



Structural Performance of Masonry Buildings Under Harsh Climate and Seismic Loads

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Abstract

Masonry buildings in Van which is a city located at the eastern border of Turkey exposed to several earthquakes in 2020 while continuously being subjected to harsh weather conditions. Two earthquakes with $M_w = 5.9$ occurred on the same day and four months after another one hit the region. Besides the earthquakes 124 freeze-thaw cycles occur annually and the region has snow and rain approximately 103 days. Both continuously and periodically applied environmental exposures due to harsh weather conditions and earthquakes that is significant in a short period, affects the structural performance of the masonry buildings adversely. During the field investigations after the earthquakes, it was realized that none of the buildings comply with the available codes, foundations, and walls damaged due to the environmental exposures and those damages either increased or end up with total collapse due to the seismic loading. Besides the damaged buildings, it was observed that some buildings were strengthened locally at the corners. L shaped reinforced concrete columns were added to the cracked corners to increase the lateral capacity of the walls. That strengthening seemed to be adequate for low seismicity because no significant damage was seen.

Keywords Masonry buildings · Earthquake · Seismic performance · Environmental loading · Rubble stone masonry

1 Introduction

Masonry buildings in Van which is a city located at the eastern border of Turkey exposed to several earthquakes in 2020 while continuously being subjected to harsh weather conditions. The first earthquake that hit the region was occurred on February 23, 2020 and the epicenter was located between the northeast of Başkale-a town of Van City in Turkey, and Khoy City of Iran. The magnitude of this earthquake was reported to be 5.9 ($M_w = 5.8$ acc. to KOERI 2020) and the depth was measured as 14.9 km according to AFAD (2020a). Another earthquake, 10 h after the previous one hit the same

region again with the same magnitude of $M_w = 5.9$ (depth was 8.1 km) as reported by AFAD (2020a). Ten people and more than 100 animals were killed, 97 people were injured in these earthquakes. 255 residential buildings and 400 barns were collapsed, and more than 600 buildings were severely damaged. The third earthquake occurred on June 25, 2020 with $M_w = 5.4$ and epicentral depth of 7.48 km at Saray, a town of Van City in Turkey (AFAD 2020b). No human loss was recorded but 5 people were injured in this earthquake however several residential buildings and barns were collapsed and severely damaged. Figure 1 shows the seismic activity during the first and second earthquake (February 2020) and during the third earthquake (June 2020). It is obviously seen that the seismic activity was dense in the region in February (KOERI 2020).

The seismic activity mentioned above affected the villages nearby. According to the visual inspections from reconnaissance team of VADUM and SARGEM (Sağlam Selçuk 2020a, 2020b), in the first and second earthquakes the villages called Bilgeç, Özpınar, Ömerdağı, Gelenler, Kaşkol and Böğrüpek experienced many damages. Since those villages mainly have masonry buildings, the effect of earthquakes was more pronounced. 70% of the dwellings

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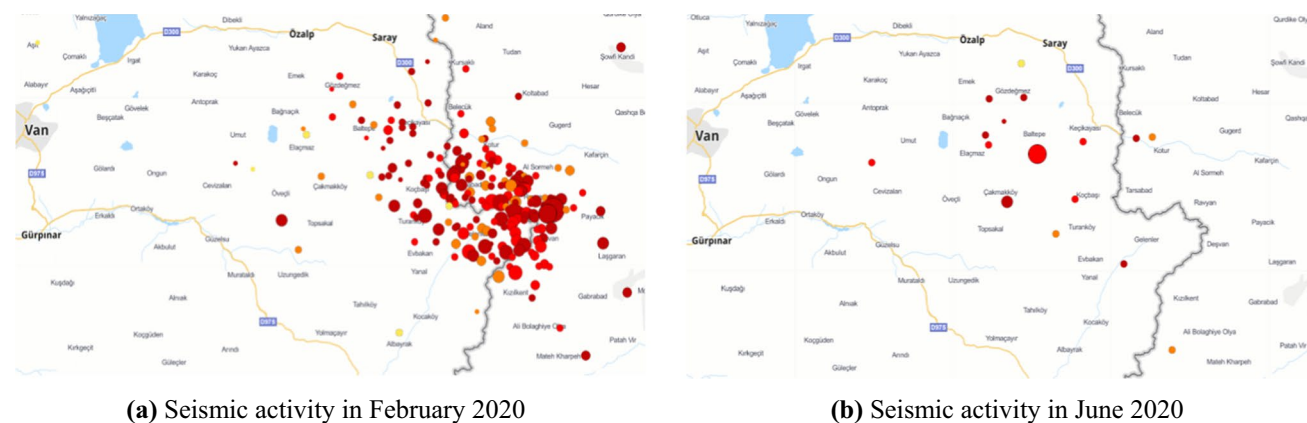


Fig. 1 Seismic activity in the region

were stated to be collapsed or severely damaged in Özpınar village (Gov. of Van 2020). The reasons for the damages were attributed to the materials used in masonry walls (rubble stones or low strength lightweight pumice blocks), the mortars (mainly mud between rubble stones, and low strength cement-based mortar between lightweight pumice blocks) and nonstandard construction techniques (buildings did not comply with the available seismic design codes) (Sağlam Selçuk 2020a). Reconnaissance team of VADUM visited the region affected from the third earthquake and stated that in the villages called Elaçmaz, Baltepe, Çaybağı, Yünkuşak, Gözdeğmez, Zincirkıran, Çardak, Örenburç, Kepir and Kargalı 612 buildings including 339 dwellings and 153 barns were collapsed or severely damaged (Sağlam Selçuk 2020b, Gov. of Van 2020).

In the reconnaissance reports written after every earthquake in Turkey in the last 40 years, it was stated that masonry buildings were not constructed considering engineering knowledge. The constructions were made using local materials with the help of local craftsmen, the connectors between masonry units did not have sufficient strength. In addition, it was stated that out-of-plane collapse was common in general, and with the occurrence of this situation, the roof became unsupported and total collapse occurred. In addition, heavy roofs were found to adversely affect the out-of-plane capacities (Erdik 1983; Gülkan et al. 2002; Karaşin and Karaesmen 2005; Askan et al. 2010; Akansel et al. 2014; Livaoğlu et al. 2018; Çağlar et al. 2020).

Similar observations are continuously recorded after each earthquake although code requirements are improved after each great earthquake. The observations after 2009 Abruzzo Earthquake revealed that the unreinforced masonry buildings in city of L'Aquila and its surroundings heavily damaged or collapsed since most of them built, modified, or structurally improved before the adoption of the 2002 updated requirements. The reasons were attributed to the lack of structural performance due to the inadequate structural system,

inadequate material quality (Indirli et al. 2013). In addition, quality of the workmanship is stated to be one of the most effective phenomena in seismic performance of masonry structures (Akhoundi et al. 2016).

This paper tries to address the reasons behind the damages and collapses considering not only structural performance during earthquakes but also long-term environmental effect. The villages suffered from earthquakes were visited and masonry buildings were investigated in detail. Figure 2 shows the epicenters of each earthquake and the investigated villages. The distance of each village to the epicenters of earthquakes are presented in Table 1. As can be seen, the distance of the epicenter of Khoy1 and Khoy2 earthquakes to Böğrüpek, Gelenler and Ömerdağ is 13–20 km, whereas it is 38–45 km to the other villages. As for the Saray Earthquake, the nearest village, Elaçmaz, is 3.4 km away from the epicenter and the distance to the farthest village investigated herein is 39 km.

