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Seismic behaviour of the historical structural system of the island of Lefkada, Greece

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Abstract

The detailed survey of damages observed in numerous historical houses of the city of Lefkada, Greece, after the strong August 2003 earthquake, as well as the parameter analysis carried out within the present work allowed for the structural characteristics of this historical system (conceived to resist earthquake) to be identified and evaluated. The pathology of the structural system was analytically verified, and indicative intervention schemes were analytically examined.

Keywords: Lefkada island; Historical structural system; Earthquake; Vulnerability; Stone masonry; Timber framed masonry; Pathology; Parameter analyses

1. Introduction

The island of Lefkada (Fig. 1), one of the Ionian Islands, is situated in the most earthquake prone region of Greece. A local structural system was developed before the 19th century in Lefkada. The strong earthquake which occurred in 1821, proved the adequacy of the system to sustain seismic actions. Thus, the British Authorities (ruling the Ionian Islands at that time) imposed rules for the construction of new houses following the main characteristics of this local structural system. These rules, further developed and completed, constituted the Code for Construction, issued in 1827. That Code provided guidance on the selection of building materials, on the thickness of stone masonry in the ground floor, as well as on the maximum storey height. In addition, a minimum distance is required between adjacent buildings to allow for better protection against fire and earthquake pounding. A considerable number of houses built according to this structural system are still in use in the city of Lefkada.

In August 14th, 2003, a strong earthquake (Mw of 6.2) occurred in the Lefkada segment of the Cephalonia Transform Fault running in a N–S direction along the Western coast of Greece. The peak ground acceleration recorded within the town was 0.35 g [1]. Extensive ground failures were observed, as well as severe damages in harbours, retaining walls and roads. Damages of varying severity were observed in modern RC structures (with one house collapsed), as well as in structures built according to the historical structural system of interest herein. With the aim to preserve the structural system surviving uniquely in Lefkada, a research project was carried out at the Laboratory of Reinforced Concrete, National Technical University of Athens. The scope of the project was (a) to check and complete the analysis of the local structural system, previously studied by Touliatos and Gante [2], (b) to study the pathology of the system, (c) to provide qualitative interpretation of typical damages observed and (d) to draft guidelines

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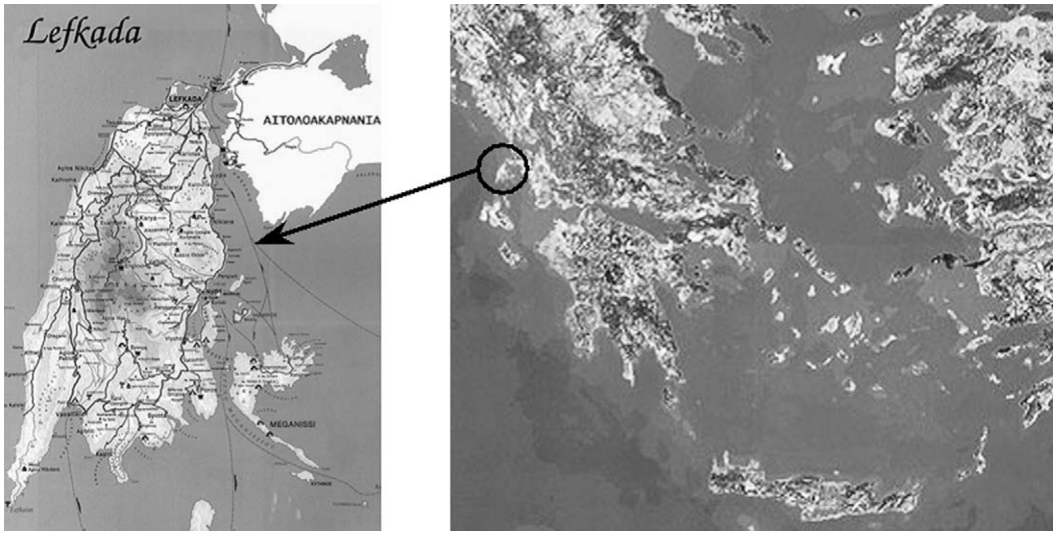

Fig. 1. Map of Lefkada.

for structural rehabilitation and strengthening of this type of buildings. The main steps taken to this purpose, as well as the main findings of this research (see also [3]) are briefly presented in this paper.

2. Short description of the structural system

The historical part of the city of Lefkada (Fig. 2) situated by the sea is developed both sides of a central street that separates the city into two parts of approximately equal areas. Lateral narrow streets start from the central street and are directed towards the sea (N–S or S–N direction). This arrangement allows for easy drainage of rainwater towards the sea. In addition, the-most frequent-north, northwest winds offer favourable conditions for reduction of humidity in the timber parts of these houses.

In what follows, the structural system of typical buildings is described, on the basis of the information provided by Touliatos and Gante [2], and completed by Vintzileou et al. [3], after extensive in situ work on a larger inventory of these buildings.

Fig. 2. The old part of the city of Lefkada.

2.1. Morphology

Typical buildings (one- to maximum three-storey buildings) consist of a stone masonry ground floor plus one or two timber framed brick masonry storeys. Intermediate floors and roof are made of timber. The roof is covered with tiles. Openings are practically symmetrically arranged in plan. To protect the timber framed masonry from humidity, the upper storeys used to be covered along the exterior façade by timber planks (Fig. 3(a)). The high cost of replacement of this cover (decayed with time) led to its replacement by plane or corrugated metal sheets (Fig. 3(b)). Covered walkways (Fig. 4) constitute a typical morphological and functional element in this city having frequent rainfalls.

