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STRUCTURAL ANALYSIS AND RETROFITTING OF THE "TZOTZAS" BUILDING IN KASTORIA, GREECE

C. Maraveas, K. Tasiouli, K. Miamis and Z. Fasoulakis

Structural Eng. Dept., C. MARAVEAS PARTNERSHIP
Athens, Greece
e-mail: c.maraveas@maraveas.gr

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Abstract. *This paper presents the assessment of the capacity and the improvement of the seismic performance of a historic masonry structure in Kastoria, Greece. The study posed several challenges, as there were uncertainties regarding the properties of the materials and the global response of the structure. The load-carrying system consisted of various structural element types, i.e. timber-framed masonry and timber or steel frames, which implicate non-linearities/complexities regarding the response of the structure in both directions. In addition, the fact that the building has been strengthened in the past complicated its assessment. The procedure followed for the proposal of the necessary improvements was in accordance with the European Standards (Eurocodes) related to earthquake, masonry, timber and steel design. Moreover, dynamic time history analyses were performed for comparison purposes. The results verify the enhancement of the building's behavior under earthquake loading.*

1 INTRODUCTION

It is widely recognized that the cultural characteristics of historical structures are of paramount importance. Therefore, when proposing structural modifications to such structures, it is necessary to provide applicable solutions which take into consideration the impact on aesthetics. Moreover, the structural design, among other parameters, should conform to reversibility and compatibility principles. The pathology identification and restoration methods used for historical structures should be separated from those pertaining to conventional or modern structures [1-3].

For the majority of historical structures, a lack of evidence pertaining to the material properties and the structural system is generally observed. Furthermore, specific types of loads (e.g. earthquakes) or environmental exposure result in accumulation of damage and deterioration of masonry structures. Although it has been stated that traditional lateral load-resisting systems were conceived to sustain seismic actions [4-5], in most cases historic structures do not meet the demands specified in the modern codes, especially in high seismic regions.

As a consequence, various methods have been developed to adequately evaluate the response of structural systems consisting of stone masonry [1-5] or timber-framed masonry [6-7]. In addition, code specifications [8-13] cover most approaches that can be applied for the seismic evaluation of complex structures, considering also the uncertainties in the distribution of the horizontal load along height.

This paper presents the structural appraisal/strengthening proposal of an existing historical masonry building. Firstly, a reliable assessment of the load-carrying capacity was performed. To this end, a finite element model was created (with the aid of commercial software [14]) to evaluate the static and dynamic response of the structure. More importantly, a realistic simulation of the mechanical properties was essential to minimize material uncertainties. Accurate modeling of the geometry and successful selection of various analysis and design parameters contributed to the identification of structural inadequacies in a major way. Afterwards, necessary strengthening modifications were proposed, based on non-destructive techniques.

2 PERFORMANCE AND DESIGN PRINCIPLES

The construction of the studied structure dates back to the second half of the 19th century. The "Tzotzas" building (named after its owner) is a traditional mansion house, known as "Archontiko" (in Greek), which contributes to Greek heritage. It has been declared as a "listed" building by the Greek ministry of culture. Therefore, it is mandatory that the retrofit proposal does not to alter its traditional architectural characteristics. Photographic evidence of the exterior of the building (facade) is presented in Figure 1.

Figure 1: Photographs of the "Tzotzas" building.

2.1 Geometry description

The total plan of the building is approximately 220m^2 (rectangular layout with approximate dimensions of 14.35m by15.55m). It consists of three floors with a height of 2.85m, 2.80m and 3.35m respectively and a 2.85m high roof. The ground floor consists of stone masonry only, while the structural system of the above floors includes external stone masonry walls (Z direction), external masonry infilled timber walls (X direction) and timber frames in both directions inside the building. Moreover, there is an octagonal steel framed core, from a previous retrofit of the structure, which carries the major portion of the loads from the floors and the roof.

The geometries of the ground floor and the second floor are schematically presented in Figure 2. The RHS columns of the octagonal steel core are also illustrated. These are based on the masonry walls (pin connections) of the ground floor, except for one which is based on the ground. The thickness of the stone masonry varies from 0.4 m to 0.8 m; the timber framed masonry thickness is 0.2 m. Furthermore, local areas of stone masonry walls with small thickness (less than 0.3m), were simulated as openings and only their weight was accounted for in the analysis.