2 Structural Properties

Turkey has 5 Seismic Design Codes in 50 years: TSC1968, TSC1975, TSC1997, TSC2007 and TSC2018. Disregarding the code in 2018 since it is relatively new and it became mandatory in 2019, the investigated buildings should comply with the remaining 4 codes. The main differences in terms of masonry structures are given in Table 2. In the table it can be seen that, general rules were remained the same in 50 years although some properties were revised. For example, wall thickness, slab and roof properties, foundation type, location of the tie beams were almost the same in all codes. Number of stories were reduced whereas distance of the side of the opening to the wall corner was increased in order to increase the shear strength of the walls. As for the unsupported length of the wall, although the maximum length remained the same, in each code some revisions were

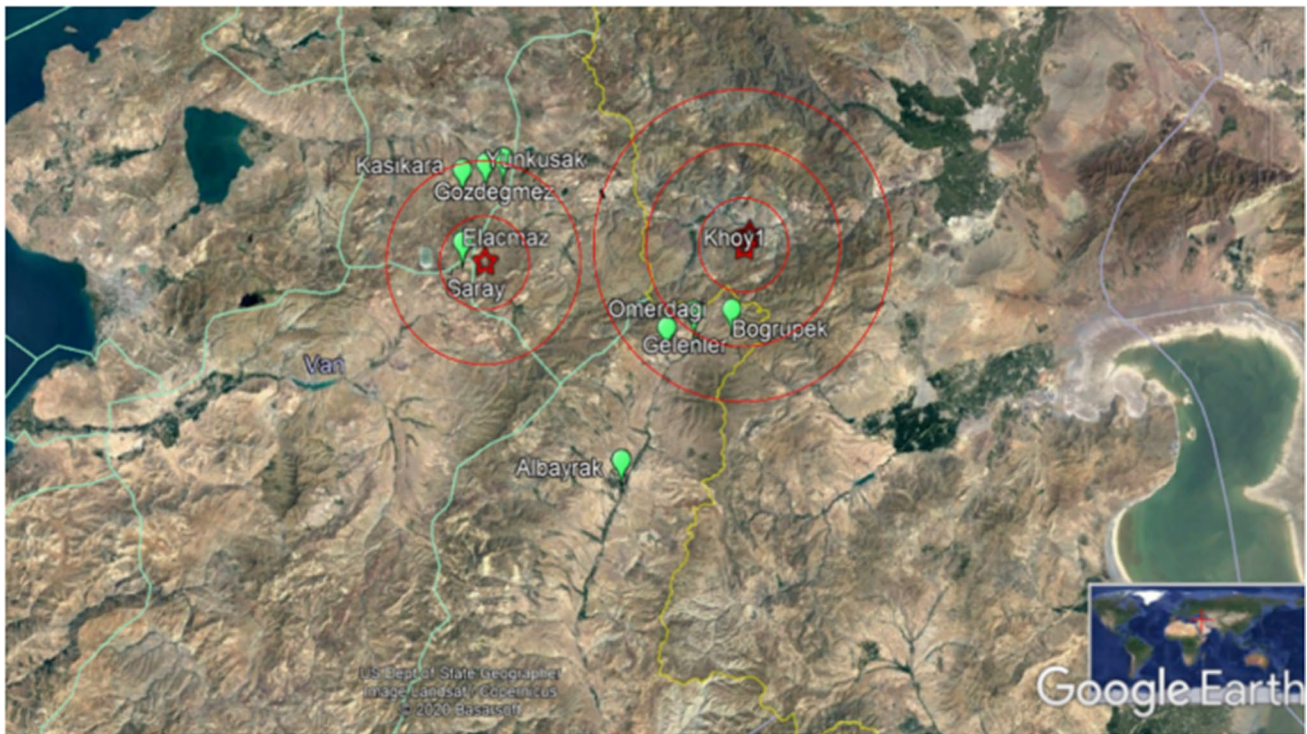


Fig. 2 The earthquakes and the affected villages (map data: U.S. Department of State Geographer, © 2021 Basarsoft, © Google, Image Landsat)

Table 1 Distance of each investigated village to the epicenter (km) (AFAD 2020a, b)

	Böğrüpek	Gelenler	Ömerdağ	Albayrak	Elaçmaz	Kaşıkara	Yünkuşak	Gözdeğmez
Khoy1	13.70	15.78	18.13	40.27	44.04	44.49	41.15	38.56
Khoy2	15.55	17.70	19.37	42.22	45.14	45.11	41.64	39.04
Saray	39.72	34.36	31.01	39.41	3.44	10.29	10.48	11.70

introduced. Codes before 2007 mentioned the vertical and lateral load analysis without detailing it, however TSC2007 introduced details of the analysis and details of the evaluation in terms of compression and shear. A study based on the regulations of the unreinforced masonry buildings given in TSC2018 revealed that it is almost impossible (even increasing the thickness of the wall) to build an unreinforced masonry building subjected to a seismic acceleration equal to or greater than 0.1 g (Kuran et al. 2020).

In the following sections properties of the investigated buildings are discussed in light of the code requirements given in Table 2.

2.1 Material

Masonry materials differ too much in the visited villages. Rubble stones (from diabase or other igneous rocks), stones (mainly calcareous sandstone) having visible surface smoothed, adobe and lightweight pumice blocks were observed in the field. Figure 3 shows the cross-section

of the walls. The walls except lightweight pumice blocks walls were more than 15 years old. Lightweight pumice blocks having low strength and more voids although were not allowed to be the main element for the load-carrying walls, were seen to be the material of choice during the last 15 years because of the low cost and being easy to handle. As can be seen from Fig. 3a–b–e the old stone walls mainly have two wythes. As for the adobe walls, they are generally 60 cm thick in which one layer of the wall has two 30 cm wide adobe units and the layer above it has 15–30–15 cm wide bricks (Fig. 3d). Although it is rare, relatively thin adobe walls were also seen in the field (Fig. 3c). This wall has one full and one-half adobe units, and can be considered as two-wythe wall and the thickness is 45 cm. In all those adobe walls, thickness comply with the code requirements. When the wall material is changed to lightweight pumice blocks then the wall become one-wythe slender wall (30 cm thick wall) as shown in Fig. 3f.

Mud clay mortar were usually used between the head and bed joints although it was not allowed in none of the codes.

Table 2 Comparison of the codes in terms of masonry structures (high seismicity case)

