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Seismic response of traditional buildings of Lefkas Island, Greece

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Abstract

This paper deals with the seismic response of the traditional buildings of Lefkas (also called 'Lefkada') Island in western Greece. These structures, with two up to four storeys, are met mainly in the old town of Lefkas, the capital of the island. The special structural characteristic of the traditional buildings of Lefkas (that is the issue of this article) is the dual load-carrying system used on ground floor level to handle vertical loads and seismic actions. The first main load-carrying system consists of single-storey stone masonry walls, while the second main load-carrying system consists of multistorey wooden frames at the inner perimeter of the ground-floor masonry walls. Therefore, the load-bearing system of the upper storeys, over the ground floor, consists of a wooden 3D frame with diagonal trusses that are infilled by walls of bricks and lime mortar. There is, also, a tiled wooden roof. During the earthquake of August 14th, 2003 ($M = 6.2$), partial collapse of the masonry walls took place in some cases at the low storey and some cracks around the openings were observed. In the upper floors, the load-bearing wooden frames did not suffer damage, but the brick infill cracked and out-of-plane falls were observed. In this paper, the analytical investigation of the seismic behaviour of such structures is developed, the identification of the main way of vibration of these systems is achieved, and both the explanation of the observed damages and the interpretation of the reasons for the development of these damages are performed. Finally, the results of a representative numerical model of a typical traditional building—from a series of examined models—with dual bearing system are presented and discussed, using as base excitation the two horizontal recorded components of the main earthquake ground motion that took place on August 14th, 2003, at Lefkas town, Greece.

Keywords: Lefkas earthquake; Traditional buildings; Structural damages

1. Introduction

Lefkas is one of the Ionian Islands at the western side of Greece with an area of 302.5 km^2. The capital of the island is the homonymous town of Lefkas, located at the North/East side, only 50 m from mainland Greece, and it is connected by a pontoon bridge (Fig. 1a, b). The area of the Ionian Islands is characterized by the highest seismicity in Greece. Thus, Lefkas has a long seismicity history. During the last two centuries many catastrophic earthquakes have occurred and serious damages were observed to traditional and new structures [1].

On August 14, 2003 at 08:15 local time (05:15 GMT) a strong earthquake of magnitude $M = 6.2$ occurred in the Ionian Sea. The epicentre of the earthquake was located very close to the North/West side of Lefkas Island (Fig. 1) [2]. The earthquake was strongly felt in the rest of the Ionian Islands (Kephalonia, Zakynthos, Ithaki, etc.) and in a large area of mainland Greece. The strong motion of this earthquake was recorded by many accelerographs of the Institute of Engineering Seismology and Earthquake Engineering (ITSAK) network, which had been installed in the surrounding area before the earthquake [3]. The maximum peak horizontal ground acceleration (PGA) was recorded at the centre of Lefkas town and it reached 0.42 of gravity acceleration ($0.42g$). In the transverse horizontal and vertical directions the maximum peak ground accelerations (PGAs) reached $0.34g$ and $0.19g$, respectively [3,4]. The long duration of the strong motion of the event (about 18 s), which was combined with the high PGA values, established this earthquake as one of the most intense ever recorded

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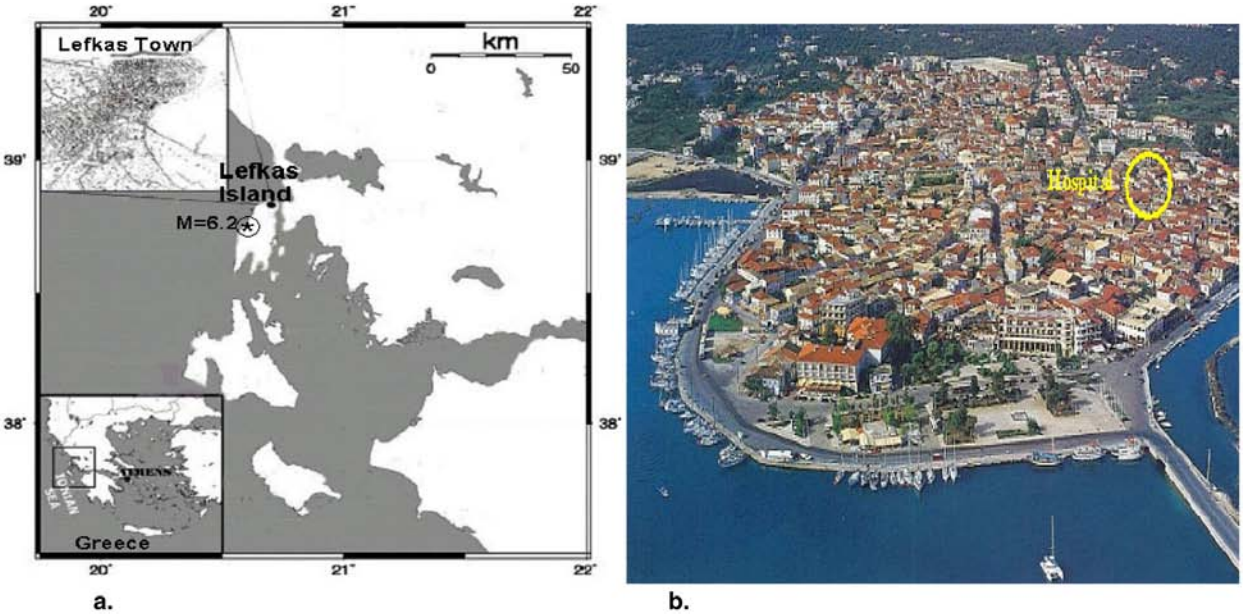

Fig. 1. a: The island of Lefkas. The epicentre of the mainshock on August 14th, 2003, is denoted by the star [3]. b: A panoramic view of the town of Lefkas. The position (Hospital) of the recorded strong motion is denoted by circle.

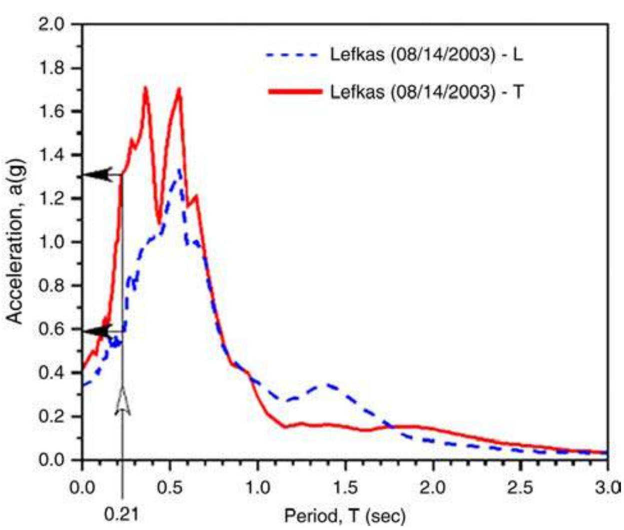

Fig. 2. Spectra of accelerations of two horizontal seismic components (L, T) of the Lefkas earthquake for viscous damping $\zeta = 0.05$.

in Greece [3]. Response spectra for 5% damping of the horizontal components of main shock have been given in previous articles [3,4]. The acceleration response spectra exhibited amplitudes $Sa > 0.9g$ for a period range from 0.2 to 0.7 s and these reveal a peak value of about $1.7g$ around 0.35 and 0.55 s (Fig. 2). In addition, we can say, practically, amplification of the accelerations values does not appear for periods less than 0.05 s, while the spectral accelerations for periods more than 1.0 s are lower than the peak ground accelerations in two horizontal directions.

