy earthen roof collapse occurs due to the combined effect of soil amplification, increasing lateral inertial forces causing heavy damages. Fig. 11 illustrates these failures.

### *6.2. Corner damages*

Corner damages occur at wall-to-wall connections when subjected to out-of-plane deformations due to the lack of connection between load-bearing walls. Bad workmanship (Fig. 12b,c) and low material quality (Fig. 12b,c) resulted in the failure of the masonry dwellings. Also, similar damages can occur at the intersection (Fig. 12a,d) of the wall and the floor or roof. To prevent these damages and increase the earthquake performance of masonry structures, the TSC requires reinforced concrete vertical bond beams. Corner damages were common in the masonry structures in the earthquake regions. Weak connections between walls and the absence of bond beams caused considerable damages. In addition, it should be emphasized that there were no suitable connections at the corners of the walls of damaged buildings. This type of damage for

a)

b)

c)

**Fig. 11.** Heavy earthen roof damage: (a) Van (Photos by B. Yön, M. E. Öncü); (b) Kovancılar (Photos by B. Yön, M.E. Öncü, A. Karaşin); (c) Bingöl (Photos by A. Karaşin, M.E. Öncü).

a).

b).

c)

cd)

**Fig. 12.** Corner damages in masonry structures, a) Bingöl (Photos by M.E. Öncü, A. Karaşin), b) Van (Photos by B. Yön, M. E. Öncü), c) Kovancılar (Photos by B. Yön, M.E. Öncü) and d) Maden (Photos by B. Yön).

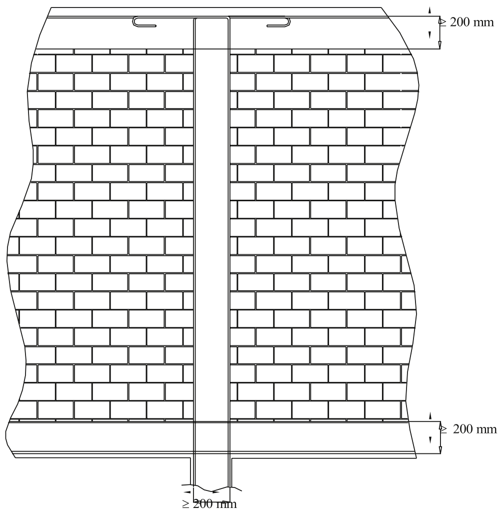



**Fig. 13.** Vertical bond beams at load-bearing walls.

masonry structures is presented in Fig. 12.

To limit corner damage effects, reinforced concrete vertical bond beams, which increase the lateral stiffness of masonry buildings, should be used. Fig. 13 illustrates vertical bond beams at load-bearing walls required in the TSC. In these beams, the compressive strength of concrete should be at least 16 MPa, and Ø8 stirrups and a maximum spacing of 200 mm should be used together with longitudinal reinforcement.

These bond beams should be used for the entire height on the corners of the buildings, along the intersections of the load-bearing walls. The cross section of these beams should be equal to the thickness of the walls that intersect at the corners of the buildings. Furthermore, the other cross-sectional dimension should not be less than 200 mm.

### 6.3. Out-of-plane failure mechanism

In-plane masonry walls show high performance compared to out-of-plane performance, and their rigidity is also high. But in the perpendicular direction, they have weak performance, and their responses generally are flexible. Therefore, the shear mechanism is an effective in-plane mechanism. Whereas the flexural mechanism affects the weak direction where the out-of-plane mechanism occurs, the out-of-plane

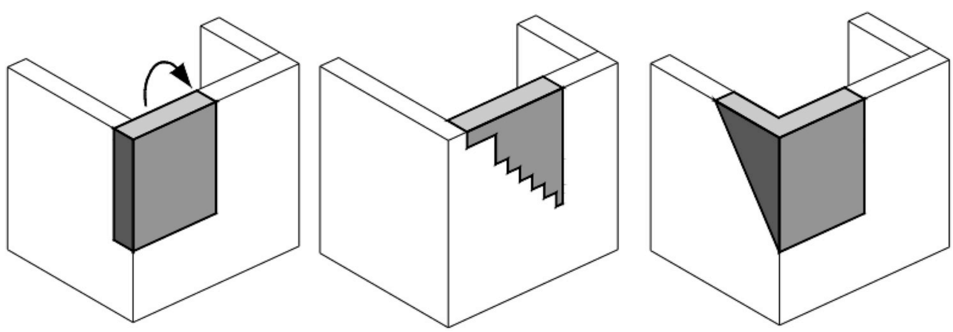


**Fig. 14.** Various out-of-plane mechanisms (adapted from Ref. [39]).


**Fig. 15.** Out-of-plane collapse of bearing walls in structures: (a) Bingöl (Photos by A.Karaşin, M.E. Öncü); (b) Van (Photos by B. Yön, M.E.Öncü); (c) Kovancılar (Photos by B. Yön, M.E.Öncü, A.Karaşin); (d) Maden (Photos by B. Yön).

