CHAPTER 11
Structural Observations

GEER/EERI/ATC Cephalonia, Greece 2014

Report Version 1

11.1 Main Structural Observations

INTRODUCTION

During the two main seismic events of January 26th, 2014 (M_w 6.1) and February 3rd, 2014 (M_w 6.0), damage occurred to a number of structures. Most of the structural damage was observed during the 2nd event and was primarily concentrated in the Paliki peninsula area (Fig. 11.1.1), on the western part of the island. This observation can be attributed to pre-existing damage which many structures already had suffered during the 1st event, in combination with the very high ground motions recorded in the Paliki peninsula area during the 2nd event. The main villages visited by the reconnaissance teams after the 2nd event include Aghios Dimitrios, Livadi, Vilatoria, Aghia Thekla, Kalata, Monopolata, Kaminarata, Mandourata, Favata, Havdata, Chavriata, Vouni and Manzavinata (Fig. 11.1.1).

Figure 11.1.1. Areas visited by the reconnaissance teams where most of structural damage occurred.

MAIN OBSERVATIONS

In general, the buildings on the island behaved well, considering the intensity of the earthquake which had ground accelerations of up to 0.75 g with Spectral Accelerations (SA) exceeding 2.5 g for periods within the 0.3-second range. This range must have been close to the period of most of the buildings, since the local code restricts building height to three stories. At the area around the epicenter, most of the buildings are low-rise with a Reinforced Concrete (RC) frame and brick infill walls. The majority of these buildings suffered negligible or minor damage at the brick infill walls, which, in some cases, were separated from the RC frame. The overall satisfactory structural performance can be attributed to good construction quality, especially of the infill walls which, in most cases, were able to withstand the largest portion of the seismic loads without cracking. Figure 11.1.2 shows an example of a building in Livadi (Fig. 11.1.1), which suffered negligible damage despite the fact neighboring structures suffered significant damage. Except from the dislocation of several roof tiles and possible displacement of the solar panel, there is no evidence of any other damages (including light cracks in the brick infill walls). The dislocation of the roof tiles, observed in many structures in the meioseismal area, is a likely result of the high accelerations that occurred.

Figure 11.1.2. A two-story RC building in Livadi which performed well, despite the fact that at this area where several buildings suffered significant damage. Evidence of high accelerations if the dislocation of roof tiles observed in several structures throughout the meioseismal area.

However, several buildings suffered medium to severe damage. In general, these buildings belong to one of the following categories:

- Reinforced Concrete (RC) buildings designed under older seismic codes. Some characteristics that possibly have contributed to their damage are: the large tie (stirrup) spacing (> 300 mm), lack of confinement at beam-column joints, and poor detailing. Examples shown in the photos of Fig. 11.1.3 (coordinates 38º13'45.60", 20º25'47.40").
- RC buildings with a soft story at the ground level. Example on Fig. 11.1.4 in Aghios Dimitrios (38º14'36.51"N, 20º25'41.91"E).
- Mixed structural systems which usually include a masonry ground floor and a reinforced concrete upper floor, built in phases over the course of several years. An example is shown in Fig. 11.1.5 (GPS coordinates: 38.229444, 20.429722.)
- Old masonry buildings which survived the 1953 earthquakes and had then been repaired (Fig. 11.1.6), including several churches discussed separately in a Section of this Chapter.
- RC buildings with masonry infill walls had minor damage on the RC structural frame and damage in the masonry infill walls. An example is shown in Fig. 11.1.7.

Figure 11.1.3. Damage of 2-story RC building in Aghios Dimitrios designed with older codes (GPS coordinates: 38.229444, 20.429722): (a,b) after 1st and 2nd events; (c,d) beam-column detail of (b).

Figure 11.1.4. Significant damage to the soft story columns of a two-story RC building in Aghios Dimitrios (38°14'36.51"N, 20°25'41.91"E). No damage was observed at the upper floor.

Figure 11.1.5. Collapse of the masonry ground floor of a two-story building in Aghios Dimitrios. The upper floor is a reinforced concrete frame, added several years after initial construction (GPS coordinates: 38.229444, 20.429722).

Figure 11.1.6. Partial collapse of a heritage masonry building in Samoli, close to Livadi village (see Fig. 11.1.1), which was probably built during the 17th century. The building had survived the 1953 earthquakes with some damage which had subsequently been repaired.

Figure 11.1.7. Severe damage of brick infill walls of a two-story RC building in Livadi (Fig. 11.1.1). The frame suffered only minor damage (38°13'48.19"N, 20°25'53.20"E).

Although most buildings performed well, a large number of nonstructural components collapsed. While details can be found further into this Chapter (and in Chapter 9), typical examples of nonstructural architectural failures are shown on Fig. 11.1.9. The photos show representative failure of numerous masonry fences and pergolas that may be attributed to lack of seismic specifications or poor quality control during construction, which does not require professional engineering inspection.

Figure 11.1.8. Examples of nonstructural architectural components failure: (a) overturning of a masonry fence in Lixouri; (b) collapse of a pergola in Livadi.

In general, damage or failure of nonstructural components was evident in the strongly-felt areas of the island, exposing the people to life threats and resulting in significant interruption of the function and impacts on the economy of the island. This earthquake reconnaissance provides detailed documentation of both acceleration- and displacement-sensitive nonstructural components in Section 11.9 and Chapter 9 of this report. The detailed information collected in the field, together with the several recorded strong ground motions in the immediate vicinity of the information, presents an excellent opportunity to enhance our knowledge on the behavior of all types of nonstructural components and develop simple, engineering and common-sense, solutions to minimize their seismic risk exposure.

11.2 Building Inventory and Construction Types

The majority of the structures of Cephalonia were rebuilt following the catastrophic 1953 earthquakes that destroyed more of the building stock. Currently, the building inventory can be grouped into four major categories according to their load bearing system:

I. One- to two-story non-monumental / non-landmark masonry buildings: These can be further subdivided into two groups based on their location:

a. Buildings constructed with clay, stone, or concrete bricks with better quality mortar, mainly found in the towns of Argostoli and Lixouri.

b. One-story masonry buildings with walls composed of roughly treated stones and low-strength clay mortar. There is only few of those buildings in small villages, survivors of the 1953 earthquakes. After the 1953 events, their use changed to barns, stables, or other auxiliary structures, or they were simply abandoned without retrofit or maintenance.

II. Reinforced Concrete (RC) buildings: Located throughout the island, these buildings were generally constructed after the 1953 earthquakes following modern seismic codes, varying from one to four stories in height. Their load-bearing system is reinforced concrete frames and shear walls and their top has wood framed roofs. Some of those buildings can be classified as monuments, especially in the largest towns of Argostoli and Lixouri.

III. Masonry monumental and other cultural heritage buildings: These are mainly churches or schools with one or two stories, constructed using traditional seismic-resistant techniques.

IV. Other buildings and structures: This category includes wood framed buildings and stone or Reinforced Concrete (RC) bridges.

The observed behavior of selected types of the above buildings have dedicated sections this report, such as bridges and churches in 8.5 and 11.8, respectively.

11.3 Damage Assessment

Damage assessment survey efforts were led by the Greek Seismic Rehabilitation Agency (YAS or ΥΑΣ in Greek for "Υπηρεσία Αποκατάστασης Σεισμοπλήκτων"). A temporary field office was established in the facilities of the Technological Educational Institute (TEI) of the Ionian Islands, a public building in Argostoli, to organize the inspections and emergency interventions. The inspections were mostly performed by teams of structural engineers who work for public agencies. A Rapid Assessment phase was performed immediately after the 1st event, followed by a Detailed Assessment phase.

Rapid Assessments have a screening role to quickly distinguish between safe and unsafe buildings. They are performed either upon the owner's request, or directly by YAS initiative for buildings where structural damage is evident from the structure's exterior. The process involved teams of two engineers each, who would fill the Rapid Inspection Forms and describe key characteristics such as number of stories, load bearing system and use. The teams would then assign a color: Green, for safe buildings, or Yellow for buildings which should temporarily be evacuated. Green-tagged buildings had minor damage only, e.g., a small number of hairline cracks in the masonry infill or bearing walls, isolated hairline cracks in Reinforced Concrete (RC) structural elements perpendicular to the element's axis.

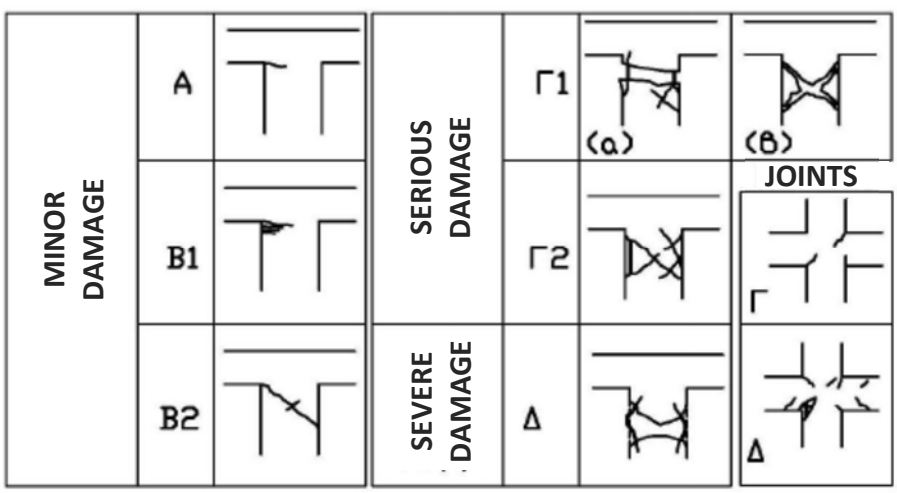

Figure 11.3.1. Typical degrees of damage in vertical structural elements of Reinforced Concrete (RC) structures according to YAS (modified from (YAS, 2014).

For yellow-tagged buildings, a Detailed Assessment phase followed by teams of three engineers, in which buildings are tagged as: Green (safe for use), Yellow (not safe for use which need retrofitting) and Red (not safe for use, decision to repair or demolish pending

detailed engineering evaluation). Sketches of typical degrees of damage in vertical structural elements of RC buildings, ranging from minor to severe, are presented in Fig. 11.3.1 (modified from YAS, 2014).

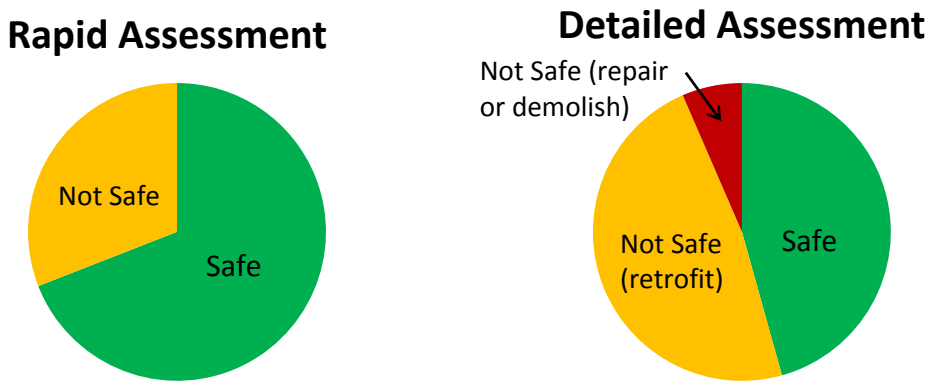

Figure 11.3.2. Results of Rapid Assessment and Detailed Assessment phases. In the Rapid Assessment, 31% or 1505 buildings were found unsafe to occupy. The Detailed Assessment of the yellow-tagged buildings found 46% (1265) safe, 48% (1325) temporarily unsafe, and 6% (180) unsafe and pending detailed evaluation of whether to repair or demolish.

Figure 11.3.2 shows color-tagging based on the initial Rapid Assessment and the Detailed Assessment that followed according to data collected by the Hellenic Earthquake Rehabilitation Services (HERS).

In the Rapid Assessment phase, 4,865 buildings were inspected, mostly in the Paliki peninsula with 31% or 1,505 buildings yellow-tagged or unsafe to immediately occupy.

During the Detailed Assessment phase, 2,770 buildings were inspected, including the yellow-tagged buildings from the Rapid Assessment phase and others that required further investigation. During this inspection, 1,265 buildings (46%) were deemed safe (green), 1,325 (48%) were considered temporarily unsafe (yellow), and 180 (6%) were unsafe (red). Of the 2,770 buildings, 1,167 were Reinforced Concrete (RC) structures that were classified by 60%, 39% and 1% as green, yellow and red, respectively. 765 buildings were masonry structures with 29%, 54% and 17% deemed as green, yellow and red, respectively and 783 were hybrid structures, with 40%, 56% and 4% considered as green, yellow and red, respectively. Finally, 85 buildings were of other types with 48%, 41% and 11% tagged as green, yellow and red.

The results of the Detailed Assessment broken down by building type are presented in Fig. 11.3.3. Considering the 180 red buildings alone, 5% were made of reinforced concrete, 73% were masonry, 17% were hybrid concrete-masonry constructions, and 5% were of other types.

It can be concluded from these statistics that the masonry buildings suffered the most. Of the regions of Cephalonia mostly affected by the earthquake, 76% of the red and 60% of the yellow buildings were located on the Paliki peninsula. When considering use, 52% of the red buildings were farm storage, warehouses, stables, etc. or had been abandoned, 39% were residential buildings and 9% were commercial buildings such as offices.

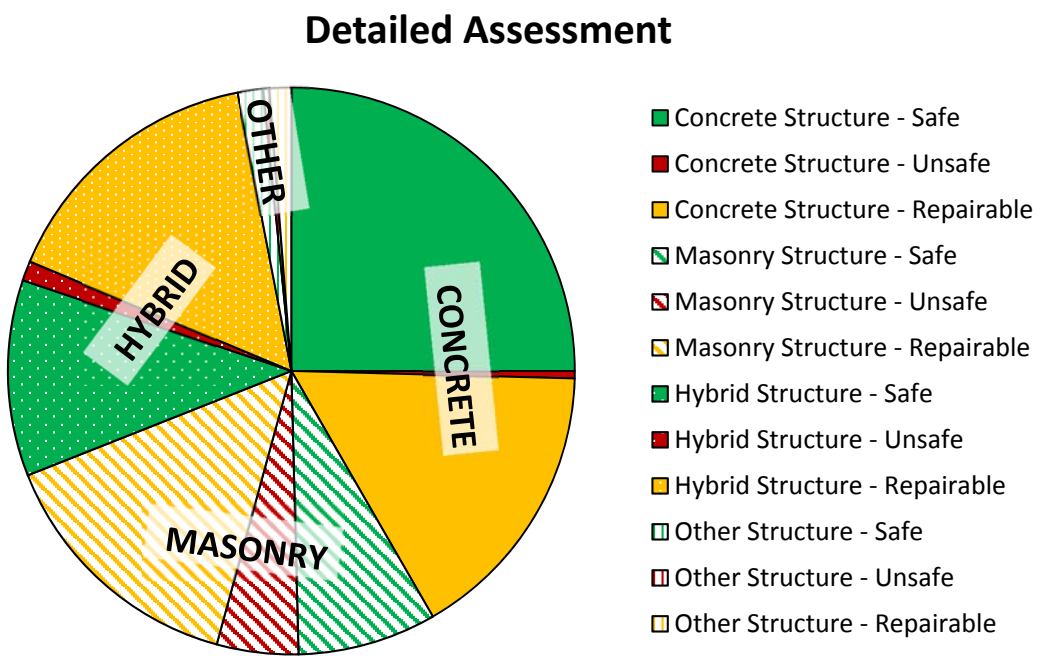

Figure 11.3.3. Detailed assessment of 2,770 buildings: 1,167 were RC (60, 39, 1% tagged green, yellow, red); 765 were masonry (29, 54, 17% tagged green, yellow, red); 783 were hybrid (40, 56, 4% tagged green, yellow, red); and 85 were other (48, 41, 11% tagged green, yellow, red).

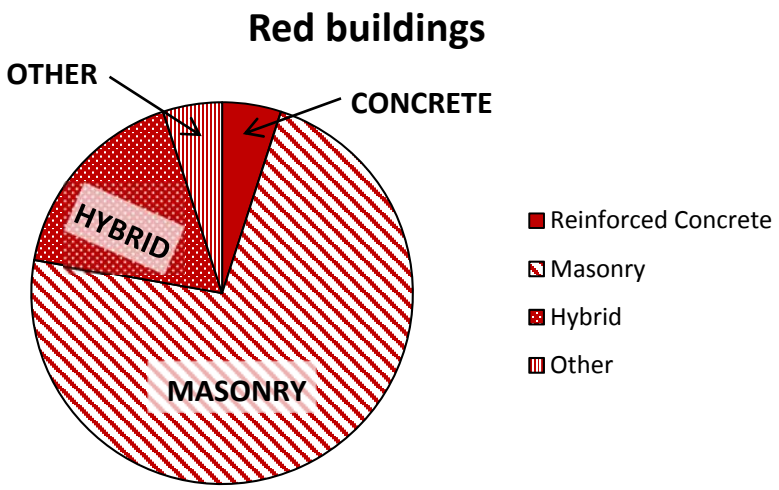

Figure 11.3.4. Types of the 180 red-tagged buildings of the Detailed Assessment: 5% reinforced concrete, 73% masonry, 17% hybrid concrete-masonry, and 5% other types.

The Greek Government was quick to respond in the aftermath of the disaster. Shortly after the main shocks it announced that aid would be offered to repair temporarily unsafe for use (yellow) or reconstruct dangerous for use (red) buildings. 80% of the aid will be in the form of free assistance and the rest will be in the form of an interest-free loan to be paid back over a period of 15 years. The government will pay up to €1,000/m^2 to rebuild residential buildings up to 120 m^2 in area. For business premises and public buildings, government contribution will be up to €500/m^2 for up to 120 m^2 in area. Churches will receive up to €800/m^2 regardless of area. Finally, farm storage, warehouses, stables, etc. will receive up to €250/m^2 for up to an area of 120 m^2. For repairing damage, the government will pay up to €450/m^2 up to an area of 120 m^2 for damage to load bearing and non-load bearing elements. This will be up to €250/m^2 for areas of up to 120 m^2 for non-structural damage. Funds will be provided in successive instalments paid upon completion of specific stages of the work. For owners of buildings classified as yellow or red, the government will subsidize rents to owners for a period of two years. For tenants, this will be for up to six months. As a first estimation, the total cost of replacing or repairing only the damaged buildings will be of the order of €100 to 200 million.

11.4 Typical Damage Patterns by Construction Type

Even in the region affected the most by the 2014 earthquake events, there is a large number of buildings which upon our inspection had no obvious or even minor damage, such as small cracks separating infill walls from the reinforced concrete bearing elements. There are, however, buildings that were severely damaged, as well as buildings that collapsed. In this section, we present typical observed damage patterns, along with some qualitative interpretation of the damage, wherever possible.