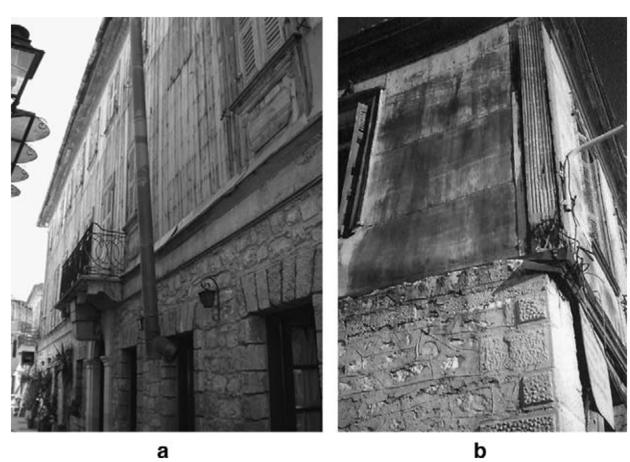

Fig. 3. Protection of timber elements: (a) by timber planks; (b) by metal sheets.

Fig. 4. Covered walkways.

2.2. Foundation

The city of Lefkada is founded on low strength alluvia. Therefore, special care was given to the foundation of buildings. Although it is not practicable to explore the footings of existing buildings, according to descriptions (based on information provided by craftsmen that worked in the construction of similar buildings), the buildings are founded on a grid of trunks (Fig. 5), at a depth of 0.6–1.0 m. The space between the timber elements is filled with sand, rubble stones and hydraulic mortar.

2.3. Ground floor

Bearing walls in the ground floor (typically, perimeter walls) are no more than 3.0 m high and they are made of stone masonry (0.60–1.00 m thick). The external leaf of walls is made with roughly cut stones, whereas cut stones are used in the corners of the building (Fig. 3(b)), as well as along the perimeter of openings (Fig. 3(a)). Rubble stones are used in the internal leaf of ground floor walls. The space between the two leaves is filled with small size stones mixed with pieces of bricks and mortar. Natural pozzolan was used in the buildings constructed up to the end of the 19th century. Later, lime was used together with straw (to improve the mechanical properties of mortar), whereas in the lower income housing clay mortar was used. In addition to masonry walls, a secondary (timber) bearing system is present in the ground floor which consists of timber columns (Fig. 6) arranged close to the masonry walls. The function of this secondary gravity load bearing system is discussed in Sections 4.3 and 4.6.

2.4. Upper storeys timber framed masonry

In Fig. 7, the typical arrangement of timber elements in the timber-framed walls is shown. The timber frame of the perimeter walls is connected to the ground floor masonry through timber beams, arranged along the perimeter of the stone masonry walls (Fig. 8(a)). Metal ties (of various geometry and size, Fig. 8(b)) are used to connect these timber elements of the floor with the stone masonry and/or with the timber frame of the upper storeys.

2.5. Floors and roof

Fig. 9 shows the typical construction of intermediate floors and roof structures. It should be reminded here that intermediate floors do not carry only their own (dead and live) loads, but they also transfer the vertical loads of the upper storey(s) perimeter masonry to that of the ground floor, as well as to the secondary timber bearing system of the ground floor.

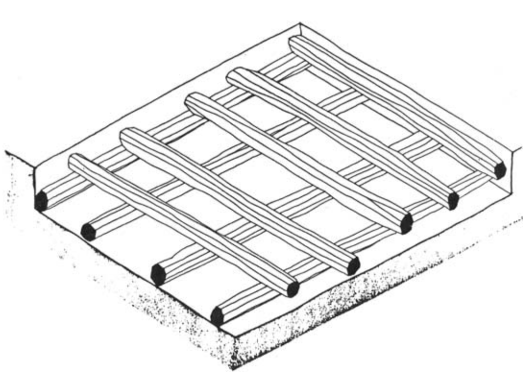

Fig. 5. Grid of trunks under the foundation of buildings (schematic [2]).

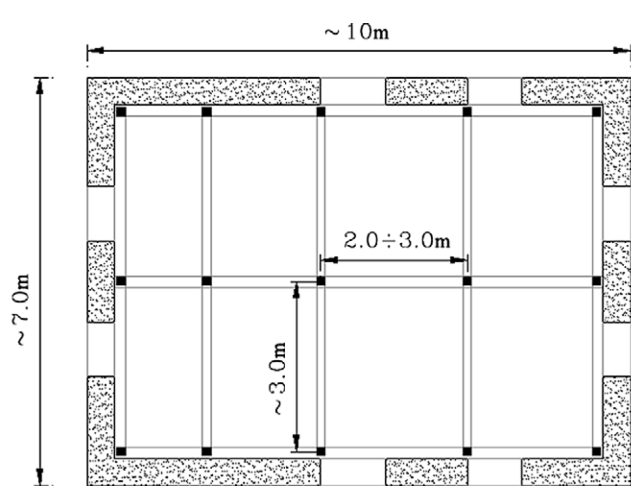

Fig. 6. Arrangement of the secondary timber system in a typical building (schematic).

Fig. 7. Typical morphology of timber-framed walls.

