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STRUCTURAL DAMAGE ASSESSMENT OF HISTORIC TRADITIONAL MASONRY BUILDINGS: A CASE STUDY

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Abstract

The work presented is concerned with the rehabilitation of a typical traditional masonry building built in Athens in the late 18th early 19th century. The building has suffered significant damage primarily in the form of vertical and inclined splitting of the bearing walls and fragmentation of the mortar used to bind together the masonry stones as a result of seismic excitation, lack of maintenance, construction defects, etc. The paper describes the damage suffered and investigates its causes through the use of numerical analysis techniques. It is found that the causes of damage are predominantly linked with structural deficiencies such as lack of diaphragmatic action and bracing and restoration methods are proposed. The latter include a reinforced-concrete layer at the wall crowning, strengthening (or replacement wherever necessary) of the floor timber beams and bracing in the form of external and internal reinforced concrete strips at the level of the basement floor.

Keywords: *Traditional masonry buildings; Building materials; Structural elements; Deterioration phenomena; Structural damage; Cracking; Finite element method (FEM).*

Introduction

It has become widely accepted to classify buildings constructed without a formal design process as traditional masonry. Their form, plan and method of construction simply follow tradition at the time and place of their conception. Historical traditional masonry is a form of architecture built by using local resources, covering materials, techniques and the building skills, and it is the fundamental expression of the culture of different communities and their relation with nature and the landscape [1-3].

Athens is one of the most famous heritage cities worldwide; it contains many historic buildings of different ages. Traditional Urban Residential Masonry (TURM) buildings in Athens are usually made of rubble (cobble) natural stones and a large volume of low strength lime mortars, while their floors/roofs are made of timber elements. Their load-bearing walls are of the single-leaf type (with various degrees of bonding and block interlocking) or of the so-called three-leaf type (with two discrete external leafs and an infill “material” of a large voids ratio) with thicknesses smaller or larger than approx. 700 mm, respectively. Therefore, the basic

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structural elements are made of a particularly ‘undisciplined’ material, to be finally handled or formed as a ‘pseudo-continuum’ medium [4-7].

Plaka is the oldest region of modern Athens located at the feet of the Acropolis with many historic buildings constructed within different centuries (see Fig. 1). It is in this region where the building that forms the subject of the paper is located, at the crossing of Aktaiou and Lykomidon streets, and will be referred to as *Aktaiou* building thereafter.

Fig. 1. Typical historic buildings in the Plaka district (Church, Mosque, and traditional houses)

The *Aktaiou* building, which was built in the early 1900’s, is considered to represent the structural and architectural trends prevailing in Athens this period (see Fig. 2). Over the years, it has suffered significant damage due to various causes, such as seismic excitation, lack of maintenance, construction defects, etc.; in recent years, although used occasionally by refugees as shelter, the building remained forlorn and derelict. As a result of the long period of neglect and lack of maintenance it has sustained damages to a considerable extent.

Fig. 2. The Aktaiou building

The work described in what follows is concerned with a numerical investigation of the causes of the damage suffered by the structural elements of the building. The work is based on the use of a widely used analysis package and the results obtained form the basis for proposing the rehabilitation work required for restoring the building.

Structural system

The Aktaiou building is a two-storey masonry building with basement; the bearing walls forming part of its structural system are shown in Fig. 3, which also shows the wall names adopted for the work. The façade of the building lies in Aktaiou Street (wall 1) and has a length of 18.40 m; its left-hand side face (wall 2), which lies in Lykomidon street, has a length of 18.00 m, whereas its right-hand side face (wall 3) sees at an internal open space and has a length of 17.00 m. Wall 4 essentially forms the fence separating the building from the adjacent property.

The basement extends within the part of the plan enclosed by walls 1, 5 and parts of walls 2 and 3. The in-plan geometry of the building has a U shape with a central part (CB) extending between walls 1, 6, 2, and 3 and two wings extending between walls 6, 4, 2, and 7 (W1) and walls 8, 3, 6, and 4 (W2), respectively. An open space forms between walls 7, 8, 6 and 4.