Detailed assessments for some of the collapsed or severely damaged buildings will be presented in future revisions of this report. These (currently underway) assessments take into account actual construction and design drawings provided to our teams, recorded ground motions and details observed in the field. At this stage, only general observations and qualitative interpretation of some damaged buildings can be provided.

MASONRY BUILDINGS (POST-1953 GOVERMENT-BUILT)

Buildings of this category typically have a single story and are constructed of cement blocks (Fig. 11.4.1), with Reinforced Concrete (RC) vertical ties/columns (at least at the corners of the structure), horizontal RC ties/beams (at least at the top of the walls), and timber roofs (see detail of Fig. 11.4.2). Most of the masonry buildings discussed in this category behaved adequately, suffering little to no damage.

Figure 11.4.1. Typical single-story masonry building constructed after the 1953 earthquake.

This category also includes the "Arogi" houses ("relief" in English), built with the intent of temporary housing by the government between 1953 and 1959, but are used until now. Several of the Arogi houses experienced damage, mostly in cases where the owners had expanded their

homes by adding masonry or reinforced concrete additions. This practice is fairly common on the island, with houses being built and expanded in several phases during many years to accommodate younger generations of the same family. Expansions were made horizontally (in plan, see Fig. 11.4.1) or vertically (in elevation, i.e., by adding a floor).

Figure 11.4.3 shows an example of an Arogi house built in phases, where a portion of it failed. In this case, the original (1950's) timber roof of the structure was removed and substituted with RC slab to accommodate a RC frame with clay brick masonry infills as a 2nd story, added in 1978. The added 2nd story did not appear to be damaged, but the masonry ground floor collapsed. The failure was probably caused due to excessive loading of the original ground floor. Sections 11.5 and 11.6 provide more cases and information on Arogi houses.

Figure 11.4.2. Smooth steel bars of a vertical Reinforced Concrete (RC) tie, typical for RC building construction during the 1950's and 1960's.

Figure 11.4.3. (a) Collapse of 1st story of a masonry Arogi building where a 2nd story was added more than two decades after the original construction. (b) Cement block masonry reinforced with vertical and horizontal RC ties (GPS coordinates 38°13'49.80"N, 20°25'49.80"E.)

MASONRY BUILDINGS (PRE-1953 OR POST-1953 PRIVATELY-BUILT)

One or two-story buildings constructed of stone or brick masonry built before 1953 or post-1953 by private entities (unlike the government-built Arogi houses), can be found in Lixouri and in neighboring villages. Typical observed damage is shown in Figs. 11.4.4 to 11.4.9.

Figure 11.4.4. Typical bi-diagonal (shear) cracks in a masonry wall**.**

Figure 11.4.5. Shear failure of bearing walls and out-of-plane failure (corner of building and top of walls) of a single-story stone masonry building. Note the double-leaf construction of bearing walls and local disintegration of the outer leaf.

Figure 11.4.6 shows typical three-leaf stone masonry construction with two exterior stone masonry leaves and intermediate filling material of poor quality. Observed damage includes separation of leaves, which probably occurred before the earthquake, and partial collapse of the wall (more extensive for the exterior leaf) due to out-of-plane bending.

Figure 11.4.6. Typical three-leaf stone masonry construction. Damage includes separation of leaves, probably present before the earthquakes, and partial collapse of the wall due to out-of-plane bending.

This category includes churches and schools, some of which are considered landmarks. The behavior of these particular structures are discussed in detail in following sections of this chapter. Indicatively, damage examples on churches are presented in Figs 11.4.7 to 11.4.9.

Figures 11.4.7 and 11.4.8 present severe damage observations after the 2nd event in the masonry church of Virgin Mary in Chavriata (38°10'57.52"N, 20°23'13.61"E). The damage pattern is affected by the damage after the 1st event and eventual collapse after the 2nd event of the stone masonry retaining wall as described in detail in Section 8.3.

Figure 11.4.7. Structural damage of the Virgin Mary masonry church in Chavriata (38°10'57.52"N, 20°23'13.61"E) after the 2nd event, affected by the stone masonry retaining wall collapse (Section 8.3).

The shear and vertical cracks shown on Fig. 11.4.8 may be due to the displacement of the central part of the church towards the failed retaining wall. Prior to the 2014 events, some seismic retrofit was made using RC jackets shown on Fig. 11.4.8a (below the window). However, it seems that RC jackets were not installed throughout the entire perimeter of the church (Fig. 11.4.8b), and the contribution (positive or negative) of this intervention to the overall behavior of the church cannot be assessed at this stage without further data.

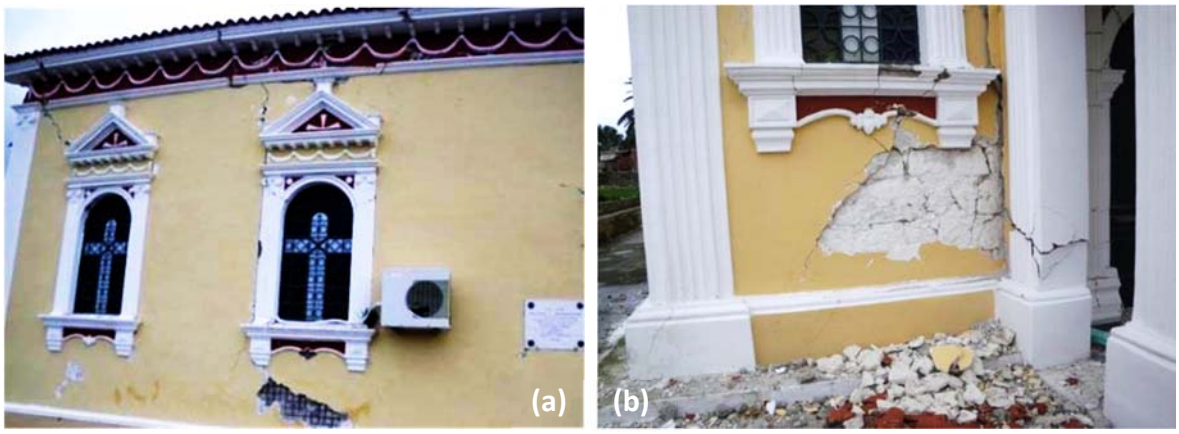

Figure 11.4.8. Observed cracks possibly due to church displacement towards the failed retaining wall. (a) Seismic retrofit RC jacket below window; (b) absence of jackets (38°10'57.52"N, 20°23'13.61"E)

Typical shear and out-of-plane damage in the stone masonry church of Aghia Thekla (38°14′41″N 20°23′06″E) is shown on Fig. 11.4.9.

Figure 11.4.9. Typical shear and out-of-plane damage in the northeast corner of the stone masonry church of Aghia Thekla (38°14′41″N 20°23′06″E).

For details and inventory of the Cephalonia churches, see Section 11.8 of this Chapter.

REINFORCED CONCRETE (RC) BUILDINGS

The Reinforced Concrete (RC) buildings cover the majority of structures in Cephalonia and they overall behaved well, regardless of their year of construction and corresponding seismic code generation followed for their design (see details in Section 11.5).

The most common type of damage, observed in numerous cases, was the detachment of the infill walls from the surrounding concrete beam-column frames. This damage raised much of the public concern since it was a main visible damage pattern for many RC structures. After detailed inspections, no cracks were observed at the RC structural elements of almost every building checked. Additionally, in most cases no diagonal cracks were observed at the infill walls. It was also noted by the reconnaissance teams that typically two horizontal concrete ties, sometimes dowelled into the concrete column-shear wall, were constructed along the height of the infill wall. Due to the stiffness of the infill walls, the inter-story drift was small, possibly explaining the absence of diagonal cracks.

In some RC buildings, damage of structural elements and/or in infill walls was observed mainly along the road north of Lixouri (towards Aghios Dimitrios and Livadi) with the most serious damage in and around the village of Livadi. However, in most cases it was found that the buildings were reinforced with an adequate number of steel bars and stirrups resulting in no observed failures due to the inelastic elongation of steel bars. Observed failures were mainly due to crushing of concrete and diagonal tension. Examples of damage patterns and failures are illustrated in the following figures 11.4.10 to 11.4.20.

Figures 11.4.10 to 11.4.12 show one of the most striking collapses of the 2014 events: the failure of the intermediate story of a three-story RC building that was completed in 2007. The damage of the RC bearing elements on the ground floor (Fig. 11.4.11a) may be attributed to shear failure of a shear wall. On one side the infill walls failed in their plane, while in the perpendicular direction the infill walls collapsed in the out-of-plane direction. An indication of different intensity of ground motion in the main directions of the building is presented in Fig. 11.4.11b. Poor behavior of infill walls with disintegration of vertically perforated bricks is shown on Fig. 11.4.12. The negative effect of “mortar keys” is evident. The RC tie beams (typical for enclosures throughout the country) did not prevent the occurrence of horizontal cracks along the tie beam/brick wall interface. This case history is under investigation and conclusive remarks regarding the collapse cannot be made until more data and analysis results become available.

Figure 11.4.10. Collapse of intermediate story of 3-story RC building completed in 2007. (GPS coordinates: 38.258888, 20.424444).

Figure 11.4.11. Building of Fig. 11.4.10 (GPS coordinates: 38.258888, 20.424444): (a) damage of RC bearing elements on ground floor; (b) indication of different ground motion in main building directions.

Figure 11.4.12. Poor behavior of infill walls with disintegration of vertically perforated bricks observed at the building of Fig. 11.4.10 (GPS coordinates: 38.258888, 20.424444).

A two-story building in Aghios Dimitrios with load bearing system designed to accommodate at least three stories as indicated by the extending steel bars extending beyond the columns top is shown on Fig. 11.4.13 (38°14'36.51"N, 20°25'41.91"E). The ground floor columns failed due to soft story without infill walls. Figure 11.4.14 shows the crushing of concrete and fracture of the closely spaced, single rectangular hoops of the columns.

Figure 11.4.13. Two-story building that experienced typical soft story damage in Aghios Dimitrios (38°14'36.51"N, 20°25'41.91"E).

Figure 11.4.14. Crushing of concrete and fracture of closely spaced, single rectangular hoops of the columns of the two-story building in Fig. 11.4.13 (38°14'36.51"N, 20°25'41.91"E).

Figure 11.4.15. Damage of 2-story RC building in Aghios Dimitrios (38°13′45.6″N 20°25′47.4″E): (a) shear cracks and separation of infills after 1st event; (b) failure of columns and joints after 2nd event; (c, d) detailing with single rectangular largely spaced hoops; and (e) detail of beam-column joint failure.

The damage of a two-story RC building in the village of Aghios Dimitrios (38°13′45.6″N 20°25′47.4″E) following both events is presented on Figures 11.4.15. The 1st event caused shear cracks at the middle column of the façade next to the door and separation of infills from the surrounding frames and shear failure of enclosures (Fig. 11.4.15a). Following the 2nd event failure of columns and beam-column joints along both building axes was observed (Fig. 11.4.15b). The deformed shape of RC elements suggests that they likely failed after the collapse of the infill walls (already separated from the frames and severely damaged during the 1st event). Evidence of poor detailing of the columns with single rectangular largely spaced hoops is shown on Figs. 11.4.15c, d and detail of the failure at a beam-column on Fig. 11.4.15e.

Figure 11.4.16. Residential 1962 "Arogi" public housing complex in Lixouri (GPS coordinates 20.434256, 38.21159) with: (a) extensive damage of infill walls and structural elements; (b) failure of a column. Note large spacing of single rectangular hoops.

Figure 11.4.17. Single story building in the public housing complex of Fig. 11.4.16 without earthquake resisting frames: (a) damage to the columns; (b) detail of slab-column joint.

The public 1962 "Arogi" residential housing complex in Lixouri (GPS coordinates: 38.211666, 20.433611) had extensive damage of the infill walls and of the structural elements

(Fig. 11.4.16). The adjacent single-story building, part of this complex, had no earthquake resisting frames and suffered severe damage to the columns and the slab-column joint (Fig. 11.4.17). Nearby, a 2-story residential building of 1978 had severe damage on the ground floor and minor damage on the upper story (Fig. 11.4.18, GPS coordinates 38.211567, 20.434134).

Figure 11.4.18. (a) Two-story residential buildings of 1978 with severe damage on the ground floor and minor damage on the upper floor; (b, c): Details of damage (coordinates 38.211567, 20.434134).

Failures of the upper part of RC belfry of churches were observed (see Section 11.8 for Cephalonia churches observations). As an example, Fig. 11.4.18 shows belfry damage at Aghios Ioannis church of Kourouklata (GPS coordinates: 38.242119, 20.473977) that could be attributed to stiff upper part, flexible lower part (poor detailing of rather short columns).

Figure 11.4.19. (a) Failure of upper part of the RC belfry of Aghios Ioannis church in Kourouklata (GPS coordinates: 38.242119, 20.473977); (b) detail showing poor detailing of rather short columns with stiff upper part, flexible lower part. Failure and crushing of concrete.

Figure 11.4.20. Typical infill wall construction from the 1960's that incorporates reinforcement with vertical and horizontal rebars that have likely contributed to satisfactory performance of many buildings of that time. This photo shows a building in Lixouri during rehabilitation.

Overall, RC buildings performed well and appear to have redundancy and reserved strength of individual structural members. The reconnaissance teams identified as beneficial factors the structural system regularity, good construction quality, and contribution of the infill walls with vertical and horizontal rebars (Fig. 11.4.20).

TIMBER BUILDINGS

The existing timber buildings and roofs performed satisfactorily, with the timber roof being independent from the main structural system. An exception is shown in Figure 11.4.21, where the superstructure displaced relatively to the foundation.

Figure 11.4.21. Two-story timber residential structures in Aghios Dimitrios. The only damage observed was a crack at the interface of the wood frame/walls with the concrete base slab.

11.5 Structural Behavior Based on Seismic Codes

GREEK BUILDING CODES

A brief historic review of the Greek aseismic code helps us better understand the behavior of the different building types during the 2014 earthquake events in Cephalonia. Greek seismic codes have evolved as major seismic events have been observed over the past decades. The different generations of codes are:

1. 1959: The first seismic code became effective in 1959, a result of the 1953 Cephalonia earthquakes which also caused destruction in the neighboring Ionian islands of Zante (Zakynthos) and Ithaca. This code was based on elastic design and simplified assumptions of the dynamic behavior of buildings. It prescribed a design horizontal force based on the building mass, seismicity zone, and soil category. Three seismicity zones were included, with Cephalonia being in the zone of highest seismicity of seismic factor ranging between 0.08 to 0.16g, depending on soil conditions.

   Prior to 1959, buildings were generally designed for vertical loads only using the 1st building code for Reinforced Concrete (RC) of 1954, an allowable stress design code which did not include any detailing for ductile behavior. However, in the seismic-prone Ionian Islands, some empirical design concepts were applied which allowed for a certain amount of lateral resistance. It has been estimated that the inherent local ductility of RC elements in buildings constructed according to the 1954 and 1959 codes is between 1.5 and 2.0.

2. 1984: Additional seismic design provisions were incorporated in the existing code as a result of the disastrous earthquakes in the two largest Greek cities, Thessaloniki (1979) and the capital Athens (1981). Among other provisions, capacity design of columns in bending and specific detailing for local ductility were introduced.

3. 1995-2000: The New Greek Aseismic Code (NEAK) and the Code for Reinforced Concrete Design (NEKOS) became in effect in 1995 and were finalized in 2000 (with an amendment in 2003) as EAK and EKOS, respectively, based on predecessors of the Eurocodes with modern concepts for ductility. A global behavior factor of 3.5 was introduced for RC frame structures (by which the horizontal seismic force is divided). In addition, an alternative design for elastic behavior was allowed. Currently in effect, EAK-2000 provides a seismic zoning map (Fig. 11.5.1) for rock conditions with three zones I, II, III with reference ground acceleration a_g of 0.16, 0.24, and 0.36 g, respectively, with Cephalonia in Zone III.

4. 2012: Eurocode EC-8 (2008) became in effect concurrently with the Greek codes EAK and EKOS in 2012, introducing more demanding detailing provisions. The seismic zoning of Greece according to the Eurocodes is the same as the one in EAK (Fig. 11.5.1).

Figure 11.5.1. Current seismic zoning of Greece (EAK, 2000) for ground type A (rock) with zones I, II, III of reference ground acceleration a_g of 0.16, 0.24, and 0.36 g. Cephalonia is in Zone III.

EC-8 defines the seismic hazard using the regionally zoned ground acceleration a_g for ground type A (rock with shear wave velocity $V_s > 800$ m/s) for an event with approximate return period of 2,500 years, and modifies for site conditions (ground type). Table 11.5.1 presents the definition of ground types according to EC-8. Site factors, S, associated with each ground type and average shear wave velocity at the top 30 m ($V_{s,30}$) are applied to produce design spectra for the various ground types. EC-8 code-based elastic acceleration spectra are presented on Fig. 11.5.2. The ground types are presented in Table 11.5, including comparisons with the equivalent ASCE 7-05 site classification (the basis of International Building Code IBC-09), discussed in detail in Section 8.1.

Table 11.5.1. Eurocode EC-8 and ASCE7-05 ground types and site classification.

EC-8			ASCE7-05		
Ground Type	Description	$V_{s,30}$ (m/s)	Site Class	Description	$V_{s,30}$ (m/s)
A	Rock	> 800	A	Hard Rock	> 1,500
B	Deep - Very Dense	360 – 800	B	Rock	760 – 1500
C	Deep - Dense to Medium	180 – 360	C	Very Dense Soil / Soft Rock	360 – 760
D	Loose to Medium Dense	< 180	D	Stiff Soil	180 – 360
E	5-20 m thick V_{s30} same as Type C or D	–	E	Soft Soil	< 180
			F	Liquefiable Soil	–

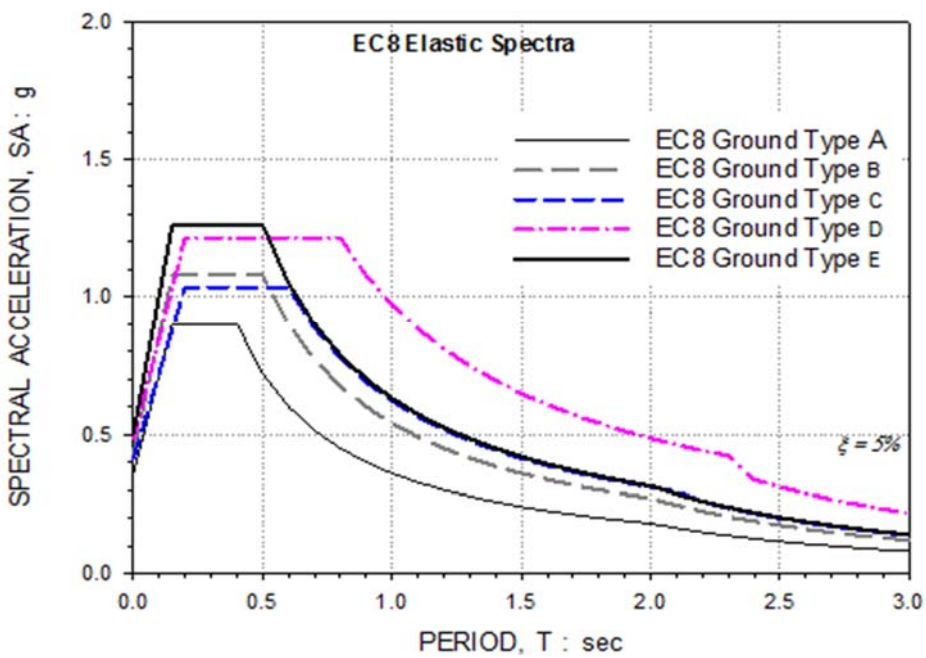

Figure 11.5.2. Cephalonia elastic acceleration response spectra for various ground types based on Eurocode EC-8 and the Greek seismic code EAK-2000.