2.6. Connection between timber elements

As shown in Fig. 10, special care was given by the constructors to the connections between timber elements in the roof, within the timber-framed masonry of the upper storeys, as well as between timber beams and columns in the ground floor. Taking into account that Lefkada has always been an island with limited financial resources, one may assume that the purpose of the constructors was to enhance the overall stiffness of buildings. This assumption is discussed in Section 4.4.

Fig. 10. Typical connections between timber elements.

3. Pathology of the structural system

As mentioned in the Introduction, after the August 2003 earthquake, a detailed survey of damages incurred

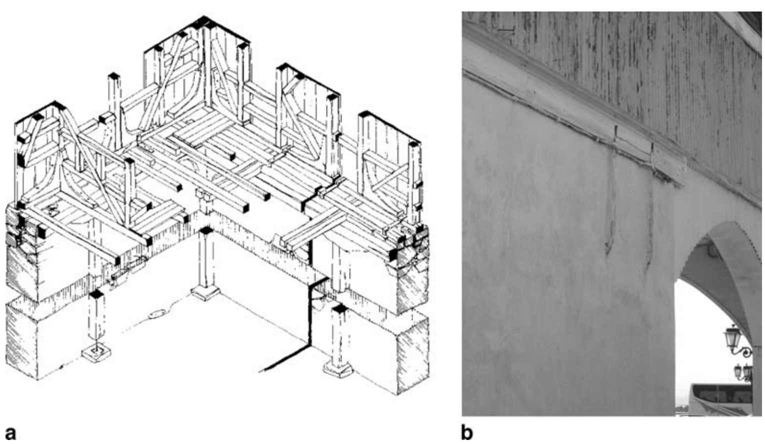

Fig. 8. (a) Connection between timber elements; (b) metal ties connecting timber floor to stone masonry.

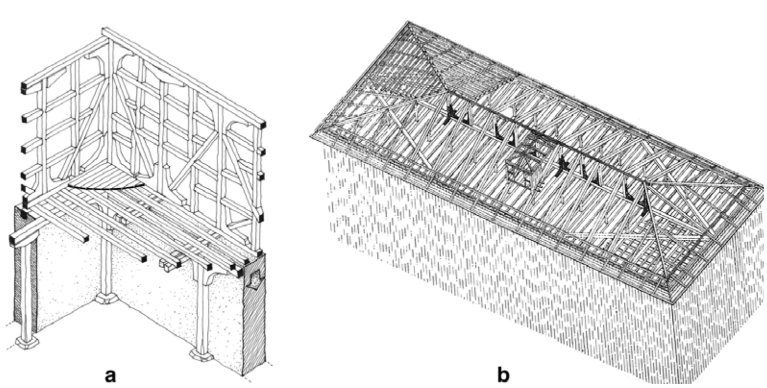

Fig. 9. Typical construction of (a) intermediate floor and (b) roof.

to the buildings of the city of Lefkada was carried out. This systematic work covers practically all houses from the outside. Furthermore, 26 buildings (in which access within was allowed) were fully investigated. On the basis of this work, the main damages are categorized in what follows. It should be noted, however, that several damages are due to physical actions rather than earthquakes. Nevertheless, they are described here, since-as discussed in the following sections-they have affected the seismic behaviour of these buildings.

Stone masonry in the ground floor: Cracks of varying width were detected, ranging from hairline cracks to several centimeters wide. The cracks were either inclined (along one or both diagonals of an exterior wall) or almost vertical close to the corners of the buildings.

Timber elements in the top of ground floor masonry: In several cases, extensive decay of the timber elements was observed due to biological attack (favoured by high percentage of humidity). The form of this damage though obviously not attributed to the earthquake is mentioned herein, since it has affected the seismic behaviour of the buildings. In fact, the reduction, which was observed in some cases, of the sectional dimensions of the timber beam serving as support to the upper storey(s) perimeter masonry walls led to the transfer of loads directly to the secondary timber system of the ground floor.

Secondary timber bearing system (timber columns and connections): In this case too, reduction of the sectional dimensions of the timber elements was observed due to decay.

Intermediate (timber) floors: The decay of timber beams due to environmental and biological actions led, in some cases, to a reduction of their cross-sectional dimensions and, by way of consequence, to large permanent deflections of the floors.

Timber framed masonry: Similar damages were observed in the timber elements of this system as well. In addition, cracks were observed in several cases between brick masonry and surrounding timber elements. In a limited number of cases, out of plane collapse of the filling masonry was observed.

Covered walkways: Extensive decay of the timber columns supporting the walkway was observed in several cases due to adverse environmental conditions. As expected, decay is concentrated close to the base of timber columns. As a result, lateral displacement of the columns at their base was observed (reaching values up to several centimeters).

Excessive lateral displacements: This is a failure observed in a limited number of buildings (Fig. 11) in which the stone masonry was partly demolished in the ground floor, before the event, when the use of those buildings was modified from residential to commercial. The demolition of masonry was done without previous design. Thus, the secondary bearing system of the ground floor became primary. Since, however, its stiffness was very limited, this system could not prevent large (permanent) horizontal displacements that led to distortion of the building as a whole at the ground storey.

Fig. 11. Permanent interstorey drift in the ground floor.

It should be noted, however, that in numerous cases the observed damages are not due only to the recent earthquake. In fact, one could distinguish damages, which obviously occurred during previous earthquakes that were never repaired and that were expectedly deteriorated. Finally, given the magnitude of the earthquake, the fact that a large number of buildings are not inhabited (and, hence, not maintained) for several decades, and taking into account that no design has preceded the various modifications made to the initial structural system of some houses, it can be concluded that the observed damages in this group of structures are of a degree that allows the vast majority of these buildings to be preserved with occasional rehabilitation work on a case by case basis.