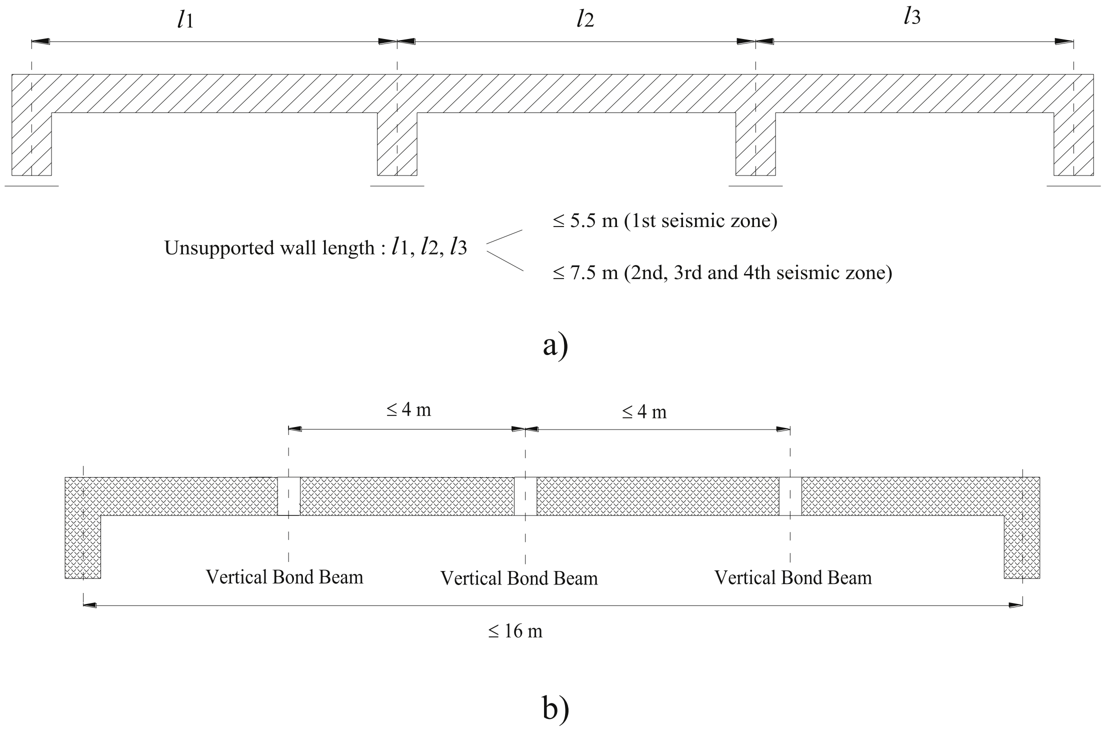



**Fig. 16.** Maximum unsupported wall length and span between vertical bond beams.

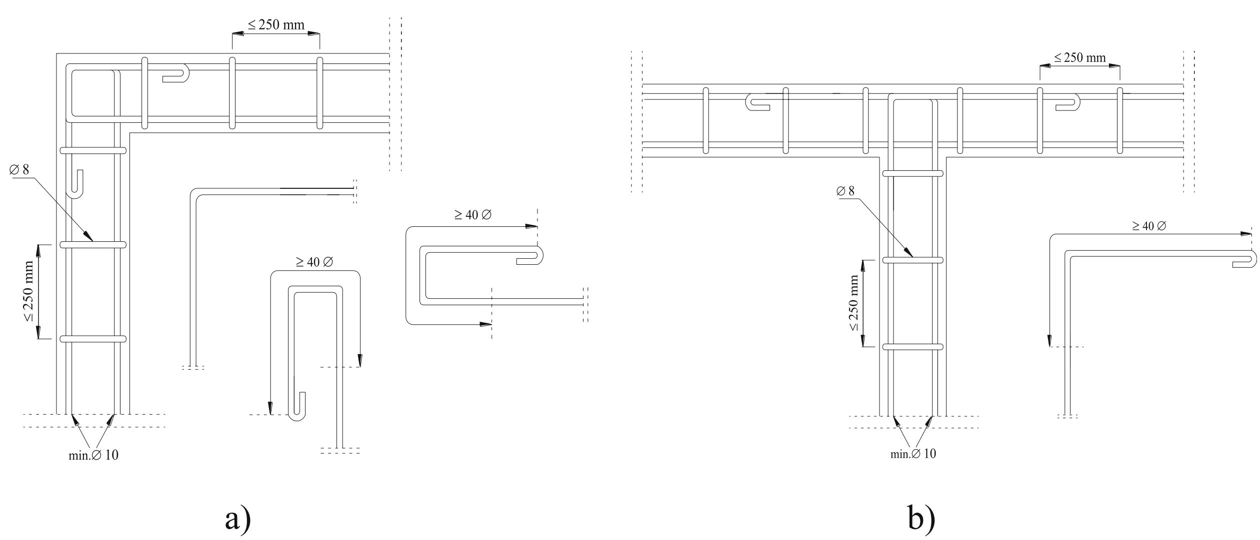



**Fig. 17.** Horizontal bond beams at the corner (a) and intersection (b).

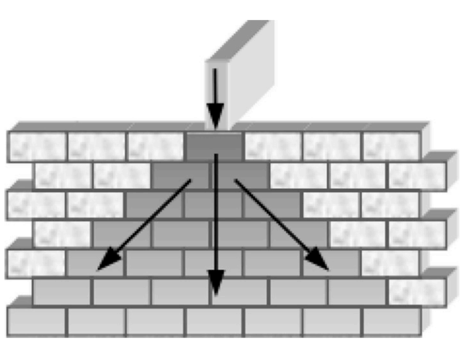


**Fig. 18.** Distribution due to in-plane loading (adapted from Ref. [41]).

mechanism represents especially brittle response compared to in-plane mechanism. Improper connections between two orthogonal walls, wall-to-floor connection or wall-to-roof connection, large unsupported wall lengths, absence of bond beams cause the out-of-plane failure mechanism. In addition, the stiff floor concept becomes important to prevent this type of failure mechanism. This floor creates a rigid diaphragm and provides unity of walls. Also, vertical and horizontal bond beams in the wall provide tensile and friction forces on the plane of the wall surface. If there is no girder on the wall, partial or complete overturning failure mechanism occurs. Fig. 14 shows various out-of-plane mechanisms. It was seen that the observed failure mechanism was commonly triggered by ignoring code requirements. Fig. 15 illustrates the out-of-plane mechanism of masonry structures.