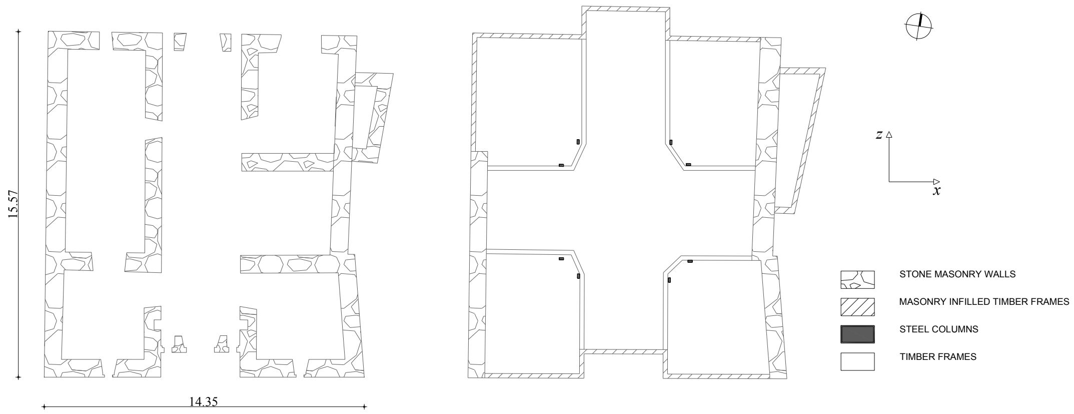

Figure 2: Plan of the ground floor (left) and 2nd floor (right).

The floors and the roof consist of closely spaced timber beams, some of which have been strengthened in flexure by steel plate attachments. Diagonal bracing from steel laminates has been tied to the timber floors, in order to increase their stiffness and ensure rigid diaphragm behavior. In addition, the stone masonry walls of the ground floor have been locally reinforced with cement-based grouts. Two interior views of the building are presented in Figure 3.

Figure 3: Interior views of the 1st (left) and 2nd (right) floor. The timber framed masonry is clearly visible.

2.2 Eurocode specifications

In order to perform the capacity assessment, the specifications of the Eurocodes were used, namely EC3 [10] for the steel members, EC5 [11] for the timber members, EC6 [12] for masonry and EC8 [13] for the seismic analysis of the structure. The imposed loads followed the specifications of EC1 [9].

On the basis of EC8, an inelastic response spectrum was adopted for soil type C (soil factor S=1.15 and characteristic response spectrum periods T_B=0.2 sec, T_C=0.6 sec, T_D=2.0 sec), design ground acceleration a_g=0.16g, importance factor γ_I=1.0 and behavior factor q=1.50 (unreinforced masonry). Two analyses were performed, namely a simplified response spectrum analysis (lateral force method) and a modal response spectrum analysis, with a damping ratio of 5%.

In addition to the self-weight of the structure, distributed dead loads of 0.5 kN/m^2 and 1.0 kN/m^2 were considered for the floors and the roof, respectively. The floor live load was 3 kN/m^2 (area Category C1 per EC1 [9]). Moreover, a 1 kN/m^2 live load was imposed on the roof, which was not combined with other actions (repair load). Snow and wind loads were applied according to the regulations of EC1 [9].

The mechanical properties of stone masonry walls were determined from laboratory tests on samples taken from the field. More specifically, tests on the constituent materials (stone and mortar) were carried out. Furthermore, the lowest class for timber members (C14) according to EC5 was selected for the analysis (conservative assumption). The allowed mean compressive strength for the brick masonry was selected equal to 2.5 MPa, which is the minimum encountered in the literature [2]. The structural steel was categorized as S235. It should be noted that, for the seismic analyses, the stiffness of the stone masonry was reduced to 50% of its initial value [13], to account for the influence of cracking.

For the design checks the following load combinations were used, as defined in EC0 [8]. Eq. 1 indicates the combination of actions other than seismic:

$$S_d = \sum_j \gamma_{G,j} G_{k,j} + \gamma_{Q,1} \psi_{0,1} Q_{k,1} + \sum_i \gamma_{Q,i} \psi_{0,i} Q_{k,i} \tag{1}$$

where "+" implies "to be combined with", the summation symbol "Σ" implies "the combined effect of", $G_{k,j}$ denotes the characteristic value "k" of the permanent action j and $Q_{k,i}$ refers to the characteristic value "k" of the variable action i. The seismic actions were combined according to Eq. 2 [13], while the combination of the horizontal components of the seismic action is defined in (Eq. 3):

$$S_d = \sum_j G_{k,j} + E_d + \sum_i \psi_{2,i} Q_{k,i} \tag{2}$$

$$E_d = \pm E_X \pm 0.30 E_Z \text{ or } E_d = \pm E_Z \pm 0.30 E_X \tag{3}$$

where E_d is the design value of the seismic action for the two horizontal components (longitudinal and transverse), respectively and $\psi_{2,i}$ is the combination coefficient for the quasi-permanent action i, taken equal to 0.60. E_X and E_Z are the horizontal seismic actions along the X and Z direction of the structure, respectively.