	TSC1968	TSC1975	TSC1997	TSC2007
Material	Concrete, adobe, rubble stone, cut stone, brick and hollow brick and other materials	Concrete, artificial and natural stone, adobe, other materials	Solid concrete brick block, natural stone, solid brick, bricks with special vertical hollows, adobe, other standardized materials	Solid concrete brick block, natural stone, solid brick, bricks with special vertical hollows, adobe, aerated concrete (AC)
Min. material strength	Cement and lime based	Artificial stone: 5 MPa Stone: 35 MPa Concrete: 14 MPa Cement and lime based Cement based	Stone: 10 MPa Concrete: 16 MPa Others: 5 MPa Cement and lime based Cement based	Stone: 10 MPa Concrete: 16 MPa Others: 5 MPa Cement and lime based Cement based
Maximum number of stories without basement (N)	Adobe: 1 Others: 4	Adobe: 1 Others: 4	Adobe: 1 Others: 2	Adobe: 1 Others: 2
Minimum wall thickness (t_{min})	Stone and RC: 50–60 cm Brick: 23–35 cm Adobe: 45 cm	Stone: 50 cm RC: 25 cm Brick: 1.5 t_{brick} cm Adobe: 45 cm	Stone: 50 cm RC: 25 cm Brick: 1.5 t_{brick} cm Adobe: 45 cm	Stone: 50 cm RC: 25 cm Brick and AC: 1.5 t_{brick} cm Adobe: 45 cm
Foundation	RC Adobe: Rubble masonry with cement and lime-based mortar	RC Adobe: Rubble masonry with cement and lime-based mortar	RC Adobe: Rubble masonry with cement or cement-lime-based mortar	RC Adobe: Rubble masonry with cement or cement-lime-based mortar
H/t_{max}	Stone: 8, Brick: 15, Adobe: 6			
Openings (A_{op} , total area of the openings in a single wall, A_w , wall area, t : wall thickness)	$A_{op}/A_w < 0.4$ Length max. 3 m Dist. btw. op. and wall corner: 1 m Dist. btw. window and door op.: 0.8 m (2t for adobe)	$A_{op}/A_w < 0.4$ Length max. 3 m Dist. btw. op. and wall corner: 1 m Dist. btw. window and door op.: 0.8 m (2t for adobe)	$A_{op}/A_w < 0.4$ Length max. 3 m Dist. btw. op. and wall corner: 1.5 m Dist. btw. window and door op.: 1 m	$A_{op}/A_w < 0.4$ Length max. 3 m Dist. btw. op. and wall corner: 1.5 m Dist. btw. window and door op.: 1 m
Max. unsupported wall length (l_u) (I: bldg. importance factor, A: floor area)	Adobe: 5 m Others: 7.5 m	Adobe: 4.5 m Others: 5.5 m	Adobe: 4.5 m Others: 5.5 m; If $l_u/A < 0.25l$ then tie columns are needed at each 4 m in wall length also at the corners	Adobe: 4.5 m Others: 5.5 m; If $l_u/A < 0.2l$ then tie columns are needed at each 4 m in wall length also at the corners
Tie beams (H: story height)	Rubble masonry: RC tie beams at each 1.5 m height Others: 1 lintel at top for $H < 3$ m, lintels at each 2.5 m height for $H > 3$ m	Rubble masonry: RC tie beams at each 1.5 m height Others: At top	Rubble masonry: RC tie beams at each 1.5 m height Others: At top	Rubble masonry: RC tie beams at each 1.5 m height Others: At top
Columns	RC or any material providing rigid diaphragm	RC or any material providing rigid diaphragm	Bldg. corner, wall connections, two sides of the openings RC	Bldg. corner, wall connections, two sides of the openings RC
Slab	RC or any material providing rigid diaphragm	RC or any material providing rigid diaphragm	RC, timber, steel Adobe: Earth roof is not allowed	RC, timber, steel Adobe: Earth roof is not allowed
Roof	Adobe: Earth roof is not allowed	Adobe: Earth roof is not allowed		

Table 2 (continued)

	TSC1968	TSC1975	TSC1997	TSC2007
Compressive strength of the wall (f_d)				Depends on masonry material and mortar strength: varies btw. 0.4 MPa and 1.8 MPa If material strength is not known: Hollow bricks: 0.5–1.0 MPa Solid brick: 0.8 MPa Stone: 0.3 MPa AC: 0.6 MPa Values reduce with increase in H/t
Shear Strength of the wall (τ_{em} ; allw. shear str. of wall, τ_o ; cracking str. of wall, μ : friction coef., σ : vertical stress of the wall)				$\tau_{em} = \tau_o + \mu\sigma$ τ_o is as follows: Hollow bricks: 0.12–0.25 MPa Solid brick: 0.15 MPa Stone: 0.1 MPa AC: 0.15 MPa $E_d = 200f_d$
E_d (modulus of elasticity of masonry units)				

Thickness of the mortar changes significantly depending on the interaction between the stone elements. If the stone surface is smooth, thickness could be as low as 1 cm, however if the surface is rough thickness could increase up to 7 cm. In adobe walls the mortar is again made from clay mud. The mud mortar was replaced by thin cement-based one (up to 2 cm) in lightweight pumice block walls which increased the bond between the pumice blocks (Fig. 3f).

Mud plaster with/without straw is also seen on the surface of all stone and adobe walls. Thickness of the plaster is also variable depending on the roughness of the surface. If the stone surface is smooth, then either plaster is not used, or its thickness is less than 2–3 cm. However, if the stone surface is rough then plaster is inevitable, and thickness varies up to 6–7 cm. Most of the mud plasters have varying straw contents as shown in Fig. 4. As for the lightweight pumice block walls, cement-lime-based thin (up to 2 cm) plaster layers was visible. It is interesting to note that although cement-based mortar is available in the region, in some buildings bed and head joints were of mud mortar, but cement-lime-based plaster was used on the walls.

2.2 Construction Technique

2.2.1 Load Carrying Walls

Although dwellings were close to each other, several different construction techniques were seen in the region (Table 3). This can be attributed to the fact that the codes did not clearly indicate the wall construction technique. Masonry material is found to be used in running (or stretcher) bond style, random rubble masonry style, coursed rubble masonry style or herringbone style as shown in Fig. 5. From the material and the construction technique, it was observed that people tried to do less work, use less and easy-to-reach material to construct the buildings or walls. Since the main aim was to finish the construction in a short time because of the harsh weather in the region, several materials with different kind and sizes were used which reveals that earthquake was ignored during the construction phase.

Rubble stone masonry construction is the easiest and cheapest technique, but it stated to have two main drawbacks: it has no shear keys, and the mortar layer may be too thick which reduces the connection between the masonry elements. As mentioned above, rubble stone masonry construction differs too much, and the shape of the masonry elements (interlocking mechanism) and the filler material is important in transferring the shear forces between the masonry elements (Fonti et. al. 2017).

Code rules given in Table 2 indicate that, masonry materials should have adequate bond and roof should be constructed such that box behavior of the walls be maintained through the vertical and lateral loads. When the code