STRUCTURAL SEISMIC DETAILING

Seismic detailing provisions that may have contributed positively to the observed structural behavior during the 2014 Cephalonia earthquakes include: (i) approximate maximum allowable spacing of stirrups of 100 mm along the height of vertical bearing elements, and 150 to 200 mm between stirrup legs in cross section. This requirement was introduced in 1984 as supplementary provision to the 1959 code, and is currently in effect, and (ii) shear walls are required to have confined columns at both ends based on the current codes. In the 1959 code, the cross-section of the corner columns should be at least 350 mm on each side for square columns, and no provisions were included for the detailing of shear walls.

COMPARISON WITH 2014 RECORDED GROUND MOTIONS

Figures 11.5.3 and 11.5.4 compare acceleration response spectra recorded ground motions from the 1st and 2nd events, respectively, with the Cephalonia EC-8 elastic spectra. Details of the stations are provided in Chapter 7. As discussed extensively in Section 8.1, local site effects and directivity have likely contributed to high amplification effects that need to be studied further with pertinent in-situ testing.

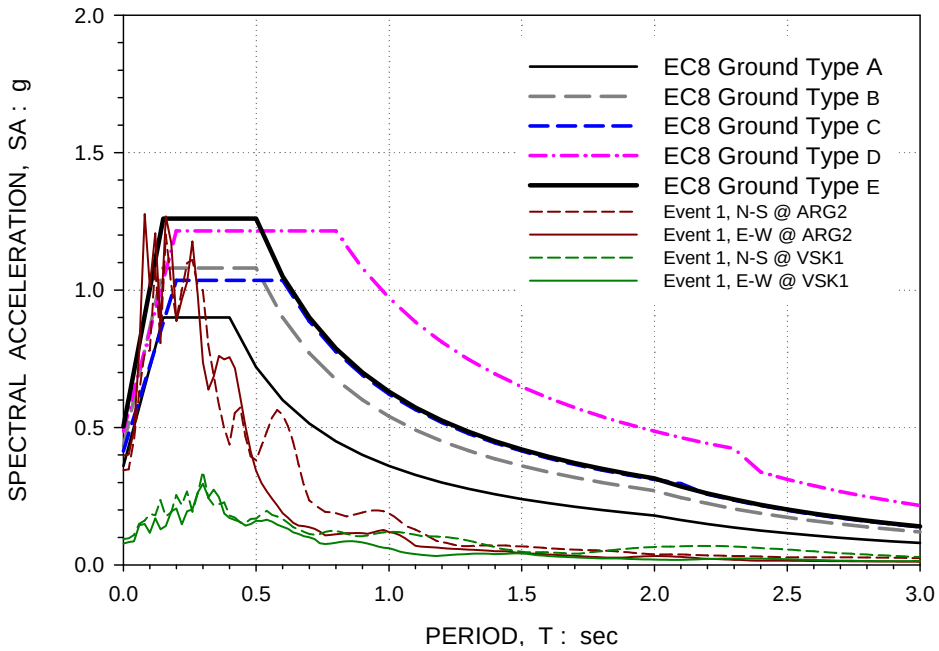

Figure 11.5.3. Comparison of elastic response spectra for various ground types, based on current seismic codes (EC-8 and EAK-2000) with spectra of recorded motions from the 1st event of 1/26/14.

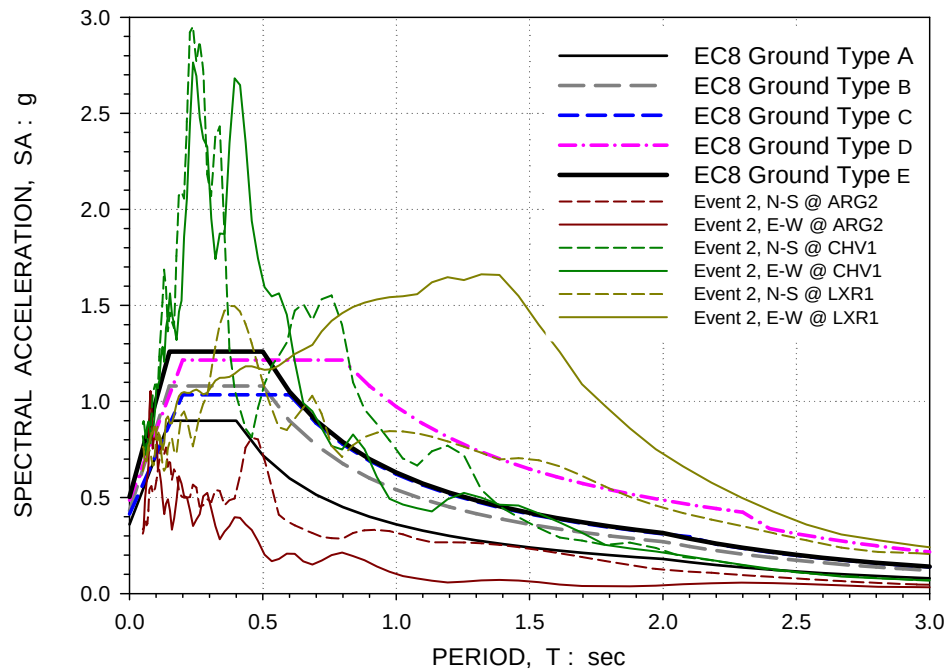

Figure 11.5.4. Comparison of elastic response spectra for various ground types, based on current seismic codes (EC-8 and EAK-2000) with spectra of recorded motions from the 2nd event of 2/3/14.

The recorded spectra far exceeded code-based spectra in the 2nd event, with the exception of the Argostoli (ARG2) records that are lower, which can be expected since this station is farther away from the epicenter and is on stiffer site conditions as compared to the other stations. Modern buildings designed to latest code are expected to have a period in the range of 0.2 to 0.4 seconds and have experienced maximum spectral accelerations reaching 3 g (at Chavriata) and 1.5 g (in Lixouri) during the 2nd event.

The resiliency of these structures need to be studied further. The Cephalonia case studies present an excellent opportunity to analyze and explain good structural behavior of RC buildings subjected to spectral accelerations more than twice their elastic spectral design values. The information provided in this report, including structural drawings for a dozen of structures provided to our teams by their owners, can be used to this end, combined with pertinent soil testing that can explain site response effects.

STUCTURAL OBSERVATIONS BY DESIGN CODE

Structural observations have been grouped based on the year of construction and contemporary applicable seismic code. Regardless of the year of construction, building infills and partitions are typically made of brick masonry that are considered non-bearing elements and are not taken into account in the seismic design.

Built prior to 1953

Most of the Cephalonia structures were built after the 1953 earthquakes that destroyed most of the building stock of the island. Buildings constructed as family homes prior to 1953, mostly found in small villages, typically have load bearing masonry walls and wooden-frame clay-tiled roofs as shown on Fig. 11.5.5. Most of them are single-story, and if there is a 2nd floor, wooden floors are used. Many churches on the island also belong in this category, as described in detail in Section 11.8.

Although these structures were not constructed for seismic loads, the pre-1953 buildings which still remain on the island have survived not only the 1953 earthquakes, but also a number of other earthquakes since then. In most cases, their use has been changed in the past decade from family homes to low-importance occupancy structures used for storage of agricultural goods or tools. Currently, these buildings have become abandoned or not maintained.

Figure 11.5.5. Typical pre-1953 masonry buildings in Cephalonia.

Damage observed in the pre-1953 buildings includes diagonal cracks on the bearing walls and/or partial collapses. In some cases, cracks were formed during previous earthquakes and were inadequately repaired with a different type of mortar and were aggravated by the 2014 earthquakes. After the 2nd event, many of these structures suffered major damage or collapsed (Fig. 11.5.5: Chavriata 38º11'0.67"N, 20º22'54.93"E; Havdata 38º12'11.62"N, 20º23'09.01"E). This observation was recorded in a small number of structures throughout the island and cannot be attributed to a particular area.

Built 1953-1959 "Arogi" Houses (Arogi = Αρωγή = Relief)

Arogi (relief) houses are single-story individual structures built after the 1953 earthquakes by the government for temporary use, but have been maintained until now and they are common in Cephalonia. Observations for these houses are discussed throughout this Chapter.

The Arogi structures are generally rectangular in plan (ranging from 30 to 40 m^2, up to 80 m^2), made of cinder-block walls with cement mortar encased in RC frames as shown in Fig. 11.5.6. The walls are reinforced with vertical steel bars running through their height typically every two cinder blocks. In addition, there is some horizontal reinforcement in the walls below the window openings (with the top of the window located at the RC beam).

Figure 11.5.6. Pre-1953 masonry construction: (a) partial collapse of a Chavriata house (38°11'0.67"N, 20°22'54.93"E), 50 m from station CHV1; (b) collapse of building Chavriata; (c) collapse of 2-story house in Havdata (38°12'11.62"N, 20°23'09.01"E); (d,e) partial collapse of secondary-use buildings; and (f,g) adjacent buildings with no damage in (g) that had reinforced concrete ties at the top.

This type of construction is light, simple, quick and economical to build and has high resistance to seismic forces. In cases where no additions/expansions were made to an Arogi house, the structure behaved well during the 2014 earthquakes as they did in the preceding events of lower intensity in the past 50 years. Few structures, however, did suffer some damage expressed as visible diagonal cracks in the infill walls.

Arogi houses often include multiple extensions constructed over the past decades with different construction methods. Typically horizontal extensions were made first (usually after 1959), followed by vertical 2nd-story additions (usually between the 80's and 90's). The performance of these structures is discussed in Section 11.6.

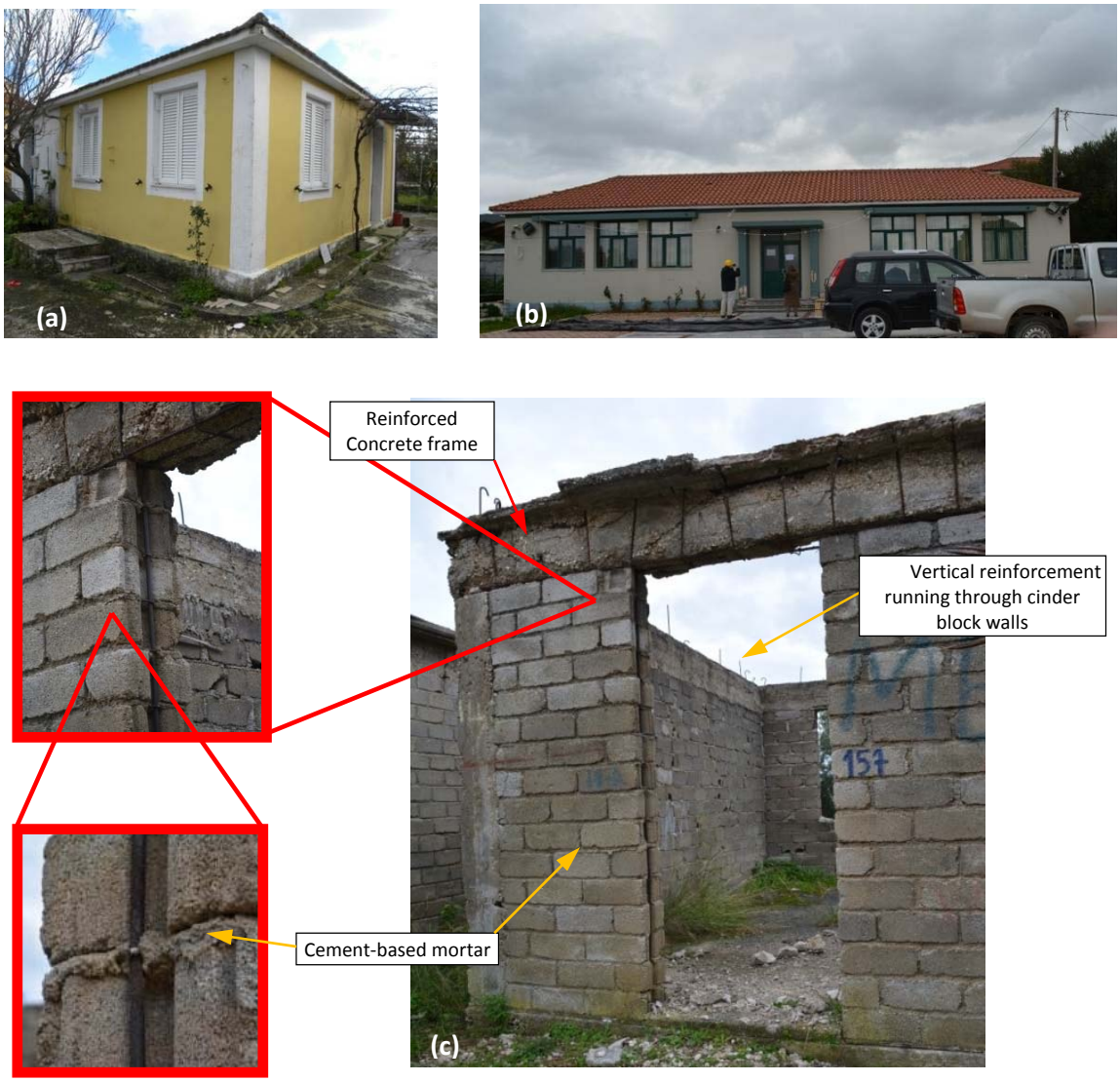

Figure 11.5.7. Arogi-type buildings post-1953: (a) typical Arogi house in Livadi; (b) Chavriata old school (38°11'00.67"N, 20°22'54.93"E) with cinder block construction with recording station CHV1; (c) details of encased reinforced cinder walls with cement based mortar.

Interestingly, the old school building in Chavriata (38°11'00.67"N, 20°22'54.93"E) shown on Fig. 11.5.7b, had the CHV1 station accelerometer installed that recorded a Peak Ground Acceleration PGA of 0.75·g with Spectral Acceleration (SA) of almost 3 g during the 2nd event (Fig. 11.5.4). The construction of this house is typical of the Arogi type (i.e., load carrying steel-reinforced cinder block walls). In a radius of 45 to 52 m from the CHV1 station (as shown on Fig. 11.5.8a), the following observations were made: (i) Arogi house of Fig. 11.5.7c (52 m away from the CHV1); (ii) partially collapsed masonry house showed of Fig. 11.5.8b; (iii) 2-story structure of Fig. 11.5.8b with no damage. Figure 11.5.9 shows the exterior and interior damage of the old school building in Chavriata, where CHV1 is located.

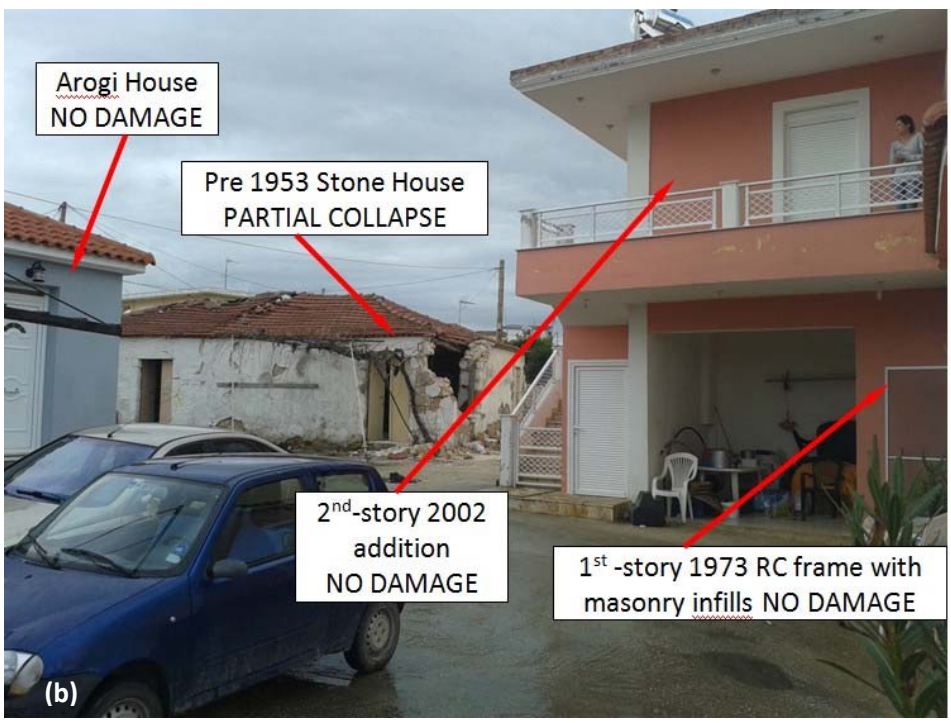

Figure 11.5.8. (a) Locations of CHV1 accelerometer in Chavriata old school building (38°11'00.67"N, 20°22'54.93"E) and four buildings within 45 to 52 meters around it; (b) three buildings east of CHV1: partially collapsed stone house, Arogi house, and two-story RC building.

Figure 11.5.9. Exterior (top) and interior (bottom) damage of the old school building in Chavriata (38°11'00.67"N, 20°22'54.93"E) where the CHV1 accelerometer is located.

Built between 1959 and 1984 (1959 seismic code)

Buildings built between 1959 and 1984 were designed based on an allowable stress design philosophy and low (relatively to current) values of the so-called "seismic coefficient," to calculate the horizontal seismic forces. Furthermore, detailing provisions were poor and, as a result the spacing of hoops in columns was not adequate and there were no provisions for detailing the beam-column joints. Examples of damage in these elements are shown on Figs. 11.5.11 and 11.5.12 (now demolished) from buildings in Chavriata and Lixouri. The building of Fig. 11.5.10 (38°11'00.67"N, 20°22'54.93"E), 45 m from CHV1 in Chavriata (Fig. 11.5.8a) was constructed in 2 phases: the 1st story was built in 1978 and the 2nd story in 2002. It suffered no structural or infill wall damage in either earthquake despite its proximity to CHV1. Only nonstructural damage was observed in a small kitchen on the 2nd floor (broken glassware, etc.).

Figure 11.5.10. Two-story reinforced concrete frame building next to the old school in Chavriata, 45 m from the CHV1 station (38°11'0.67"N, 20°22'54.93"E) that experienced no structural damage.