In addition, serviceability requirements were taken into account, by combining actions (Eq.4) according to EC0 [8]:

$$S_d = \sum_j G_{k,j} + Q_{k,1} + \sum_i \psi_{0,i} Q_{k,i} \tag{4}$$

3 NUMERICAL ANALYSES

3.1 Finite element model

The existing structure was simulated as a spatial model, using beam elements for the beams / columns and plate elements for the walls. In Figure 4, a 3D view of the global model is presented, while the octagonal steel core and the stone masonry walls are depicted in Figure 5.

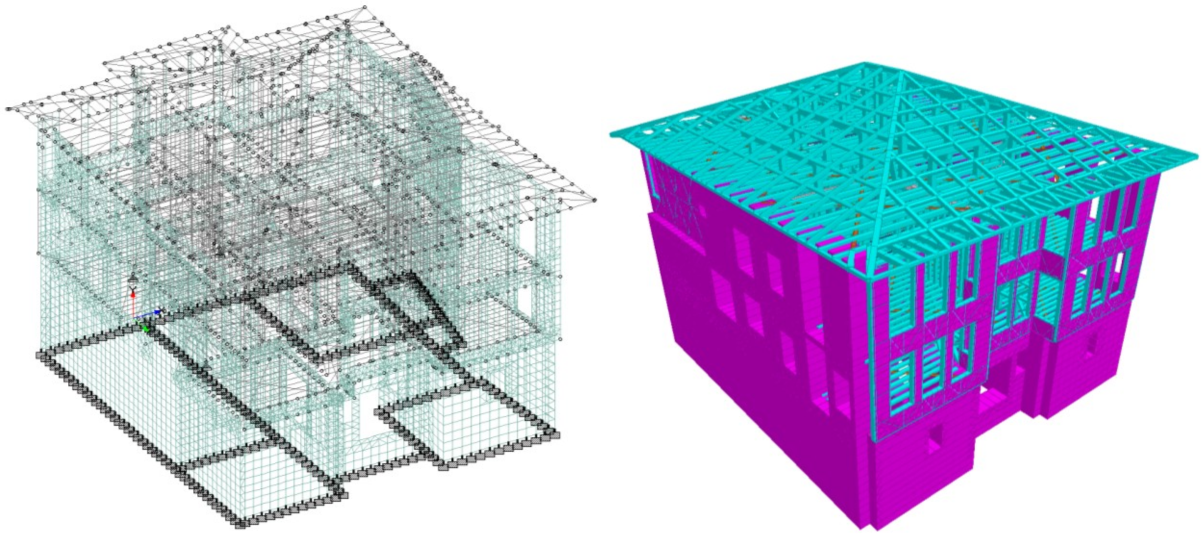

Figure 4: 3D view of the model used to simulate the structure.

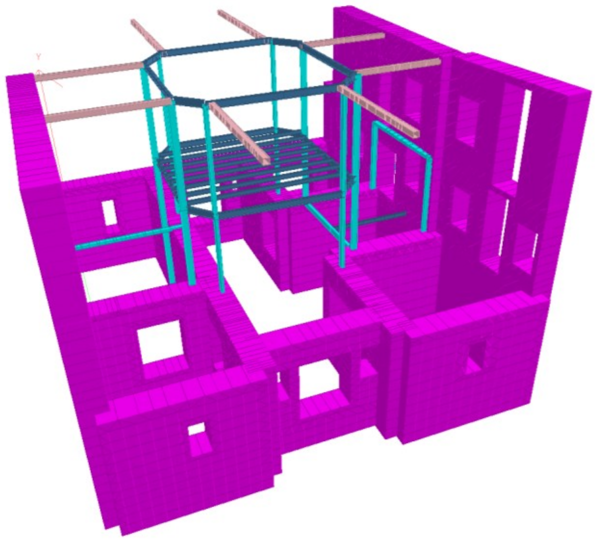

Figure 5: 3D view of the steel core and the masonry walls of the existing structure.

