

Seismic Assessment of Traditional Houses in the Balkans— Case Studies in Xanthi

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Abstract- Field evidence from recent strong earthquakes in the Balkan peninsula has reinforced the belief that the Local Historical Structural System of the region, known as timber-laced masonry construction, shows dependable resistance and significant resilience under earthquake action, rarely reaching the state of catastrophic collapse that typifies other old buildings such as brittle (poorly constructed) reinforced concrete structures. This paper examines the seismic behavior of example traditional houses from the old city of Xanthi, by using simulation models to study the dynamic characteristics of the structures, exploring the possibilities of restoration through pertinent modification of the connecting material properties. The effectiveness of the interventions is gauged through a comparative evaluation of the results obtained from the initial and the post-intervention models, with particular emphasis placed on the global modification of the dynamic characteristics and the anticipated mitigation of localized deformation demands throughout the structures. The variability of the results owing to parameter changes enables a first assessment of the uncertainties associated with the actual details, geometry and state of materials and the mechanical properties thereof, on the dynamic properties and dependable deformation capacity of the structure at the life-safety performance limit state.

Keywords- *Timber-Laced Masonry Construction; Local Historical Structural Systems; Non-invasive Interventions; Seismic Behavior; Simulation Models*

I. INTRODUCTION

Throughout the Balkan Peninsula and Asia-Minor abounds with a certain structural system comprising timber-laced masonry which has been used almost exclusively throughout the ages for construction of traditional family dwellings up to about sixty years ago. This structural system, known by various local trade names, actually draws its origin from the ancient Minoan times; timber lacing is used as a metaphor in the Bible, where the strength of soul secured by faith at times of trial, is compared to the strength of houses imparted by timber lacing in the event of an earthquake. In Roman times timber-lacing was known as *Opus-Craticium* [1], [4], and was considered as a form of low-cost construction: examples of this type of construction are found in houses that survived the eruption of Vesuvius of 78 A.D. (Such examples are still standing in the excavated site of Herculaneum near Pompei in the greater region of modern Naples).

Timber-laced masonry was displaced by the widespread adoption of reinforced concrete around the middle of the previous century, on the apparent premise that the new technology at the time held promise for better, safer structures. The experience over the past fifty years has led the profession to revise this unquestioned optimism: field experience and evidence illustrate that well-maintained, well-tied traditional historical dwellings not only perform well in terms of service life conditions, but also show impressive resilience, deformation capacity and strength to earthquake load, rarely reaching the conditions of catastrophic collapse which have been seen in older reinforced concrete construction, which is deemed brittle by today's earthquake standards [2], [3], [5], [6], [7], [8], [9]. In light of these observations, the initiative for preservation of historical construction as a bearer of important cultural heritage for the region has gained renewed significance and priority.

In the framework of a comprehensive preservation strategy, a decisive step is the thorough identification of the internal force path implicit in the formation of the structural system: this is useful not only both for assessment of the deficiencies accumulated by deterioration due to ageing and systemic inadequacies, but also for evaluation of the effectiveness of alternative options for intervention and restoration. Here, simulation through Finite Element modeling may enable insightful interpretation of the global building workings and load transfer. (The relevance of this type of analysis may be diminished at a local level owing to the inherent uncertainties with regards the accuracy of modeling of member-to-member interaction, connectivity between structural components, and the actual mechanical properties of the constituent materials and material phases.)

Computer-aided modeling for seismic assessment of a typical historical structural system used in construction of urban residences up to the early 1900's is used in the present paper as a tool in order to gain understanding into the structural function of three different traditional building structures taken from the core of the old-town of Xanthi in Thrace, N.E. Greece. There is a particular interest in verification of the potential utility of computer modeling in all phases of engineered preservation technologies of such structures, including assessment, restoration, rehabilitation and reuse, by enabling improved understanding of the structural function of the system of construction, but also in guiding the retrofit strategy through informed targeting of the retrofit objectives. By definition, when referring to preservation and restoration of structures that convey a historical value for the community, a certain type of compromise must be negotiated between the need to preserve the structure as

a surviving heritage exhibit, and the need to secure the safety of its inhabitants against loss of life or property in the event of a significant earthquake.