Figure 11.5.11. Three-story building in Lixouri (with partial 4-story extensions) probably constructed before 1984 following the 1959 code (38°12'14.17"N, 20°26'01.45"E). Collapse of some infill walls on 3rd and 4th floors (incomplete, no door/window frames) and cracks at column-beam joints on 4th floor.

Figure 11.5.12. Three-story building in Lixouri (Aravantinou Krasopatera St., 38°11'36.81"N, 20°26'11.42"E), probably built before 1984. The two upper floors did not appear to suffer any structural or infill wall damage. The columns and shear walls (or wide columns) of the 1st floor were severely damaged (soft-story failure). The structure was later demolished.

Built between 1984-1995 (1959 seismic code, with 1984 supplementary provisions)

The 1984 additional provisions to the 1959 seismic code included capacity design of columns in bending and specific detailing to guarantee local ductility. Structures complying with the 1984 supplement to the 1959 code provisions behaved satisfactorily.

For example, Fig. 11.5.13 shows observations at the three-story Palatino Hotel in Argostoli (38º10'80.00"N, 20º29'14.40"E). The three-story building has one basement level and it consists of two parts with dimensions 23m × 12m and 23m × 14m, separated by a thermal expansion joint that opened during the earthquakes (Fig. 11.5.9b). The building had some nonstructural damage in contents and infill walls.

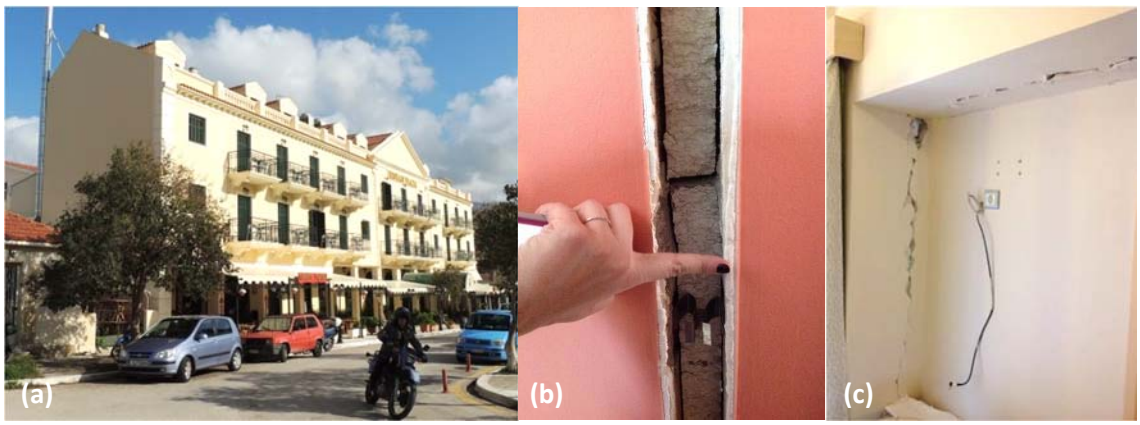

Figure 11.5.13. (a) Three-story hotel building in Argostoli (38º10'80.00"N, 20º29'14.40"E) designed in 1989 using the supplementary provisions to the 1959 building code; (b) width of the construction join opening; (c) broadening of joint after the 2nd event.

The two-story building of Fig. 11.5.14 is located in Aghios Dimitrios (38º14'36.51"N, 20º25'41.91"E). It has a reinforced concrete frame structure designed to accommodate three stories, although only two of them are built. The ground floor does not have infill walls (soft story). Reconnaissance teams observed soft story failure with disintegration of concrete and fracture of the closely spaced, single rectangular hoops of the columns. The further away columns were from the stair, the greater the building rotation and the column damage.

Figure 11.5.14. Two-story RC frame with infills in 2nd story in Aghios Dimitrios (38º14'36.51"N, 20º25'41.91"E). The building plan is rectangular with one side twice as long as the other.

Built from 1995 to present (current Greek code EAK-2000)

Given that Cephalonia is in the highest of the three seismic zones in Greece, the current seismic code EAK-2000 does not allow soft stories (open ground floors). The vast majority of structures built using the current code behaved well with practically no damage at all (including no damage to infill walls). Figure 11.5.15 shows typical examples of these newer buildings in Lixouri.

Figure 11.5.16 shows a two-story RC framed structure with shear walls in both directions and clay-brick infill walls in Livadi (38º15'24.81"N, 20º25'31.77"E). It was completed in 2004 and no structural damage was observed in any of the RC frame elements. However, extensive infill wall cracking and failure was observed as shown in the photos of the interior and exterior.

Figure 11.5.15. Typical buildings in the city of Lixouri designed according to the current seismic code EAK-2000 had no visible damage.

Figure 11.5.16. Two-story house in Livadi (38º15'24.81"N, 20º25'31.77"E), east of the Lixouri-Argostoli road. (a) infill wall cracking from the exterior; (b) infill wall failures from the interior.

11.6 Special Cases of Structural Interest

MUTLIPLE ADDITIONS WITH MULTIPLE CODES

The Greek society of Cephalonia has a family structure where several generations live together under one building. To accommodate this, the original building (usually of Arogi type described in the previous section) typically has had horizontal and vertical additions at different time periods and under different seismic codes. As a result, unique case studies of structural interest were identified of mixed construction methods using different seismic codes in the same building. Three of those cases are documented in this Section, discussing two-story buildings consisting of the original structure and additions (Figs. 11.6.1 to 11.6.3).

Case 1: The Livadi building of Fig. 11.6.1 (38°15'26.13"N, 20°25'21.38"E) suffered damage on the upper floor, but no visible exterior damage to the lower floor. It is a RC frame structure with clay brick walls constructed in the 1980's. The north part of the 1st floor was an Arogi house built in 1953. In 1973, both a lateral addition and a 2nd floor were constructed. The Arogi house was fully enclosed within the 1st floor of the addition. As shown on Fig. 11.6.1, all upper floor columns developed plastic hinges at their top, where they connect to the beams. The smooth steel reinforcement bars were rusted where exposed. Transverse column reinforcement were spaced at 25-30 cm and did not continue through the joint.

Figure 11.6.1. Two-story house in Livadi (38°15'26.13"N, 20°25'21.38"E). Damage is evident in every column of the 2nd floor and in some infill panels. No damage to the 1st floor was visible from the exterior.

Case 2: Figure 11.6.2 shows photos and a sketch of the plan and side views of a building in Aghios Dimitrios (38°14'36.51"N, 20°25'41.91"E) which was constructed in three phases. The 1st phase was an Arogi house built in 1954, the 2nd phase was a lateral addition completed in 1981, and the 3rd phase was a 2nd floor addition built in 1990. This structure suffered no structural damage. It appears that the building was founded on backfill soil supported by a 1-m tall cinder-block retaining wall (black line on Fig. 11.6.2). The retaining wall failed and the whole structure displaced 1-2 cm towards the dashed line. Due to the fact that the L-shaped (green color) columns on the south side of the structure were founded on isolated spread footings, the columns were separated from the old Arogi house columns by approximately 1 cm at the bottom as shown on the lower left part of the figure.

Case 3: The two-story building of Fig. 11.6.3 in Aghios Dimitrios (38°13'50.45"N, 20°25'49.78"E) was constructed in two phases and partially collapsed during the earthquake events. The 1st story is an Arogi type construction (concrete frame encasing cinder block walls) and the 2nd story (vertical addition with columns that extended from the 1st story into the 2nd) is a RC frame with clay brick infills built in 1978. The structure was damaged but remained standing after the 1st event showing moderate shear cracking in the walls and in some of the columns, prompting the owner and his professional engineer to use some shoring to support the building in case a 2nd earthquake occurred. The column longitudinal reinforcement consisted of four bars with stirrups approximately at every 50 cm. After the 2nd event, large shear cracks occurred in the columns as well as the walls of the 1st story resulting in the significant tilting of the 2nd floor.

It can be assumed that if the 2nd story addition had not been constructed, the single-story Arogi structure with its encased reinforced cinder block wall to the RC frame could have resisted the seismic loads from the two seismic events, although its members were lightly reinforced. This hypothesis can be further supported by the observed good response of Arogi houses throughout the Paliki peninsula area, as demonstrated by the three single-story Arogi houses within 50 m from the CHV1 accelerometers that survived the earthquakes with minimal damage, discussed in the previous section (see Fig. 11.5.7). When the 2nd story was added (without any additional strengthening of the members in the 1st floor) the seismic load doubled. This caused moderate shear cracking in the walls and columns of the first story during the 1st event. When the 2nd earthquake event occurred, the already weakened first story failed.

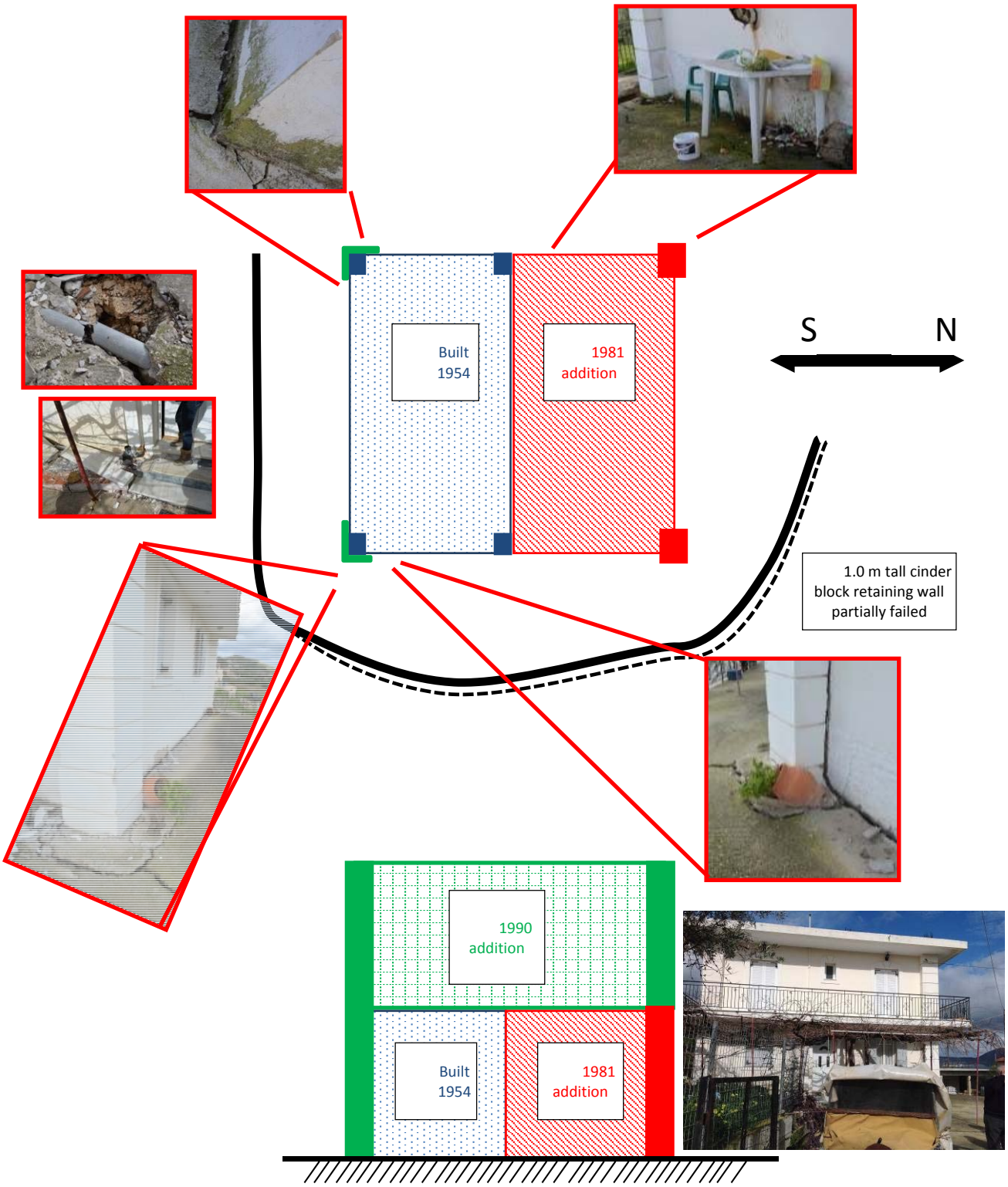

Figure 11.6.2. Top: sketch of the building plan view in Aghios Dimitrios (38º14'36.51"N, 20º25'41.91"E) and pictures showing the structure and its connection to the foundation. Bottom: sketch of the building elevation. Colors indicate the three phases of the construction.

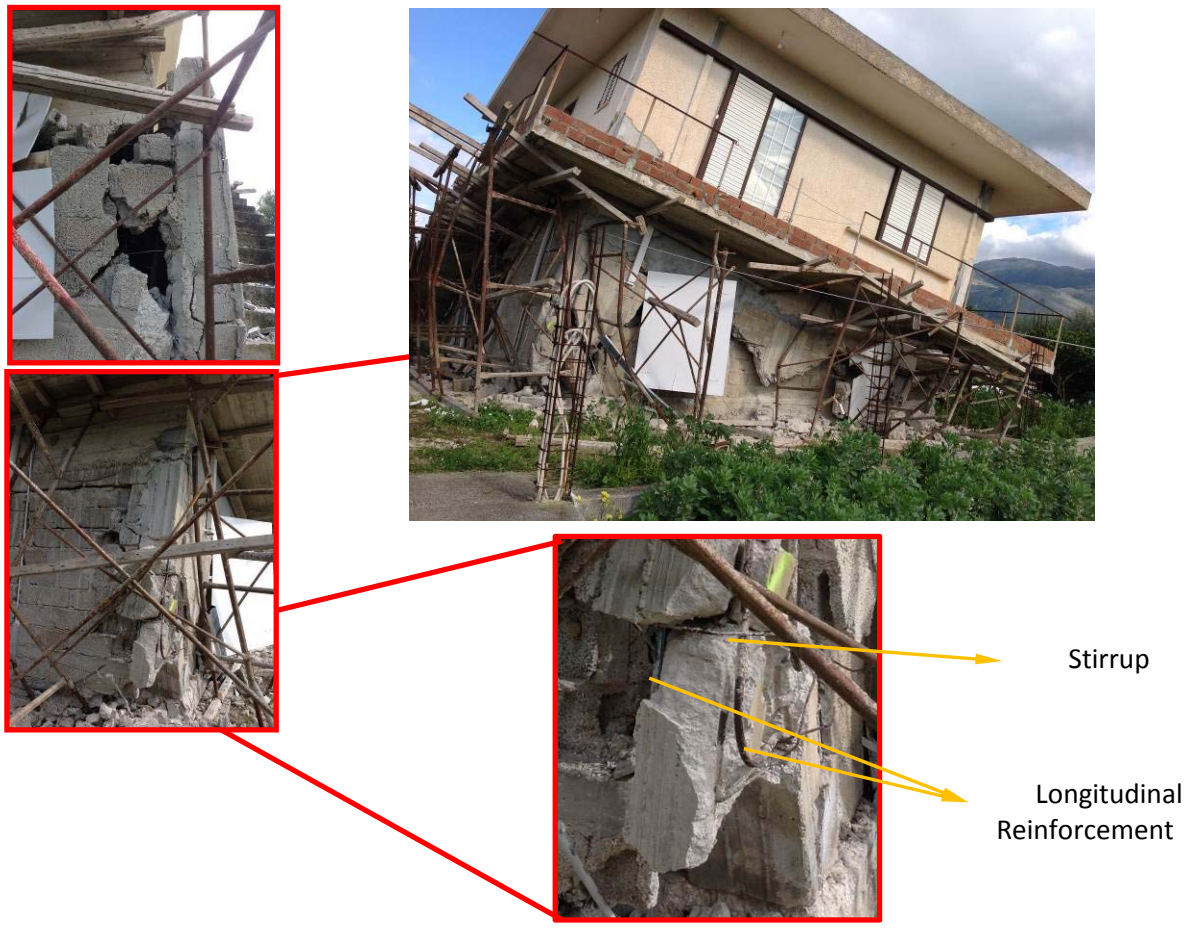

Figure 11.6.3. Tilted 2-story building in Aghios Dimitrios (Arogi house on 1st floor and 1978 RC clay-brick infill frame on 2nd floor). Failed columns and walls of the 1st story showed few stirrups along the columns and only 4 longitudinal reinforcing bars in the cross section (38°13'50.45"N, 20°25'49.78"E).

Overall, structures built in two or three phases behaved well. The buildings that experienced damage were constructed before 1984 (under the seismic code of 1959) with limited transverse reinforcement in the columns and no transverse reinforcement in the joints.

PUBLIC LOW INCOME HOUSING COMPLEX

A public low income housing complex was one of the most heavily damaged buildings in Lixouri (38°25'81.19"N, 20°26'01.84"E). The complex is a series of two- and three-story structures built during the 1960's under the first 1959 seismic code. The structures suffered extensive, non-repairable flexural and shear damage in most vertical RC structural members (Fig. 11.6.4, top). A number of buildings, primarily those strengthened in the vault with fiber-reinforced polymers, had moderate or minor damage and were classified as repairable.

In contrast to the behavior of the public housing complex, all of the newer buildings in the immediate vicinity remained essentially intact as they were built after 2000, in accordance with

the current seismic code. In many cases, the new buildings had code-imposed design differences that likely contributed beneficially to their good behavior, e.g. mat foundation rather than spread footings, or RC walls on the ground floor rather than brick masonry infills.