To prevent this type of damage to adobe buildings, timber bond

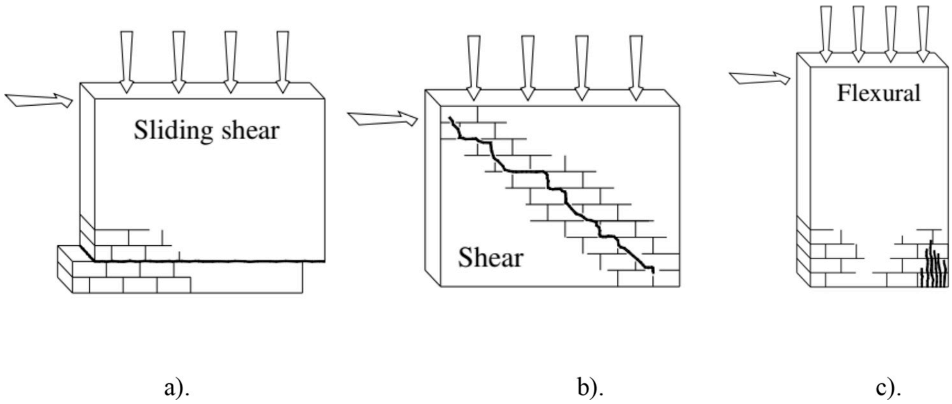



**Fig. 19.** Typical failure modes of masonry walls due to in-plane seismic load (adapted from Ref. [42]).

a) Shear and sliding with shear cracks to masonry structures in Maden (Photos by B. Yön)

b) Flexural and shear damages to piers and walls at masonry structures in Van (Photos by B. Yön, M.E. Öncü)

c) Shear and sliding damages to masonry structures in Kovancılar (Photos by B. Yön, M. E. Öncü, A. Karaşin)

**Fig. 20.** In-plane mechanisms of the masonry structures.

beams should be used. According to the TSC, timber beams should be two elements of square sections of size 100 × 100 mm and should be placed with the outer faces coinciding with the exterior and interior wall surfaces. These pieces should be tied to each other every 500 mm with nail-jointed timber elements with a cross section of 50 × 100 mm.