Both the ground level and the 1st storey floors were made of timber beams with a 17 cm high x 11 cm wide cross section arranged in parallel at distances of 50 cm and covered by planks with a 21 cm wide x 2.5 cm thick cross section. The beams are simply supported within recesses formed in opposite walls (see Fig. 4).

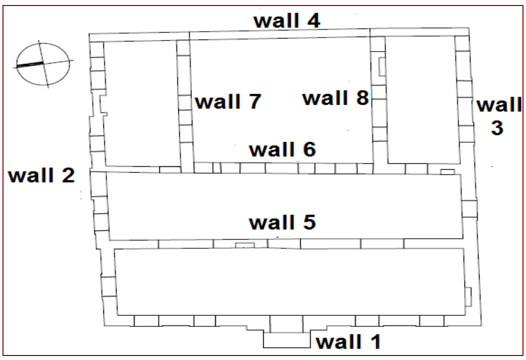

Fig. 3. Masonry wall bearing system

Fig. 4. Floor wooden beams

Fig. 5. Roof of Aktaiou building

The roof of the central part (CB) of the building (see right-hand side of Fig. 5) is two-way supported and comprises single and double slope trusses (see Figs 6 left and right), whereas the roof of the wing parts (W1 and W2) of the building (see left-hand side of Fig. 5) are one-way supported and comprise double slope trusses (see Fig. 6 left). The former comprises one vertical (king post) and two inclined struts, a horizontal tie and two diagonal struts (rafters) as indicated in Fig. 6 left. The rafters support 2 cm thick purlins extending in parallel to the supporting walls, with the purlins being covered by planking which underlies the byzantine tiles. The trusses of the wing roofs comprise inclined struts and horizontal tie only (see Fig. 6 middle).

Fig. 6. Truss of central (left and right) and wing (middle) parts of the roof

The main building materials used were limestone, marble, and volcanic stone, whereas semi fired bricks were in between the main building materials and under the windows (see Fig. 7).

It is important to note at this stage that the bearing system of the building may be vulnerable to earthquake actions mainly due to the following reasons:

- The elongated shape of plan view of the central part of the building which consists of two bearing walls (W1 and W5) with a length of approximately 18.00 m without transverse walls that would reduce the likelihood of out-of-plane displacements
- The lack of floor and roof diaphragms
- The lack of bracing that would ensure monolithic response of the building

Observed damage

Masonry whether of natural stone or of fired clay bricks, is the most durable of the traditional forms of construction [8]. In masonry buildings, structural and architectural damage occurs when the stresses due to the action of external forces exceed the strength of the materials in structural elements, either because the forces increase beyond expected limits or because of building materials deterioration [9, 10]. The investigation of the causes of damage suffered by historic traditional masonry is a prerequisite for selecting suitable alternative materials and avoiding past mistakes during the restoration [11-13].

The damage suffered by the building is clearly seen by visual inspection. It is most likely that it initiated at the bearing walls as a result of strong seismic excitation and aggravated due to structural deficiencies, lack of maintenance, environmental actions, aging, etc. Typical types of damage are shown in Figs 8 to 12.

More specifically, Fig. 8 shows the vertical splitting that occurred near or at the joint of intersecting walls, whereas the lack of bracing tying together the individual trusses of the roof at their support in the wall recesses allowed excessive out-of-plane displacement under horizontal actions, which led to near vertical cracking at the top edge (crowning) of the wall (see Fig. 9). In-plane horizontal actions appear to be the cause of the inclined cracking shown in Figs 10 and 11. Such cracking often initiates at the corners of openings (windows and doors) due to the stress concentrations developing at such locations (see Fig. 11).