II. DESCRIPTION OF THE LOCAL HISTORICAL STRUCTURAL SYSTEM

With regards to the old-town of Xanthi the traditional timber-laced masonry buildings comprise a vital portion of its historical fabric, identifying the city (Fig. 1). Primary construction materials are stone (natural blocks, usually in the foundation and the lower floors, and man-made solid clay-bricks in the upper levels) and timber (such as timber structural elements, floor ties, timber lacing elements, etc.), often tied in strategic locations with iron clamps and ties to improve member connectivity (Fig. 2). The art of construction with these materials, with the builder functioning in a complex role of an empirical engineer, evolved over the years in an improved system whose performance has stood the test of time in a region of active seismic activity.



Fig. 1 Typical Samples of traditional houses in the historical center of the city of Xanthi

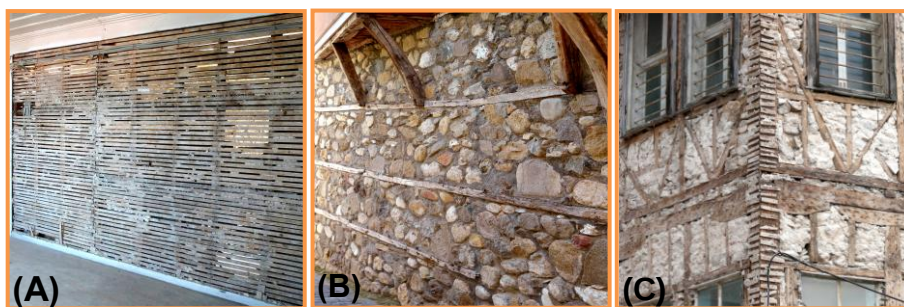


Fig. 2 (A) Uncoated timber weaved wall, (B) Stone masonry timber laced wall, (C) Timber laced wall

The traditional system studied in the present paper combines a stiff load-bearing timber-laced stone-masonry wall system for the lower floor, with the upper floor made of infilled timber frame (this was *opus craticium* in roman times [1], [4], but later became known as *fachwerk*, *chatmas*, or *half-timbered system* in the various parts of Europe and Asia where it was found [2], [3], [5], [6], [7], [8], [9]). The load bearing structure comprises stone masonry foundation with connecting mortar; in some cases, to improve the redundancy of the foundation particularly in compliant soils, a supporting substrate layer made of treated timber is provided under the foundation. Load bearing walls in the first floor including the major interior divisions are made of stone masonry with lime-type connecting mortar and carefully tied timber-laces. Frequently, the connections between timber laces reveal many techniques borrowed from the local ship-making industry. Secondary interior dividing walls were made of light timber-woven gages coated with a lime-based mortar (mud-based mortar was used in poorer dwellings), usually reinforced with straw or animal hair; this is also evident in ancient monuments, but its use is found throughout southern Europe and Asia. Therefore, in

construction of a traditional house these three structural forms are used selectively, combined in an overall structural system and expanded in space following well-defined rules depending on their weight, load-carrying capacity, and stiffness. The system described represents the historical city quarters of Xanthi in its entirety and is marked for the carefully organized geometry (meant to avoid irregularities and large ratios of slenderness in plan or in height, and to exploit the benefit of symmetry, the optimal distribution of mass, stiffness and deformation compliance).

One important distinguishing characteristic is the relative compliance of connections between members, despite their robustness. This means that all members participate to lateral load resistance as there are no singularly stiff paths for the seismic load to reach the ground. Therefore, structures of this class survive the earthquake by practically not “taking”, or “attracting” significant forces; instead, they comply with a controlled fashion, developing wide-spread, low-intensity damage, particularly owing to the frictional interactions between the various components which comprise different materials and types of elements. This creative combination of contact-based resistances, contributes to diffuse the strain energy imparted by the earthquake throughout the building (Fig. 3).