The town of Lefkas consists of the historic centre district, which has older traditional buildings (up to 200 years old) and soft soil conditions. In the surrounding area the more recent Bei and Neapoli districts exist, with typically modern r/c buildings of better quality, founded on better soil conditions (Fig. 1b). The whole town of Lefkas has an estimated number of 2100 buildings. From an in situ investigation on approximately 10% of the total building stock of Lefkas town the following distribution of building categories was recorded: Ratio 6% masonry buildings, 15% wooden buildings, 34% traditional buildings with dual structural system (as explained below and examined in this paper), while the remaining 45% of the total building stock consists of r/c buildings. A two-level inspection of 2100 buildings of the town of Lefkas resulted in 1435 without damages (68.34%), 315 “green” (15.02%), 337 “yellow” (16.02) and 13 “red” (0.62%). The classification, referred to the degree of damage (“red”, “yellow”, “green”), is briefly described as follows [4]:

“Green”: Original seismic capacity has not been decreased, the buildings are immediately usable and entry is unlimited.

“Yellow”: Buildings in this category have decreased seismic capacity and should be repaired. Usage is not permitted on a continuous basis.

“Red”: Buildings in this category are unsafe and entry is prohibited. Decision for demolition will be taken on the basis of more thorough inspection.

As mentioned above, the traditional building category, which is examined in this article, represents 34% of the total building stock of the town and it exists in the historic centre. As mentioned by Prof. Touliatos, a member of many European Committees for wooden and traditional buildings, at the 2nd Hellenic Conference on “Appropriate Interventions for the Safequarding of

Monuments and Historical Buildings” [5], this category concerns one of the three special designed seismic systems in Europe and is acknowledged by the European Scientific Community [6–9]. The three special seismic systems, which are the most elaborate and completed seismic structural systems, are the following: (a) the traditional building with dual structural system of Lefkas in Greece, (b) the well-known “Casa Baraccata” in Italy and (c) the traditional building of Portugal that had been constructed after the big earthquake in Portugal in the 18th century. The architectural morphology, the construction materials of these structures, the dual structural bearing system, as well as the observed damages from previous strong earthquakes were investigated by other researchers in the past [6–9]. The dual structural system of these structures is described in the following sections, parallel to the damages observed due to the earthquake of August 14th, 2003. Moreover, the analytical investigation of the seismic behaviour of such structures is developed, the identification of the main way of vibration of these systems is achieved, and both the explanation of the observed damages and the interpretation of the reasons for the development of these damages are performed.

The modelling of the masonry walls of the ground floor has been achieved using suitable shell elements. Different shell elements are used for the infill masonry walls of the upper floors, while for modelling of the wooden frames beam elements are used. Also, suitable failure criteria for masonry walls are adopted. In this paper, using as base seismic excitation the two seismic recorded components of the main seismic shock in the town of Lefkas during the earthquake of August 14th, 2003 [3,4], a series of numerical models of such traditional buildings with dual bearing system have been examined. Qualitative conclusions as well as quantitative results of analyses that have arisen from the investigation of the above series of numerical models are presented and discussed. Also, these results and conclusions contribute to the qualitative comprehension of the seismic response of these buildings, which possess similar seismic structural bearing systems that also occurred in many other countries.

2. Structural system of the investigated traditional buildings

2.1. General description

In this paper, the traditional buildings of Lefkas town, with usually two or three storeys, found mainly in the centre of the old town (Fig. 3a), are examined. The special characteristic of seismic design of these traditional buildings is the dual load-carrying system (Fig. 3b). The first main load-carrying system consists of a wooden multistorey 3D-frame. The wooden 3D multistorey frame system, on the ground floor, is enveloped by stone masonry walls (Fig. 3b, c) and it is connected on the top of the ground floor and on the perimeter of the building with the second load-carrying system that consists of a single-storey stone masonry wall. On the upper floors, the wooden 3D frame possesses diagonal wooden trusses. Single bricks with lime mortar fill the wooden trusses. The majority of these buildings are resident houses and the rest of them are public buildings such as schools, the old Town Hall, etc. Despite the age of these buildings (many of them have been existing for more than 200 years), almost all of them are in use nowadays. The differences between resident houses and public buildings are the plan dimensions and the quality of the materials, especially those related to the strength of the masonry walls at the ground floor (type and shape of stones and quality of lime mortars).

The most common plan dimensions of the resident houses, usually with two or three storeys that have height of 2.8–3.0 m, are 4.0–5.0 m along the horizontal x-axis and 7.0–15 m along the normal y-axis. The external format of these traditional buildings is usually orthogonal. The width of stone masonry walls at the ground floor fluctuates from 0.5 to 0.7 m and at the upper storeys the width of brick infill masonry is 0.10–0.15 m. Only a few openings appear to the external walls, usually in symmetric positions both in the plan and the elevation. However, there are significant differences in the distribution of the mass in elevation of the structure, because in these buildings the mass of the ground floor is 5 to 6 times larger than the mass of one typical upper floor. Also, the mass is distributed continuously on the vertical walls of the building and therefore it is not concentrated at the geometric centre of the plan. This means that the equivalent static method of seismic design is not suitable for use, because the mass of each floor does not concentrate at the geometric centre of the plan of the building.

2.2. Structural characteristics

Poor soil conditions and the high level of underground water characterize the centre of Lefkas town. So, the traditional builders applied a specific type of foundation for these buildings (Fig. 4). The same or similar types of foundation occur in many other places in Greece with similar soil conditions (e.g. for the foundations of stone buildings and stone bridges) [10,11]. According to the bibliography and observations which took place during repairing or reconstructing works in this area the depth of the footings is estimated to be approximately 1.0 m and the width up to 1.5 m. The lower level of the footings that is usually covered by underground water is built on a sub-foundation of horizontally placed tree trunks in three levels (Fig. 4).

The traditional builders have applied special techniques for the preparation of the tree trunks in order to protect them from organic destruction [10]. At the same time the setting of tree trunks offers the technicians enough good conditions to work despite the poor soil conditions and the high underground water level. However, the more important role of this part of foundation, from the structural point of