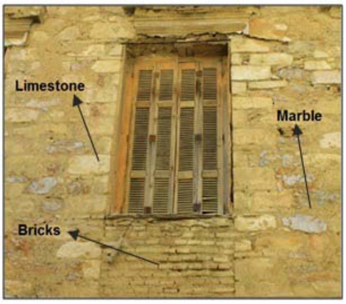

Fig. 7. Building materials

Fig. 8. Near vertical cracking of walls near their joint and splitting of wall-to-wall joint

Fig. 9. Vertical cracking at to edge (crowning) of the façade of the building

Fig. 10. Inclined splitting of masonry wall

Fig. 11. Cracking at openings' corners

It is also important to note that the loss of the plastering of many of the walls (see, for example, Fig. 12) revealed fragmentation (bleeding) of the mortar used to bind together the stones of the masonry. This is considered to weaken considerably the structural system not only in terms of strength, but also in terms durability, since it reduces the stability of the walls in future seismic excitations.

Fig. 12. Loss of plaster revealing mortar fragmentation (bleeding)

It is considered that the main causes of the damage described above are associated with the building's structural characteristics discussed in the preceding and this is investigated numerically in what follows.

Numerical investigation

The aim of the numerical work described in the following is, on the one hand, to establish whether the deficiencies in the building's structural characteristics identified in a preceding section are indeed the underlying cause of the damage suffered, and, on the other hand, if this is indeed the case, to show that a significant improvement of the structural performance can be achieved by eliminating these deficiencies. This is achieved by comparing the results obtained by numerical stress analysis of the building (assuming it in an undamaged state) before and after eliminating the deficiencies of the structural system under the action of the loading conditions specified by current codes for the design of earthquake-resistant structures [13, 14].

Details of numerical analysis

The numerical analyses of the building was carried out through the use of ETABS v9.7.4, a finite-element analysis (FEA) package produced by CSi and widely used in practical structural analysis and design in Greece. The building was subjected to a combination of vertical and horizontal loading as specified by current codes of the design of earthquake-resistant structures [15-17]. Two loading cases were considered: horizontal loading in the x- and y- directions (orthogonal to each other) combined with the same vertical load, with the x-direction coinciding with the orientation of the Aktaiou Street. The vertical load corresponds to the dead weight and part of the live load acting on the building and the horizontal load is the static equivalent of the inertial forces developing as a result of the seismic excitation.

Finite element discretization

The masonry bearing walls were modeled through the use of four-node shell elements combining membrane and plate-bending behavior expressed in the form of an isoparametric formulation that includes translational in-plane and rotational stiffness components in the direction orthogonal to the plane of the elements [18]. The linear elements of the floors and roof were modelled through the use of beam elements. Full details of the above elements used can be found in the CSI analysis reference manual [19].

Material properties

Since the analysis performed was linear, elastic properties are sufficient for providing an adequate description of material behavior. The masonry bearing walls were assumed to be homogeneous and isotropic; their mechanical characteristics were assessed through the use of the formulae given in Table 1 [20]. As indicated in the table, the formulae are expressed as functions of the mean values of the uniaxial compressive strength of the limestone, marble and mortar constituents of the masonry. These values, given in Table 2, were calculated from the results obtained from uniaxial compression tests on specimens cored or chiseled out from the walls of the Aktaiou building [21]. The mechanical characteristics of the masonry assessed by replacing the values of Table 2 in the formulae of Table 1 are given in Table 3. It should be noted that the values of the tensile strength of the masonry shown in the table were assumed to be of the order of 10% of the compressive strength. Finally, the mechanical properties of the wood are given in table 4.

Results of stress analysis

The results obtained are provided in Figs 13 to 22. Figures 13 to 16 show the results obtained from stress analyses of the building before attempting to eliminate the deficiencies of its structural system, whereas the results obtained after implementing modifications to the structural system considered likely to improve its response to the induced loading conditions. In all cases, the figures provide an indication of the deformed shape of the building and the distribution of principal tensile and compressive stresses exceeding the strength of the masonry of the bearing walls. The tensile stress distributions are shown in Figs 13 to 20, whereas the compressive stress distributions in Figs 21 and 22.