For the simulation of this complex structural system, several issues had to be taken into consideration. In particular, the slip at the nailed ends of timber beams and the cracking and unstucking of masonry infills result in non-linear behavior. For this purpose, the diagonal wooden members were simulated to resist compression forces only (compression members), whereas the stiffness of the brick masonry infills was reduced to 20%, according to the literature [7]. Furthermore, the horizontal diagonal steel laminates were not allowed to buckle; they were simulated as tension members only.

The aforementioned non-linearities have significant effects on the earthquake resistance of the structure. For the above reasons, it was essential to perform a simplified response spectrum analysis.

3.2 Dynamic time history analysis

For comparison purposes, the seismic action was also investigated in terms of ground acceleration time histories. To this end, acceleration recordings of two earthquake events in Greece (Aigion (1995) and Athens (Syntagma, 1999)) and that of Northridge (1994) were considered. The original accelerograms were scaled, in order to adjust the ground motion records to the spectrum defined in the design code (target spectrum).

However, according to the literature [15], there are no uniform criteria for record scaling. EC8 [13] recommends that artificial records be generated from the scaling of at least three real records and also sets other scaling requirements. These were fulfilled for the scaling of the original accelerograms selected for the purpose of this study.

The original (black colored) and scaled (red colored) accelerograms, which correspond to the studied seismic motions (both horizontal directions), are presented in Figure 6. The response spectra for the Aigion seismic event are compared in Figure 7.

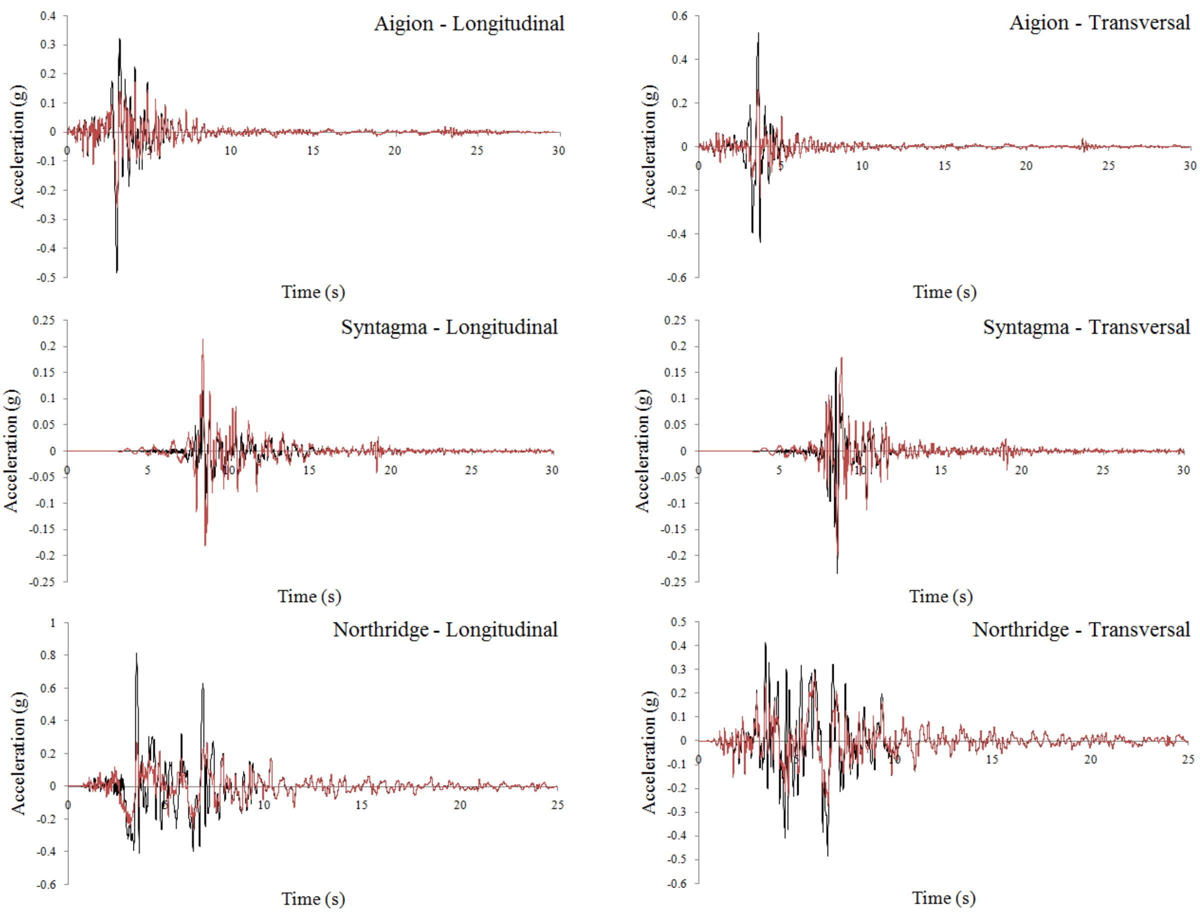

Figure 6: Time histories that were used for the dynamic analyses: original (black colored) and scaled (red colored) accelerations.