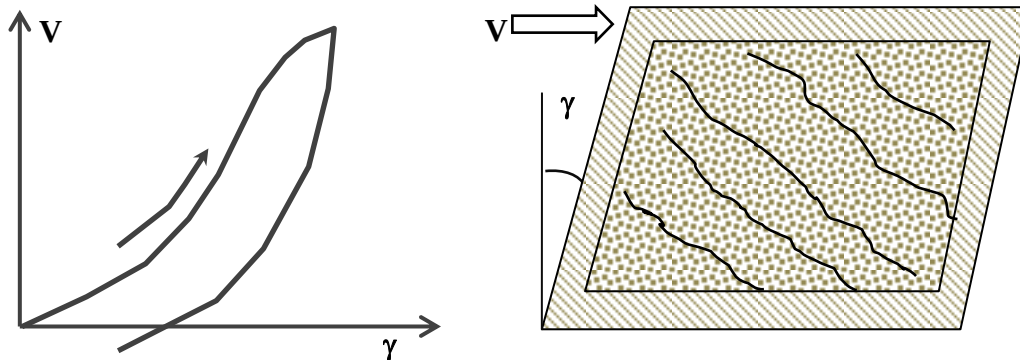


Fig. 3 (a) Continuous hardening behavior of encased masonry as distortion forces further locking and friction between components thereby increasing the effective stiffness. (b) timber-laced panel undergoing lateral distortion

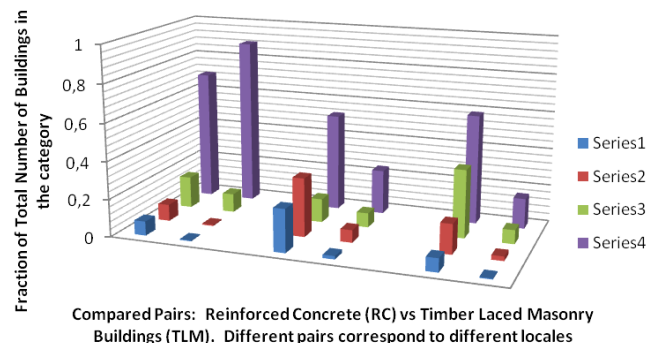


Fig. 4 Comparative Damage Assessment for Residential Buildings (Greater region of Golcuk - Kocaeli (Turkey)). Series 1 corresponds to heavy damage or collapse, Series 2 to Moderate Damage, Series 3 to Light damage, whereas Series 4 to No damage. Each bar represents the number of buildings in the category, normalized by the total number of buildings in the category (e.g. number of R.C. buildings that sustained heavy damage, normalized by the total number of R.C. Buildings in the locale under consideration.) Each pair of bars organized along the horizontal axis represents RC structures to the left, and TLM Structures to the right. The three groups of pairs in the horizontal axis refer to three different locales

Through this function, the absorbed energy is consumed by the extensive internal frictions that develop along the interfaces and the connections of the individual elements, materials and structural forms, leading to a large value of *equivalent* or, *effective* damping (quantified by the area enclosed by the hysteresis loops as shown in Fig. 3. Note the peculiar characteristic of a concave-upwards resistance curve representing the typical behavior of a timber-laced panel in shear, which represents the increasing degree of “locking” and gradual mobilization of resistance with increasing shear deformation). Energy dissipation through internal friction is a characteristic mechanism for all three structural forms described (laced masonry, infilled timber frames and timber-woven walls). The process of energy dissipation may continue over a large range of deformation capacity prior to failure. Furthermore, the system is self-adjusting, since failure of the individual building components does not influence to a critical extent the other members of the system. This type of behavior to seismic loads is enhanced by the diaphragm action of the floor system, to a degree that depends on the robustness of its structure and the manner of its connection or attachment to the load bearing walls. In many of these buildings the roof timber truss is elastic and does not contribute by diaphragm action to the structure.