In addition to this, different applications should be used for masonry

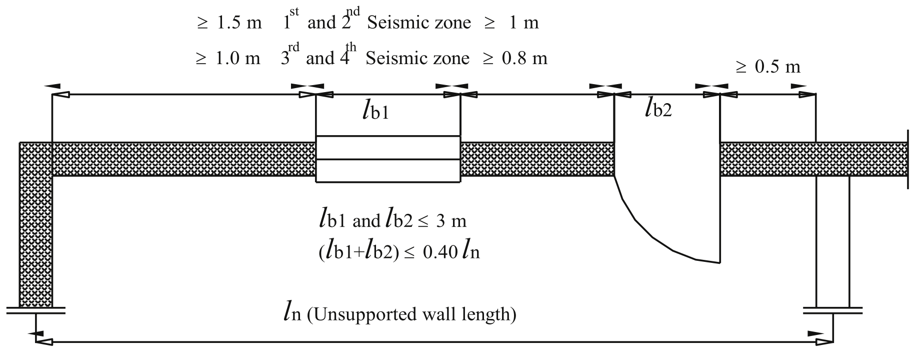



**Fig. 21.** Maximum openings in load-bearing walls according to the TSC.

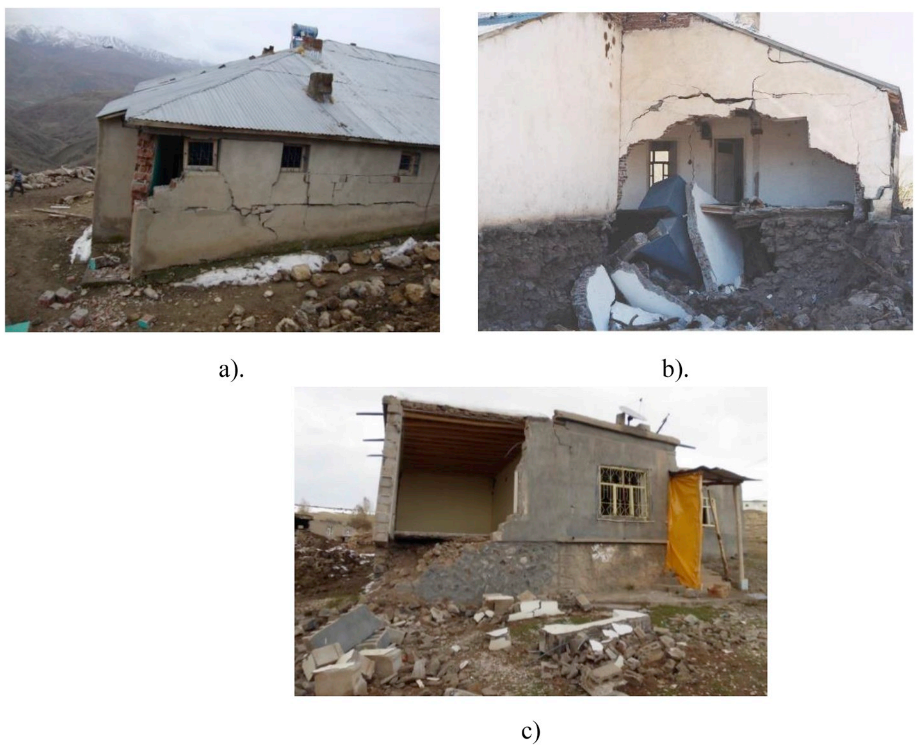



**Fig. 22.** Damages to structures located on sloping land a) Kovancılar (Photos by B. Yön, M. E. Öncü, A. Karaşin), b) Bingöl (Photos by M. E. Öncü, A. Karaşin) and c) Van (Photos by B. Yön, M. E. Öncü).

structures. The code requires that the maximum unsupported length of a wall should not exceed 5.5 m in the first seismic zone in the plan and 7.5 m in the second, third and fourth seismic zones in the plan. However, this unsupported length should not exceed 4.5 m in adobe buildings in all seismic zones (Fig. 16a). In addition, if these requirements are not provided, reinforced concrete vertical bond beams should be constructed along the full-storey height at the corners. Furthermore, these beams should be used every 4 m along the in-plane wall, and the unsupported wall length should not exceed 16 m (Fig. 16b).

The TSC requires that reinforced concrete horizontal bond beams should be used on the walls that support floors. These bond beams should be cast monolithically with the floors. The width of the horizontal bond beams and wall should be equal. The height of the horizontal bond beams should not be less than 200 mm. The concrete quality for the bond beams should be at least 16 MPa, and Ø8 stirrups and a maximum spacing of 250 mm should be constructed together with the longitudinal reinforcement. Figs. 17a,b illustrate the horizontal bond beams at the corner and intersection of the walls, respectively.