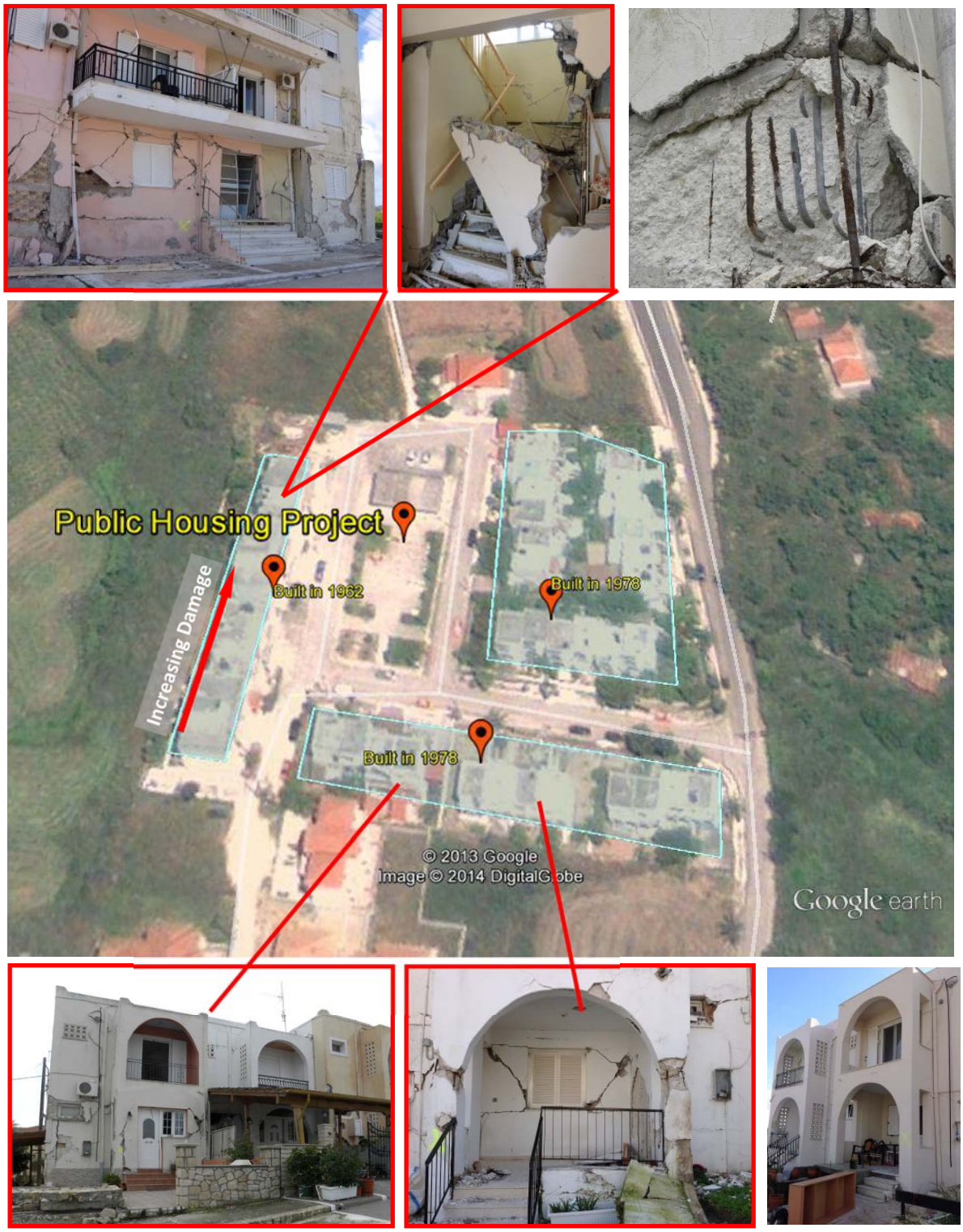

Figure 11.6.4. Public low income housing complex. Top: severe damage of the three story buildings built in 1962. Top right: lack of shear reinforcement. Center: public housing blocks/units. Buildings with red roof on the lower portion are undamaged two-story houses built after 2000 with the current seismic code. Bottom: Repairable damage at two-story buildings built in 1978.

TYPICAL DESIGN DRAWINGS AND DOCUMENTATION

In Greece, the building owners usually have a copy of the design drawings and construction permit. Our reconnaissance teams have obtained copies of these documents for approximately a dozen of buildings, several of which are in the immediate vicinity of recording stations and behaved satisfactorily (Fig. 11.6.5, 38º11'0.67"N, 20º22'54.93"E). The documents, combined with photographs and recorded motions can be used to further study the behavior of the buildings.

Figure 11.6.5. Typical drawings collected for the 2-story RC building shown on Fig. 11.5.7, about 50 m from CHV1 accelerometer in Chavriata (38º11'00.67"N, 20º22'54.93"E). Construction was done in two phases, in 1978 with an addition in 2000. The building suffered no visual structural damage.

Future versions of the report will provide organized information on the collected structural documentation, which is currently underway.

BULDING INSTRUMENTATION

Following the 1st event, the EPPO-ITSAK team installed a mobile accelerometer instrumentation array in a two-story (with one basement) building in Lixouri to record potential aftershocks. The building selected is the new administrative building of the Lixouri hospital (Fig. 11.6.6) that was erected in 2009, designed with the latest Greek seismic code in combination with the Eurocode EC-8 provisions. The construction was made possible with a grant from the Stavros Niarchos Foundation (snf.org). The record of station ARG2 in Argostoli showed a PGA = 0.38 g and had similar spectral shape as the design response spectrum, as discussed in Section 11.5 and shown on Fig. 11.5.2. The building behaved very well in the 1st event, with essentially no damage to either its load bearing structural system or the infill walls.

The particular structure was selected for instrumentation because: (i) it has a more or less "regular" structural and architectural system, both in plane and height (i.e., no soft story, static eccentricities or other factors that would lead to particularities in its seismic response); (ii) its plan dimensions and height (two stories) are representative of Cephalonia buildings; (iii) it is stand-alone, with no adjacent buildings that could affect its response (e.g., through pounding); (iv) it is a public building, with easier accessibility than private structures; (d) it does not have a high occupancy that could accidentally disrupt the operation of the instrumentation system. The ground floor was not instrumented as it was temporarily used as an emergency medical center, since the nearby main hospital ("Mantzavinateio", discussed in Section 11.7) building was evacuated until its seismic safety was evaluated by professional engineers; (vi) as-built drawings were available that can facilitate numerical modeling that will be further calibrated using the recorded response of the building in the aftershocks and the 2nd event.

Fig. 11.6.6. The administration Lixouri building (left) and photos of instrumentation with special accelerometer array (right).

The special structural array used in the instrumentation consists of a central recording unit (type K2© by Kinemetrics Inc.), that can support up to 12 sensors (uniaxial, ± 2g full scale, Episensor© accelerometers). The recording unit has a 19-bit resolution, a sampling rate capacity of up to 200 sps and a dynamic range of 108 dB @ 200 sps. The system can accommodate setting independent triggering threshold for each sensor (from 0.01% to 100% full scale), while the user can predetermine the sensors, or combinations of them, that will trigger the system. Recordings are stored in the system's flash memory, and can be retrieved either in situ or through a modem.

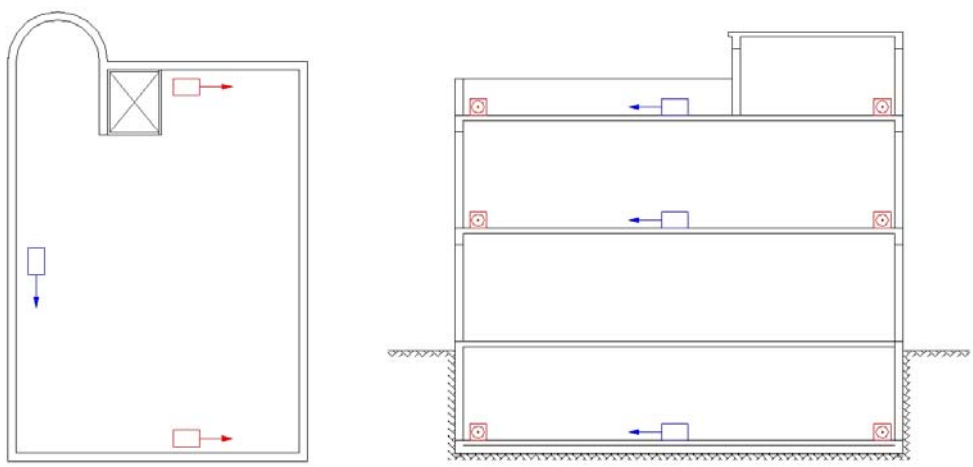

Figure 11.6.7. Instrumentation layout of Administration building at Lixouri.

The instrumentation, shown on Fig. 11.6.7, includes 9 uniaxial sensors in sets of 3, at 3 building levels: basement, 1st floor (what would be called 2nd floor in the US) and terrace. As previously described, the ground floor level (or 1st floor in the US) was not instrumented. At each level, 2 uniaxial sensors were placed in parallel along the floor's edges (Fig. 11.6.7 in red color), and the 3rd (Fig. 11.6.7 in blue) was placed in an orthogonal direction along one of the other two edges. In this way it is possible to record the structural response in the two translational orthogonal and torsional directions.

The recorded response of the building will be used to assess its dynamic characteristics (eigenvalues, eigenmodes, and damping ratios) that will assist in properly calibrating finite element models of the structure that will be used to further investigate the response to seismic excitations, including the 1st event. This EPPO-ITSAK research effort aims at contributing in the advancement of knowledge of seismic response of civil engineering structures (Karakostas et al. 2003, 2005, 2006, Lekidis et al. 1999, 2005, 2013, Sous et al. 2004). The research effort is ongoing and results will be presented in future versions of this report.

11.7 Public Buildings

INTRODUCTION

This section presents reconnaissance information for public buildings of Cephalonia, including administrative, critical, and school structures. Overall, the structural performance of the public buildings inspected was satisfactory given the intensity of the earthquake ground motions. As a result, immediate occupancy and next day serviceability was possible for most of these buildings, also attributed to acceptable behavior of nonstructural components, with the exception of the International Airport Terminal that had to close for three weeks due to nonstructural components damage. In some cases, including schools, conservative measures of evacuation and detailed engineering assessment were carried, partially due to feelings of population insecurity due to their (still vivid) experience of the devastating 1953 earthquakes.

ADMINISTRATIVE AND CRITICAL BUILDINGS

The public administrative and critical buildings that were visually inspected by the reconnaissance teams are all of Reinforce Concrete (RC) summarized in Table 11.7.1. Constructed after the destructive 1953 earthquakes, they generally behaved well and with few exceptions, became operative one or two days after the events. Some observations are discussed in this section.

Table 11.7.1. Public reinforced concrete buildings visually inspected during reconnaissance.

ID	Building	Location	No. of Stories	Structural Type	Year of Construction	Design Code	Damage (visual observation)
1	Hospital "Mantzavinateio"	Lixouri	2	Reinforced Concrete (RC)	1937 / 1952	N/A	Minor
2	Hospital	Argostoli	2		N/A	N/A	Negligible
3	Town Hall	Lixouri	2		1968	1959	Minor
4	County Court	Argostoli	2		1962	1959	Minor
5	County Court	Lixouri	4		1979	1959	Negligible
6	Tax Office	Argostoli	4		1994	1985	Negligible
7	Tax Office	Lixouri	3		2000	1995	Negligible
8	Port Authority (main/auxiliary)	Argostoli	2		1960	1959	Minor/Moderate
9	Public Realty Office	Argostoli	4		1999	1995	Negligible
10	Naval Academy (administrative)	Argostoli	3		1973	1959	Minor
11	Archaeological Museum	Argostoli	2		1957 / 2000	N/A	Moderate
12	Archaeological Museum	Lixouri	2		N/A	N/A	Negligible
13	Prefectural Division of Ionian Islands	Argostoli	3		1957	RC1954	Minor
14	Airport Terminal	Argostoli	2		N/A	N/A	Minor
15	Police Station	Argostoli	3		>2000	2000	None

The Cephalonia International Airport terminal building (ID #14, coordinates 38°7'10.0"N, 20°30'17.7"E) suffered nonstructural damage, described in detail in Section 11.9. The interior masonry walls cracked and parts of the roof cladding fell. The terminal remained closed to the public for three weeks. (Fig. 11.7.1)

Figure 11.7.1. Cephalonia International Airport terminal building (ID #14, coordinates 38°7'10.0"N, 20°30'17.7"E): (a) aerial photo; (b) nonstructural damage after the 2014 events resulted in closing the terminal for three weeks.

The Lixouri City Hall (Fig. 11.7.2, ID# 3, coordinates 38°12'3.53"N, 20°26'14.92"E) suffered damage following both events. Figure 11.7.2a shows the building before the earthquakes and Figs. 11.7.2b,c show damaged columns and overturned statue after the 2nd event. Further details on statues overturning is provided in Chapter 9 of this report.

Figure 11.7.2. Lixouri Town Hall. (ID#3 in Table 11.7.1; coordinates 38°12'3.53"N, 20°26'14.92"E). (a) before the 2014 events; and after the 2nd event: (b) column damage; (c) overturning of statue.

The main and auxiliary buildings of the Port Authority in Argostoli (ID #8, coordinates 38°10'44.8"N, 20°29'23.3"E) remained in operation despite extensive ground deformations in

the overall bay area. Settlements of 5 to 10 cm and tilting of 0.7^{o} were measured. Construction joints with adjacent buildings opened by up to 5 cm (Fig. 11.7.3).

Figure 11.7.3. Open construction joint at Argostoli Port Authority (ID#8 in Table 11.7.1; coordinates 38°10'44.8"N, 20°29'23.3"E).

The Merchant Naval Academy (Fig. 11.7.4, ID# 10) is located 700 m north of the Argostoli's main square (coordinates 38°11'4"N, 20°29'9"E) and has been operating since 1975. It is considered one of the most prominent naval academies in the country. The complex has two main blocks, each one of which consists of three buildings separated by joints (Fig. 11.7.4a). Minor diagonal cracks were observed mainly on a few external masonry walls and extensive pre-existing corrosion was evident (Fig. 11.7.4b).

Figure 11.7.4. (a) Merchant Naval Academy building (ID#10 in Table 11.7.1) in Argostoli (coordinates 38°11'4"N, 20°29'9"E; photo from kefalonia.net.gr); (b) Observation following the 2nd event of lack of shear reinforcement and corrosion of rebars.

The Argostoli Archaeological Museum (Fig. 11.7.5, ID# 11, coordinates 38°10′40″N 20°29′17″E) was constructed in 1957 to replace the original one which was destroyed during the 1953 earthquakes that resulted in loss of several findings from the first Marinatos archaeological excavations. The Ministry of Culture conducted a rehabilitation and repair

project from 1998 to 2000. The reconnaissance teams observed concrete spalling due to reinforcement corrosion at the base of the columns. A number of exhibits overturned or fell within their showcases.

Figure 11.7.5. Argostoli Archeological Museum (ID#11 in Table 11.7.1; coordinates 38°10′40″N 20°29′17″E). (a) photo taken after the 2000 rehabilitation but before the 2014 events (mygreece.travel); (b) concrete spalling; and (c) minor cracking at short columns. Both photos taken after 2nd event.

The main General Hospital of Lixouri "Mantzavinateio" (ID #1 in Table 11.7.1, coordinates 38°10'44.8"N, 20°29'23.3"E) is a 2-story RC building completed in 1957 with floor plan area of 625 m^2. Its construction began in 1937 but from 1937 to 1952 the construction stopped with only the structural frame in place (Fig. 11.7.6a). A seismic retrofit program for the hospital started in 2004 by collecting data, including performing geotechnical subsurface and laboratory investigation and non-destructive testing on the foundation and superstructure (Figs. 11.7.6b,c). The retrofit was completed in 2012. The hospital was evacuated preventively, although no major damage was observed, following the 1st event and was back in operation on February 6th, few days after the 2nd event (Fig. 11.7.6d, inkefalonia.gr, 2014) following engineering assessment (Bardakis, 2014). Visual inspection of our reconnaissance teams after the 2nd event did not reveal any significant damage to the structural load bearing system.

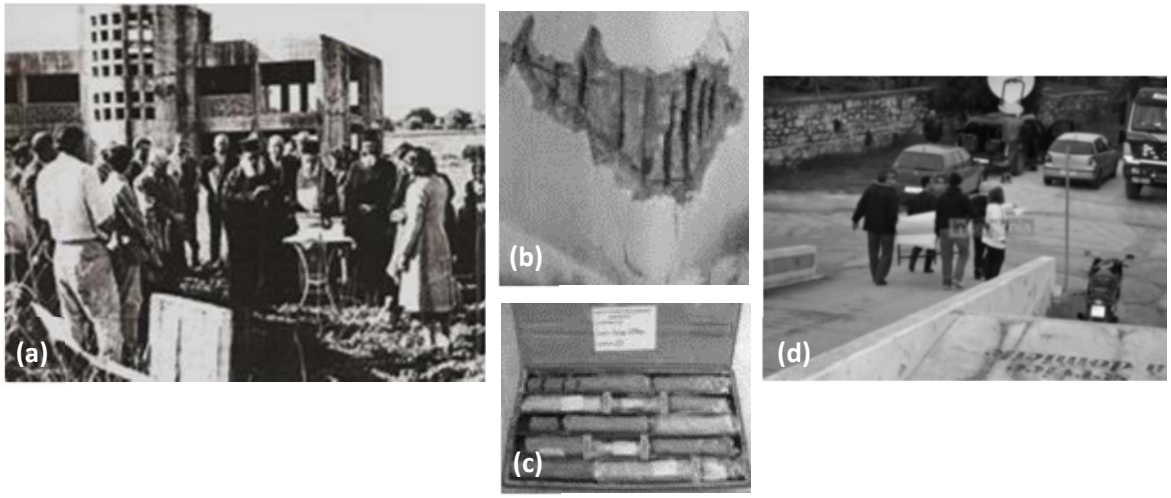

Figure 11.7.6. Lixouri "Mantzavinateio" General Hospital (ID #1, 38º10'44.8"N, 20º29'23.3"E): (a) historic photo of structural frame completion prior to 1953; (b,c) structural and geotechnical investigations for 2004-12 seismic retrofit; (d) evacuation after 1st event (inkefalonia.gr, 2014). Following assessment, the hospital was deemed safe and operations resumed few days after the 2nd event. Retrofit and historic information and photos a,b,c from Barthakis (2014).

Examples of public buildings with satisfactory structural responses include a three-story building of the Prefecture (ID #13) constructed in 1957 which exhibited only a limited number of hairline cracks in the infill walls and a two-story building of the Court House in the center of Argostoli (ID #4, coordinates 38°10'37"N, 20°29'18"E), constructed in 1962 (Figure 11.7.7).

Figure 11.7.7. Examples of satisfactory behavior of Argostoli administrative public buildings: (a) Prefecture building (ID #13); and (b, c) Court House (ID #4, coordinates 38°10'37"N, 20°29'18"E).

SCHOOLS AND OTHER EDUCATIONAL BUILDINGS

Following assessment by the Organization of School Buildings (Ο.Σ.Κ.), 37 educational units in Cephalonia were classified as "A" (immediate occupancy), 19 as "B" (immediate occupancy with rehabilitation after school hours), and 9 will restore operation after the damage is repaired. Table 11.7.2 shows a summary of the most important educational structures that had some level of damage. For comparison purposes, we include the Technological Educational Institute (T.E.I.) which accommodated the Argostoli crisis management center and the GEER/EERI/ATC reconnaissance teams meetings, and remained intact after the two seismic events. Damage at "Petritsio" High School of Lixouri (ID#1) is shown on Fig. 11.7.8.

Table 11.7.2: Educational buildings visually inspected during reconnaissance and assessed by Ο.Σ.Κ.

ID	School	Location	No. of Stories	Structural Type	Year of Construction	Design Code	Damage (visual observation)
1	"Petritsio" High School **	Lixouri	1-2	RC, Masonry	1959	N/A	Minor-Moderate
2	2nd and 3rd High School	Argostoli	2-3	RC, Masonry	1954, 1959	N/A	Minor-Moderate
3	1st Preliminary School	Lixouri	1	RC, Precast	1959, 1985, 1995	1959	Minor-Moderate
4	2nd Preliminary School	Lixouri	2	RC	1959	1959	Minor-Moderate
5	2nd Preliminary School	Argostoli	2	RC	1959	1959	Minor-Moderate
6	2nd Primary School **	Lixouri	2	RC	1985	1985	Minor-Moderate
7	4th Primary School **	Argostoli	1-3	RC	1959	1995	Minor-Moderate
8	Preliminary School	Keramies	1	Masonry	1959	1959	Minor-Moderate
9	Technological Educational Institute	Argostoli	1	Precast	1995	1995	None

** *multiple buildings*

Figure 11.7.8. "Petritsio" high school, Lixouri (School ID#1, 38°12'12.87"N, 20°26'15.43"E): (a) prior to 2014; after 2nd event: (b) concrete spalling at external beam; (c) beam-column joint failure.