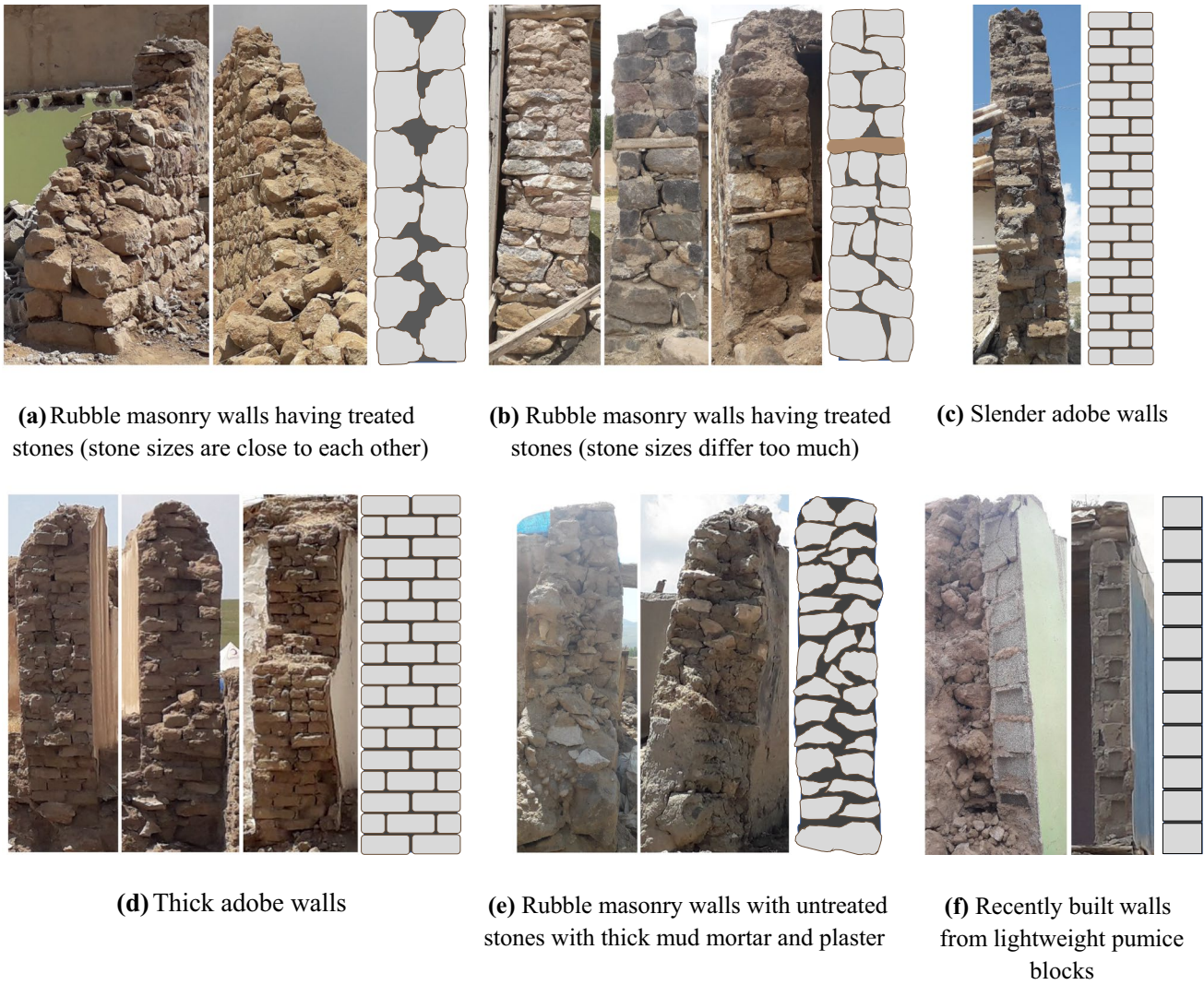


Fig. 3 Observed masonry wall cross-sections

Fig. 4 Plasters on the walls

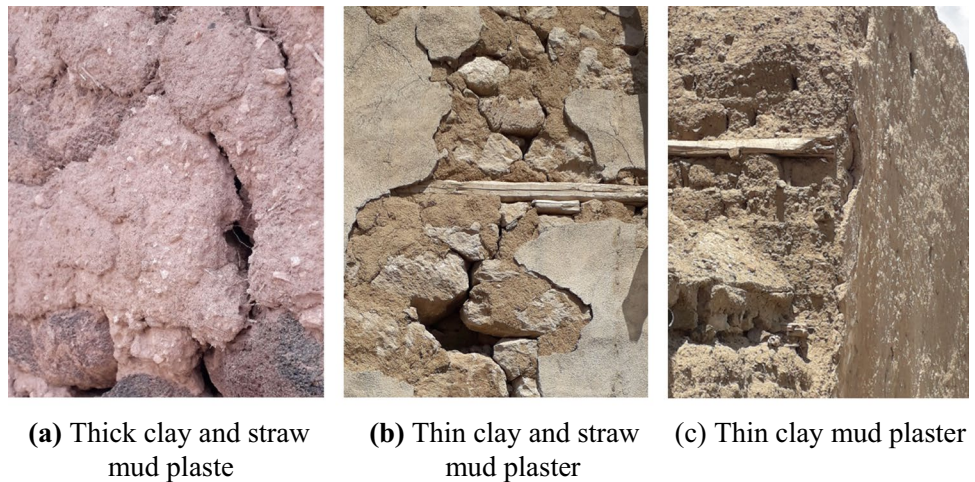


Table 3 Properties of the investigated masonry buildings

Masonry type	Structural walls						Foundation		Roof
	Masonry Material	Wall configuration	Mortar	Tie beams	Tie columns	# of Wythes	Masonry material	Mortar	
URM	Rubble stone	Coursed herringbone random	Mud	Common	No	2	Coursed rubble stone	Mud—Lime	Earth
URM	Treated stone	Coursed running bond	Mud—Lime	Rare	No	2	Coursed rubble stone	Mud—Lime	Earth
URM	Adobe	Running bond	Mud	Rare	No	2–3	Coursed rubble stone	Mud	Earth
URM and Confined Masonry	Pumice blocks	Running bond	Lime	Rare	Rare	1	Coursed rubble stone	Lime	Metal sheet

**(a)** Herringbone and coursed rubble masonry walls**(b)** Herringbone rubble masonry wall**(c)** Random rubble masonry wall**(d)** Running bond masonry wall**(e)** Mixed style masonry wall (Ashlar corners, coursed base, coursed and random body, herringbone top)**(f)** Mixed style masonry wall (Coursed-random bottom and herringbone top)**Fig. 5** Different wall design

requirements were met, then the compressive and shear strength of load-carrying walls were of concern. Since wall construction was inelaborate in the region, mortar was not adequate and rules were neglected, it has become impossible to determine the compressive and shear strengths. Although the earthquakes that hit the region had PGA almost equal to 0.1 g, the damages seen in most of the buildings indicated that load carrying walls had inadequate lateral strength.

In some buildings, it was observed that some load-carrying walls contain tie beams embedded in the wall made of wood or rubble masonry. In all the codes given above, RC

tie beams were mandatory at half and full height of the rubble stone masonry buildings and at the story level in other types of buildings. However, a few buildings were observed to comply with this rule.

In adobe walls wooden tie beams were placed at slab level and roof level. Most of the adobe buildings had one story but there were also two-story buildings, although it was not allowed in the codes. Tie beams generally are used to distribute the vertical force uniformly to the lower level of the wall. Tests on timber laced masonry buildings indicated that timber laces prevent the opening of the cracks formed

due to the out-of-plane bending and also the cracking load increased significantly when timber laced masonry was subjected to diagonal compression (Vintzileou 2008; Karantoni et al. 2016; Mouzakis et al. 2018). Although wooden tie beams are common all over the world, the rubble masonry tie beams are rare. The building shown in Fig. 6b has two rubble masonry tie beams in the external walls.

Besides the construction technique, the walls in some buildings were built using different materials. Figure 6c shows a wall which has rubble masonry at the bottom, adobe at the body and lightweight pumice blocks at the top.

2.2.2 Foundation

Almost 50–100 cm raised foundations were built from rubble stone masonry. The buildings have neither basement, nor foundations buried into the ground. The reason for the raised foundation can be attributed to the rain, snow, and the topographic effects. The region has harsh climate and snow season lasts almost 5 months. Therefore, to prevent the dwellings from the water caused by rain and snow, the base was raised.

In the past the upper level of the base, i.e., the ground floor was covered by clay and that clay was compacted and leveled to create a smooth surface. However, as it became easy to reach the cement, earth ground floor was replaced by concrete but in this case, most of them built without steel reinforcement. Few buildings had reinforced concrete floors although in the codes the foundation is stated to be built from reinforced concrete to ensure the box behavior at the foundation level.