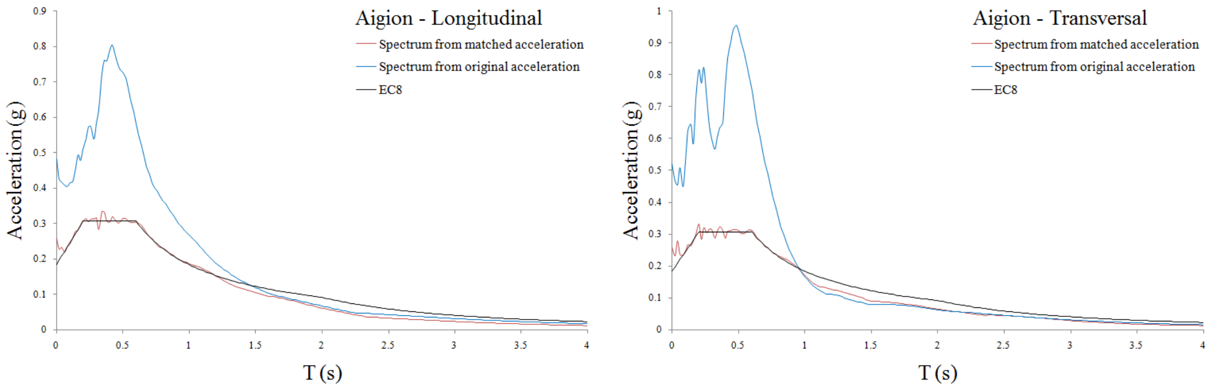

Figure 7: Response spectra derived from the Aigion earthquake (1995); comparison with the corresponding one from EC8 a) for longitudinal and b) for transverse direction.

4 STRUCTURAL EVALUATION OF THE EXISTING BUILDING

Particular attention was given to the structural adequacy of masonry. In order to detect the critical areas, the stress distribution of the overall structure was firstly obtained from all load combinations. Subsequently, the code-based criteria were applied to evaluate the resistance (flexural, shear and axial) of masonry and perform the required adequacy checks. Results showed that, for seismic action, capacity was exceeded in many locations. In certain cases, demand exceeded resistance by as much as five times. It must be mentioned that, in many local areas, high tensile stresses were observed during the seismic excitation of the structure. The stress distribution and displacements of the existing structure (earthquake loading) are schematically illustrated in Figures 9a and 10a.

Regarding the timber structural elements, flexural failures and excessive deformations were observed in a significant portion (approximately 35%) of the floor and roof beams which are not strengthened by steel plates. On the contrary, steel elements were proven to be adequate for both static and dynamic loading.

A general observation is that the seismic loads are resisted mostly by the masonry walls, with insignificant participation of the beam elements/frames to the global stiffness.

5 STRENGTHENING PROPOSAL

Based on the analysis results and the global response of the existing structure to the applied loads, a strengthening strategy for improving its performance is proposed.

The global rigidity of the structure, especially in one direction, is enhanced, in order to receive earthquake forces. More specifically, along the X direction (where the maximum out of plane moments were observed) four timber frames are added (Figure 8) to improve the resistance to lateral forces in that direction. This will result in a reduction of the out-of-plane bending for the masonry walls. Strengthening of the floor-diaphragms (at all levels) is also proposed, in order to increase their rigidity and contribution to the horizontal stiffness of the structure.

Additionally, to increase the stiffness of stone masonry, cement grouts will be injected in entire wall regions, while the tensile strength at local areas (mainly above openings) will be increased by attaching fiber reinforced polymer (FRP) composites on the surface masonry. Appropriate connection modifications are suggested to ensure that the diagonal timber elements are capable of carrying tensile loading.