4. Analytical investigation of the structural system

4.1. General information

The aim of the analytical work carried out within the project was the following: (a) to evaluate the basic dynamic characteristics of typical structures, (b) to interpret the major damages observed after the recent earthquake, (c) to assess the role of timber floors and roof, as well as the effect of the timber connections on the seismic behaviour of buildings and (d) to formulate general guidelines for interventions to the structural system of Lefkada. To this purpose, a typical building was selected as shown in Fig. 12(a). This is a building presenting the major features of the structural system without the quite numerous variations that a structural system developed in time expectedly presents. Table 1

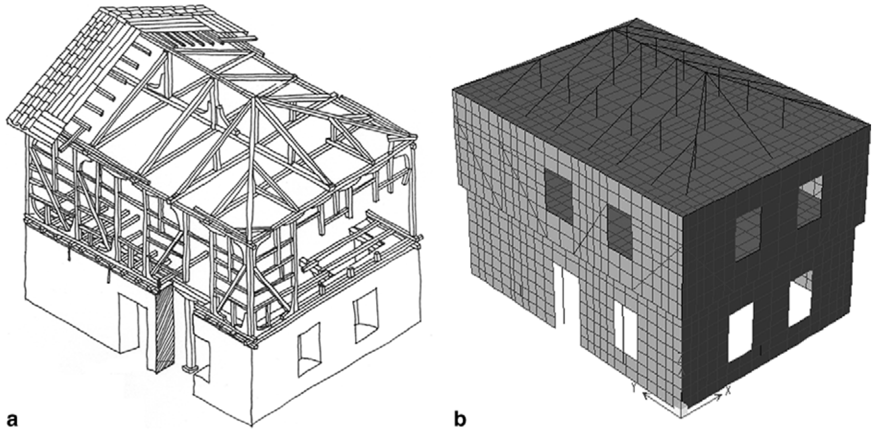

Fig. 12. (a) Typical building [2]. (b) Model of the building used for parameter analyses.

Table 1
Cases of analytically investigated buildings

Building case	Building designation		
	1st term	2nd term	3rd term
Two storey building	K2		
Three storey building	K3		
Stone masonry along all four sides of the building		4	
Masonry demolition along one long side		3 M	
Masonry demolition along one short side		3 m	
Masonry demolition along two consecutive sides		2	
Full demolition of stone masonry in the ground floor		0	
One missing side of masonry substituted by timber elements type 1			T1
One missing side of masonry substituted by timber elements type 2			T2
Partially demolished masonry along one side			TM
Ground floor masonry substituted by steel frame			St

summarizes the cases of buildings that were analytically investigated.

The dimensions of all structural members were selected on the basis of the in situ survey carried out by Touliatos and Gante [2], as well as within the framework of the present work [3]. Regarding the mechanical properties of the structural materials, representative values were assumed for timber and masonry properties (Table 2), based on the information available in the literature (e.g. [6,7]). In situ evaluation of actual materials of these buildings was not in scope. The buildings were analyzed (a) for vertical loads ($1.35g + 1.5q$) and (b) for seismic actions, combined with dead loads and a portion of the live loads ($g + 0.3q$). It should be noted that according to the Greek Aseismic Code [4], the seismic action along one principal direction is combined with 30% of that along the transverse direction..

Seismic actions were calculated according to [4]. Assuming a behaviour factor, q, equal to 1.50, corresponding to a low ductility system, a maximum design spectral acceleration equal to 0.60(g) is estimated, corresponding to a peak ground acceleration for the city of Lefkada, equal to 0.36(g). On the basis of the elastic response spectrum of the above earthquake record (Fig. 13), the actually demanded spectral acceleration, corresponding to a first mode period of vibration of approximately 0.1 s, is about 0.5(g), which is well below the above design value, specified in [4]. Although the

Table 2
Assumed mechanical properties of materials

	Stone masonry	Brick masonry	Timber elements (C30)
Density (kg/m^3)	2200	1900	380
Modulus of elasticity (MPa)	2500	1500	12,000
Compressive strength (MPa)	2.5	1.5	23.0
Tensile strength (MPa)	0	0	18.0
Shear strength (MPa)	0.15–0.18 (depending on the normal stress)	0.15–0.18 (depending on the normal stress)	3.0

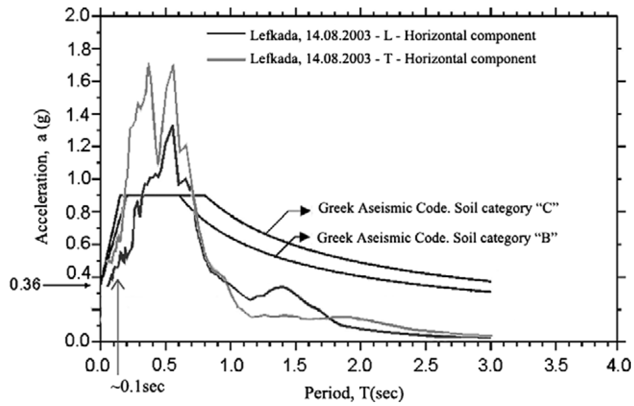

Fig. 13. Linear elastic response spectrum of the August 14th, 2003 earthquake [1].

structures were not designed in the modern sense of the term, and considering the spectral demands for these buildings under the subject earthquake, one may observe, on the basis of the registered damages, that the available margins of safety of this historical system are quite large.