### 6.4. In-plane mechanism

The in-plane mechanism consists of various types of failure patterns, such as sliding, shear and flexural cracks. However, these cracks can occur together [40]. Fig. 18 shows the vertical load distribution in the plane of the wall. If a heavy vertical load affects the wall, cracks will occur. However, the in-plane mechanism arises from shear effects while acting forces in-plane direction of the wall.

It is possible to classify the in-plane loading as shear behavior and flexural behavior. Fig. 19a shows sliding failure, where incrementally or horizontally cracks progress along the lengthwise plane of the masonry panel, which is divided into two parts that slide along their fracture surface. Fig. 19b illustrates diagonal shear cracking, in which tensile cracks progress along a diagonal of the lengthwise plane. Finally, the masonry panel is divided into two parts that move away from each other. The cracks that occur are usually X-shaped. When shear cracks increase in the plane of the wall and become unsuitable, a part of the wall collapses. Fig. 19c presents flexural failure, in which compression vertical fissures and tensile horizontal fissures accumulate at the toes of

a) b) c) d)

**Fig. 23.** Damages to masonry walls for various earthquakes, a) Bingöl (Photos by A. Karaşin, M.E.Öncü), b) Kovancılar (Photos by B. Yön, M. E. Öncü, A. Karaşin), c) Van (Photos by B. Yön, M. E. Öncü), d)Maden (Photos by B. Yön).

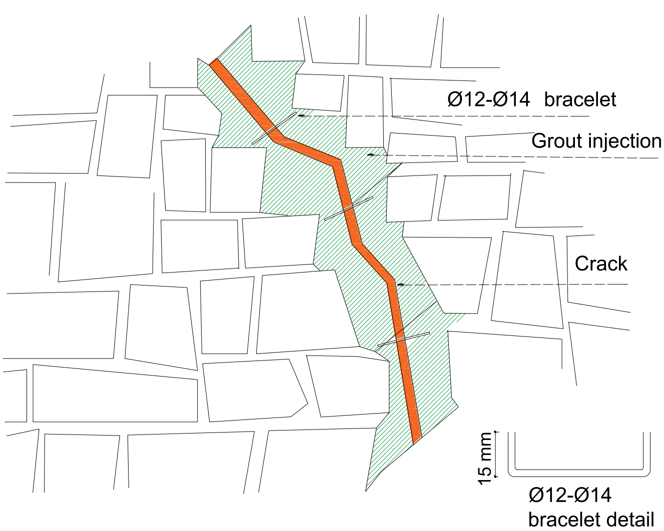



**Fig. 24.** Repairing minimal cracks with U-shaped metal units and grout injection (Adapted from Ref. [45]).

the panel geometry due to yielding of masonry.

The authors observed that the in-plane mechanism occurred mostly in masonry buildings because their walls did not have sufficient shear strength. Fig. 20 illustrates the in-plane mechanism of the masonry structures.

Many masonry buildings did not have sufficient and proper bond beams to enhance the lateral strength of the walls affected by earthquake. Furthermore, large openings that decrease the stiffness of the walls increased the shear effects. To limit this type of failure, the TSC requires that reinforced concrete bond beams should be used (Figs. 13 and 17). Also, the requirements of the openings of load-bearing walls are illustrated in Fig. 21.

### *6.5. Damages arising from soil and foundation*

Structures located on sloping land or made-up ground can be damaged seriously. Foundation elevations of structures on slopes vary along the length of foundations in the plan. While one side of the structure can be on stiff soil, the other side can be on made-up ground. The side of the structure that with elevation lower than ground level is forced by lateral pressures arising from the soil. The increasing pressure during ground motion causes lateral movement of the structure. Fig. 22 shows damages arising from soil conditions.

### *6.6. Local damages*

In Turkey, especially in the rural areas, stone masonry buildings are constructed in multi-layers, which can contribute to their disintegration. In this type of wall, spaces between layers are filled with pitch-faced stones and bonded together with mud mortar. The out-of-plane mechanism causes disintegration of the wall because of inadequate connections between the inner and the outer layers of the walls. Additionally, the low quality of construction, poor workmanship, and using improper materials cause the disintegration. This type of failure generally occurs in the upper segment of the wall because, at the top of the building, the normal stress distribution is low due to gravity loading. However, this type of failure depends on the low shear strength of the wall. Fig. 23 presents these failures in stone masonry buildings for various earthquakes.