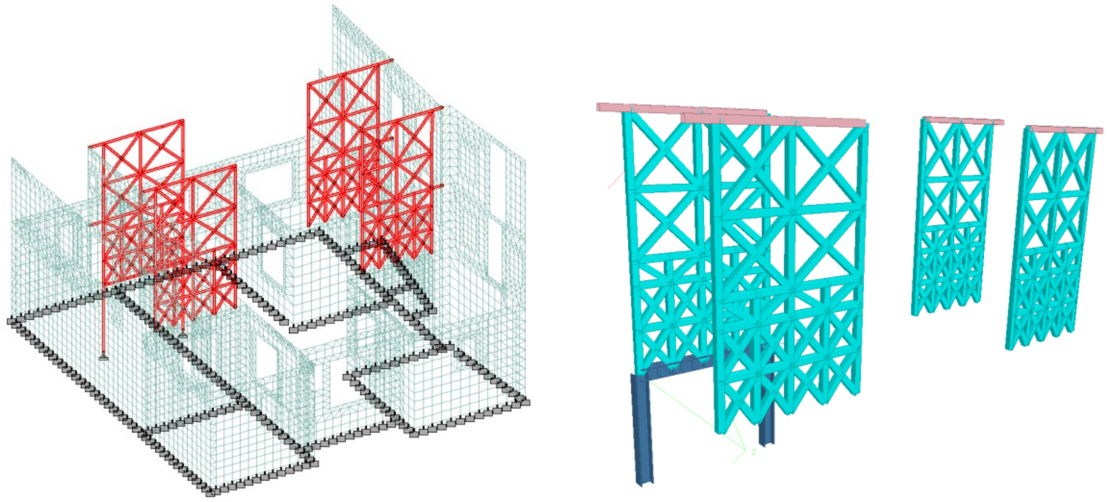

Figure 8: Strengthening proposal (addition of timber frames) for stiffness increase along the X direction.

Other techniques that would provide greater bending resistance, without intervening with the architectural characteristics of the building (minimally-invasive options), have been investigated by researchers [16-17]. These involve the use of prestressing tendons located at both sides of the walls or implementation of vertical oriented near surface mounted (NSM) FRP strips. Even though both solutions are efficient, the latter would involve cutting through the stone units, which is not allowed in this case, due to architectural constraints.

6 COMPARATIVE RESULTS

An overview of the results is illustrated in the figures below. The stone masonry deformations, resulting from earthquake loading combination along the X direction (simplified response spectrum analysis), are presented for both the initial (Figure 9a) and retrofitted (Figure 9b) structure. Figure 10 shows, respectively, a comparison of the principal major stress distribution from the aforementioned load combination. It should be noted that high stresses develop in the stone masonry walls of the existing structure, especially above the ground floor level, due to the existence of out-of-plane moments. In addition, the relief of the tensile stresses is evident in the retrofitted building (Figure 10b), with the maximum stresses being reduced from 1.2 MPa to 0.3 MPa.

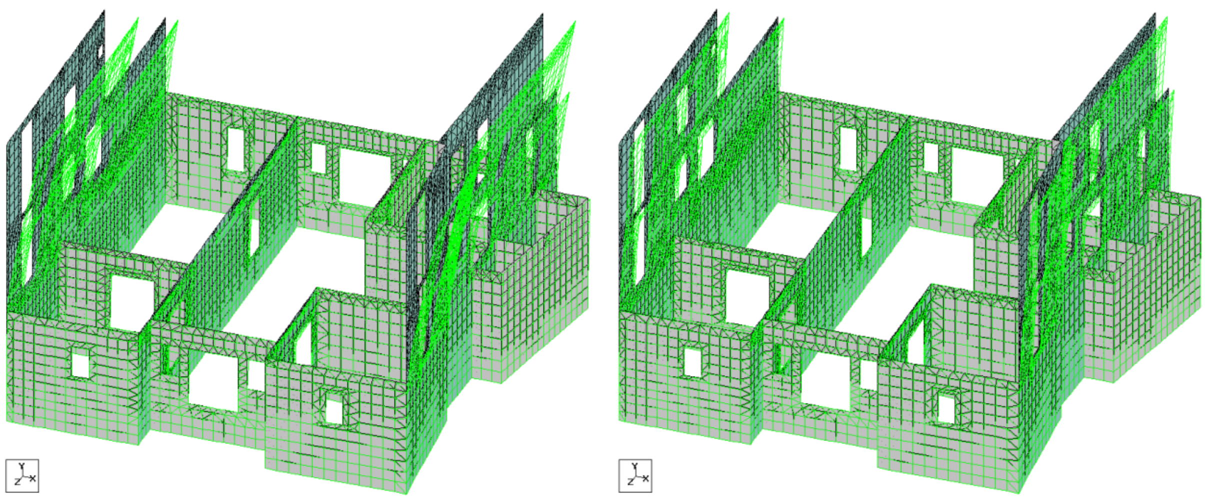

Figure 9: Stone masonry deformation under earthquake loading along the X direction , a) before and b) after strengthening.