4.2. Modelling

In order to evaluate the expected response of these structures, linear elastic analyses were performed using the program SAP2000 [5] (CSI, 1995). The three-dimensional model of the structure (consisting of shell elements-for stone and brick masonry and linear elements-for timber) is shown in Fig. 12(b). Special care was given to modeling of connections between timber elements: In terms of their ability to transfer bending moments, connections stiffened by means of "L"-shaped elements (Figs. 9 and 10) were modeled as fixed. All the other connections (roof trusses, upper floor perimeter timber trusses, column base of ground floor secondary system) were modeled as hinges.

As part of a sensitivity analysis to bracket the spectral response predictions, different extreme connection models were assumed for (a) the lateral fixity of the floor planks to the stone masonry walls (either fully fixed to the wall or sliding) and (b) the roof truss bottom chord members to the timber framed perimeter walls (in a similar function). In this way, lateral slip of the timber connections of the roof and secondary system and the perimeter walls was considered. Furthermore, vertical load transfer in these connections was assumed fully effective, with the exception of the case where the ground floor stone masonry walls were excluded from the model, in which case all vertical and seismic loads from the superstructure were transferred to the secondary system, connected to the perimeter collector beams with appropriate (eccentric) connections (Fig. 8(a)). In-plane flexibility of the wooden floor was also explicitly modeled assuming the actual configuration and timber members dimensions of the floor grillage (without relative slip), with or without the addition of a 2 cm thick layer of wooden planks, for in-plane rigidity, modeled using thin membrane elements.

In what follows, the main findings of the analytical work are presented and commented.

4.3. Fundamental mode periods of vibration

The typical reference building (K2-4) proves to be very stiff, with the first mode period about 0.1 s. The period increases substantially when stone masonry is provided only in some of the four sides of the building (Table 3). Based on the elastic response spectrum of the 2003 earthquake (Fig. 13), this increase in the natural period leads to higher accelerations imposed to the building. In case of the masonry being demolished completely in the ground floor, the natural period is several times higher than that of the reference building. The calculated values of the natural periods indicate that the stone masonry of the ground floor is-expectedly the major stiffening factor for the buildings. Furthermore, it is obvious that the secondary bearing system of the ground floor (i.e., the system of the timber columns) does not contribute to the seismic response of the buildings. This seems to confirm the hypothesis put forward by Touliatos and Gante [2] that the secondary bearing system plays a considerable role in case stone masonry of the ground floor is heavily damaged or even partially collapsed due to the earthquake. In fact (see also Section 4.6), this secondary system is able to transfer safely the vertical loads of the entire building superstructure until the masonry is repaired to be ready to resist the next earthquake. This hypothesis seems to be confirmed also by the fact that the constructors provide a well detailed connection between the perimeter wall collector beams at the base of the upper storey(s) and the secondary bearing system (Fig. 8), even though this is not needed when the perimeter stone masonry walls are present in the ground floor.

4.4. The role of floors and roof

One of the parameters that were investigated through the parametric analysis herein is the contribution of the timber floors and roof to the seismic behaviour of the buildings. As mentioned previously, the study of

Table 3
Periods of vibration

Building	T_1 (s)	T_2 (s)
(K2-4)	0.0728	0.0596
(K2-3M)	0.1027	0.0774
(K2-3m)	0.1199	0.0684
(K2-2)	0.2277	0.0986
(K2-0)	1.7797	1.5212
(K3-4)	0.1041	0.0888

the buildings in the city of Lefkada shows that both intermediate floors and roof were very carefully constructed with timber elements of large cross-sectional dimensions and very meticulous connections between them. In addition, the timber flooring in the floor levels (made of timber planks arranged perpendicular to the main beams) is carefully attached (by closely spaced nails) to the beams. This holds true also for the timber cover of the roof (needed to accommodate the roof tiles). The respective results of the analytical investigation are summarized in Figs. 14 and 15. One can see that the roof of the building is quite stiff in space even without taking into account the timber roofing for stiffness. In fact, it exhibits almost full diaphragm behaviour, thus, ensuring equal displacements of the walls at their top when fully connected to these walls. The addition of the timber roofing improves further the diaphragm action of the roof.

The beneficial effect of the timber roofing is quite apparent in the case of intermediate floors as well. In fact, as shown in Figs. 14 and 15, the bearing system of the floor seems to be quite flexible, since the horizontal displacement at mid-length of the longitudinal wall is 10 times that of the corners of the building. On the contrary, when the timber roofing is also taken into account and the floors are assumed connected to the walls, the horizontal displacements of the building as a whole are substantially reduced, whereas the relative displacement of the mid-length of the longitudinal wall to its ends is reduced by an order of magnitude.

One could not overemphasize the contribution of the horizontal bearing elements of the structure to the box action of masonry buildings. In fact, thanks to the box action, the (brittle by nature) masonry buildings are subject to smaller in- and out-of-plane deformations during an earthquake. In addition, the efficient interconnection of all bearing elements in-space allows for redistribution of actions, thus, preventing the building from excessive damage and collapse.