2.2.3 Roof

The codes do not allow heavy earth roof in adobe buildings. In other buildings, the roof is stated to have adequate stiffness in in-plane direction to ensure rigid diaphragm. During the field investigation, it was observed that old buildings, i.e., more than 15 years old, have heavy roof consisting of wooden logs and earth roof as shown in Fig. 7a-b. One way spanning wooden logs were placed directly on top of the load-carrying walls, wooden laths or wickers were placed on the wooden logs and finally almost 20 cm clay earth was spread over. It is obvious that those roofs are flat and keep



Fig. 6 Different tie beams and wall materials

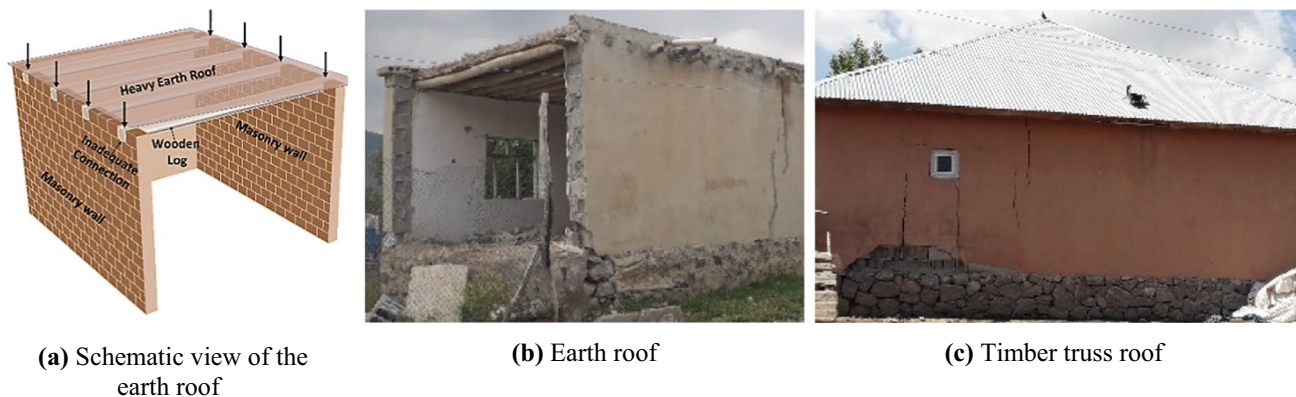


Fig. 7 Different roof design

the heavy snow on top if not plowed. That heavy snow and earth make the wooden logs displaced downwards continuously. Since the mid span has more displacement, each time residents place more earth and thickness at the midspan increases regularly. This is a vicious cycle because as the load at the midspan increases so the displacement and as the displacement increases at the midspan it creates a pit and that pit is filled with additional earth.

Besides the earth roof, some new buildings especially less than 15 years old and built from lightweight pumice blocks have timber truss roof with metal sheet covering (Fig. 7c). Since this roof is inclined, the maintenance cycle is low and no need to deal with snow. In addition, since this roof is light in weight, the axial load transferred to the walls is low. However, the number of this roof type is rare because it is rather expensive in the region.

3 Damage Patterns

As previously noted, although the PGA is low in those seismic events, many buildings severely damaged and collapsed. During the field investigation it was realized that the reasons of the damages and collapses could not be attributed merely to the seismic events. Damages were due to the several loading combinations before and during earthquakes. Considering the construction techniques, damages are explained in several aspects.

3.1 Pin Support Foundations

The aforementioned foundations have some problems:

They were built from rubble masonry

The top (ground floor) has no rigid diaphragm since it is either leveled by earth or concrete without reinforcement. Mud mortar is used between the rubble masonry
The ground that the foundation is set is inclined at some regions

Foundations of the investigated masonry buildings are not stiff enough due to the reasons mentioned above because rubble masonry has mud mortars and mortars deteriorate because of wet-dry cycles and freeze–thaw cycles each year due to the rain and snow. 29-year freeze–thaw frequency and average daily temperature cycles of the region which were provided by the General Directorate of State Meteorological Affairs in Ankara are given in Fig. 8. From the 29-year data, it was found that approximately 124 freeze–thaw cycles take place in the region per year and the freeze–thaw period lasts 6 months. 104 days were reported to be rainy or snowy meaning that the foundation is wet in those days causing the disintegration and deterioration of the mortar. In other words, those adverse effects force mud mortar to wash away from the joints. Figure 9a shows the highly deteriorated joints in the foundation. If the bed and head joints of the foundation are preserved and maintained as shown in Fig. 9b, then the foundation may behave stiffer and ensure the box behavior at the bottom layers of the walls.

Freeze–thaw cycles and wet-dry cycles continuously deteriorate masonry elements and mortars. Mortars are reported to be deteriorated even at low cycles of freeze–thaw. The level of deterioration in mortars was seen to be related to the mixing materials and porous structure. Lanas et al. (2006) testing aerial lime-based mortars and hydraulic lime-based mortars under freeze–thaw cycles and varying relative humidity levels reported that the porous structure of the mortar influences the strength and durability considerably. They found that the strength and durability reduce

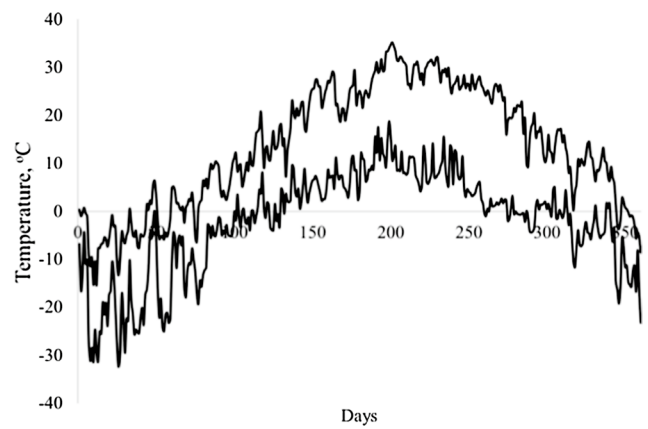
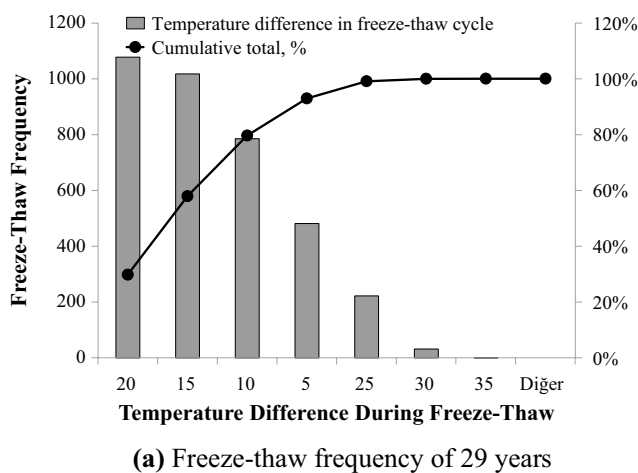
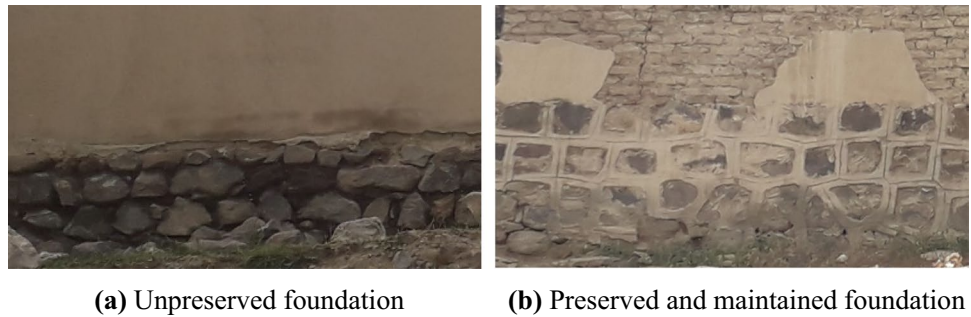


Fig. 8 Freeze–thaw frequency and average daily temperature of the region (General Directorate of State Meteorological Affairs, Ankara)

Fig. 9 Preserved and unpre-served foundations



(a) Unpreserved foundation

(b) Preserved and maintained foundation

significantly (in some cases %100 reduction was seen) after even 5 cycles of freeze–thaw cycles between -5° and $+35^{\circ}$. They also stated that flexural strength reduces with increasing relative humidity. Uranjek and Bokan-Bosiljkov (2015) stated that freeze–thaw and moisture slightly affect the compressive strength of lime mortar but their effect on flexural strength were significant. The mortar with a porosity of 33%, experienced 43% of reduction in flexural strength which led to reduction in modulus of elasticity of the wallets. Wiehle and Brinkmann (2020) reached similar results while investigating the earthen masonry. They observed that when relative humidity increases from 50 to 80%, the compressive strength of the earthen structural materials reduces 25–35%. Reduction in mechanical properties of mortars and resulted decrease in stiffness (especially modulus of elasticity) of the structural walls due to the environmental loadings make the masonry building vulnerable to seismic loadings (Vargas Neumann 2010). Alam (2019) investigated several ancient masonry buildings in Bangladesh. He found that abnormal rate of change in temperature and moisture results in gradual parallel and vertical cracks and powdering in masonry materials due to the exceeding the limit of the extension and the contraction stress. Besides some materials were observed to be deteriorated and dissolved because of water absorption. For example, mud mortar was stated to be highly soluble in water. Also, calcite or limestone, thin bricks and lime-mud mortar are soluble when exposed to medium level of water impact. Since those materials dissolve because of water, they erode, and strength of the binder reduces significantly.