11.8 Churches

INTRODUCTION

The Greek Orthodox Churches of Cephalonia are of the Basilica type, which consists of a longitudinal single nave (Fig. 11.8.1) with windows at both sides. The nave is separated from the chancel by the iconostasis, a timber screen covered with icons of the Eptanisian (Ionian Islands) Baroque era (Fig. 11.8.2). The chancel is positioned at the liturgical east end in the Greek Orthodox churches. Cemeteries are often located in the area surrounding the churches, separated at various levels by stone masonry or reinforced concrete retaining walls. The behavior of cemeteries is extensively covered in Chapter 9 of this report.

Figure 11.8.1. Façade of typical Greek Orthodox Basilica church.

The Cephalonia churches have two types of belfries, both of which were inspected at the locations shown on Fig. 11.8.3: (i) freestanding towers (Fig. 11.8.4), and (ii) belfries incorporated into walls (Fig. 11.8.5), either free standing or connected to the main building.

Figure 11.8.2. Typical church nave and iconostasis.

Reconnaissance work in the churches was led by Prof. Harris Mouzakis of the National Technical University of Athens (NTUA) who performed inspections shortly after the 1st and 2nd events. The NTUA team collaborated with reconnaisance teams from the Institute of Engineering Seismology and Earthquake Engineering (EPPO-ITSAK), 35th Ephorate of Prehistoric and Classical Antiquities (LEEPKA) and Aristotle University of Thessaloniki (AUTH). A total of 49 churches were inspected, mostly concentrated on the western of the Paliki peninsula, as shown on Fig. 11.8.3 with details on Table 11.8.1.

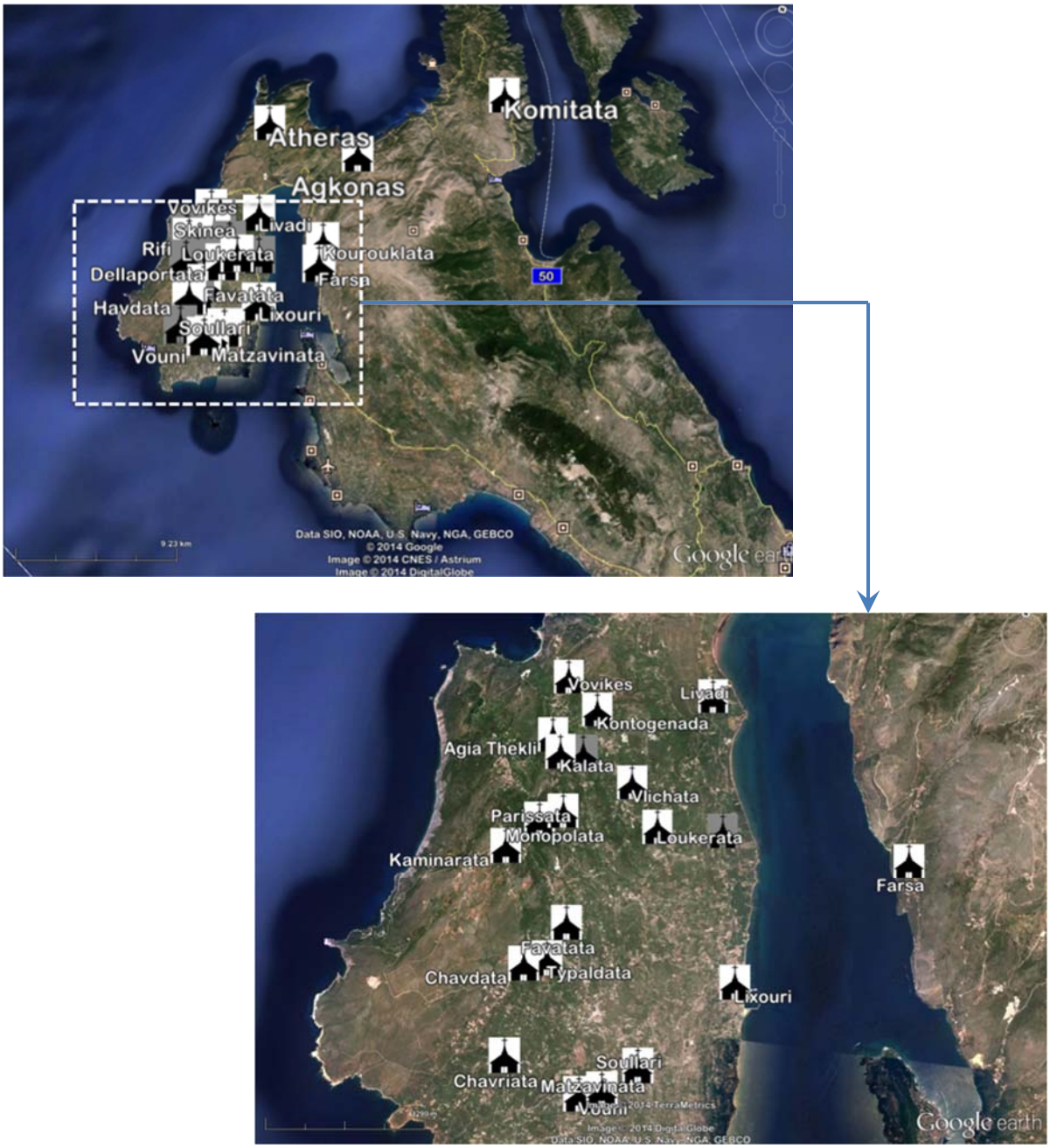

Figure 11.8.3. Church locations inspected during reconnaissance efforts (top) with details in the western part of the Paliki peninsula (bottom).

CHURCH STRUCTURAL CATEGORIES

The churches can be classified in 7 categories based on their Load Bearing (LB) system:

1. Stone masonry dated back to the 12th century. The oldest one is a two-leaf stone masonry with lime mortar. Those structures have been subjected to strong ground motions, including the destructive earthquakes that took place in 1867 and 1953.
2. Stone masonry with internal timber structure that supports the ceiling, built after the 1867 earthquake and which have experienced the 1953 earthquake.
3. Stone masonry built after the 1867 earthquake which have experienced the 1953 earthquake.
4. Stone masonry of Category 3 which was repaired and strengthened with cast in-situ Reinforced Concrete (RC) members (columns, beams and lintels).
5. Stone masonry of category 3 which was repaired and strengthened with internal RC wall in contact with the masonry wall.
6. Reinforced concrete frames with infill walls made of clay or cement units, built after 1953.
7. Fully reinforced concrete structures built after 1953, with concrete strength of 16 MPa (2.3 ksi) and smooth longitudinal and transverse reinforcing bars of 220 MPa (32 ksi) tensile strength.

Figure 11.8.4. Freestanding Tower belfry in Chavriata (38°10'57.47"N, 20°23'14.4"E).

Figure 11.8.5. Belfry incorporated into walls.

CHURCH RECONNAISANCE AND DAMAGE RATING SYSTEM

For the Unreinforced Masonry (URM) Churches (Categories 1 to 3 above), we used the following notations to classify the Degree of Damage (DD):

DDURM0	No damage
DDURM1	Slight damage (hairline cracks in a few walls)
DDURM2	Moderate damage (fall of large pieces of plaster)
DDURM3	Severe damage (large and extensive cracks in walls)
DDURM4	Very severe damage (wall collapses)
DDURM5	Collapse

For the Reinforced Concrete (RC) churches (Categories 4 to 7) the damage was classified as:

DDRC0	No damage
DDRC1	Negligible damage (hairline cracks in columns and beams of frame)
DDRC2	Slight damage (shear cracks in non-structural walls)
DDRC3	Moderate damage (shear cracks in columns and beams and in structural walls)
DDRC4	Severe damage (spalling of concrete cover, buckling of reinforcing rods)
DDRC5	Collapse (collapse of total or parts of building)

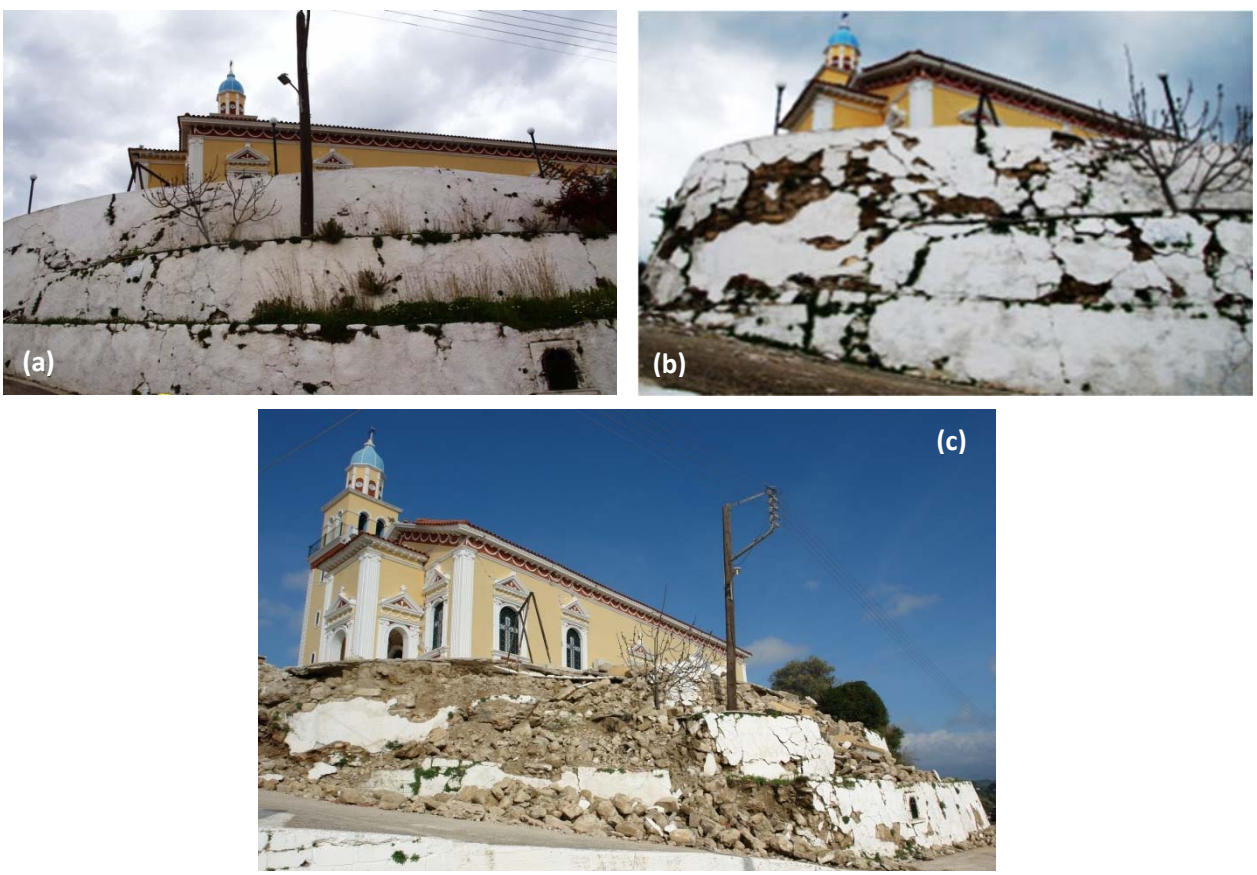

Figure 11.8.6. Collapse of masonry retaining wall at courtyard of The Virgin Mary church at Chavriata (ID#1 38°10'57.47"N, 20°23'14.4"E): (a) condition before the 2014 events; (b) damage after 1st event; (c) collapse after 2nd event.

Table 11.8.1. Cephalonia Orthodox Churches, including name, location, century of construction and repair, Category of Load Bearing (LB) system and Degree of Damage (DD) from the 2014 events.

ID	Name	Location	Century of Construction / Repair	LB Category	DD
1	The Virgin Mary	Chavriata	17 /20	4	DDRC3
2	Apostles	Havdata	19	2	DDURM3
3	Birth of Virgin	Tipaldata	20	6	DDRC2
4	Aghios Ioannis Prodromos	Favatata	19 / 20	5	DDRC3
5	Aghios Athanasios	Favatata	20	6	DDRC3
6	The Virgin Mary	Kechrionas	17 / 20	4	DDRC3
7	Aghia Paraskevi	Monopolata	19 / 20	4	DDRC3
8	Aghios Constantinos	Monopolata	19	2	DDURM4
9	The Virgin Mary of Rongoi	Monopolata	17	1	DDURM3
10	Birth of Virgin Mary	Kominata Kalata	18	1	DDURM4
11	Aghios Nikolaos	Aghia Thekla	19 / 20	4	DDRC5
12	Aghia Thekla	Aghia Thekla	17 / 20	2 - 4	DDRC3
13	Aghios Dimitrios	Kalata	18 / 20	4	DDRC3
14	The Virgin Mary	Skinea	19 / 20	4	DDRC3
15	Aghios Dimitrios	Vlichata	20	7	DDRC3
16	Aghios Panteleimon	Loukerata	20	6	DDRC2
17	Aghios Dionisios	Livadi	20	6	DDRC2
18	Aghios Christoforos	Farsa	20	6	DDRC2
19	Aghios Ioannis	Kourouklata	20	6	DDRC1
20	Aghios Nikolaos	Livathinata	20	6	DDRC2
21	Aghios Dimitrios	Aghios Dimitrios	20	6	DDRC2
22	Aghia Paraskevi	Atheras	19 / 20	4	DDRC3
23	Evangelistria	Atheras	19 / 20	4	DDRC3
24	Aghios Ioannis Theologos	Kontogenata	19 / 20	4	DDRC3
25	The Virgin Mary	Kontogenata	18 / 20	4	DDRC3
26	Aghios Georgios	Kontogenada	12	1	DDURM1
27	Aghios Vasilios	Kontogenada	17	1	DDURM3
28	Aghia Erini	Vovikes	20	6	DDRC0
29	Aghios Dimitrios	Vovikes	20	6	DDRC2
30	The Virgin Mary	Dematora	19 / 20	4	DDRC4
31	Aghios Vlasios (Blaise)	Dematora	18	3	DDURM4
32	Aghios Nikolaos	Rifi	19 / 20	4	DDRC2
33	Aghios Dionisios	Damoulianata	20	6	DDRC2
34	The Virgin Mary	Damoulianata	19 / 20	4	DDRC4
35	Prophet Elias	Kaminarata	20	6	DDRC0
36	The Virgin Mary	Parissata	19 / 20	4	DDRC3
37	The Virgin Mary	Delaportata	20	6	DDRC3
38	Aghios Nikolaos Miniaton	Lixouri	20	6	DDRC2
39	Aghios Spiridon	Lixouri	20	6	DDRC2
40	Aghios Charalampos	Lixouri	19 / 20	4	DDRC3
41	The Virgin Perligou	Lixouri	20	6	DDRC2
42	Pantokrator	Lixouri	20	6	DDRC2
43	Aghia Marina	Soullari	17	1	DDURM4
44	Aghios Nikolaos	Soullari	20	6	DDRC3
45	Aghios Spiridon	Matzavinata	20	6	DDRC2
46	Aghios Dimitrios	Soullari	19 / 20	4	DDRC2
47	Aghia Sophia	Matzavinata	20	6	DDRC2
48	The Virgin Mary	Matzavinata	20	6	DDRC3
49	Aghios Dimitrios	Vouni	20	6	DDRC2

Table 11.8.1 summarizes the name, location, century of construction, and repair, type of load bearing system and the observed degree of damage for the 49 churches visited during reconnaissance. Note Aghios and Aghia stand for Saint, male and female, respectively.

Figure 11.8.7. Typical damage of stone gables in the churches of: (a, b) The Virgin Mary of Rongoi (ID#9) – (a) shows original condition prior to the 1953 earthquakes; (b) The Virgin Mary at Kechrionas (ID#6); and (c) Aghios Ioannis Theologos at Kontogenada (ID#24).

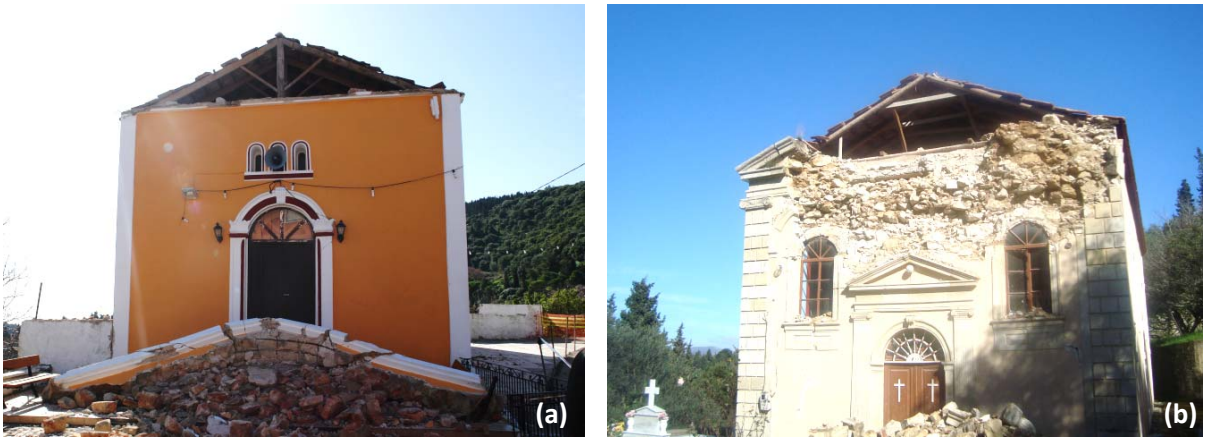

Figure 11.8.8. Collapse of stone gable due to loss of continuity between stone wall and gable through installation of a RC lintel in the churches: (a) Aghios Nikolaos at Aghia Thekli (ID#11); (b) Aghios Vlasios (Blaise) at Dematora (ID#11).

Figure 11.8.9. Typical damage to Category 4 (strengthened masonry) churches to the top part of the walls due to inadequate overlap of the lintel reinforcment.

Figure 11.8.10. Typical damage in Category 5 churches: Collapse of the outer stone leaf of the west gable, at a church strengthened with internal RC wall.

Figure 11.8.11. Typical damage to Category 4 churches at top part of the walls of a strengthened masonry church due to inadequate overlap of the lintel reinforcment.

Figure 11.8.12. Damage in Category 7 church: Permanent out of plane displacement of approximately 7 cm at the top of the walls along the long sides of the structure.