It should, therefore, be appropriate to recognize that the constructors that invented the structural system of

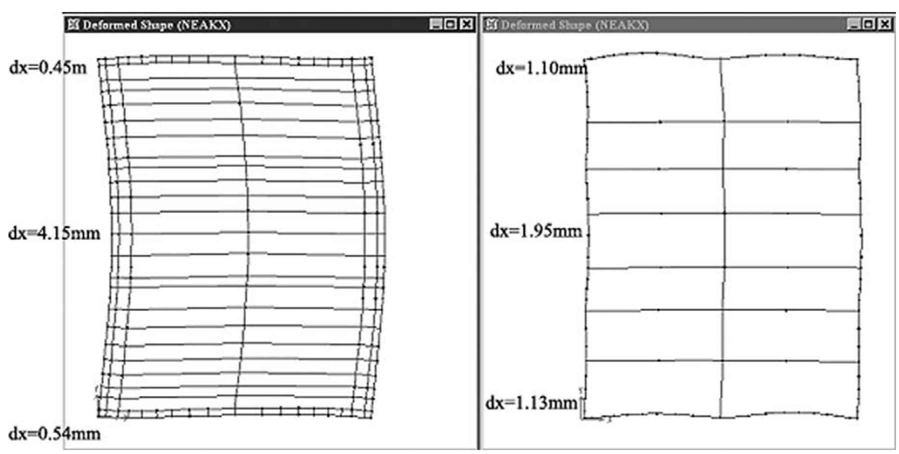

Fig. 14. Building (K2-4): Horizontal (in-plane) deformations of intermediate floor and roof without contribution of timber pavement for seismic action along the short dimension of the building.

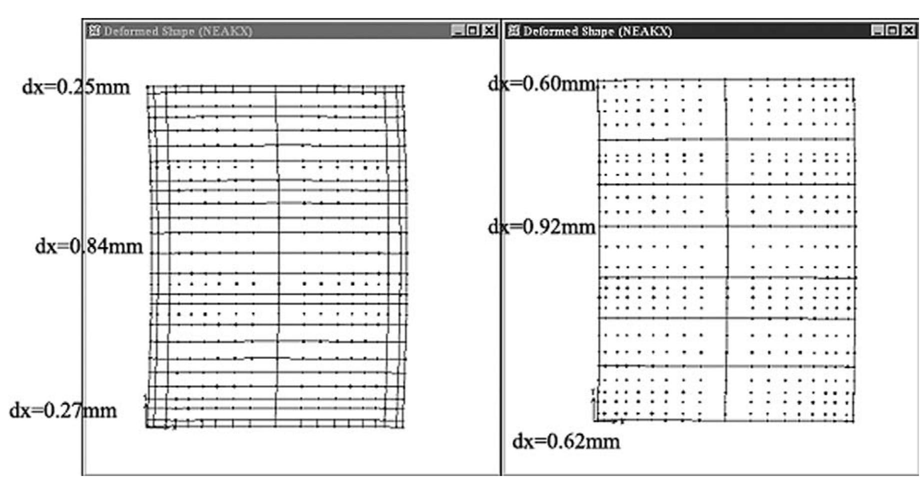

Fig. 15. Building (K2-4): Horizontal (in-plane) deformations of intermediate floor and roof in presence of timber pavement for seismic action along the short dimension of the building.

the Lefkada historical buildings were aware of the role of the horizontal bearing system.

4.5. The behaviour of the timber framed perimeter walls

It should be noted that modeling of the perimeter walls in the upper storey(s) did not include some aspects that are of major importance for the behaviour of this type of construction. Thus, the confinement of the filler brick masonry thanks to the surrounding timber elements, as well as secondary mechanisms of energy absorption (such as friction due to slip along timber–masonry interfaces) were not taken into account. In fact, overall energy absorption within the building was only considered in the average sense assuming a (conservative) amount of 5% critical damping. A detailed modeling of timber framed walls was beyond the capabilities of the software and it was out of the scope of the present work. The analysis that was carried out proved that timber the framed walls exhibit high in-plane stiffness. Thus, interstorey drifts are limited, preventing extensive damage. In addition, the limited dimensions of brick panels infilling the timber frame, as well as the adequate bearing capacity of the overdesigned timber elements and their efficient connections, led to very satisfactory out-of-plane behaviour of the approximately 12 cm thick timber framed walls. This is corroborated by the fact that very limited damages were observed in the upper storeys of the buildings (Fig. 16).

4.6. The contribution of timber elements

Examination of the critical member stresses in all the elastic analyses carried out, has proven that the timber elements (both in the secondary bearing system of the ground floor and in the timber framed walls) are overdimensioned (Fig. 17). In fact, even under the most adverse parametric study conditions, the axial load the vertical secondary system members have to carry is by an order of magnitude smaller than their estimated resistance, allowing therefore adequate factors of safety even for the case of some cross-section reduction due to environmental or biological effects. Therefore, the following observation can be made: As far as the timber columns of the ground floor are concerned, it is proved that they can safely carry the vertical loads, even in case that a complete removal of the stone masonry in the ground floor is considered. This is an indirect verification of the role the timber columns are called to play during the seismic event after collapse of the masonry.

Regarding the timber elements of the timber framed walls, one may assume that their dimensions are selected more to contribute to the out-of-plane than to the in-plane behaviour of the walls.

Fig. 16. Typical damage of timber framed wall.