Stiffness of the foundation in both in-plane and out-of-plane directions is important because foundations are responsible for the box behavior of the masonry building at the base. If the foundation is not stiff enough and it enables the walls translate or rotate individually, tensile stresses at the corners will increase and result in separation of the walls. The rotation and translation in foundation harm the fixed end support situation ending up with pin or roller support. Several damages on the walls were observed to be due to the rotation in the foundation. Those damages were available before the earthquakes because of the reduced mud mortar which resulted in the rotation of the individual rubble masonry elements. When the individual masonry elements

rotate, they force the base of the wall to displace out-of-plane. The out-of-plane displacement creates tensile stresses and if the wall fails to carry the tensile stresses either cracks develop in the mortar providing that the masonry element is strong (Fig. 10a and b), or cracks penetrate through mortar and masonry elements when both materials have similar strengths (Fig. 10c).

Recent study by Erdil et al. (2020) revealed that low strength lime mortar has 0.031 MPa shear strength and 0.3 MPa tensile strength (found from three-point bending test). Besides, the shear strength of mud mortar in adobe walls is 0.024 MPa and the mortar bond strength is measured as 0.007 MPa. It is believed that those strength values do not differ too much in the buildings investigated in this study. Therefore, it can be said that the walls will experience vertical cracks through the wall when the tensile stress reaches 0.3 MPa in lime-mortar-based walls and almost 0.2 MPa in mud-mortar-based walls. Figure 11 illustrates the corner cracks and body cracks in longitudinal direction caused by the rotation and translation in foundation levels. As it is seen those are not earthquake-related cracks and residents tried to repair several times either filling the cracks with cement-based mortar or mud mortar. Most of the buildings experienced cracks due to the movements in foundation. Even a little foundation movement may cause the tensile stress level in the wall corner to be exceeded.

During the field investigations, it was observed that available cracks either widened or resulted in collapse. Besides the available cracks, new cracks due to the foundation movement were seen to be developed. Cracks again developed at the corner resulting in separation of the walls (Fig. 12). In the region, the wooden logs at the roof level were mainly placed on a solid wall rather than the walls with openings. The reason can be attributed to the reduced vertical and horizontal capacities because of the openings and inadequate bending capacity of the lintels. The lack of axial load on the front wall with opening reduces the friction between the masonry elements thus increases the rotation ability of the foundation below those walls. The rotated walls created tensile stresses at the corner, and this resulted in out-of-plane displacement of the unloaded wall. Thus, those unloaded and displaced walls were the most vulnerable ones.

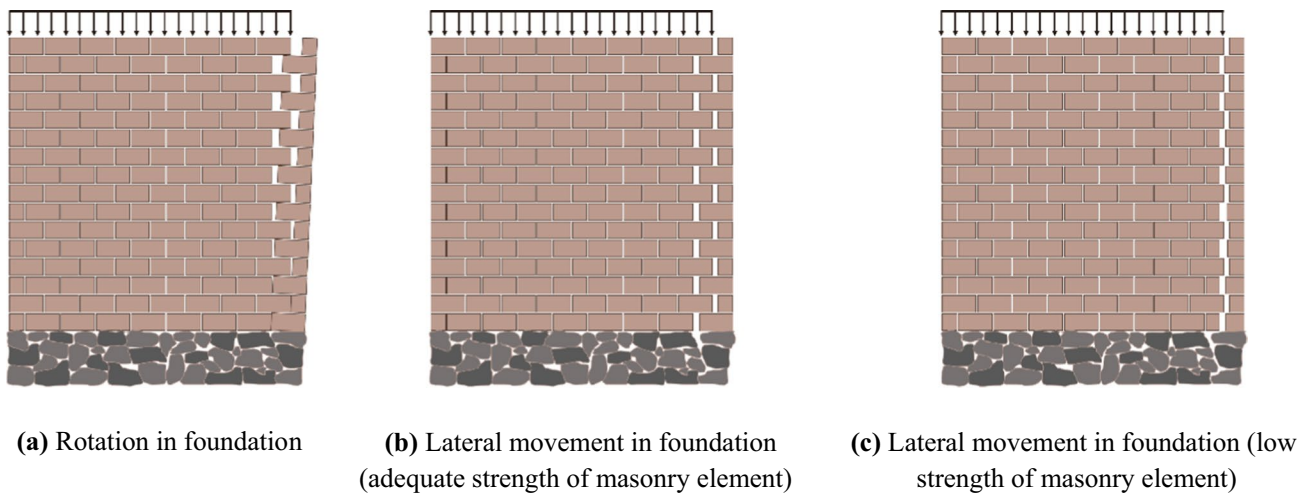


Fig. 10 Damage illustrations due to foundation movement

Fig. 11 Wall cracks due to the foundation movement



3.2 Roof Collapse

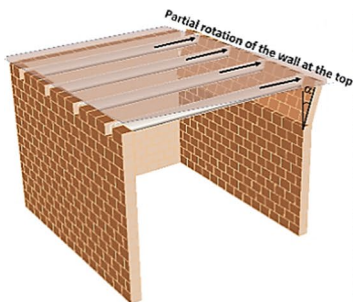
As previously noted, heavy roofs on the walls were built from wooden logs and thick earth slabs. The logs were not securely connected to the walls, instead they only placed between some masonry elements which created roller support. Considering the vertical load case only, it can be said that heavy roofs impose high vertical loads and increase the compression stresses thus increase the friction between the masonry elements. This seems to be positive contribution to the walls but roof itself is adversely affected because of the inadequate connection at the ends which enables the ends to translate or rotate under high vertical distributed loads. The main adverse effect is seen in earthquake loading since in this case the great mass of the roof vibrates and this vibration forces the so-called beams of wooden logs in lateral direction. The wooden logs force the walls to displace out-of-plane direction provided that the friction at the end connections is adequate (Fig. 13a). In this case two possible

actions may be seen: if the out-of-plane stiffness of the wall is inadequate, the partial or total collapse in out-of-plane direction is inevitable, however, if the out-of-plane stiffness of the wall is adequate (this was seen in thick masonry walls – more than 50 cm thickness) then no adverse effect is seen at the roof. In this case the wooden logs displace individually, and they slip over the wall and fall over. Since they fail, the earth slab also fails (Fig. 13b).

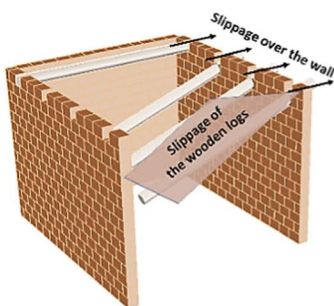
The wooden logs and their connection behave like a seismically isolated roof in the direction of longitudinal axis of the wooden logs because the roof can freely vibrate provided that the friction between the wood and the masonry is low. This indicates that the locals unintentionally use the seismic isolation phenomenon. However, since the friction is not low enough, the displacement in longitudinal direction, forces the wall in out-of-plane direction.