Table 1. Empirical formulae used for assessing the masonry mechanical characteristics

	formulae
Horizontal mortar layers – Volume of mortar	25-30mm, Vmortar≈0.30/0.40
Materials afety factorγ_m	2.00
Wall compressive strength normal ($\perp$) to horizontal masonry layers (f_{wc}, $\perp$)	$\left(\frac{2}{3}\sqrt{f_{bc}}-a\right)+0.5\cdot f_{mc}$
Wall compressive strength parallel (//) to horizontal masonry layers (f_{wc}, //)	For incomplete mortar filling $f_{wc,//}=(0.50-0.65)\cdot f_{wc,\perp}$
Wall tensile strength normal ($\perp$) to horizontal masonry layers ($f_{wt,\perp}$)	$f_{wt,\perp}=f_{mt}$
Wall tensile strength normal (//) to horizontal masonry layers ($f_{wt,}$ //)	$f_{wt\,//}=2\cdot f_{mt}$
Shear strength(horizontal sliding)	$f_{wv,0}=0.05+0.25*\left(3/4\cdot\sigma_o\right)\approx 0.05+0.20\cdot\sigma_o$
Shear strength(diagonal cracking)	$f_{wv,d}=2/3\cdot f_{wt,d}\cdot\sqrt{1+\frac{0.85\cdot\sigma_o}{f_{wt,d}}}=f_{mt}\cdot\sqrt{1+0.5\frac{\sigma_o}{f_{mt}}}$
Modulus of elasticity	$E=800\cdot f_{wc,\perp}$
Shear modulus	G=0.40E
Poisson's ratio	0.25

Table 2. Mean values of compressive strength and specific weight of materials comprising the masonry walls

Building materials	**Compressive strength f_{bc}(MPa)**	**Specific weight (kN/m^3)**
Marble	**25.21**	**27.63**
Limestone	**33.48**	**20.18**
Volcanic stone	**5.31**	**10.63**

Table 3. Masonry mechanical characteristics

material	**Stone compressive strength,f_{bc} (MPa)**	**Masonry compressive/tensile strength,f_{mc}/f_{mt} (MPa)**	**Modulus of Elasticity,E (MPa)**	**Poisson's ratio, v**	**Shear modulus, G (MPa)**
		Ground floor			
Marble	25,21				
Limestone	33,48	1,81/0,18	1448	0,25	296
Mean value	29,35				
		1st floor			
Marble	25,21				
Limestone	33,48	1,25/0,12	1000	0,25	400
Volcanic stone	5,31				
Mean value	21,33				

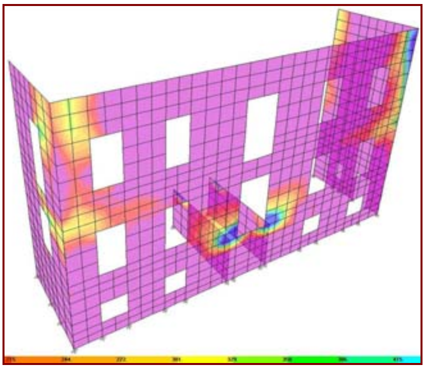

Fig. 13. Tensile stresses (in kN/m²) exceeding the tensile strength of masonry developing at the wall crowning under seismic action in the y-direction (orthogonally to Aktaiou Street)

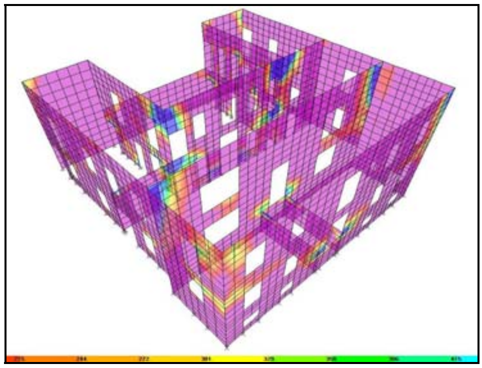

Fig. 14. Tensile stresses (in kN/m²) exceeding the tensile strength of masonry developing in the region of the wall intersections under seismic action in the y-direction (orthogonally to Aktaiou Street)