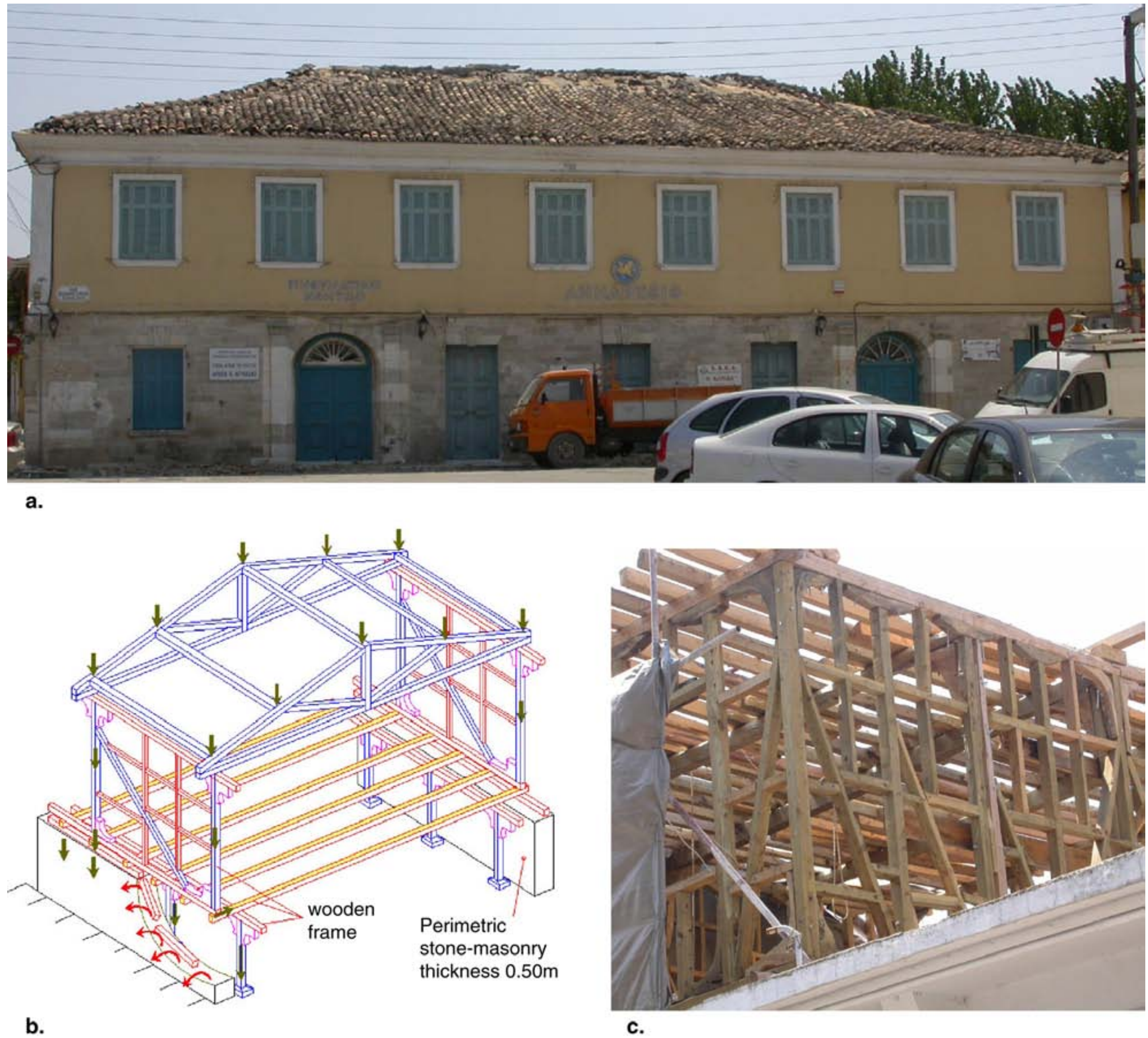

Fig. 3. a: Traditional building (old Town Hall) of the town of Lefkas (Greece) with dual bearing system. Large percentage of tiles at the wooden roof detached during the Lefkas earthquake (14/08/2003). b: The dual bearing system of traditional buildings in Lefkas. c: The wooden 3D frame of the upper storey.

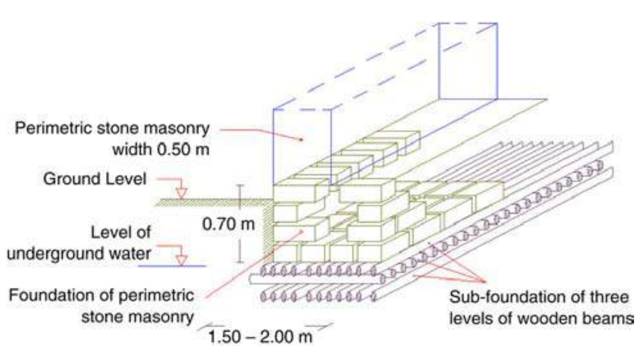

Fig. 4. Details of the foundation of traditional buildings of the town of Lefkas (Greece).

view, is its contribution in avoiding the differential settlings of the footings usually occurring in structures that are founded on poor soil conditions. The traditional builders, over the sub-foundation of tree trunks, construct stone masonry foundation utilizing lime mortars with pouzolan giving a high strength to the foundation and moisture protection to the walls of the ground floor level. Also, this sub-foundation of three levels of horizontal wooden beams resembles (approximately) an ancient (primitive) seismic base isolation system, without a ditch in the perimeter of the building. In order to examine whether this sub-foundation offers a real seismic base-isolation role (if it exists and if it is activated) further investigation must be performed. According to Naeim and Kelly [12], similar sub-foundation systems play the role of the seismic base-isolation system and had been used in other countries by 1870, such as in Tokyo (Japan) [12]. In Lefkas town of Greece, an extended and systematic use of that sub-foundation system was taking place by 1800 (nowadays in Lefkas town there are several existing traditional buildings, which were constructed by 1800).

As mentioned above, one load-carrying system at the ground level consists of stone masonry walls, while the other load-carrying system consists of a 3D wooden frame (from cypress trees) at the inner perimeter of the stone masonry walls (Fig. 5a). The main concept of this structural system is that in the case that the stone masonry walls fail due to seismic actions (e.g. heavy cracking and/or partial collapse leading to reduced load bearing capacity), the wooden 3D frame becomes more active in order to carry more and more permanent loads of the upper floor (Fig. 3b). Two wooden beams lie at the top level of the external walls connecting them at the corners. Transverse wooden beams lie over the previous beams carrying the loads from the upper floor(s) connecting the opposite walls and increasing the capacity of the structure in case of out-of-plane motion. These transverse beams, that are also supported by the beam of the redundant wooden frame, transfer part of the dead loads to the wooden columns (Fig. 3b). In some cases the wooden columns are free standing at the ground floor and in other cases they are fixed on it. According to the bibliography [10, 11], it is clear that the traditional builders have established and used two methodologies for the construction of the ground floor. In the first methodology, they constructed stone masonry walls at the ground floor level and after that they continued with the multistorey 3D wooden frame (from cypress trees). In the second methodology, the builders started the construction with the multistorey 3D wooden frame and afterwards they continued with stone masonry walls.

In any case, following a numerical analysis of the dead loads of the structure, it is found that the wooden columns support a part of the upper floor(s) dead loads, fluctuated from 5% (in the first methodology) to 85% (in the second methodology), related to the methodology of construction. It is clear that after the first earthquake the distribution of the dead loads changed, due to stone masonry walls and 3D wooden frame interaction.

The load-bearing system of the upper floors, over the ground floor, consists of wooden frames with diagonal trusses and tiled roofs (Figs. 3c, 5b). Main emphasis was given to the stability of the wooden frame (from cypress trees) utilizing wooden corner (from olive trees) parts increasing the capacity of the wooden element joint connections (Fig. 5c). The majority of these wooden frame structures appear to have been completed with brick infill (Fig. 5d). Usually, the internal and external sides of the walls of upper storeys are covered with plastered limes. Moreover, the external sides of the external walls are typically cladded with zinc sheets for rain protection (Fig. 3a).

The diligence of the foundation construction, and the dual bearing system at the ground floor, combined with the relatively low mass of the upper storeys, exhibit a remarkably reduced vulnerability of this type of structures to earthquake actions. The damages that occurred after the August 14th, 2003, earthquake with a brief interpretation of the reasons to the occurrence of these damages accompanied with an analytical investigation of the seismic behaviour of such structures are presented in the next section.