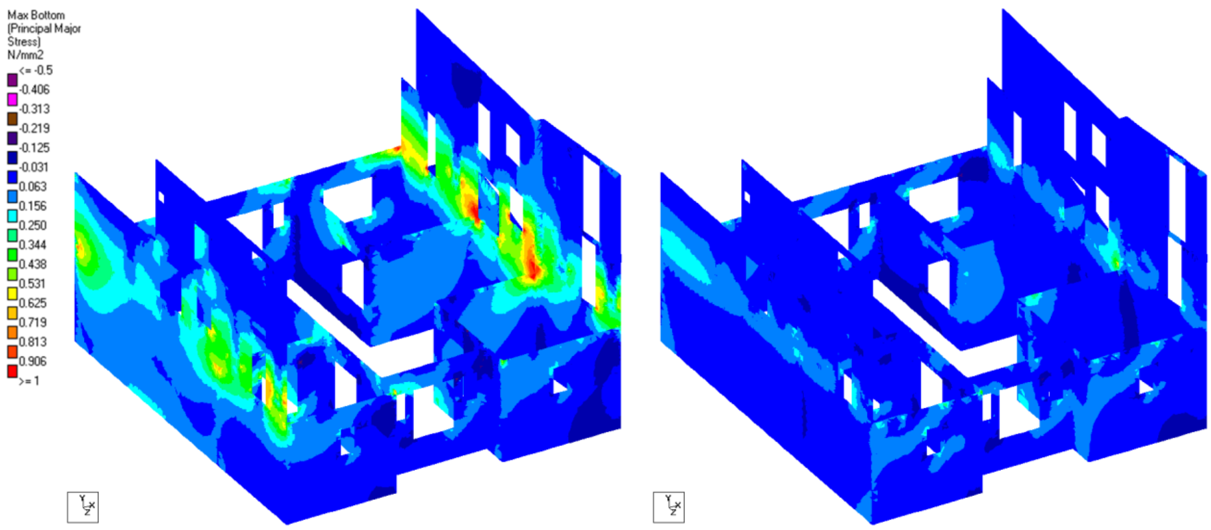

Figure 10: Major principal stress distribution under earthquake loading along the X direction, a) before and b) after strengthening.

In Table 1, a comparison of various results obtained from the analysis of both the existing and the retrofitted structure is shown. The results refer to the axial force of a typical timber column, the out-of-plane and in-plane moments (first and second value respectively) of a stone masonry segment (of a critical, high-stressed area), the maximum base shear force along the X direction (first value) and Z direction (second value) and the displacement of an edge node at the top of the masonry wall.

It should be noted that 600 modes were considered in the dynamic analysis of the structure. The modal mass participation exceeded the 90% of the total mass for both the existing and the strengthened structures. It is also observed that greater out-of-plane moments developed in the masonry walls for the Athens (1999) time history analysis, most probably because of inaccurate scaling near the region of the low periods (in which most of the mass is excited). More details regarding the modal response spectrum analysis are presented in graphical form in the figures below.

Table 1: Comparison of forces and displacements for the studied seismic analysis cases.

	Analysis	Axial Force Timber column (kN)	Moment Masonry (kNm)	Base Shear Force (kN)	Displacement at the top (mm)
Before Retrofit	Lateral force	22	338/1669	3282/3282	12.09
	Modal	34	363/2474	2212/1878	8.89
	Aigion	24	451/498	2085/2139	13.12
	Athens	49	414/2186	2139/1643	12.76
	Northridge	28	429/886	2164/1881	14.12
After Retrofit	Lateral force	79	246/1844	3382/3382	4.39
	Modal	97	197/1862	2589/2201	5.48
	Aigion	134	142/619	2980/3056	7.74
	Athens	120	324/1448	2489/2579	6.65
	Northridge	140	175/582	2916/2834	8.93

In Figure 11a, it can be observed that the modal frequencies of the strengthened structure are higher than those of the existing one, due to stiffness increase. The decrease of the first period (from 1.19 sec to 0.34 sec) is remarkable. The effect of stiffness enhancement is also easily noticed in Figure 12, in which the increase (more than 50%) of the total base shear is obvious. Moreover, a step-wise increase of the base shear is observed for the strengthened structure (Figure 12), due to the high mass participation factors of certain modes (6, 53, 129).