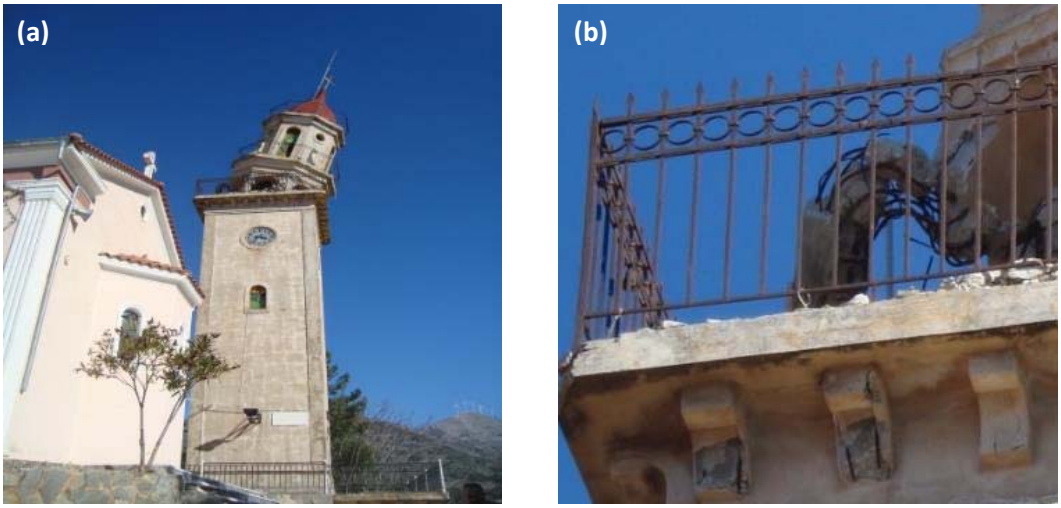

Figure 11.8.13. (a) Collapse of RC belfry top due to column failure (38°14'32.25"N, 20°28'26.40"E); (b) Detail of deformed rebars.

Figure 11.8.14. Category 1 Church of Aghia Marina at Soulari (ID#43): (a) before the earthquakes (b) Collapse of bell tower (c) Partial collapse of the east stone gamble.

Figure 11.8.15. Category 6 church experienced moderate damage of the RC frame infills. The roof tiles located at center of the building were displaced but the ones closer to the edges remained intact.

Figure 11.8.16. Severe nonstructural damage was observed in many churches: (a) fallen objects and out of plain deformation of iconostasis; (b) damage due to falling stones.

SUMMARY OF CHURCH PERFORMANCE

Unlike the residential and public buildings which, in their majority, suffered minor to moderate damage under the two strong earthquake events, the Cephalonia churches exhibited extensive structural damage (even partial collapse), and severe nonstructural damage. This can be attributed to their construction type, retrofit history, and lateral force resisting system of 7 main categories presented in this section. Most of the churches are very old, tracing back to the 17th century, and have accumulated structural strain from the several historic earthquakes in the past, which must have played a significant role in their response to the 2014 event. For example, in many cases in-plane masonry failure initially propagated within the wall following the 1st event and caused out-of-plane structural damage or collapse after the 2nd event.

Church behavior related to the Load Bearing (LB) Category can be summarized as follows for the total of 49 churches inspected by the reconnaissance teams:

- Categories 1 to 3 (stone masonry) suffered significant damage, with five being severely damaged. In many cases the stone gables partially collapsed (Fig 11.8.7 and 11.8.14).
- Category 4 churches with Reinforced Concrete (RC) members added after the 1953 earthquake, performed better than Categories 1 to 3 with slight or moderate damage. In cases where a RC lintel was constructed between the stone wall and the gamble, loss of continuity appeared (Fig. 11.8.9). Damage of RC members due to insufficient reinforcement overlap extended to the nearby masonry walls (Fig. 11.8.11).
- Category 5 churches typically experienced collapse of the outer masonry leaf as they disconnected from the inner one which remained attached to the RC wall (Fig.11.8.10).
- Category 6 churches suffered slight damage with shear cracks in the nonstructural walls.
- Category 7 churches typically experienced out of plane permanent displacement of the wall top along the long sides of the structure (Fig. 11.8.12).

Other general observations of damage include: (i) partial failure or collapse of retaining walls in the church perimeter (Fig. 11.8.6); (ii) complete or partial collapse of many belfries (Figs 11.8.13 and 11.8.14); (iii) moderate roof damage with falling tiles (Fig. 11.8.15) and severe nonstructural components damage (Fig. 11.8.16), which could have caused human injuries or loss of life that fortunately did not take place due to the time the earthquakes hit.

11.9 Nonstructural Components

INTRODUCTION

Damage of nonstructural components was extensive in the Lixouri area and significant in Argostoli. This section presents observations of damage to nonstructural components in buildings that did not have significant damage. These observations were made during reconnaissance efforts in the period from February 8 to 11, 2014, primarily by the practicing engineers of Easy Facilities, GMS, MRCE. The impact of this damage was substantial in the function and the economy of the island. Nonstructural damage caused business to stop operating, including banks, restaurants, and stores, and shut down the only airport in the island for more than 10 days. On the other hand, the local owners of businesses, knowing first hand their exposure to seismic hazard, often had applied empirical, common sense measures that worked well in protecting their safety and valuables.

NONSTRUCTURAL CATEGORIES AND PERFORMANCE

We have grouped the nonstructural components observed during reconnaissance in three categories, as per the commonly used references of FEMA E-74 and ASCE7-05/10:

- Architectural
- Building utility systems (Mechanical, Electrical, and Plumbing)
- Furniture and contents

Table 11.9.1 summarizes the observed damage per nonstructural component Category and indicates the photo that corresponds with the observations. Detailed descriptions of the damage and potential deficiencies that led to the damage follow.

During reconnaissance, both acceleration– and displacement– or drift–sensitive nonstructural components were inspected. Acceleration–sensitive components include suspended ceilings, lights, mechanical equipment, etc. Drift-sensitive components include partitions, façades, etc. Bracing and anchorage to structural elements is important in the behavior of the nonstructural elements. The presence, type, and general conditions of the bracing or anchorage were documented by the reconnaissance teams. Specific observations with possible explanations for selected nonstructural components in each category are presented in the following paragraphs. Special types of nonstructural components such as the

infrastructure network piping systems, and rigid objects such as statues and objects placed in churches and on tombs in cemeteries have been addressed in Chapters 9 and 10 of this report.

Table 11.9.1. Nonstructural components damage observations and construction or design deficiencies by Category of components: architectural, building utilities, furniture and contents.

Category	Type	Component	Figure
Architectural	Egress Means - Stairs	Stairways	11.9.2
		Heavy railings	11.9.1
	Freestanding frames	Fences	11.9.3
	Glazing	Façade and overhead (typically safety glass)	11.9.4
	Interior Partitions	Heavy, usually of bricks, full-height	
		Heavy, usually of bricks, half-height	
		Light, gypsum board	
	Lighting	Recessed light fixtures	11.9.6
		Short polls in public sidewalks	11.9.7
	Suspended Ceilings	Acoustic lay-in tile or gypsum board	11.9.6
		Bracing	11.9.8
	Roofs	Ceramic or plastic tiles	11.9.10, 11.9.11
	Unreinforced Masonry	Parapets	11.9.12
		Chimneys	
	Veneers	Adhered or anchored veneer	11.9.13
		Tile veneers (usually adhered to shear walls)	
Building Utilities (MEP)	Piping	Gutter pipes	11.9.14, 11.9.15
	Lifeline Infrastructure	Potable and wastewater networks	
	Equipment	HVAC	11.9.16, 11.9.17
	Suspended Mechanical Space	Bracing	11.9.6, 11.9.18
	Electrical	Power transformer	11.9.19
	Electrical panels	Distribution panels	
Furniture and Contents	Accessories	Desktop computers, printers, etc.	11.9.20
	Office furniture	Bank teller desks	11.9.21
	Rigid Objects	Heavy bank vaults	11.9.23, 11.9.24, 11.9.25
		Monuments attached to cemeteries	See Ch. 9
		Sculptures	11.9.22
	Storage	Bookcases	11.9.26
		Computer racks	
		File cabinets	11.9.27, 11.9.29
		Objects on shelves	11.9.30, 11.9.31
		Office shelves	
		Restaurant rolling storage units	11.9.32

The following architectural components and potential causes of damage are discussed and illustrated in Figures 11.9.1 through 11.9.9: egress means (stairs); freestanding frames; glazing; interior partitions; lighting; suspended ceilings; unreinforced masonry; and veneers.

Egress Means – Stairs: As the primary means of egress, the performance of stairwells is critical following an earthquake. Observations of damage in stairways often involved lack of rolling supports at either end to accommodate interstory drift. Failure of base anchorage occurred in heavy railings, as shown in Figure 11.9.1.

Figure 11.9.1. Base anchorage failure at heavy stair railings in Lixouri.

Large, heavy contents along egress routes could potentially fall over or block pathways and exits. Figure 11.9.2 shows a series of unbraced filing cabinets and storage units that could tip over and restrict access to the adjacent stairwell (38°10’27.3”N, 20°29’26.4”E).

Figure 11.9.2. Heavy, unbraced storage units with the potential to block means of egress in the National Bank branch of Lixouri. (GPS coordinates: 38°10'27.3"N, 20°29'26.4"E).

Freestanding fences experienced collapse from rotation due to lack of foundations or reinforcement. Figure 11.9.3 shows observed damage to fences not anchored to the ground.

Figure 11.9.3. Freestanding fences without foundations or reinforcement were damaged. (GPS coordinates: (a) 38°11'0.8"N, 20°22'53.9"E, (b) 38°12'41.0"N, 20°26'1.4"E).

Glazing: Façade and overhead glazing, although typically made with safety glass, were not designed for seismic drift. The observed lateral frame deformation was the likely cause of the significant cracking in glazing, as shown on Figure 11.9.4. FEMA (E-74) guidelines for nonstructural components provides recommendations for adding more space around the pane of glass where it is mounted between stops or molding strips in order to accommodate greater frame distortions without cracking the glass.

Figure 11.9.4. Glazing failure due to lack of proper design for lateral frame deformation.

Interior Partitions: Heavy half-height partitions lack lateral bracing to the structure above and were not engineered as cantilevers to the base. Light gypsum board partitions were not isolated from building deformations. Figure 11.9.5 shows a typical method for mitigating damage to partitions by installing diagonal bracing (FEMA).

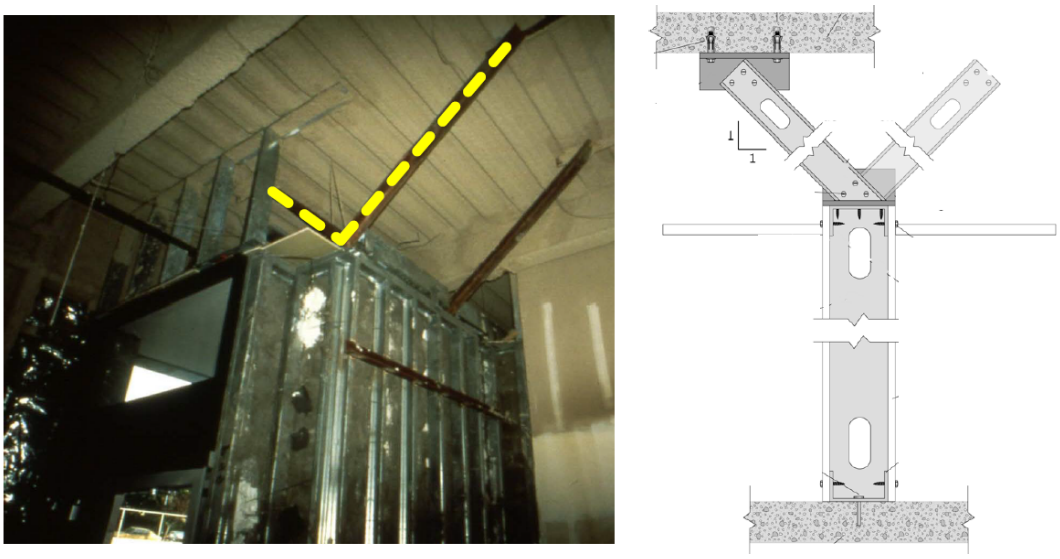

Figure 11.9.5. Mitigation method for bracing interior partitions as recommended (FEMA, E-74).

Lighting: Recessed light fixtures did not have positive support from hanging, or any special safety devices for the attachment of lens covers. Figure 11.9.6 shows damage to recessed lighting in the National Bank branch of Lixouri.

Figure 11.9.6. Lack of support for ceiling tiles and recessed light fixtures in the ground level of the National Bank branch of Lixouri. (GPS coordinates: 38°10'27.3"N, 20°29'26.4"E).

Street lights along public sidewalks are poorly anchored to the pavement. The supporting metal columns lacked sufficient strength to withstand the earthquake movements. Figure 11.9.7 shows damage to two lamp posts that experienced foundation failure and damage midway up the column in the Lixouri and Argostoli port areas.

Figure 11.9.7. Poorly anchored sidewalk lighting columns in: (a) Lixouri port (Easy Facilities team members George Tsalakias and Angeliki Psychogiou in the photo) and (b) Argostoli (GPS coordinates: (a) 38°12'7.2"N, 20°26'18.2"E; (b) 38°12'1.0"N, 20°26'18.2"E).

Suspended Ceilings: There was no bracing of the suspension grid for acoustic lay-in tile or gypsum board ceilings. Figure 11.9.6 shows panels that fell from the ceiling at the National Bank branch of Lixouri (38°10"7.3"N, 20°29'26.4"E), similar to damage recorded at several locations in the meioseismal area.

The bracing system of the mechanical space in the suspended ceiling of Eurobank branch of Argostoli (38°10'29.4"N, 20°29'28.2"E) suffered damage following the 2nd event. The structural system is a moment frame structure erected in 1996 and had visible damage to several nonstructural components. In the second floor mechanical space, a 20 m strut buckled, as shown on Figure 11.9.8 (38°10'29.4"N, 20°29'28.2"E). The strut on the other side of the ceiling bent but did not buckle. Figure 11.9.9 illustrates a mitigation option recommended by FEMA to install diagonal bracing for ceiling supports.

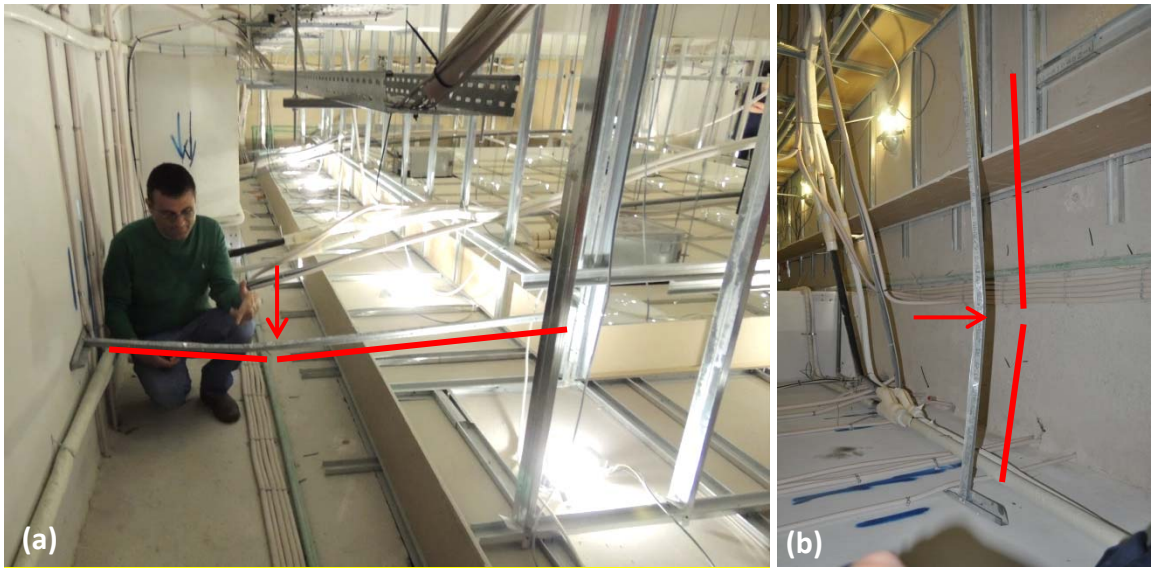

Figure 11.9.8. Ceiling brace buckling in the Eurobank branch of Argostoli: (a) horizontal strut and (b) vertical brace. (GPS coordinates: 38°10'29.4"N, 20°29'28.2"E).

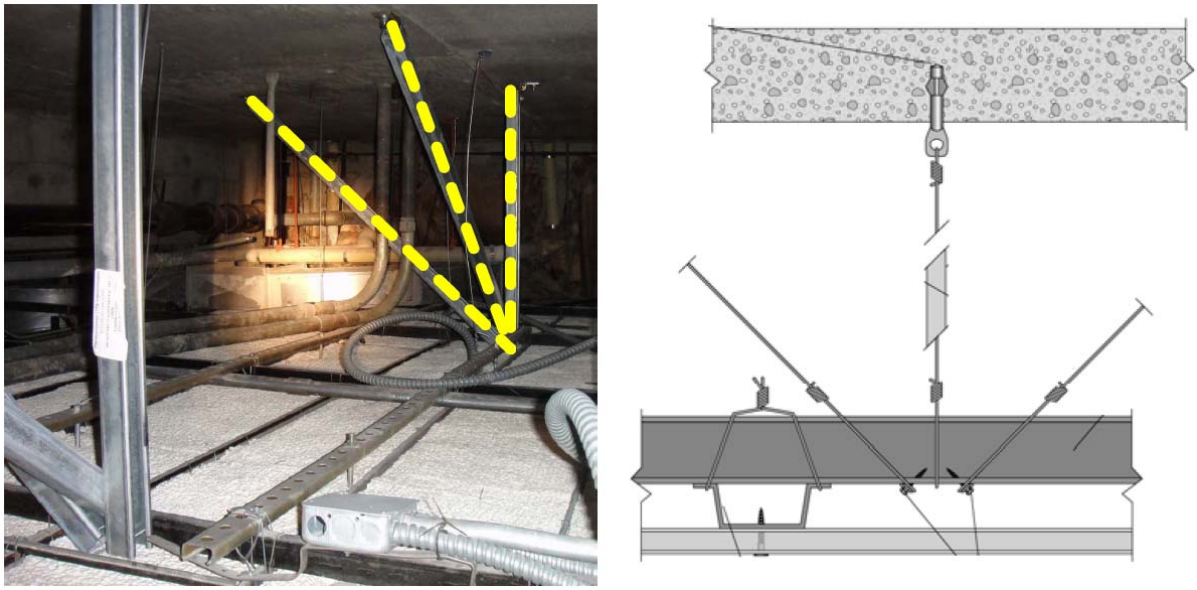

Figure 11.9.9. Mitigation option to provide diagonal bracing for ceiling systems in FEMA E-74.