3. Observed damages after earthquake

Despite the long strong duration of the earthquake event of August 14th, 2003, (estimated at about 18 s) and the high peak ground accelerations values, the traditional buildings with dual structural system, such as those examined in this paper, behaved in a rather satisfactory way. Although there was a case of a three-storey reinforced concrete building with total collapse after the earthquake, not one case of a traditional building developed similar behaviour [4]. In addition, the serious damages observed in modern structures, and especially to infrastructures by foundation settlings [3], were not observed in any traditional buildings, despite the poor soil conditions of the old town district of Lefkas town, where the majority of such type of buildings exists. This was possible due to the use of the extended wooden footings as sub-foundation, which was described in the previous section, and the relatively small mass of these buildings. A notably common—albeit nonstructural—damage was met in all types of buildings with tiled roofs. A high percentage of tiles (Fig. 3a) on wooden roofs was detached due to the spectral amplification of the acceleration of the vertical seismic component. Indeed, the PGA of the vertical seismic component of ground accelerations recorded was $0.19g$ and its maximum spectral acceleration was $1.10g$ (because, for vertical vibrations with small vertical amplitudes of wooden roofs, the equivalent viscous damping is less than $\zeta = 1.5\%$, Fig. 6). The value of spectral vertical accelerations $1.10g$, which is greater than g, justifies totally the damages of the tiles (in fact when the spectral accelerations are larger than 0.60–$0.70g$ then tiles detach due to the slope of the wooden roof). The structural damages of the traditional buildings observed after the earthquake can be classified in two main categories, related to the structural system: (a) damages to the stone masonry walls of the ground floor and (b) damages to the wooden frame with brick masonry infill on the upper storeys.

3.1. *Damages to the stone masonry walls of the ground floor*

As was expected, the damages of the masonry walls occurred very close to the corner of the ground floor or very close to the openings where a concentration of stresses can be developed by horizontal seismic actions. The observed damages can be classified in two types:

(a) Cracking at the masonry stones or at the joint of mortars. This can be explained due to the high level of stressing or due to the low quality (or capacity) of the masonry work such as the poor and non-diligent connection of stones between old and new parts of the wall (Fig. 7a).

Fig. 5. a: Inside view of the ground floor of a traditional building. b: The wooden frame of the upper storey. c: Detail of the wooden joint elements. d: Inside view of the upper storey. The wooden frame with masonry infill.

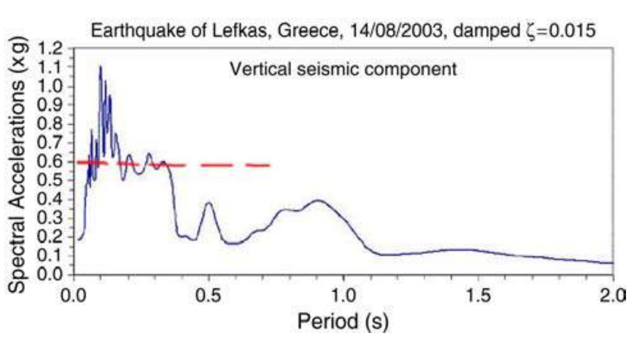

Fig. 6. Spectrum of accelerations of vertical seismic component of the Lefkas earthquake.

(b) Partial collapse of the masonry walls (Fig. 7b). This type of failure took place in a few cases and the structural stability of the building was ensured by the activation of the other structural seismic bearing system, namely the wooden 3D frame. From the in situ investigation, this type of damage was observed in a limited number of buildings and it was due to the old age and poor maintenance as well as the low quality of the masonry materials (stones and mortars) determining the very low strength of the masonry wall.

3.2. Damages to the wooden frame with brick masonry infill on the upper storeys

In the upper floors, the load-bearing wooden 3D frame did not appear to be damaged. In contrast, damages to the brick masonry infill were observed, including cracks and out-of-plane falling of wooden frames. These damages were difficult to be noticed from outside, since the external walls of the upper storeys are typically cladded with zinc sheets

Fig. 7. a: Cracking of masonry walls at the ground floor. b: Local failure of masonry walls. Vertical loads from upper floor carried by the wooden frame on the ground floor.

(for rain protection). Damages could thus be observed only on the interior face of the walls, which are usually plastered with lime. An interior investigation has shown that these damages developed at the middle of the height of the upper storeys, very close to the openings and the corners of the storeys (Fig. 8a). The observed damages on brick masonry infill can be classified in four types:

(a) Shear failure at the interface between plastered lime and bricks. Usually, this type of failure was followed by cracks and sometimes by falling of the plastered lime (Fig. 8b).
(b) Shear failure at the interface between bricks and wooden frame. This type of failure can be observed both during in-plane and out-of-plane response of the wooden frame (Fig. 8c).
(c) Crushing of the brick infill. This type of failure can be developed under diagonal compression and usually occurred at the corners of the wooden frame (Fig. 8d, f).
(d) Total collapse and out-of-plane falling of the brick infill. This type of failure follows the previous types and it is easier to be developed during the out-of-plane response (Fig. 8e, f).

4. Numerical investigation

In the frame of the present paper, a suitable analytical investigation of the seismic behaviour of such traditional buildings is performed in order to explain the observed damages and interpret the reasons for failures, which were developed during the Lefkas earthquake on August 14th, 2003. The recorded time histories of the acceleration along the two horizontal directions at the centre of Lefkas town are used as base excitations for the analysis. Next, the numerical results of representative numerical models of two/three-storey traditional buildings are presented and discussed. From the in situ investigation in Lefkas town the special characteristics (dimensions, used materials, type and strength of masonries, existing conditions, etc.) of the traditional buildings are recorded. The numerical models, which are examined in this paper, possess similar (as mean values) special characteristics as the above in situ investigation (Table 1). So, it has to be clear that the numerical models that were examined in this paper do not represent real structures but their geometrical and other characteristics that describe many typical resident buildings of such traditional structures in the town of Lefkas. Therefore, for the purpose of the present paper twelve (12) different models (6 two-storey & 6 three-storey models) of traditional buildings have been analyzed. Six (3 two-storey & 3 three-storey) of the 12 models are used with reference to cases where the wooden columns of the ground level take over 5% of the dead loads of upper floors and the other six (3 two-storey & 3 three-storey) are used with reference to cases where the wooden columns of the ground level take over 85% of the dead loads of upper floors. Also, 9 different directions (0°, 30°, 45°, 60°, 90°, 120°, 135°, 150° and 180°) of the two horizontal seismic components, as base excitations, have been assumed in order to build an envelope of stresses. In addition, two different conditions of the foundation are examined. In the first condition suitable dynamic springs were used such as those used in a previous relevant paper [4]; these were the results of a geotechnical soil study using test drillings at Lefkas town. These springs are used under the foundations of the examined numerical models. In the second condition,

Fig. 8. a: Cracking of masonry walls at the ground floor. b: Failure of the plastered lime at the upper floor. c: Shear failure at the interface of wooden elements and masonry infill. d, f: Crushing of masonry infill at the corners of the wooden frame. e, f: Out-of-plane collapse of masonry infill.