4.7. Interpretation of major damages

The analytical work allowed for qualitative interpretation of the major damages observed after the 2003 earthquake. Since typical damages of masonry buildings (such as diagonal shear cracks in walls and vertical cracks near the top of walls at their connection with the transverse walls) are easily interpreted and analytically reproduced, the respective results are not presented here (see [3]). Thus, special attention is given to some damages typical of the particular structural system under examination:

(a) Oblique cracks in stone masonry in the ground floor: In some cases, in buildings with masonry demolished along one of the sides of the building, oblique cracks along the whole length of the adjacent side were observed. Those cracks (Fig. 18(a)) opened along the whole diagonal of the wall, their axis passing through openings as well. The analysis showed (Fig. 18(b)) that those cracks are due to out of plane action of the longitudinal wall. In fact, due to the demolition of the transverse wall, the longitudinal wall behaves like a vertical slab fixed only along two sides (i.e., at the foundation level and to the existing transverse wall).
(b) Excessive horizontal displacement of the building as a whole: This was a damage observed in buildings in which the stone masonry of the ground floor was partially demolished (without previous design) to accommodate shops in the ground floor (Fig. 11). The analyses proved (Fig. 19) that-expectedly-interstorey drifts are large in the most flexible floor, thus, leading to excessive horizontal displacements and to distortion of the building as a whole.

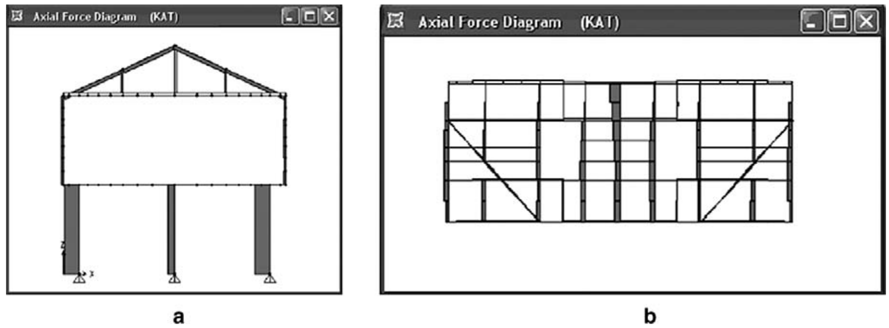

Fig. 17. Axial forces in timber elements under vertical loads. (a) Columns in ground floor for fully demolished stone masonry: maximum calculated axial force = 55 kN, minimum resistance = 500 kN. (b) Timber framed walls: maximum calculated axial force = 11 kN, minimum resistance = 225 kN.

Fig. 18. (a) Typical damage of stone masonry, in case of demolished masonry along one transverse façade. (b) Distribution of principal tensile stresses (in kPa), calculated for a similar case. Seismic action perpendicular to the wall under consideration.

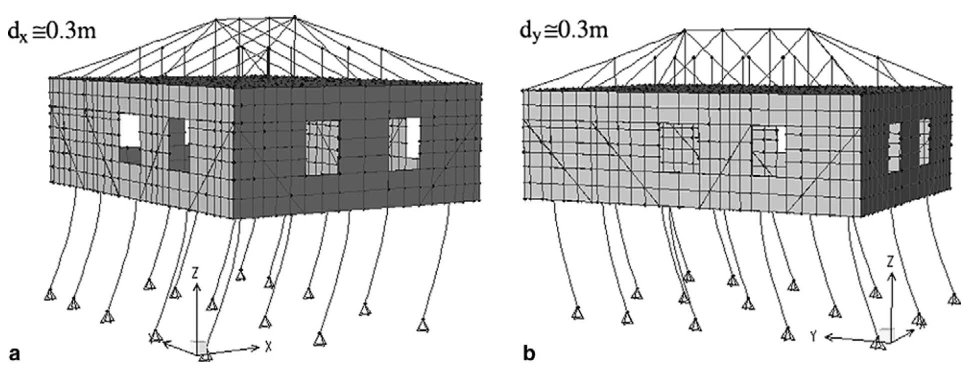

Fig. 19. Deformed shape of building, for fully removed ground floor masonry: (a) seismic action along the transverse axis of the building; (b) seismic action along the longitudinal axis.

(c) Permanent horizontal displacement of timber columns: In some cases, a permanent loss of verticality was observed in the timber columns of the secondary system after the earthquake, with the stone masonry in the ground floor intact. This damage was hard to interpret. Nevertheless, the survey of buildings to which this damage was observed has proven (Fig. 20) that extensive decay (due to biological attack) of the timber beam through which the upper storeys are resting on the ground floor masonry has led to loss of support. Thus, the upper storey(s) were supported only by the flexible secondary bearing system (i.e., by the timber columns). Therefore, the damage can be interpreted exactly the same way as the previous one presented in paragraph 4.7(b).

5. Indicative intervention schemes

As mentioned in the previous sections, both in situ survey and analytical work have proved the major role of the ground floor stone masonry. Extensive damage of ground floor masonry or, alternatively, removal of masonry even along one single façade of the building lead to large displacements that the system could not sustain. Therefore, one basic rule would be that damaged masonry should be immediately repaired, whereas demolition of masonry should not be permitted. Nevertheless, since the continuation of the use of buildings in the old part of the city constitutes a prerequisite for their preservation, one should try to suggest interventions that would serve new needs of the local society. To this purpose, some indicative intervention schemes were analytically investigated for the representative building configuration. In all of these schemes, an effort is made to partially substitute the strength and, mainly, the stiffness of masonry in the ground floor, by systems that allow for relatively large openings in the ground floor (Fig. 21). All intervention schemes were applied in the shorter façade of the model building, since analyses have proven that demolition of masonry along this side is the most unfavourable for the seismic behaviour of the entire building. Furthermore, such an arrangement of the intervention systems is in accordance with real conditions, since most of the houses are arranged with their short side facing the street.