Note the above observations and interpretations concern systems where the structural function has not been altered or reversed by arbitrary interventions by either users through additions or removals of building components, inadequate or misguided retrofit of older earthquake-induced damages, or lack of maintenance. All of the above contribute to the notable performance of the traditional building system to seismic action as documented in reconnaissance studies cited in the literature ([3], [5], [6], [7], [8],

[9]). These present comparative evaluations of load-bearing, timber-laced masonry with the parallel performance of lightly reinforced concrete construction in major seismic events which, upon collapse, have proven lethal to human life.

With regard to the objective of seismic upgrading, the need to secure a level of protection to the users of a heritage building under accidental actions that is consistent with contemporary perceptions of acceptable risk, is potentially orthogonal to the spirit of international treaties for preservation and non-invasive restoration of important monuments. These usually address both the use, accessibility, appearance of the edifice, but also the structural function of the various components (such as foundations, redundancy for stress redistribution, compliance of the individual elements and connections thereof, reversibility of the intervention, compatibility in terms of compliance and physical characteristics of the new materials with the originals that serve as a substrate, etc.). A determining factor is the layout of the building itself and its state of deterioration and damage.

Systematic evaluation of the implications of major seismic events in neighboring countries that also have several examples of timber-laced construction confirms this favorable preconditioning of the traditional structural system when this has been properly constructed (see Fig. 4 which summarizes observed damages in R.C. buildings and in traditional, Timber-Laced Buildings during the 1999 Izmit - Kocaeli earthquake with a 7.4 Magnitude on the Richter scale); this, provided that past interventions or previous damages did not either compromise the integrity of the structural system or counteract the essential principles of the structural system described, which secure the seismic resilience and toughness of the system.

To demonstrate the efficacy of non-invasive rehabilitation protocols, their performance as a means of modifying the dynamic response and the seismic demands of the structural systems on which they are used, is evaluated through modal analysis of a number of representative traditional houses from the historical center of Xanthi. These are referred to hereon as Buildings A, B and C for convenience. Although built with similar methods of construction, each one of the three structures possesses distinguishing features that are called on here to test or to illustrate the issues of concern in the basic simulation methods employed for the purposes of assessment. Interventions considered in the analytical models are outlined below.

III. PROPOSED REHABILITATION WITH NON-INVASIVE METHODS

In the context of “preservation” of heritage construction international treaties generally require, (1) non-invasiveness of the intervention (i.e. it should disrupt or interfere with the traditional structural form and appearance), (2) reversibility (i.e. in light of the advent in technology and material qualities any intervention should be replaceable in the future with better solutions should such become available), (3) compatibility with the existing materials (particularly when new materials are placed on a brittle substrate such as stone or masonry, where chemical reactions or differences in compliance to stress between adjoined materials may cause disintegration of the more vulnerable old material). Enforcement of these requirements limit and restrict in practice the breadth and extent of the interventions. The only methods that are generally acceptable within this framework of restrictions are, (a) deep repointing of masonry, (b) replacement of deteriorated timber components, and (c) in some cases, homogenization of multi-leaf masonry walls. In these applications it is expected that materials chosen must be entirely compatible in terms of the rate of strength gain in time, stiffness, porosity, adhesion, while at the same time possessing resistance to biological and chemical corrosion agents.

In this work three different traditional houses are considered for seismic assessment of the implications of non-invasive interventions. The structures are selected to represent the range of structural systems encountered in the historical core of the city of Xanthi, and as such they generally possess different degree of frame-like and wall-like action and connectivity. The remedial plans considered include a combination of the following measures.

(a) Deep repointing in the perimeter stone walls (after removal from the joints of any old crumbled mortar and cleaning with water pressure), replacement of decayed timber lacing, repair of all timber connections.

(b) Fiber reinforced mortar coating on wall surfaces where functionally and aesthetically acceptable by the utility of the rooms and structure.

(c) Replacement of decayed timber in floor joists, diaphragms and trusses. Where needed intermediate timber beams are included in order to increase the floor stiffness. Replacement of flooring and roofing interlocking planks and dense nailing on the timber beams in order to secure composite action of the floor beams is also required.