Table 1
Mechanical properties of the materials (mean values)

Material	Young's modulus (kN/m^2)	Poisson ratio	Mass density (t/m^3)	Compressive strength (MPa)	Tensile strength (MPa)
Wood	9000 000	0.30	0.5	25	46.5
Stone masonry	4325 000	0.15	2.7	4	0.4
Clay brick infill walls	1708 000	0.15	2.1	4	0.4

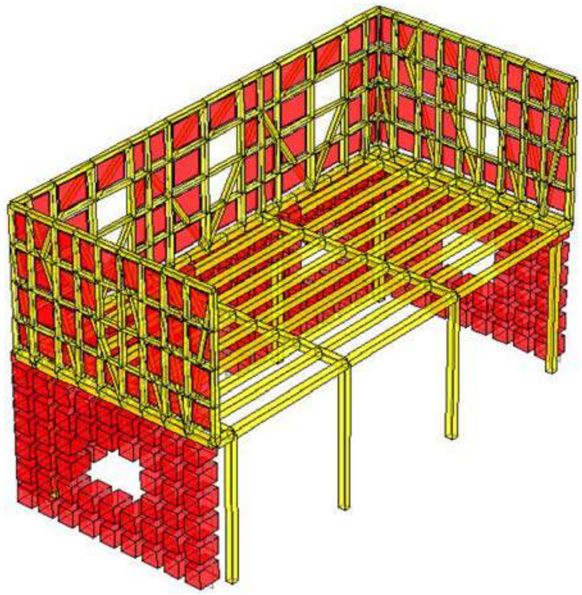

Fig. 9. An open view of the 3D finite element numerical model (façade elements not shown).

fixed foundations are considered on numerical models in order to write the relative deviations. So, 216 (12 models $\times 9$ different directions $\times 2$ cases of foundation conditions) dynamic linear time history analyses have been performed. All these models have common geometrical characteristics and material properties as explained below. Finally, from the series of numerical models (that correspond with a series of traditional buildings that are examined) an indicative two-storey numerical model is presented.

4.1. Geometry and properties of materials of the numerical model

The plan dimensions of the investigated two-storey numerical model are 5 m $\times$ 10 m in both storeys and the height of each storey is 3.0 m. The width of the stone masonry walls at the ground floor is 0.5 m and the width of the brick masonry walls at the upper floor is 0.10 m. These dimensions form a rectangular box similar to the shape of a typical real structure. The openings at the ground and the upper floor are in symmetrical positions with dimensions similar to those of real structures (1.0 $\times$ 1.0 m for the windows and 1.0 $\times$ 2.20 m for the entrance door). The diaphragmatic operation, around the vertical axis, of the wooden truss roof of the building is considered in the examined models. Also, geometric parameters such as the distances between the vertical elements of the wooden frame at the upper floor, the position of the diagonal wooden trusses and the positions of the wooden columns at the ground floor are applied to the models. All these parameters are applied to the series of numerical models following the practical rules as established by the traditional builders [6–11].

For the mechanical properties of the materials involved, the values listed in Table 1 were used for Young's modulus E, Poisson ratio υ and mass density ρ, based on existing literature and past experience of the authors. It is well known that the compressive strength of the masonry is related to the corresponding strength of the stones or bricks and the lime mortars. From the in situ investigation it was found that the masonry walls of traditional buildings in Lefkas town consist of various types of stones and quality of mortars, which determine a large variation of the compressive strength of the real structures. Therefore, for the main requirements of this investigation, the values of the compressive and tensile strength listed in Table 1 are adopted as mean representative values of the existing structures. However, it is well known that the adopted values of the compressive and tensile strength do not influence the numerical results of the analysis but are used in order to introduce the failure criteria (see Section 4.4 below).

4.2. Simulation and dynamic properties of the numerical model

In traditional buildings of this type, the largest portion of the mass is distributed continuously on the exterior walls and the ground storey has 5 to 6 times the mass of a typical upper storey. This mass distribution differs from that of a reinforced concrete building, where most of the mass is found on the floor slabs. Therefore, shell elements are used for modelling the stone masonry walls (width 0.50 m) of the ground floor and other shell elements are used for modelling the brick masonry infill walls (width 0.10 m) of the upper floor. All the wooden elements (wooden frame of the upper floor, wooden columns at the ground floor, wooden diagonal trusses and the horizontal wooden beams at the top of the stone masonry walls of the ground floor) are simulated with beam elements. The stone masonry walls and the wooden columns of the ground floor are assumed to be fixed at the ground level or with suitable ground springs, which were used successfully in a previous relevant investigation [4]. An open view of this numerical model is shown in Fig. 9.

From post-earthquake observations in the majority of cases, due to strong ground motion of the Lefkas earthquake (2003), it appears that the dual structural system of these traditional buildings responded into the linear and elastic area. Indeed, wooden frames remained in the elastic area, while damages were observed on the brick masonry infill walls only of the upper floor. However, it is worth noting that these infill masonry walls are secondary elements, which do not belong to the main dual bearing system. In very few cases, damages were observed on small parts of the stone masonry walls of the ground floor, while in the majority of traditional buildings, the stone masonry walls of the ground floor remained in the elastic area, as well. So, the most suitable method of analysis is the linear dynamic time-history analysis, using the corrected recorded accelerogramms (time-histories) of the Lefkas

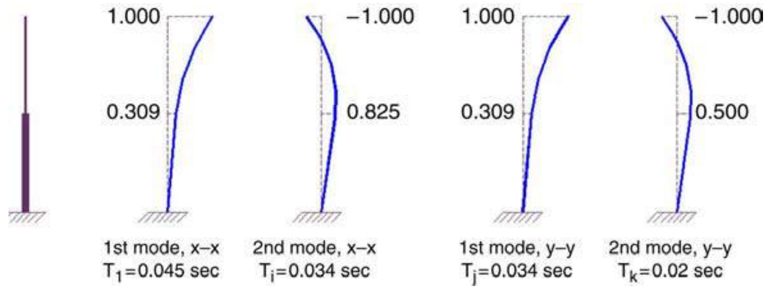

Fig. 10. Shapes of the first and second global modes along the x–x and y–y directions.

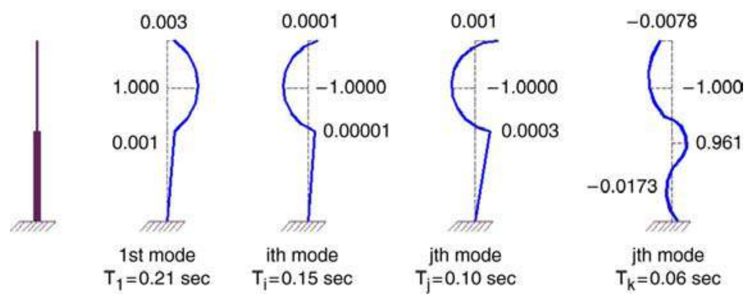

Fig. 11. Local modes of wooden elements at the upper floor (out-of-plane).

earthquake [3,4]. Also, for the investigation of the dynamic characteristics of a traditional building the most suitable method is modal analysis. Therefore, for the analysis of the two/three-storey numerical models, the well-known general structural analysis program SAP2000 [13] was used. At first, a modal analysis of each numerical model was developed in order to investigate the predominant shape modes of the response. The very important findings of that analysis can be classified in two categories of shape modes: global and local modes. In the present model, the global shape modes describe the response of it as a whole structure and refer to the translational shape modes in two perpendicular directions (x-axis and y-axis) (Fig. 10) as well as the torsional shape mode (around the z-axis).

The local modes describe the response of specific elements of the upper floor wooden frame that can be excited independently of the global modes (Fig. 11). It has to be underlined that a large number of the wooden elements are excited and all these local modes develop an out-of-plane response. At the same time, it is also very interesting that the corresponding periods of the global modes (along the x-axis, along the y-axis and around the z-axis) have very small values and by comparing with the response spectrum of the main shock, all these values located in a range of periods practically without spectra amplification (less than 0.01–0.02 s for fixed foundation and 0.05–0.10 s for spring foundation). In contrast to that, the corresponding period values of a large number of local modes are located in a range of periods with significant spectral amplification (about 0.10–0.20 s for fixed foundation and about 0.20–0.40 s for spring foundation) (Fig. 2). It is clear that during the August 14th, 2003, earthquake, the local modes with out-of-plane response had a significant contribution to the seismic response of such structures and can be accepted as one of the main reasons for the observed damages.