As for the transverse direction of the roof, since the translation is prevented by the masonry elements that the wooden logs embedded within, the vibration at the roof level creates

Fig. 12 Wall cracks due to the foundation movement during earthquake



(a) Wall deformations caused from roof displacements



(b) Partial or total collapse of the roof

Fig. 13 Wall deformations caused from heavy roofs

great lateral forces due to the high mass of the roof. The vibration and resulting force transfer directly to the walls and resulting in high lateral forces and displacements which cause the out-of-plane collapse in axial load-free-walls.

3.3 Out-Of-Plane Failure of Structural Walls

Out-of-plane failure is the common failure type in all over the villages. Since mortar was not adequate between the masonry elements, the connection from the friction which is positively affected from the heavy roof starts to play an important role. If the wall lacks these connections, then it is inevitable to see severe damages (Vlachakis et al. 2020).

It is known that H/t or L/t or H/L ratio directly affects the out-of-plane capacity of the wall. Karantoni et al. (2014) stated that H/L ratio and H/t ratio affect the vulnerability in buildings with stiff floors when compared to the ones with flexible floors. Walls with $H/t > 1$ were found to be more vulnerable. After the investigations on masonry buildings after the 2006 earthquake in Silakhor city of Iran, it was observed that the damage decreased with the decrease in H/t and L/t ratios, and the risk of damage increased as the slenderness increased. It is stated that the walls of $H \leq 3$ m, $L \leq 5$ m and $t \geq 60$ cm were not severely damaged. The walls with a H/t ratio of more than 5 and a L/t ratio of more than 9 were observed to be severely damaged. In addition, it has been stated that the probability of damage increases in walls that are less than 50 cm thick and built at a height of one floor (Salek Zamankhani et al. 2011). Another study indicates that when the wall has damages in in-plane direction, its out-of-plane capacity decreases 3 times when the wall has a slenderness (H/t ratio) of 20 and an aspect ratio (L/H ratio) greater than 2 (Agnihotri et al. 2013).

Most of the out-of-plane failures observed in the region were in lightweight pumice block walls in which the wall was slender (H/t is between 13–15), the mortar was inadequate to connect the wall to the transverse walls and in some cases even the tensile strength of lightweight pumice blocks was so low that crack penetrated from mortar through the blocks. In those buildings the out-of-plane displacement resulted in the total collapse of the walls. However, when the connection between the masonry elements was increased and slenderness was reduced (H/t is less than 7) then the contribution of the transverse walls increased, and the wall partially collapsed (Fig. 14).

The vertical roof force is observed to be important in masonry buildings. As previously noted, the wooden logs of the roof run in one direction and exert vertical forces to the walls at their ends. Since they are one directional element, they only tie two opposite walls remaining the adjacent walls free from the vertical forces. In addition, the vertical force-free walls are not directly attached to the roof. Besides, if they are not adequately attached to the transverse walls, they can freely rotate at the base in case of a lateral force (Fig. 14).

3.4 Corner Failure

Corner failure was also common in the region which can be attributed to the lateral force on the walls caused from the heavy roofs. Since the wall vibrates under the seismic action, it is loaded from the top because of the roof and through the wall height because of the mass of the wall. As mentioned above if the connection between the masonry elements is not adequate then out-of-plane failure is inevitable. However, if the connection is adequate in the case of adobe walls, total failure is shifted to the partial failure. The buildings shown



Fig. 14 Out-of-plane failures

in Fig. 15 experienced partial corner failure because the walls had adequate connections at the corner, the slenderness was low, and the mortar strength was adequate (Erdil et al. 2020). Although the connections were adequate, still the tensile strength of the mortar was low and tensile stresses developed due to the out-of-plane forces. The corner being exposed to two-way loading and two-way displacements during lateral loads, and out-of-plane displacement being great at the roof level, stress concentrations will occur at the top corners resulting in partial collapse (Bansal et al. 2017, Casapullo et al. 2018, Vlachakis et al. 2020).

3.5 Breakage of The Link Between The Walls

In some buildings, it was observed that connections between the walls at the corners were failed and significant cracks throughout the wall height became visible (Fig. 16). Although those cracks were significant, the separated wall did not collapse because of the great mass and stiffness of the wall which resisted the seismic load. Since the seismic

load was low, those walls were able to absorb the seismic energy. If the seismic energy were great, then the out-of-plane collapse would be inevitable.

In most of the buildings, external walls had one window opening placed at the center. However, there exist some buildings which had door and window openings on the same wall, and some buildings having window openings without having adequate spacing between sides of the wall. From the field investigation, it was observed that some buildings experienced in-plane damages at their walls with openings. Either the cracks developed at each side of the opening, or diagonal cracks started at each corner of the opening. In all cases, the cracks were wide (almost 4 cm) and adversely affected the capacity of the walls resulting in severe damage condition (Fig. 17).

3.6 In-plane Shear Failure of Structural Walls

Magenes and Calvi (1997) stated that in-plane structural walls started to be forced and damaged when the out-of-plane



Fig. 15 Corner failures



Fig. 16 Breakage of the link between the walls



Fig. 17 Damage around openings

capacity of the wall is adequate to prevent collapse. In-plane damage was mentioned to be seen by rocking, shear cracking and sliding. In rocking failure great overturning moment is necessary to create cracks at the bed joints, in shear cracking the cracks can develop at bed and head joints in diagonal form, and finally sliding occurs through bed joints in case of reduced vertical stress levels (Wu et al. 2017). The inertial friction and bond between masonry elements resist the lateral loads, but even at small lateral displacements large diagonal cracks (Wang et al. 2018). It is known that vertical stress level is important for in-plane shear capacity. When the vertical stress level is between low to moderate which is common in most low-rise buildings, the shear strength is found to be independent of stone arrangement, however, when the vertical stress level is high, the random arrangement of masonry elements and their bond affect the shear capacity of the wall adversely (Lourenço et al. 2006).

Since in-plane capacity of a masonry wall is greater than its out-of-plane capacity, in-plane failures were not common in the region. Pictures given in Fig. 17 show some examples of in-plane damages. As can be seen, in-plane damages became visible in diagonal shear cracks. In all the buildings although the roof is heavy, the maximum vertical stress is almost 0.2 MPa which would have limited contribution to the shear strength of the wall. If the walls had adequate out-of-plane capacity, then in-plane damage becomes visible and either X-cracks develop on the wall (especially the wall segments between the openings), or stepped or continuous diagonal shear cracks become significant on the walls without openings. In some cases, high in-plane deformations cause the cracked wall to be partially collapsed in out-of-plane direction (Fig. 18).

3.7 Separation of External Leaves

Multi-leaf walls are generally exposed to leaf-separation by either horizontal seismic loads or high compressive forces (Candela et al. 2016). In both case, the partial or total separation is seen at the external leaf since this leaf is subjected to environmental exposures throughout its lifetime. As previously noted, the region is exposed to approximately 124 freeze–thaw cycles annually and almost 1/3rd of the

year there is rain or snow indicating that wet-dry cycles are also great which creates adverse environmental effects to the external leaf. The continuous environmental cycles decrease the strength and bond between the wythes and since the outer wythe is susceptible to the harsh environment, it deteriorates continuously and delaminates easily even under low level of stress (Vintzileou and Tassios 1995). The other reason is that the inner wythe carries loads from roof increasing the bonding of the masonry units and their flexural strength. Additionally, the out-of-plane deformation of the inner wythe is prevented to deform inside the structure by the roof or floor beams. Another reason for the leaf separation is the lack of shear keys that connect the leaves and take care of the shear forces. The repair and retrofit of this type of collapse are stated to be hard in rubble masonry buildings since there is no significant and patterned configuration (Corradi et al. 2007).