(d) Removal of roof tiles, replacement or treatment of decayed timber elements, restructuring of the roof with water insulating membranes under the original or properly replicated tiles.

For the needs of computer assisted simulation of the rehabilitated structures, and with reference to the characteristic compressive strength of masonry defined according with EC 6 (2005) the material properties are prescribed as follows.

- Average masonry wall compressive strength is taken as, $f_{wk} = K f_b^{0.7} f_m^{0.3}$, whereas the corresponding short term elastic modulus, $E = 1000 f_{wk}$. Parameter K depends on the type of the stone-blocks (sb) and joining mortar (jm), whereas f_b is the compressive strength of the stone-blocks, and f_m is the average compressive strength of the joining mortar.

- Stone masonry comprising lime and granite natural stones, $f_b = 20$ MPa, $f_m = 2$ MPa, $K = 0.45$. Characteristic strength

$f_{wk}=4.5$ MPa, $E=4.5$ GPa, and density $\gamma=2200$ kg/m³.

- Solid clay bricks, $f_{wk}=2.25$ MPa, $E=2.25$ GPa, and density $\gamma=1800$ kg/m³.
- Voided clay bricks, $f_{wk}=2.25$ MPa, $E=2.25$ GPa, and density $\gamma=1800$ kg/m³.
- Timber lacing components, $E=10$ GPa, density $\gamma=800$ kg/m³.
- Repairing mortar, quality M10 (EN998-2), $f_m=10$ MPa, composition (lime-cement-sand volume ratios): 0.5:1.0:5.0 which is expected to give rise to an average compressive strength for the joining mortar to 3.55 MPa (based on the preceding strength equation), which is expected to impact the mechanical properties of the finished masonry wall as follows: $f_b=20$ MPa, $f_m=3.55$ MPa, $K=0.45$, $\delta=1$, and a 19% increase in the characteristic strength to $f_{wk}=5.358$ MPa, and $E=5.358$ GPa.

It is further assumed that the sectional area of the timber diaphragm elements will be doubled from the initial situation by interpolating additional joists, replacement of the flooring planks and addition of roofing layers in each floor (under the joists) and the overall stiffness of the diaphragm will be increased by means of strengthening all connections.

IV. DESCRIPTION AND SIMULATION OF BUILDING A

The building is depicted in Fig. 5. It is located in the Metropolis square, with a total height of 9.05 m from road level (i.e. 9.95 m from basement floor), with plan dimensions of 10.25 by 9.5 m, floor heights 2.4m (basement), 2.8m (first floor), 3.0m (second floor) and 1.75 m (roof). Simulation was based on the idealization of the two facades parallel to the seismic action in each case, using a series of planar frames linked by horizontal nonlinear springs that represented the diaphragm stiffness; nonlinear frame members were located on the centroidal axes of the piers, whereas connectivity was based on rigid links between members to account for the actual depth of the piers in the direction of seismic action (Fig. 6).