4.3. Displacement response

Time history analyses are also performed, using the acceleration recordings in two horizontal directions of the main shock in the town of Lefkas. It is known that the critical loading orientation of two horizontal seismic components does not exist [14]. So, nine different base excitation directions of 0°, 30°, 45°, 60°, 90°, 120°, 135°, 150° and 180° are assumed, in order to build an envelope of displacement response and the possible maximum stress states of the models. The relative displacements response to the base of the structure is investigated in three stages.

In the first stage, main emphasis is given to the determination of the structural model areas where the maximum values of the displacements take place. An envelope of these values is presented in Fig. 12 utilizing the numerical results from the nine different base excitation directions. As can be seen from this figure, maximum displacement concentrations are observed in mid-height of the infill walls of the upper floor, agreeing with the positions of the observed damages from the in situ investigation (Fig. 8). The results included in that envelope figure come from both in-plane and out-of-plane responses of the walls in both storeys. Therefore, one of main issues of the present paper is the investigation of the response of masonry walls with in-plane and out-of-plane vibrations.

Table 2
Maximum displacement response along the height of the model

Position along the height	Description of the position	Maximum displacement (m)		Ratio
		In-plane (m)	Out-of-plane (m)	Out-of-plane/In-plane
1	Top of the ground floor	0.00007286	0.0001659	2.28
2	Middle of the upper floor	0.0001039	0.02146	206.54
3	Top of the upper floor	0.0001432	0.0004341	3.03

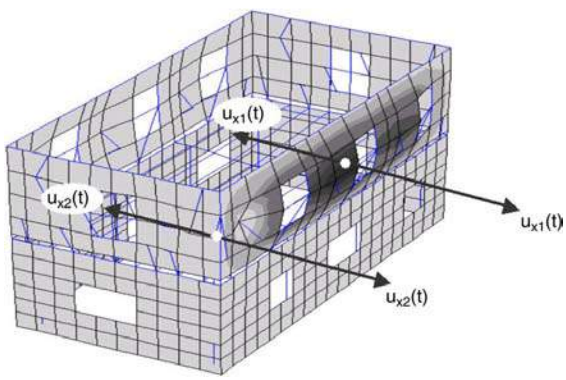

Fig. 12. Distribution of the maximum relative displacement response. Contours lines of displacements u_x (out-of-plane) over 0.001 m. Concentration of out-of-plane-displacements at the middle of the height of the upper storey.

In the second stage, the distribution of the displacement response along the height of the structural model during time excitation is investigated. For this reason, a vertical cross section of the model is examined and the displacements of three specific points along the height of that section are recorded (Fig. 13). The first point (no. 1) corresponds to the top level of the ground floor, the second one (no. 2) to the mid-height of the upper floor and the third one (no. 3) to the top level of the upper floor. The investigation of this subject is based on the numerical results from an analysis with the acceleration base excitation of two simultaneously horizontal directions parallel to the main axis of the model (x-axis and y-axis). The values of maximum displacement response of the three investigated points across the height of the examined model, as described above, are listed in Table 2. In the last column of this table the ratio of the out-of-plane maximum response to the corresponding in-plane response is given. As can be seen from this table the maximum displacement response occurred at the mid-height of the upper storey due to the out-of-plane response.

In addition to that and according to the last column of Table 2, it can be seen that, at any point across the height of the structure, the out-of-plane response is much higher than the in-plane response. On the other hand, in both cases (in-plane and out-of plane-response) the maximum displacement at the top of the upper floor is much higher than those at the top of the ground floor, confirming the large flexibility of the upper floor, consisting of a wooden frame with brick masonry infill, than that of the ground massive masonry structure.

The distribution of the out-of-plane response of the examined vertical cross section of the model at different time steps during base excitation is shown in Fig. 13. As is seen from this figure and according to the conclusions from the modal analysis of the examined model (explained in the previous paragraph) the out-of-plane response on the upper floor is governed by the local modes.

In the third stage, the distribution of the displacement response in a horizontal cross section at the level of the mid-height of the upper floor is examined. The investigation is limited to the out-of-plane response in two different positions of the cross section ($u_{x1}(t)$ and $u_{x2}(t)$ along the x-direction) (Fig. 12). The first one corresponds to a position very close to an opening (window) and at a significant distance from the corner of the structure ($u_{x1}(t)$) of position (1). The second one is exactly at the corner of the structure ($u_{x2}(t)$) of position (2). It is clear that the out-of-plane displacement response along the x-direction of the second position is equivalent to the in-plane response of the transverse wall (along the y-direction). The time history of the displacement response of the two different positions is shown in Fig. 14. By comparing the response of these positions, it can be said that the out-of-plane displacement response increases with the distance from the corner of the structure. In the presented model, at position 1 (close to the window), the maximum $u_{x1}(t)$ displacement is 82.7 times larger than the maximum $u_{x2}(t)$ displacement at the corner of the structure (position 2).

4.4. State of stress and failure criteria of masonry

In Fig. 15, the envelope of the tensile stresses in two principal directions (s11 and s22) are presented, with contours indicating stress levels $>$ 0.4 MPa (equal to the mean tensile strength adopted in this paper). Tensile stress concentrations are observed in mid-height of the infill walls of the upper floor agreeing with the positions of the observed damages from the in situ observations (Fig. 8). At the same time the wooden frames remain in the elastic range, presenting no structural failure, a fact also confirmed from in situ investigations. It has been mentioned that the observed structural damages on this type of buildings from the in situ investigation were caused on the ground floor stone masonry walls and the brick masonry infill of the upper storeys. Therefore, several failure criteria of masonry are required in order to describe the observed damages, utilizing the numerical results of this study. Since

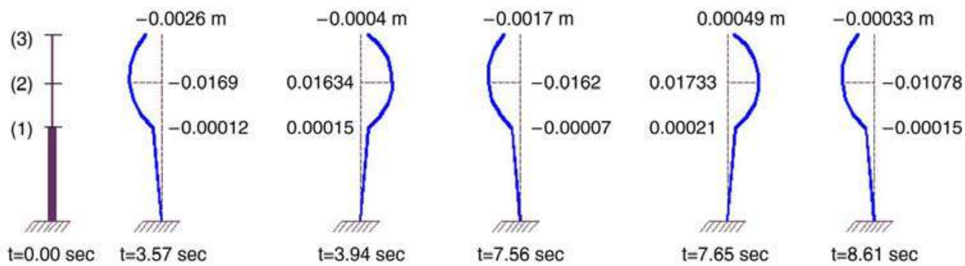

Fig. 13. Out-of-plane displacement shapes of a cross section along the height of the model at different time steps.

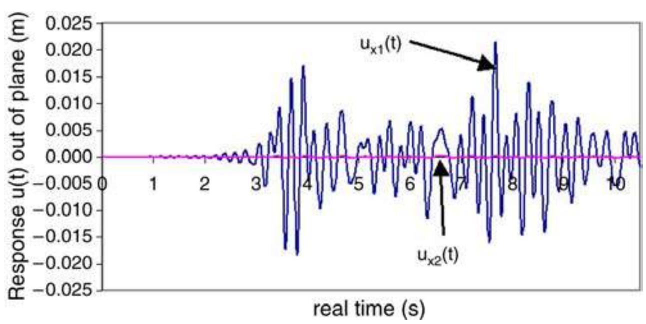

Fig. 14. Time history of out-of-plane displacement response of two different positions in the horizontal section of the upper storey (see Fig. 12).

both the stone masonry wall and the brick masonry infill are two-dimensional structural elements, conventional (two-dimensional) failure criteria have to be adopted. According to the literature, the shear stress—compressive stress (τ–s) and the stresses in two principal directions (s11–s22) are the most common two-dimensional states of stress, which are compared with corresponding failure criteria for masonries.