As shown in Table 1 and in Fig. 21, two alternative timber frame schemes were considered (Fig. 21(a) and (b)), a steel frame (Fig. 21(c)), as well as partial demolition of masonry along one façade (Fig. 21(d)). The criteria applied to evaluate the efficiency of the building under the examined intervention schemes were (a) their first mode period and (b) their lateral displacements, compared to those of the undamaged reference building with masonry along the entire perimeter of the ground floor. As shown in Table 4, all proposed schemes are correct in principle and they serve the purpose for which they were conceived, since both period and displacements are substantially reduced. It should be noted, however, that the results of this investigation are indic-

Fig. 20. Advanced decay of timber elements along the perimeter of stone masonry.

Table 4
Buildings after intervention

Building	T_1 (s)	T_2 (s)	Maximum displacement (mm)
(K2-4)	0.0728	0.0596	0.27
(K2-3m)	0.1199	0.0684	2.70
(K2-3m-T1)	0.0953	0.0642	1.40
(K2-3m-T2)	0.0943	0.0646	1.30
(K3-3m-St)	0.0816	0.0634	0.70
(K2-3m-TM)	0.0836	0.0657	0.80

Periods of the first two modes of vibration and maximum horizontal displacement for seismic action parallel to the short axis of the building.

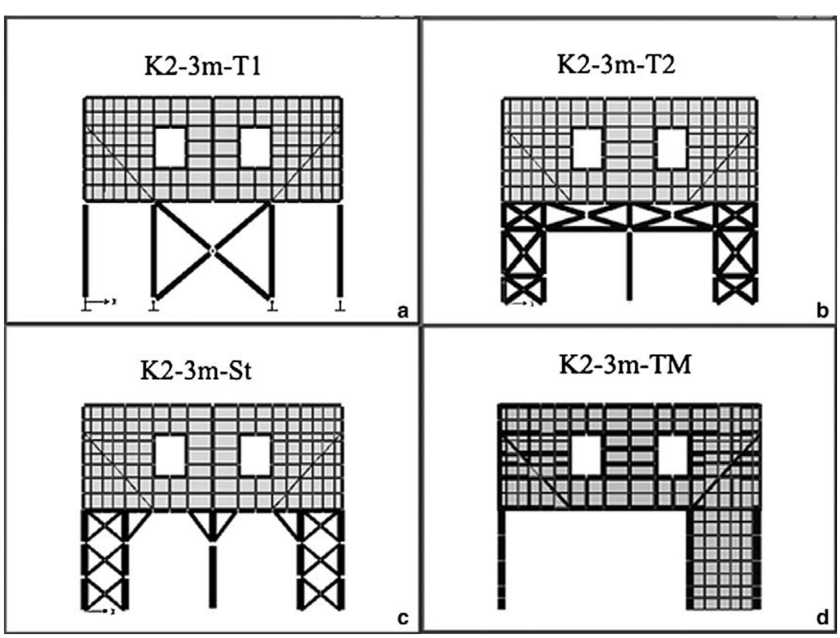

Fig. 21. Intervention schemes analytically investigated.

ative. In fact, the final responsibility lies within the hands of the Structural Engineer who will provide the design needed for each specific building on a case-by-case basis. It should also be noted that the examined systems were assumed to be properly connected with the existing structural system, since this is a prerequisite for their composite action.

6. Conclusions

The local structural system of the historical part of the city of Lefkada was systematically investigated. The systematic survey of numerous buildings, the study of the pathology due to the August 2003 strong earthquake, as well as the analytical work carried out, allow for the following conclusions to be drawn.

The structural system of the city of Lefkada was conceived to resist seismic actions. This is proven by the survival of the system after several strong earthquakes, even though several buildings have not been maintained at all, the damages due to previous earthquakes were not repaired, and in a number of cases the structural system was altered without previous design.

The study of the system allowed for a better understanding of several features that play an important role in its seismic resistance, namely the stiff stone masonry in the ground floor, the secondary timber system (able to safely carry vertical loads after extensive damage of masonry), the reduction of self weight thanks to the use of timber framed masonry in the upper storey(s) and light wooden floors, the diaphragm action of these intermediate floors and the roof, as well as the highly efficient connections between the structural members of the system.

The typical damages observed in the buildings are analytically interpreted; it was also proven that they are in general repairable (with the exception of limited cases of buildings distorted due to large interstorey drifts in the ground floor). The most severe among them are due to the decay of timber elements, as well as to alterations of the initial lateral and gravity resisting system. Therefore, a prerequisite for the preservation of the system is to protect and maintain all the timber elements for environmental and biological attack, to replace those severely decayed and to ensure the quality of their connections. This is because of the contribution of all the timber elements to the seismic behaviour of the buildings.

The guidance for preservation of the local system in Lefkada includes several alternative intervention schemes that aim at substituting the stiffness and the bearing capacity of part of the stone masonry in the ground floor. Those schemes prove to be efficient. Nevertheless, the results presented in this paper do not intend to substitute the design required for each specific building by the Structural Engineer in charge.

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