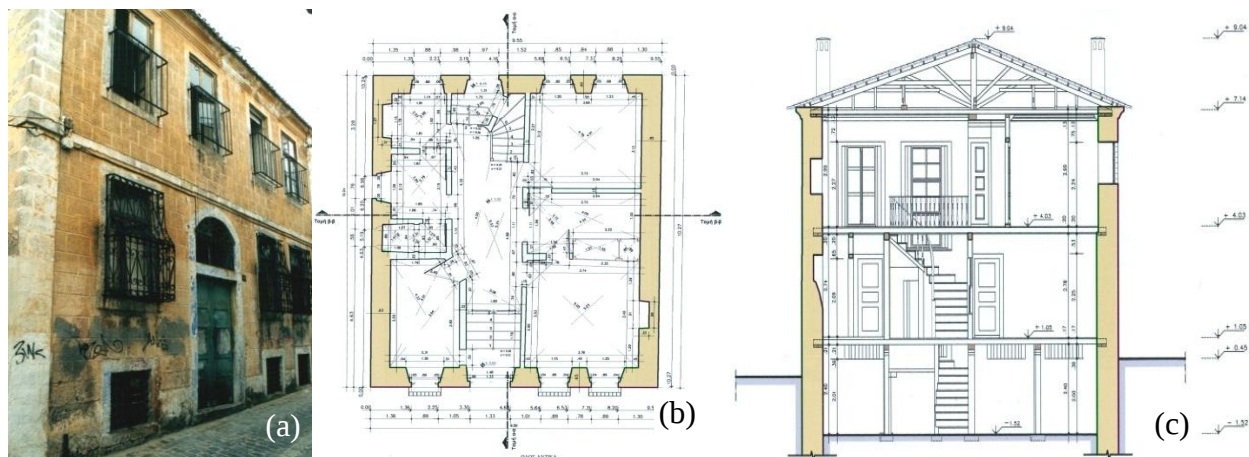
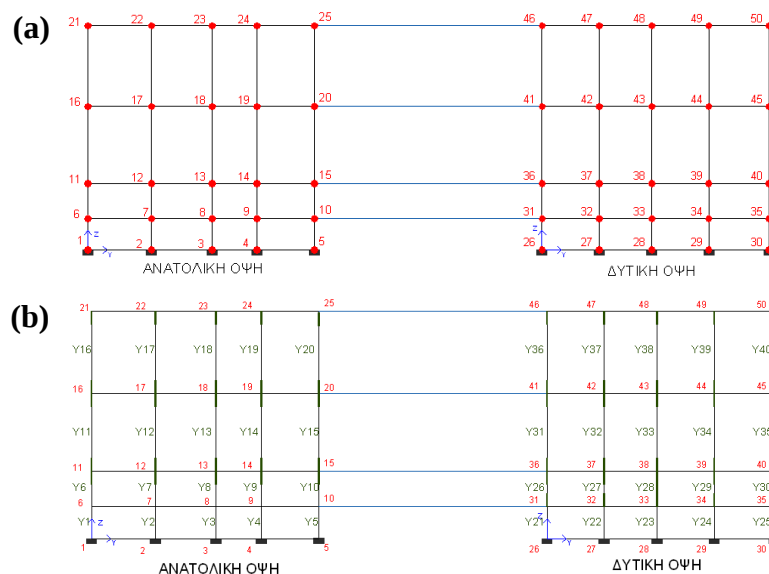


Fig. 5 Building A: (a) eastern view, (b) plan, (c) cross section



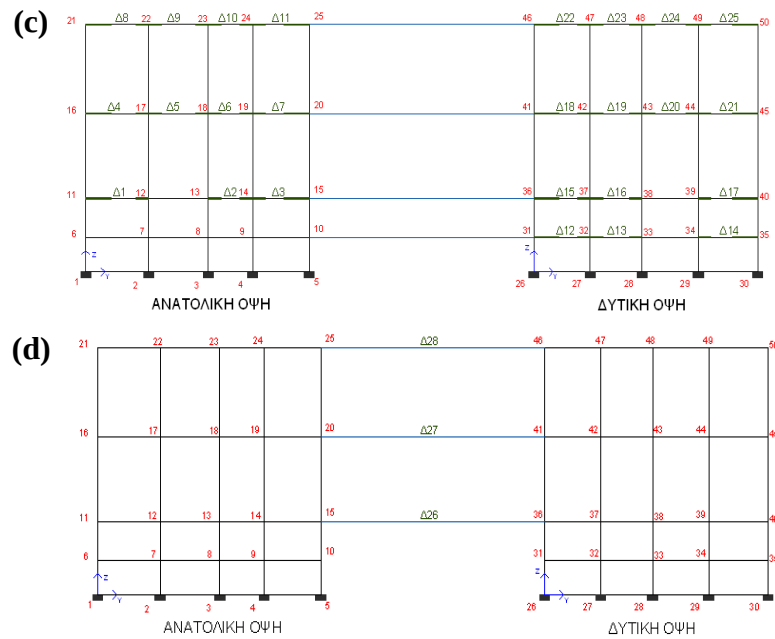


Fig. 6 Model used in simulation: (a) node numbering, (b) discretization of idealized columns, (c) idealized Beams, (d) connection of parallel facades using elastic links to represent diaphragm action