In addition, the stone masonry walls and the brick masonry infill consist of stones, bricks and lime mortar. Since stones, bricks and lime mortars are brittle materials with similar properties to concrete, conventional concrete failure criteria can be adopted to model failure to the stone masonry walls or brick masonry infill [15,16]. Although there are various failure criteria according to the bibliography, for the purposes of this paper, a simple von Mises failure criterion is adopted, as shown in Fig. 16, based on stresses in two principal directions (s11–s22) [17]. The parameters needed for the definition of the surface shown in Fig. 16 are $f_c = 4000\,\mathrm{kN/m^2}$ (compression strength) and $f_t = 400\,\mathrm{kN/m^2}$ (tension ultimate strength), namely f_t assumed equal to $0.10\,f_c$. The stress state in two principal directions at any position of the structural model is compared with the failure envelope described in this figure and if it exceeds the envelope than the following types of failure are accepted:

Cracking of masonry in both principal stress directions (s11 and s22) occurs when the state of stress is of the biaxial tension–tension type and both of the tensile principal stresses are beyond the tensile-failure envelope, which is designated as zone z1 in Fig. 16. In this situation the material loses its tensile strength completely. Cracking of masonry in one direction occurs when the state of stresses is of the tension–compression type and a principal stressing s11 or s22 direction exceeds the limiting value prescribed by the tensile failure surface (see zone z2 or z3 in Fig. 16). In this case, the material loses its tensile strength in the direction parallel to s11 or s22, respectively. Crushing of the masonry occurs when the state of stress is biaxial compression–compression and the stress level is beyond the simplified von Mises failure surface, shown as zone z4 in Fig. 16. Under this condition, the material also loses its strength completely.

For a further investigation in order to explain typical types of failure, the stress state of many positions of the model, in two principal directions, as it is developed during a time history analyses is compared with the adopted failure envelope. The investigation of this subject is based on the numerical results from analyses with the acceleration base excitation in two horizontal directions parallel to the main axes of the model (x-axis and y-axis), with application in three cases.

In the first case, the stress in two principal directions of the more stressed area of the ground floor stone masonry walls is compared with the failure envelope as shown in Fig. 17. As can been seen from this figure, failure can be developed in the case of very small compression, much lower than those values of strength that are adopted in this paper. This is true, because the tensile stresses are often exceeding the tensile strength of the masonry wall (especially when a part of stone masonry wall is practically unloaded of gravity loads as in the case when the wooden column of the ground floor take over 85% of the dead load of upper floors). The above conclusion was verified numerically at the different positions of the high tensile stress in the upper storeys of the examined models the above conclusion is verified numerically. It was already mentioned above from the in situ investigation that the observed damages on stone masonry walls of the ground floor (diagonal cracks or out-of-plane collapse) occurred in the cases of masonries with very poor materials, which exhibit very small compressive and tensile strength. This observation is in good agreement with the conclusion of the interpretation of the previous numerical results.

In the second case, the numerical results of stressing in two principal directions at the interface between the wooden frame and brick masonry infill of the upper storey

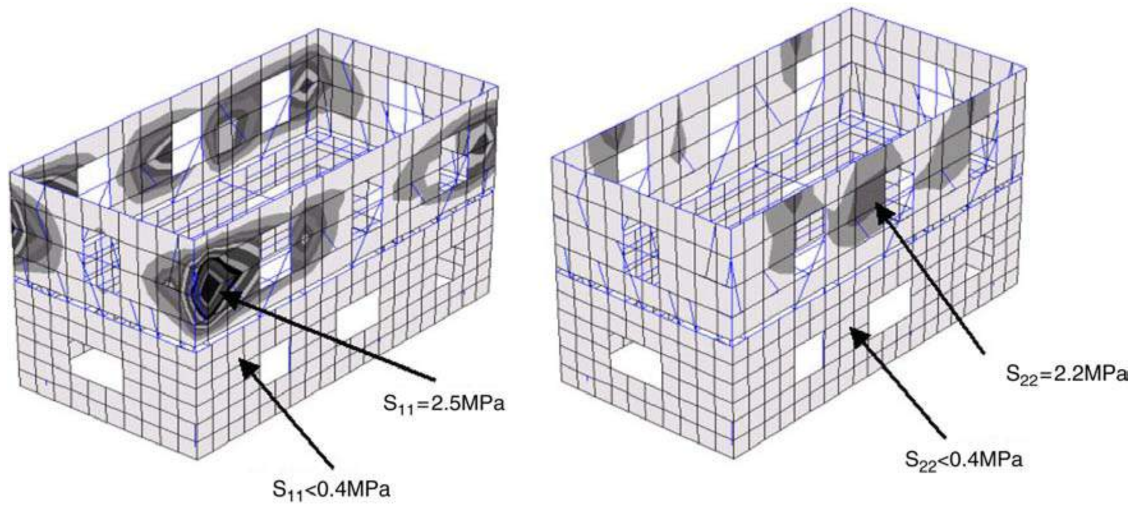

Fig. 15. Distribution of the maximum normal tensile stresses—Concentration at the middle of the height of the upper storey.

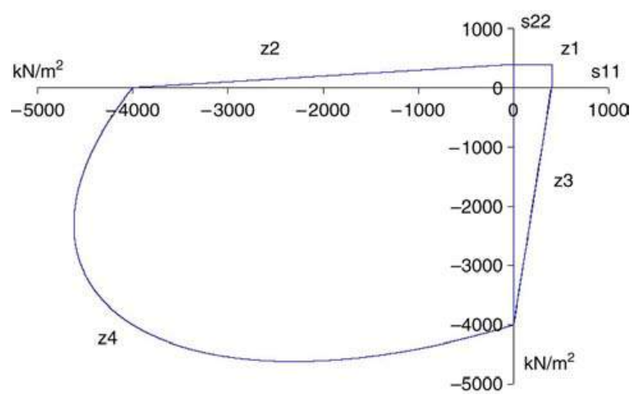

Fig. 16. Envelope of adopted failure criterion based on von Mises [17].