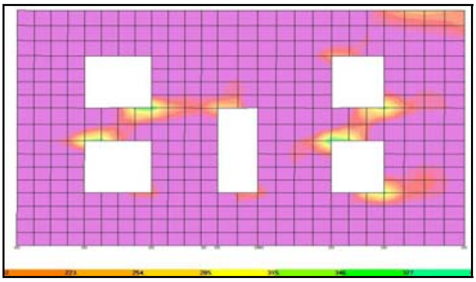

Fig. 15. Tensile stresses (in kN/m²) exceeding the tensile strength of masonry developing in the region of the openings of a typical internal wall in the x- direction (i.e. parallel to Aktaiou Street)

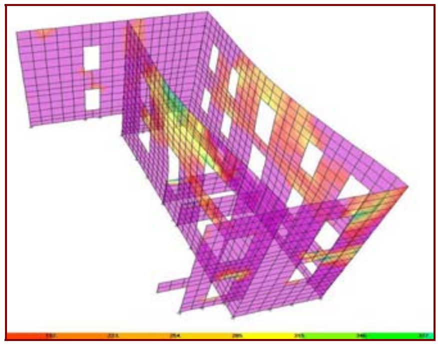

Fig. 16. Deformed shape and related tensile stresses exceeding the tensile strength of masonry (in kN/mm²) under the combined action of the service load and seismic excitation in the y-direction (orthogonally to Aktaiou Street)

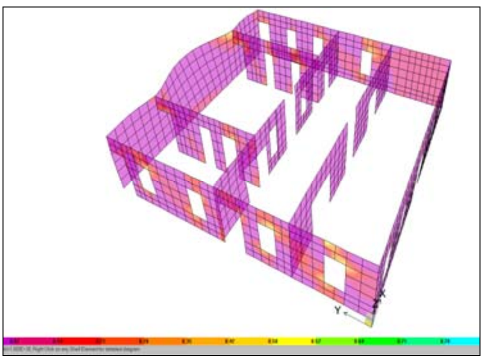

Fig. 17. Deformed shape and distribution of principal tensile stresses (in MPa) exceeding the tensile strength of the masonry of the walls of the first floor of the building under seismic excitation in the y-direction

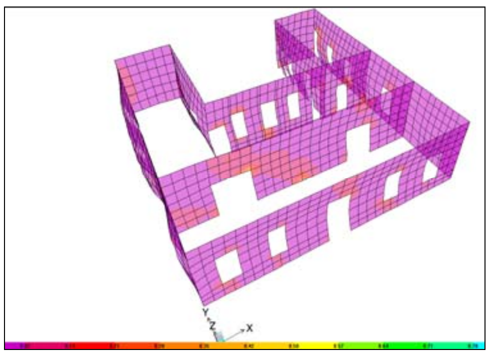

Fig. 18. Deformed shape and distribution of principal tensile stresses (in MPa) exceeding the tensile strength of the masonry of the walls of the first floor of the building under seismic excitation in the x-direction (ACC3)

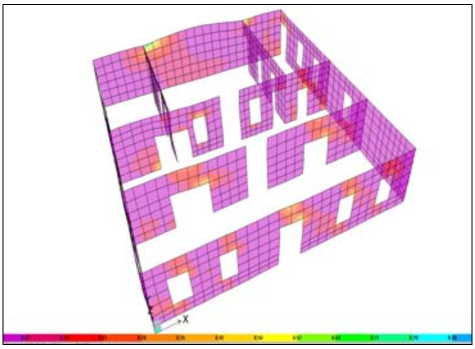

Fig. 19. Deformed shape and distribution of principal tensile stresses (in MPa) exceeding the tensile strength of the masonry of the walls of the basement of the building under seismic excitation in the y-direction (ACC5)

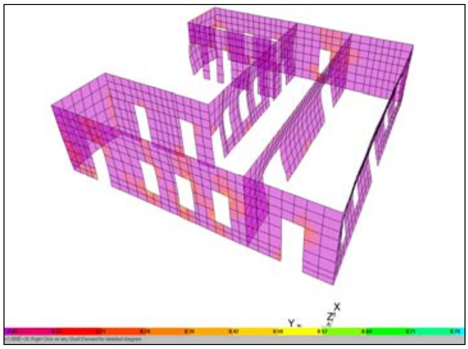

Fig. 20. Deformed shape and distribution of principal tensile stresses (in MPa) exceeding the tensile strength of the masonry of the walls of the basement of the building under seismic excitation in the x-direction (ACC3)