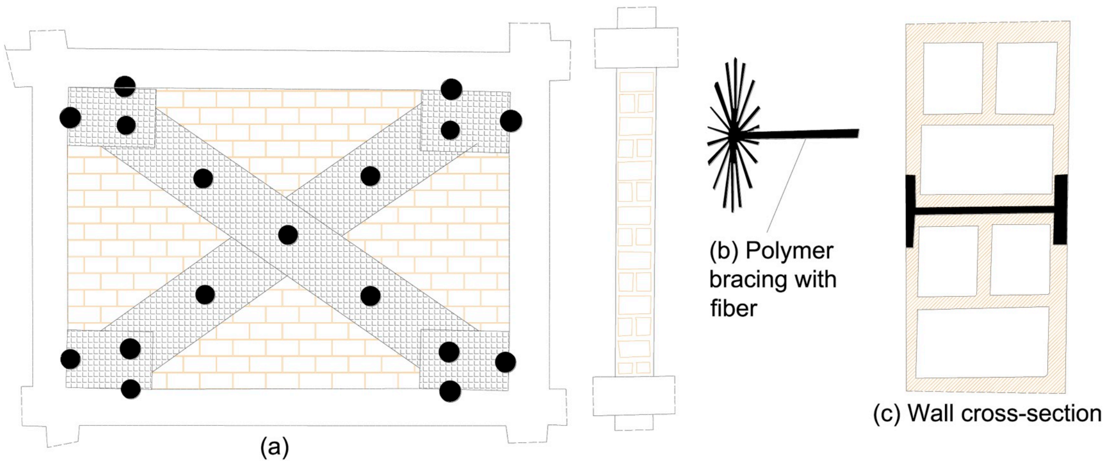



**Fig. 25.** Reinforcing a masonry wall by using polymer strips.

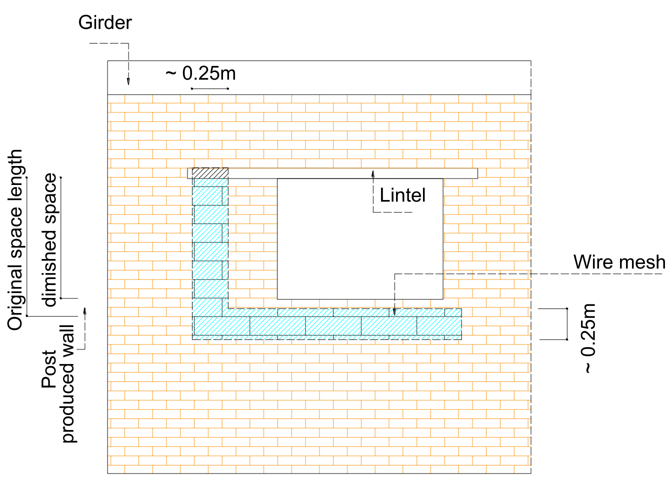



**Fig. 26.** Repair technique for open spaces in masonry walls [45,46].

## 7. Reinforcing techniques for existing masonry structures

The basic philosophy of strengthening and implementation, through the techniques proposed below, is long proven and present in many masonry dwellings [43,44]. These proposed techniques include the combination of decreasing additional mass, replacing an earthen roof with a rather light roof material and increasing the stiffness of structural elements. i.e. repairing minimal cracks with U-shaped metal units and grout injection (Fig. 24). Moreover, a masonry wall enclosed with a girder can be reinforced by using polymer strips from corner to corner (Fig. 25). Therefore, the in-plane bearing capacity can be increased, and out-of-plane failure can be prevented.

To increase the safety of a masonry wall, one of the reinforcing techniques used is to decrease the useless dimensions of openings or close the extra openings by bonding, which can be applied generally (Fig. 26). Thus, stiffness can be increased by decreasing openings.

The connection between two masonry walls perpendicular to each other (Fig. 27a) and the connection between a masonry wall to a vertical girder (Fig. 27b) can be anchored to each other by using a steel plate and nail to fix. Then, gaps through the repaired area should be filled with ready-mixed grout.