Roofs: Typical architecture of Cephalonia includes tiled roofs. The tiles are mostly heavy ceramic, with few newer tiles made of plastic that simulate the same look as the ceramic. It was observed in numerous cases inspected in the meioseismal area that tiles fell from the roof during the earthquakes as shown on Figure 11.9.10 (GPS coordinates: (a) 38°13'46.2"N, 20°25'48.0"E; (b) 38°13'45.6"N, 20°25'48.7"E). In most cases, the tiled roof base was not properly anchored to the underlying concrete structure. Figure 11.9.11 shows an extreme case of complete collapse of a tiled roof. Additional evidence of tile damage is presented throughout this Chapter.

Figure 11.9.10. Roof tiles damage (GPS coordinates: (a) 38°13'46.2"N, 20°25'48.0"E; (b) 38°13'45.6"N, 20°25'48.7"E).

Figure 11.9.11. Collapse of tiled roof. Photo by V. Plevris of ASPETE team.

Significant damage to parapets due to lack of bracing and pounding with adjacent buildings during the earthquake events was observed in Lixouri as shown on Figure 11.9.12. Chimneys also typically did not have proper bracing or anchorage to withstand the shaking caused by the earthquakes.

Figure 11.9.12. Typical parapet gable frame failures in Lixouri.

Veneers: Adhered or anchored veneers experienced damage due to the pounding with adjacent buildings. Figure 11.9.13 shows a marble veneer that was damaged from interaction with the adjacent structure. Deformation of backing substrate was observed in tile veneers, usually adhered to shear walls.

Figure 11.9.13. Architectural marble veneer damage due to building pounding.

BUILDING UTILITY SYSTEMS

Observations were made in MEP (Mechanical Electrical Plumbing) building utility systems, namely piping; lifeline infrastructure; equipment; and electrical panels. Detailed observations on lifeline infrastructure of potable and wastewater networks are presented in Chapter 10. Characteristic damage in the remaining systems is illustrated in Figures 11.9.14 through 11.9.20 and discussed below.

Piping: Shown in Figure 11.9.14, gutter pipes separated from the external walls and cracked surrounding supports likely due to inadequate anchorage. At the Lixouri Branch of Piraeus Bank shown in Figure 11.9.14a, it appears that the ground settled and the structure (supported by a grid of 20 steel piles) remained in place, causing the separation in the piping. The reconnaissance teams noted absence of flexible joints that would allow for movement of the pipes independent of the structure and mitigate the damage potential.

Figure 11.9.14. Improperly anchored gutter pipes with rigid connections. (GPS coordinates: (a) 38°12'7.2"N, 20°26'19.8"E; (b) 38°11'0.1"N, 20°22'55.9"E).

At a 2-story Reinforced Concrete (R/C) building located near the shore south of Lixouri (38°11'36.08"N, 20°26'19.90"E), free-field soil settlement was observed in addition to the collapse of its chimneys and roof tiles. Figure 11.9.15 shows a drainage pipe that was dislocated from its collective pool as a result of surrounding ground settlement.

Figure 11.9.15. Dislocated drain pipe at the perimeter of a 2-story R/C building located near the shore south of Lixouri. (GPS coordinates: 38°11'36.08"N, 20°26'19.90"E).

Equipment: Heating, ventilating, and air conditioning (HVAC) equipment was typically not properly braced or anchored. Large air conditioning (AC) units are typically placed on building roofs. When poorly supported by simply sitting on bricks, the AC units displaced or fell over, as shown on Figure 11.9.16 (38°10'27.0"N, 20°29'26.5"E). Figure 11.9.17 shows appropriate bracing and bolting to the exterior wall of HVAC units in Argostoli that suffered no damage. Distribution panels did not have proper anchorage and were often damaged due to failure of the supporting partition walls.

Figure 11.9.16. Rooftop AC unit that fell off brick supports. (GPS coordinates: 38°10'27.0"N, 20°29'26.5"E).

Figure 11.9.17. Mechanical external AC equipment with adequate bracing that suffered no damage in Argostoli.

Suspended Mechanical Space: Both horizontal and vertical bracing elements buckled in few areas of the second floor mechanical space at the Eurobank branch of Argostoli, as shown in Figure 11.9.8 (38°10'29.4"N, 20°29'28.2"E). Figure 11.9.18 shows cable trays in the same mechanical space that were well braced and did not experience damage.

Figure 11.9.18. Well-braced cable trays in second floor mechanical space in the Eurobank branch of Argostoli. (GPS coordinates: 38°10'29.4"N, 20°29'28.2"E).

Transformers: A power transformer pillar in Lixouri (38°11'55.63"N, 20°26'20.56"E) suffered differential settlement (Fig. 11.9.19). One of the support columns appeared to have punched in the ground for 1.7 cm, while the other one remained in place. This behavior could partially be attributed to Soil-Structure Interaction (SSI) effects, as discussed in Chapter 8.

Figure 11.9.19. Differential settlement in power transformer pillar (GPS coordinates: 38°11'55.63"N, 20°26'20.56"E). See also SSI Section of Chapter 8.

FURNITURE AND CONTENTS

Desktop computers, printers, and other accessories were not anchored or tethered. Figure 11.9.20 (38°12'8.4"N, 20°26'17.4"E) shows how many of these items were displaced or fell onto one another.

Figure 11.9.20. Desktop computers and accessories without anchorage or support. Team member Ramon Gilsanz of GMS in the photo (GPS coordinates: 38°12'8.4"N, 20°26'17.4"E).

Heavy office furniture experienced rotational movement due to lack of floor connections. The displacement of a bank teller desk is calculated on Figure 11.9.22 and shows the dimensions of the unit and the direction of movement in relation to its original position.

Figure 11.9.21. Rotational movement of a heavy bank teller desk with no anchorage (GPS coordinates: 38°12'6.6"N, 20°26'17.4"N).

Several cases of rigid body motion (i.e., displacement and/or rotation) were observed in marble statues and other large rigid objects, as detailed in Chapter 9 of this report. For example, two marble statues standing opposite to each other at the entrance of Lixouri City Hall were of particular interest (Fig. 11.9.22, 38°12'3.53"N, 20°26'14.92"E). Following the 1st event, the statue located north of the building entrance (Fig. 11.9.22b) displaced and slightly rotated towards the North without collapsing, since it was supported by the wall behind it. The trace of the original base location is visible, indicating an almost uniaxial displacement of about 40 cm. The top portion of the statue eventually toppled after the 2nd event towards the SE direction (Fig. 11.9.22). A similar response was recorded for the opposite standing statue at the south of the building entrance, which also overturned in the same (SE) direction (Fig. 11.9.22a).

Figure 11.9.22. Overturned statues at the entrance of Lixouri City Hall. (GPS coordinates: 38°12'3.53"N, 20°26'14.92"E).

Rigid body motion was observed at several heavy bank vaults that did not have floor connections or supports. Figure 11.9.23 contains an illustration of how the vault moved as a rigid block at the National Bank (38°12.139'N, 20°20.318'E). Similarly, Figure 11.9.24 shows measured vault rotational displacement, evident from the original marks on the floor prior to the unit moving due to the earthquakes.

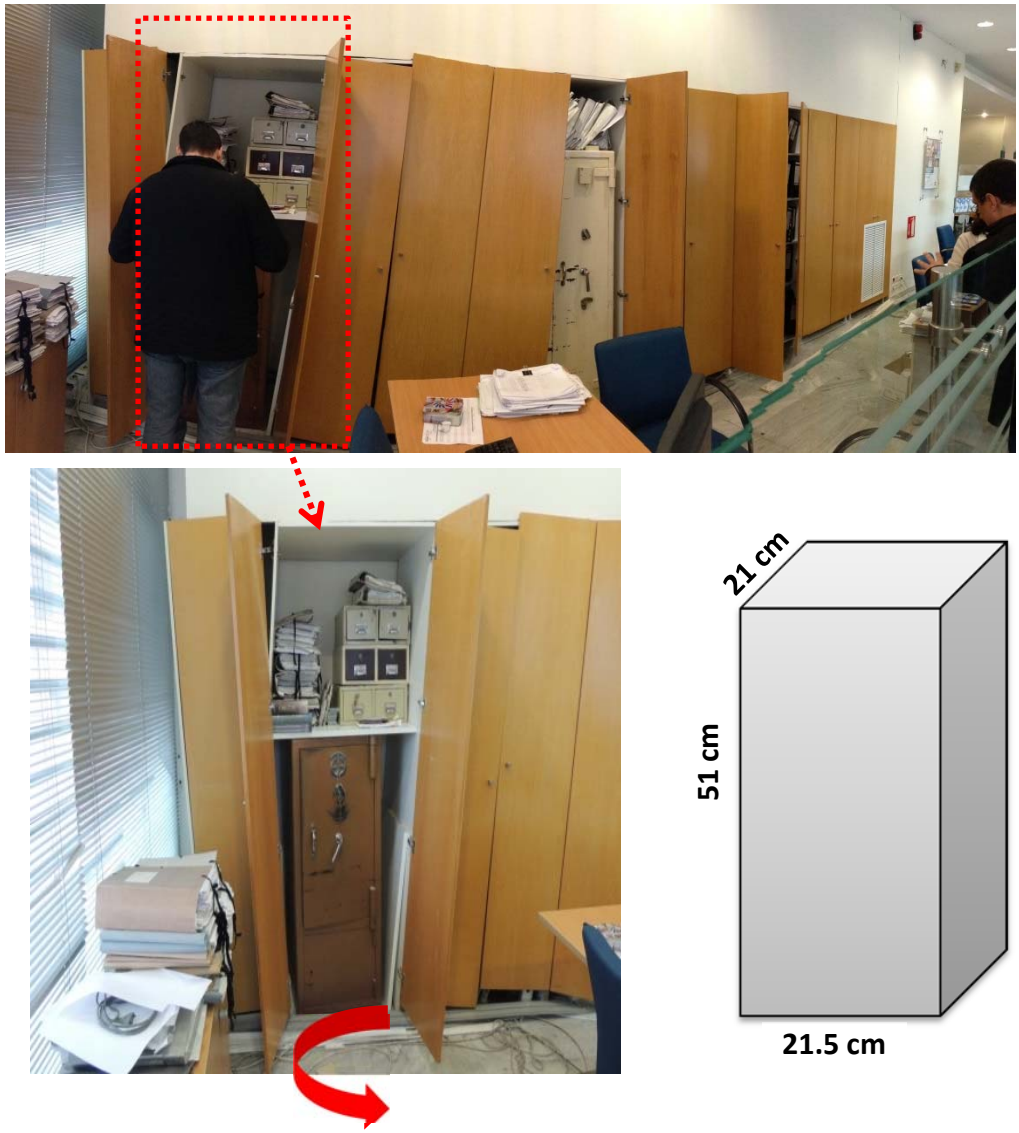

Figure 11.9.23. Sliding and rotation of aheavy bank vault with no anchorage on the floor or ceiling. (GPS coordinates: 38°12'7.8"N, 20°26'19.2"E).

Figure 11.9.24. Rotational movement of heavy bank vault at the Eurobank branch of Argostoli. (GPS coordinates: 38°10'29.4"N, 20°29'28.2"E).

A row of heavy vaults at the Eurobank branch of Argostoli (38°10'29.4"N, 20°29'28.2"E) were not anchored or braced to one another and experienced rotational displacements of approximately 2.5 cm. The dimensions recorded for one of the vaults are given in Figure 11.9.25.

Figure 11.9.25. Measurements taken (by Dimitri Kopanos of Easy Facilities team) at a row of heavy vaults at the Eurobank branch of Argostoli show approximately 2.5 cm of displacement. (GPS coordinates: 38°10'29.4"N, 20°29'28.2"E).

Bookcases were overturned in many locations due to lack of support or connections to adjacent walls. At the National Bank (38°12.139'N, 20°20.318'E), bookcases were laterally braced to one another at the top and did not fall over. Contents fell off the shelves, but as shown in Figure 11.9.26, the units were still standing.

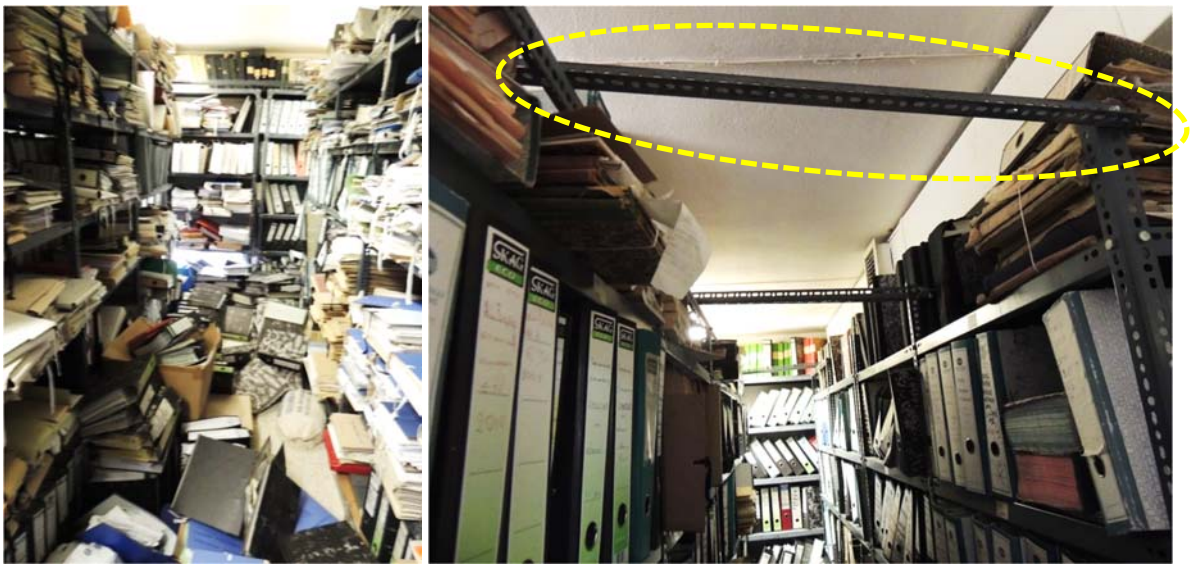

Figure 11.9.26. Bookcases braced to one another for support but lacked shelving restraints. (GPS coordinates: 38°10'27.3"N, 20°29'26.4"E).

Computer racks were typically unanchored and unbraced. File cabinets had notable rotational displacements due to lack of floor or wall supports. Figure 11.9.27 shows measurements for a large file cabinet that rotated as a rigid block at the National Bank branch of Lixouri (38°12'7.8"N, 20°26'19.8"E). This cabinet was against the wall parallel to the shoreline, as shown from the exterior in Fig. 11.9.28. Many cabinet doors and drawers did not have latches to prevent them from opening with their contents spilling during the earthquakes. Collected data for this structure relating to rigid blocks is also presented in Chapter 9 of this report.

Following the 1st event on January 26th, the National Bank branch of Lixouri (38°12'7.2"N, 20°26'19.8"E) installed corner hooks to the back of large cabinets for wall support. As shown in Figure 11.9.29, this simple non-engineered solution prevented significant damage during the 2nd earthquake event of February 3rd.

Figure 11.9.27. File cabinet with no anchorage (GPS coordinates: 38°12'7.8"N, 20°26'19.2"E).

Figure 11.9.28. National Bank branch of Lixouri located in the front row of buildings parallel to the shoreline (GPS coordinates: 38°12'7.8"N, 20°26'19.8"E). See also Chapter 9 for this case study.

Figure 11.9.29. Corner hooks installed after the 1st event at top of large storage units and bookcases at the National Bank branch of Lixouri prevented movements in the 2nd event. (GPS coordinates: 38°12'7.2"N, 20°26'19.8"E).

Objects on shelves fell and broke in many instances because items were not secured properly. Common sense solutions by the owners were adequate for most of these cases to prevent significant damage in restaurants and stores. Figures 11.9.30 and 11.9.31 show such examples in the areas that experienced the high levels of accelerations in Havdata village and Lixouri port, respectively.

Figure 11.9.30. Simple rod restraints installed by a restaurant owner (shown here with team members) at Havdata village prevented toppling of bottles. (GPS coordinates: 38°12'14.3"N, 20°23'12.0"E).

Figure 11.9.31. Shelving with simple string restraints at a gas station and supermarket in Lixouri.

Rolling storage racks in restaurants did not have seismic stoppers or brakes that may have reduced damage during the earthquakes. Figure 11.9.32 shows overturned racks that displaced contents and broke dishes and glassware.

Figure 11.9.32. Lack of seismic protection for rolling restaurant shelves and refrigerator units in Lixouri.

There is one (international) airport on the Cephalonia island that remained closed for more than 10 days following the earthquakes mainly due to non-structural damage since no significant structural damage was observed. Operations were limited with security checks done manually in open spaces near the runways and public buses used as waiting areas. Figure 11.9.34 shows the interior conditions of the airport and Figure 11.9.35 shows the exterior of the structure (38°7'10.0"N, 20°30'17.7"E).

Figure 11.9.33. Cephalonia International Airport (38°7'10.0"N, 20°30'17.7"E).

Figure 11.9.34. Interior conditions of the airport. (GPS coordinates: 38°7'10.0"N, 20°30'17.7"E).

Figure 11.9.35. Exterior of the airport, no significant structural damage was recorded. (GPS coordinates: 38°7’10.0”N, 20°30’17.7”E)

CONCLUSIONS

Nonstructural components experienced extensive damage in the Lixouri area and also in Argostoli. This damage made a significant impact on the operations and economy of the island. Businesses could not operate, including banks, restaurants, and stores, and the only airport on the island was shut down for more than 10 days. The damage shown in the photos could have a greater effect on the economy of the island and future tourism.

In some cases, local business owners knew from personal experience that their risk and exposure to seismic hazard was high, and applied empirical, common sense solutions that successfully protected their safety and valuables. Falling objects could have caused significant injuries and even loss of life, considering that the earthquakes occurred at times when public areas were likely to be active and occupied. Specifically, the 1st event occurred at 5:55 pm on a Sunday (January 26) followed by the 2nd event on February 3 at 5:08 am).

Most of the observed damage has been identified in the FEMA guidelines where suggested measures are recommended. As the reconnaissance team members observed, and as is the case in most countries, many of the nonstructural components were installed by non-engineers after construction without consideration of earthquake hazard although the local code has seismic provisions and prescribes calculations for expected deformations and loads depending on the type, weight, and location (in floor height) of nonstructural components.

A common misperception is that building utilities and critical systems are heavy enough to withstand earthquake shaking, but in reality these nonstructural components can cause significant damage from pounding against adjacent objects or falling over, especially under near-field pulse-like motions of high accelerations as in the Cephalonia events. It is critical to increase public awareness of the risks and hazards of nonstructural systems in earthquakes.

The observed damage following the 2014 Cephalonia earthquakes highlights the importance of seismic design of nonstructural components for life safety and post-earthquake operations and the need to increase educational efforts in this direction. The detailed reconnaissance information presented in this report for both acceleration- and displacement-sensitive nonstructural components, together with several recorded strong ground motions in the immediate vicinity of the collected information, presents an excellent opportunity to enhance our knowledge on the behavior of nonstructural components and develop simple, engineering and common-sense solutions to minimize their seismic risk exposure.