Isfeld and Shrive (2015) created finite element models of multi-wythe walls to understand the bond between the wythes and realized that mortar quality is important in maintaining the stability of the multi-wythe stone masonry walls. It was also stated that failure associated with the mortar deterioration is time dependent, and combined effects of freeze–thaw and wet-dry cycles and altered climate conditions worsening any initial weakness in the construction. The environmental effects lead to reduction in strength and bond of mortar, that may result in instability or collapse if left untreated.

In rural masonry structures generally there exist limited shear keys between the leaves or no shear key is used in the entire structure. Most of the investigated rubble stone masonry buildings had no shear keys and only mud or crushed rubble stones were placed between the inner and outer leaves (Fig. 19). Therefore, no shear distribution is possible between the leaves in the buildings. The separation will take place when the wall has inadequate capacity in out-of-plane direction in case of a low seismic event.

3.8 Other Observations

Many abandoned adobe buildings observed in the region had light damages although they were stated to be built



Fig. 18 In-plane damage in the walls

50–60 years ago. Their roofs were demounted and used in newer buildings. Some buildings had roof collapse. The local people stated that since adobe buildings need more maintenance, the internal space is inadequate in those buildings and mud plaster results in dirty and uneven internal surface, they built new houses from lightweight pumice blocks and abandoned those adobe buildings. Although they thought to live in a better house, they did not pay attention to the seismic event and none of the lightweight pumice block-wall buildings comply with the regulations given in the codes and neither they were designed and built by engineers. Therefore, poor seismic performances of those so-called new buildings are inevitable.

During the field investigation, most of the lightweight pumice block buildings were observed to have damages. Some lost their walls due to the out-of-plane action, some experienced separation (Fig. 20). However, the abandoned adobe buildings behaved well instead. Of course, the effect of the roof is important in the behavior, but it should not be forgotten that those adobe buildings were lack in box behavior. Some adobes were deteriorated too much due to

the environmental exposures, but their seismic capacity was still adequate (Fig. 21). In both figures, roof weight is almost the same, but the overall performance is totally different which indicates that thickness of the walls, i.e., slenderness of the walls should be low enough to have adequate seismic performance in masonry buildings. Otherwise, high slenderness results in low strength and low performance.

4 Local Strengthening

Behavior of masonry buildings is a complex phenomenon and needs observations after extreme loadings. There were two extreme loadings in the region: environmental effects and earthquakes. Environmental effects force and deteriorate the structures continuously throughout the year. They have definite periods and one can observe and estimate the time of loading. However, earthquakes are not observable and predictable, they do not have a definite period, and no one knows the loading level.

Fig. 19 Separation of external leaves



Fig. 20 Damaged newly constructed lightweight pumice block buildings



Fig. 21 Well behaved old adobe buildings

The local people in Gözdeğmez village observed the environmental effects and realized that corners continuously crack, and those cracks widen year by year. That village sometimes experiences flood because it was located on a valley where flood is inevitable when certain conditions (amount of rain or snow) are met. Some local people tried to repair and increase the bond at the corners of the walls with lime or mud-based mortars. However, there exist some people applying significant structural interventions at the corners. They strengthened the cracked corners by adding L shaped reinforced concrete columns as shown in Fig. 22. They did not use those additional corner columns at all corners, instead they used at the cracked corners only. Some people added additional columns at the midspan of the wall where a transverse wall connected. Although there are no engineering calculations for those additions, those buildings were found to behave adequately, and no earthquake related damage was observed. The reason can be attributed to the increase in lateral strength and stiffness, and also the partial confinement of the masonry. The additional reinforced concrete columns tie the walls at the corner, prevent the separation and provide additional stiffness in in-plane and out-of-plane directions. With those improvements, the lateral deformation reduces, and box behavior is guaranteed at low level of seismic acceleration.

Besides, one other reason is the low PGA level of the earthquakes. Behavior of those strengthened buildings during a strong earthquake is still questionable because not adding columns at each corner, torsional irregularity may be induced, and additional shear forces may develop at the unstrengthened corners. However, since that strengthening seemed to be adequate for low seismicity, it can be said that even inadequate strengthening has great impact on the lateral capacity of the non-engineered masonry buildings. Therefore, since the masonry buildings in rural areas have one or two stories, additional lateral load bearing elements can be used at critical locations externally to ensure box behavior and by this way their seismic performance can be improved practically.

5 Conclusions

Masonry buildings have complex behavior because several materials are brought together and tied with mortar. Each material has its own characteristics, and those characters play important role in the behavior. It is not an easy task to develop certain regulations because behavior is affected from several factors such as bond, material size, unsupported wall length, height, roof type, foundation type etc. All the factors have significant effect on the behavior. The codes try to give safety regulations considering those parameters. Although the codes regulate the seismic resistant rules, it was observed that many rules were disregarded in the region. Following conclusions can be drawn:

Masonry materials being accessible and cheap are widely used in rural areas. Although the codes give many applicable regulations, in rural areas those regulations are disregarded, and buildings are constructed based on the experience of the workmen. Workmen not having adequate knowledge about the earthquake (since earthquakes are rare), they only consider vertical loads and environmental effects. Therefore, the building becomes durable against environmental effects for some years but weak against lateral loadings.

Many structural materials were observed in the region. The reason can be attributed to the accessibility. Different materials led to different structural configurations of the walls. In a single village, several configurations can be seen which indicates that local workmen have knowledge in detailing of the walls but lack in knowledge of code regulations.

Since lime and concrete are hard to reach many buildings were built from mud mortar. Mud mortar being weak in terms of tension and shear and being not durable, needs continuous maintenance. If the maintenance is not adequate, then the masonry elements become prone to translations and rotations which create cracks in the walls.

Buildings were mostly built on a rubble stone masonry foundation in which mud mortar is the main bonding element. Since the region experiences harsh weather conditions and great number of both freeze–thaw and

Fig. 22 Locally strengthened masonry buildings



wet-dry cycles, mud mortar deteriorates and causes masonry elements at the foundations to translate or rotate due to the axial forces from the walls and roof. Displacement at the base creates high tensile stresses and cracks at the corners of the walls. Many buildings were observed to have such cracks. As the cracks reduced the connection at the corners and reduced the stiffness, even the low seismicity resulted in further openings or collapses.

Roof was also observed to be important for the seismic performance. The primitive seismic isolation technique was observed at the roofs constructed from wooden logs and earth. In those roofs, the wooden logs were found to be placed on the walls without having a firm connection. Instead, they were free to translate in longitudinal direction which reduced the lateral earthquake force exerted to the walls in that direction. However, since the wooden logs have firm connection in transverse direction the heavy roof imposed high lateral seismic forces in that direction which resulted in partial or total out-of-plane collapse of the walls.

Non-engineered masonry buildings were observed to damage even under low seismic actions due to reasons given in the paper. To reduce the damage and provide safe dwellings, seismic performance of those buildings need to be increased with proper techniques. It was observed that some buildings were strengthened locally at the corners. L shaped reinforced concrete columns were added to the cracked corners to increase the lateral capacity of the walls. That strengthening seemed to be adequate for low seismicity because no significant damage was seen. From this observation, it can be said that even the inadequate strengthening has great impact on the lateral capacity of the non-engineered masonry buildings. Therefore, since the masonry buildings in rural areas have one or two stories and are relatively light in weight, additional lateral load bearing elements can be used at critical locations externally to ensure box behavior and by this way their seismic performance can be improved practically.

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