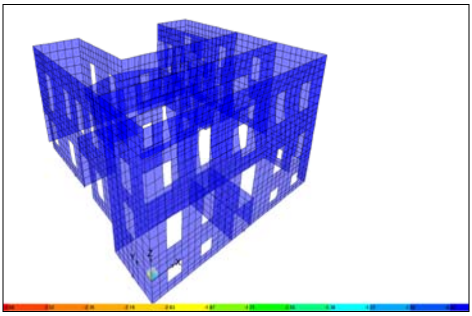

Fig. 21. Deformed shape and distribution of principal compressive stresses (in MPA) exceeding the compressive strength of the masonry walls of the building under seismic excitation in the x-direction (ACC3)

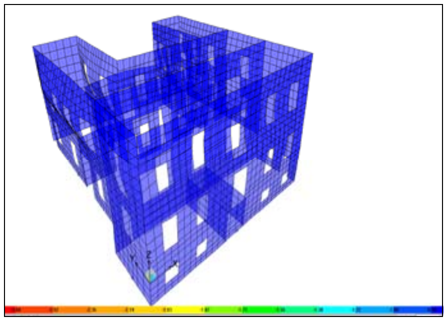

Fig. 22. Deformed shape and distribution of principal compressive stresses (in MPA) exceeding the compressive strength of the masonry walls of the building under seismic excitation in the y-direction (ACC5)

Discussion of the results

Current structural system

From Figs 13 to 16, it can be seen that the most critical cracking develops when the horizontal load acts in the y-direction. Fig. 13 shows that the most critical locations of the cracking are expected to be on either of the side regions of the top edge of wall 1 (façade in Aktaiou Street); in fact, at the right-hand side region cracking is predicted to occur at the joint of wall 1 with wall 3. Cracking is also predicted to occur at the bottom corners of the main entrance in Aktaiou Street. Cracking similar to that of wall 1 is predicted to be suffered by walls 5 and 6 (walls parallel to wall 1, see Fig. 3). As for the case of wall 1, they are predicted to occur on either side of the top edge of the walls, as well as at the bottom corners of the doors (see Fig. 14).

Under seismic excitation in the x-direction, the internal walls are found likely to suffer cracking in the region of the diametrically opposite corners of door and windows; in fact Fig. 15 shows that the largest tensile stresses develop at such locations of the door and windows to the backyard of the building enclosed by walls 6, 4, 7 and 8. Moreover, Fig. 16 shows that seismic excitation orthogonal to walls 1, 5 and 6 causes out-of-plane displacement of the walls in the form of bending of their top edge (due to insufficient roof diaphragm) and, therefore, tensile stresses at mid-length which are likely to cause flexural cracking.

The locations of the maximum principal tensile stresses presented above are found to coincide with the locations of the observed cracking of the walls discussed in a previous section.

Modified structural system

From Figs 17 to 20, it can be seen that allowing for diaphragmatic action at the levels of the floors and roof leads to a significant reduction of not only the values of the principal tensile stresses developing in the masonry walls, but also of the locations at which these values are larger than the tensile strength of masonry. In fact, diaphragmatic action is found to reduce the out-of-plane displacements of the walls to such an extent that cracking as a result of such displacement seems unlikely. Moreover, since allowing for diaphragm action also causes conditions of bracing at the levels of the floors and roof, the values of the tensile stresses developing at these locations due to the lack of bracing (see Figs 13 to 16) reduce well below the tensile strength of masonry.

Tensile stresses exceeding the masonry strength are found to develop at the diametrically opposite corners of the openings (doors and windows) (see Figs 17 to 20); smaller tensile stresses are also found to develop diagonally within the walls under the action of in-plane horizontal loading. It should be noted, however, that, in spite of the development of tensile stresses exceeding the masonry strength at localized regions, the structural system of the building is found capable of withstanding the actions specified by current codes for the design of earthquake-resistant structures (see Figs 21 and 22). Nevertheless, the interventions proposed in the following section are intended to minimize the likelihood of cracking that will violate the code requirements under both service and ultimate limit-state conditions.