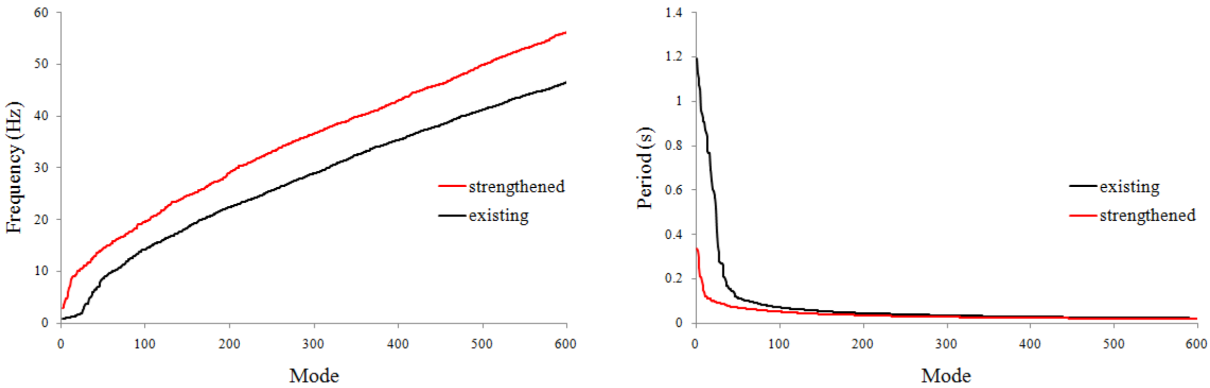

Figure 11: Comparison of mode frequencies between the existing and the retrofitted structure.

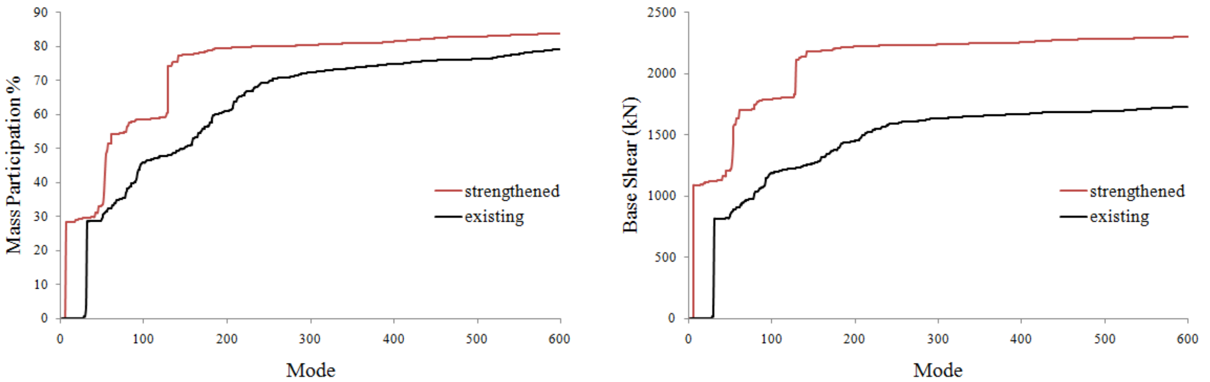

Figure 12: Comparison of base shear forces between the existing and the retrofitted structure.

7 CONCLUSIONS

The pathology and retrofit of an existing historical masonry building in Kastoria, Greece, was presented here. For the assessment of the structural behavior, the European standards were implemented, and analyses were carried out via appropriate finite element simulations. It must be highlighted that, in order to take into consideration the various uncertainties pertaining to the structural system or the materials, conservative assumptions were made.

The challenge of this study was to provide the required strengthening for the structure with the minimum impact on its aesthetics. The proposed modifications included the addition of a timber framed structural system, floor diaphragm improvements, FRP-strengthening and cement grout injections in the masonry walls. The results verify the reduction of the stresses in the stone masonry walls and the increase of stiffness.

In conjunction with the modal response spectrum analysis, time-history analyses were carried out for comparison purposes. The actual recordings used were scaled, in accordance with the Eurocode specifications, to create artificial accelerograms. The retrofitted structure displayed better response to seismic loading and complies with the regulations of the Eurocodes.

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