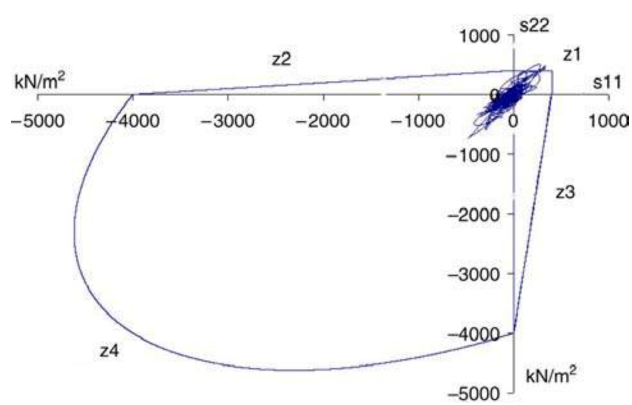

Fig. 17. State of stress of the more stressed area at the masonry walls of the first storey.

are compared to the failure envelope (Fig. 18). It is clear that for long times of base excitation the state of stress is biaxial tension–tension and at some times it exceeds the tensile-failure envelope. This situation leads to cracking at the interface in both principal stress directions (s11 and s22), even in cases with higher tensile strength than that adopted

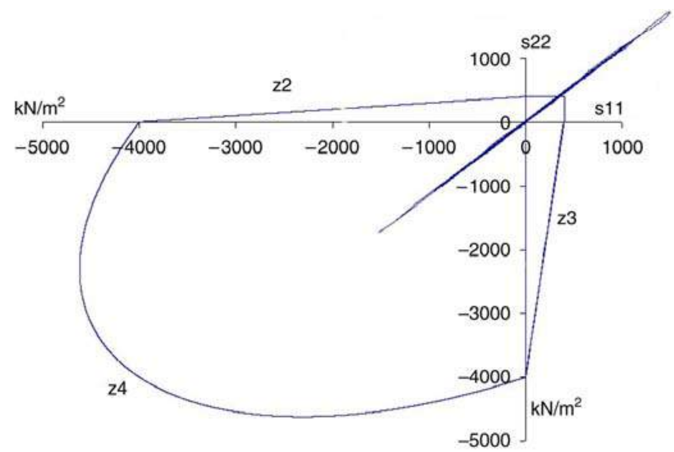

Fig. 18. State of stress at the interface between masony infill and wooden element at the upper storey—shear failure.

in this paper ($f_t = 0.10 f_c$). There are several such positions near the ends of the masonry wall of the upper storey. Indeed, as can be seen from Fig. 18, the stresses in two principal directions have almost the same value, and the loops possess very small width, which define a shear failure at the interface between the wooden frame and the masonry infill, exactly as the type of failure observed in situ (Fig. 8c).

In the third case, the possibility of crushing of masonry infill is investigated. A state of stress as biaxial compression–compression followed by crushing of masonry infill can be developed at the corners of the wooden frame elements where the material may lose its strength completely (Fig. 8d, e, f). For this reason, several of these positions at the mid-height of the upper storey have been examined. As can be seen from Fig. 19, crushing of the masonry infill can be developed in the case with small values of compressive strength, much lower than that adopted in this paper ($f_c = 4.0\,\text{MPa}$).

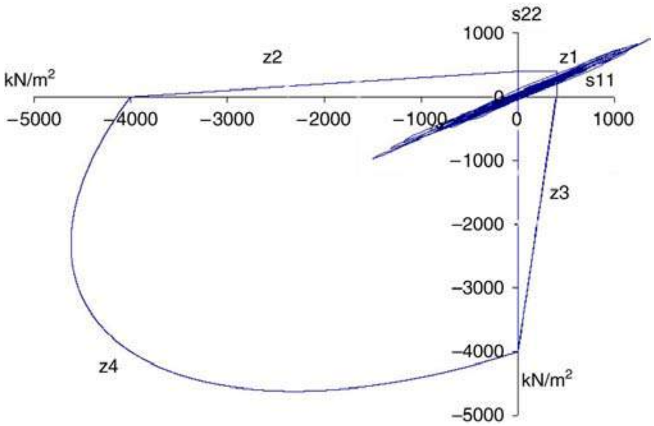

Fig. 19. State of stress of masonry infill at the corner of wooden elements at the upper storey.

As a final conclusion from the previous investigation, we can say that cracking at the masonry walls of the first floor and crushing at the masonry infill of the upper storey can be developed in the case of masonry with small values of compressive strength, which is in good agreement with the in situ observations. On the other hand, the shear failure at the interface of masonry infill and wooden elements of the upper storey can be developed even for masonry with higher tensile strength than that adopted in this paper. It is clear that, after these failures, the resistance of the masonries to the out-of-plane response is reduced and the occurrence of the out-of-plane collapse is more likely to occur, as observed from the in situ investigation (Fig. 8e, f).

5. Discussion and conclusions

a. The island of Lefkas is characterized by the highest seismicity in Greece. Due to the frequent shaking, the local population takes special care in the quality of workmanship in buildings. This experience led to the creation of traditional buildings in the area with a dual (stone masonry and wooden frame) system to handle seismic actions at the ground floor and wooden frame with brick masonry infill on the upper floors.
b. Despite the long strong duration of the earthquake event of August 14th, 2003, and the high peak ground acceleration values ($0.42g$), the traditional buildings with a dual structural system, such as those examined in this paper, behaved in a rather satisfactory way.
c. The structural damages of the traditional buildings observed after this earthquake can be classified in two main categories, related to the structural system: cracking and partial collapse of the ground floor stone masonry walls (in a few cases) and shear failures, crushing and out-of-plane collapse of the upper storey brick masonry infill (in most cases). The wooden frame, both in the ground and upper floor, did not appear to be damaged. Serious damages were observed in modern structures and especially to those with foundation settlings; no such damages were observed in the traditional buildings, despite the poor soil conditions at the old town district of Lefkas, where the majority of such type of buildings exist, because wooden footings of three levels under the stone foundation have been used, extendedly.
d. The observed damages to the brick masonry infill of the upper floors were systematically concentrated around the middle of the height of the floors and were usually followed by out-of-plane collapse.
e. According to modal analyses of representative numerical models of such structures, it can be said that many local modes excite the elements of the wooden frame out-of-plane. Also, the contribution of these modes to the out-of-plane response of the upper storeys has been set off.
f. According to the numerical results of the numerical investigation, the maximum displacement and state of tensile stress wer concentrated around the middle of the height of the upper storey, and this is a good verification and explanation of the in situ observations.
g. By comparing the numerical results of the state of stress with an adopted simple failure criterion that is based on stresses in two principal directions, the following important findings have been underlined: cracking at the stone masonry walls of the ground floor and crushing of the brick masonry infill of the upper storeys can be developed in cases of masonry with small values of compressive strength, and this is a good verification and explanation of the in situ observations. On the other hand, the shear failure at the interface between brick masonry infill and wooden elements of the upper storeys can occur even if the masonry has a higher tensile strength than that adopted in this paper. It is clear that, after these failures, the resistance of the brick masonries to the out-of-plane response is reduced and the occurrence of the out-of-plane collapse is more likely to take place, as observed from the in situ investigation.
h. The single-storey masonry by stones and mortars 3D-system (the first bearing system) has an uncoupling fundamental period of about 0.015–0.02 s for fixed foundation, while the wooden multistorey 3D-system (the second bearing system) has an uncoupling fundamental period of about 0.20–0.23 s for fixed foundation; the difference is first order of magnitude. In addition, the two mode shapes of the first bearing system are characterized as global mode shapes, while the first two mode shapes of the second bearing system are characterized as local mode shapes (vibration of infilled walls out of the their plane). The coupling of the two bearing systems appears to be a very smart idea and an important issue that technicians and expert builders of Lefkas Island have been using in the last 200 years, giving to a traditional building multiplex lines of capacity during earthquake events, and the damages have appeared and have been concentrated on the secondary elements, such as the infilled walls.

i. The previous conclusions were based on the one hand on the in situ observations after the Lefkas earthquake of (first methodology) and on the other hand on the results of numerical investigation (second methodology) using as base excitation the two horizontal recorded seismic components of the main shock in the town of Lefkas during the earthquake of August 14th, 2003. Both methodologies agree absolutely, because all types of appeared failures have been interpreted. A future analytical investigation with various types of base excitations as well as the seismic role of the sub-foundation of wooden footing of three levels will give more general and useful conclusions of the seismic response of such traditional structures.

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