Proposed interventions

The basic principle of the proposed interventions is to improve the static and seismic behavior of the building without changing its historic and aesthetic fabric (where possible). A wide variety of intervention techniques can be considered for strengthening and repair of masonry structures that have suffered damages. A rough distinction can be made among the traditional and the modern ones. Traditional techniques employ the materials and building processes used originally for the construction of ancient structures. Modern techniques aim at more efficient solutions using innovative materials and technologies [13, 22, and 23].

It was seen at the preceding section that it is possible to minimize the structural deficiencies described earlier though modifications intended to combine bracing with diaphragm action at the levels of the floors and roof.

Bracing can be achieved through the casting of a reinforced-concrete (RC) layer at the wall crowning that will form the support of the roof which will be replaced with a new one with the geometric characteristics of the old. Moreover, the timber beams of the 1st level floor will be strengthened (or replaced wherever necessary) and anchored with metal rods penetrating the walls and forming anchors at their external side. Bracing in the form of external and internal RC strips at the level of the basement floor is also proposed.

As regards safeguarding diaphragm action at the level of the roof support, this is achieved through the use of diagonally arranged steel elements, whereas at the 1st level it is considered sufficient to stiffen the timber floor with two layers of cress-crossing plywood. Finally, at the basement diaphragm action is safeguarded by replacing the timber floor with an RC slab 25 mm thick.

However, as discussed in the preceding section, in spite of the above modifications, the development of tensile stresses larger than the tensile strength of concrete cannot be prevented in localized regions of the walls. In such locations cracking may be minimized, or even prevented, if the tensile strength of the masonry is improved. The latter can be achieved through grouting that will also restore the continuity of the masonry which was disrupted by the formation of cracking. Alternatively (or concurrently), the walls may be covered both internally and/or externally with a suitable reinforced coating.

Conclusions

Historic traditional masonry structures are the cultural reflections of a society; they create a strong link between the past and the present by describing the economic, social and technical situation of the ancestors of a society.

It has become widely accepted to classify any buildings put up without a formal design process as traditional masonry, Their form, plan and method of construction simply follows tradition aspects. Traditional Urban Residential Masonry (TURM) buildings in Athens are usually made of rubble (cobble) natural stones and a large volume of low strength lime mortars, while their floors/roofs are made of timber elements.

Visual inspection of a historical building in Athens of the late 18^{th}-early 19^{th} century revealed structural deficiencies, such as lack of diaphragmatic action and bracing. These deficiencies were considered to underlie the damage suffered by the building during its life span to date. The structural damage occurred primarily in the form of vertical and inclined splitting of the bearing walls and fragmentation of the mortar used to bind together the masonry stones. Although aggravated due to lack of maintenance, environmental conditions, aging etc., the damage was considered to have primarily resulted from past seismic excitations due to the structural deficiencies of the building.

The above hypotheses regarding the weakness of the structural system of the building when subjected to earthquake action were confirmed from the results of a comparative numerical study carried out through the use of a finite-element package widely used in current design practice in Greece. It was in fact shown that safeguarding diaphragmatic action and bracing results in a significantly improved structural performance, with cracking developing in a considerably smaller number of localized regions.

Practical recommendations regarding the manner in which the above structural improvements can be materialized were proposed and suggestions for minimizing the likelihood of cracking were made. Bracing can be achieved through the casting of a reinforced-concrete (RC) layer at the wall crowning that will form the support of the roof which will be replaced with a new one with the geometric characteristics of the old. Moreover, the timber beams of the 1^{st} level floor will be strengthened (or replaced wherever necessary) and anchored with metal rods penetrating the walls and forming anchors at their external side. Bracing in the form of external and internal RC strips at the level of the basement floor is also proposed. As regards the improvement of the tensile strength of the masonry, this can be achieved through the use of grouting (which will also restore the continuity of the masonry which was disrupted by the formation of cracking) and the covering of the walls; both internally and/or externally, with a suitable reinforced coating.

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