In the former type of repair, the masonry wall is coated with steel mesh and anchored with nails. Then, the shotcrete application includes

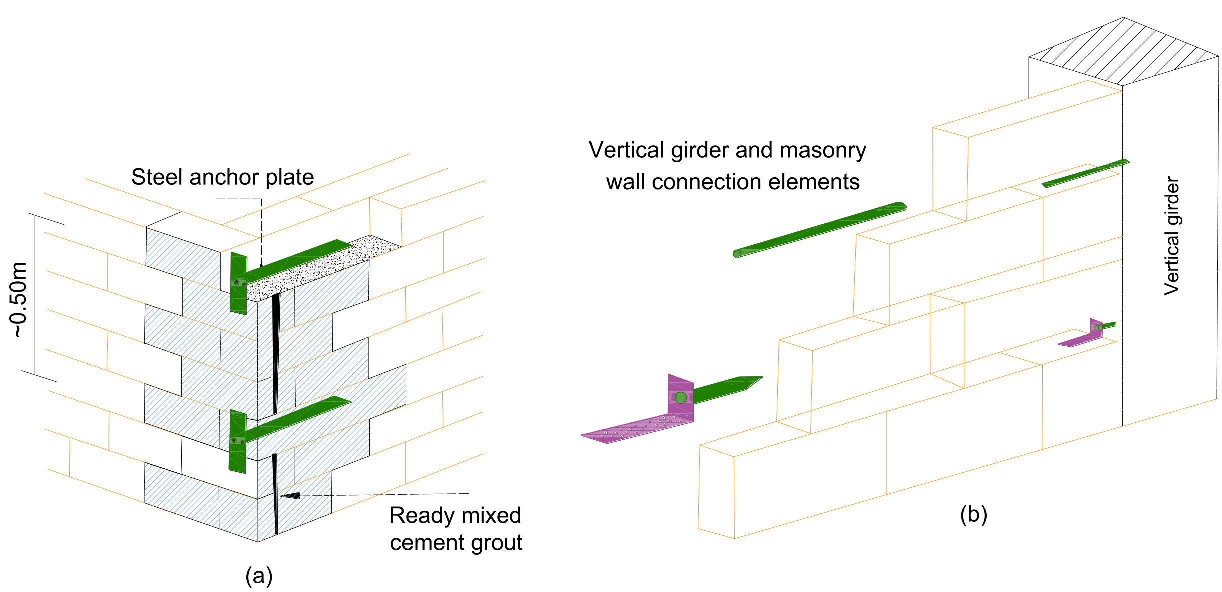



**Fig. 27.** a) Corner repair wall-to-wall connection and b) Repair wall-to-girder connection [45,46].

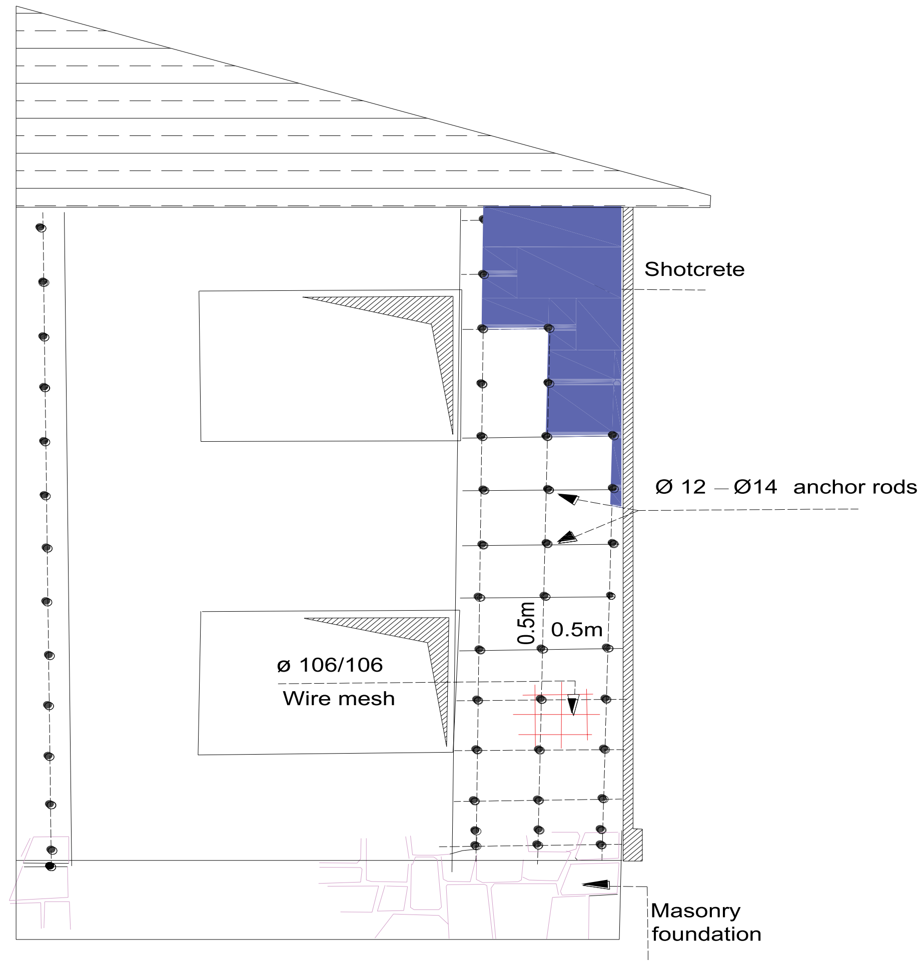



**Fig. 28.** Shear transfer system with surface application.

spraying cement mortar by a nozzle onto the masonry surface. The sprayed shotcrete provides a homogeneous layer on the wall. The weak or cracked surface of the members are bonded together with sprayed shotcrete layer. The shear transfer of this application is ensured with shear keys. Fig. 28 demonstrates a schematic of the process.

## 8. Conclusions and final remarks

As a final remark, it has to be emphasized that the EAF is also active fault compared to the NAF. According to the updated seismic map, nine different faults are available on the EAF. Most of the fault ruptures are triggered by tectonic-sourced strike-slip movement. Unfortunately, rarely reported seismic activity is not associated with a certain tectonic reason. Moreover, landslide-sourced seismic activities were also observed in the past. This vague reason of movement on the fault trace changes the diversity of the failure reason of the structure stock. Many masonry structures were damaged severely or collapsed due to a variety of damage and collapse reasons. The depths of the EAF earthquakes are shallow, and this situation causes serious damages to masonry structures. The main significant reasons of damages and collapse of structural stock are listed as follows:

- Heavy earthen roofs
- Insufficient corner connection, and therefore propagation of out-of-plane mechanism
- Weak in-plane bearing capacity
- Soil–structure interaction
- Minor local damages

From the observations, it is seen that the heavy earthen roofs caused heavy damages due to increasing the mass of the structures and caused large inertial forces during the earthquakes.

Corner damages were common in the masonry structures in the earthquake regions. Weak connections between walls and the absence of bond beams caused considerable damages. In addition, it must be emphasized that there were no appropriate connections at the corners of walls in the damaged buildings. The solution procedure for existing structures can be solved by applying anchorage wall-to-wall and wall-to-vertical-girder by using a steel plate and nail to fix.

There were many reasons of the out-of-plane failure mechanism such as improper connections between two orthogonal walls, walls and floors or roofs, large unsupported wall lengths, and absence of bond beams. This type of mechanism occurs when the entire wall or a significant part of it falls down during ground motions. This problem can be eliminated for existing masonry walls enclosed with a girder by reinforcement using polymer strips from corner to corner. Moreover, decreasing the useless dimensions of openings or closing the extra openings by bonding is another alternative solution to increase in-plane bearing capacity and prevent out-of-plane failure.

The in-plane mechanism was mainly observed as shear cracks. The earthquake resistance of masonry buildings is related to the shear strength of walls. Increased shear forces during earthquakes caused damages at walls and their connections. The suggested procedure for available structures propagating this type of damage on the masonry wall is to coat with steel mesh and anchor with nails and then to spray with shotcrete.

The structures located on sloping or made-up ground were damaged seriously due to the local soil features. The increased stress during the ground motion causes lateral movement of the structure. To prevent this movement, a retaining wall should be constructed.

The layers of walls were separated from each other during earthquakes because of the inadequate connections between inner and outer layers of the walls. Additionally, the low quality of construction, bad workmanship, and not using proper materials cause the disintegration. To prevent this type of local damage, minimal cracks should be repaired by using U-shaped metal units and grout injection.

As a result, according to observations, damages to the masonry structures in various regions were affected by the earthquakes in the same manner as listed in the above-mentioned section. The common point of damaged structures was explained as changing the seismic loading condition with different rupture of fault. Whereas the common point of collapsed structural stock not built with respect to the requirement of the code, so the demand performance of the structures was not provided for medium magnitude earthquakes. To prevent human casualties and to decrease property losses, existing structures must be retrofitted, and new buildings have to be constructed according to the current codes. The construction period has to be checked by structural engineers.

## Declaration of competing interest

The authors do not declare any.

## CRediT authorship contribution statement

**Onur Onat:** Methodology, Data curation, Writing - original draft, Writing - review & editing. **Abdulhalim Karaşin:** Investigation, Writing - review & editing.

## Appendix A. Supplementary data

Supplementary data to this article can be found online at https://doi.org/10.1016/j.soildyn.2020.106126.

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