The earthquake record used to assess the peak dynamic response of the structure is the well-known ElCentro (1940) NS component, which, being a shallow ground motion (with matching site characteristics as those of the greater region of Thrace) which is often used as a standard design earthquake motion (peak ground acceleration of 0.33g).

V. DESCRIPTION AND SIMULATION OF BUILDING B

In the same neighborhood as Building A, this structure is 12 m high (measured from the basement floor to the top), with a slightly irregular plan, having dimensions of 10.25 m (north side), 11.05 m (south side), 8.80 m (west side), and 8.35 m (east side). The building is depicted in Fig. 7. Clear floor height is 2.90 m (basement), 3.4 m (first floor), 3.3 m (second floor), and 1.75 m (roof). Modeling was based on a three-dimensional idealization of the structure using a nonlinear frame-type model for each façade, where the frame elements modeling the masonry piers were located along the axes of the piers with rigid zones securing the connectivity of nodes of adjacent beams and piers (Figs. 8, 9). Diaphragm action was modeled using diagonal braces in the horizontal plane (Fig. 10), whereas seismic hazard was represented by the Montenegro earthquake (1979).

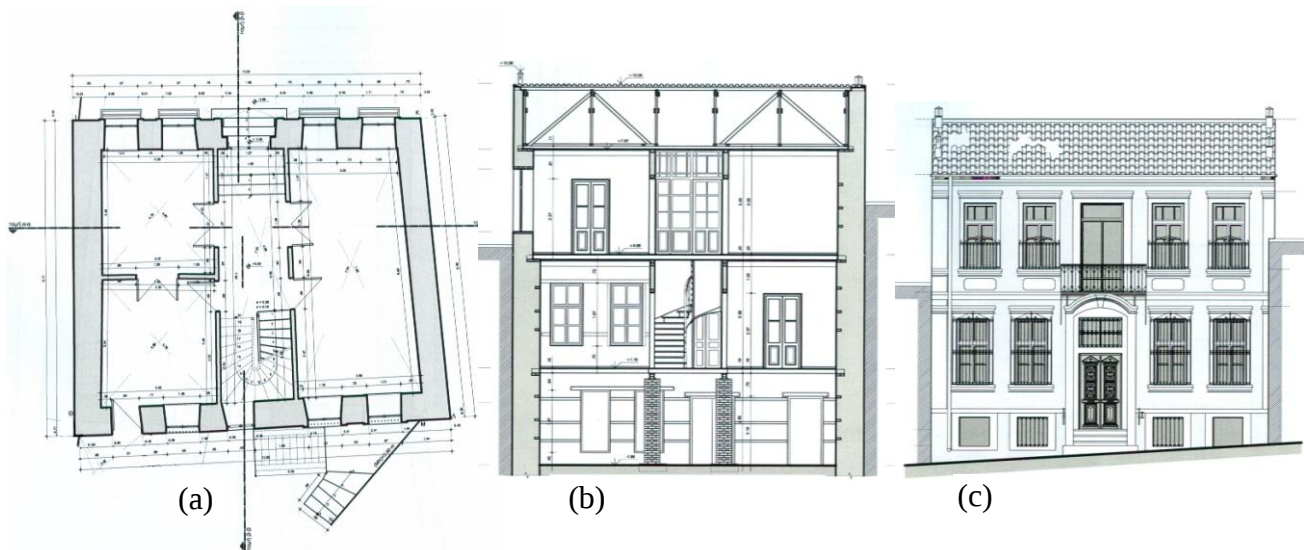


Fig. 7 Building B: (a) Plan view, (b) Cross Section, (c) View

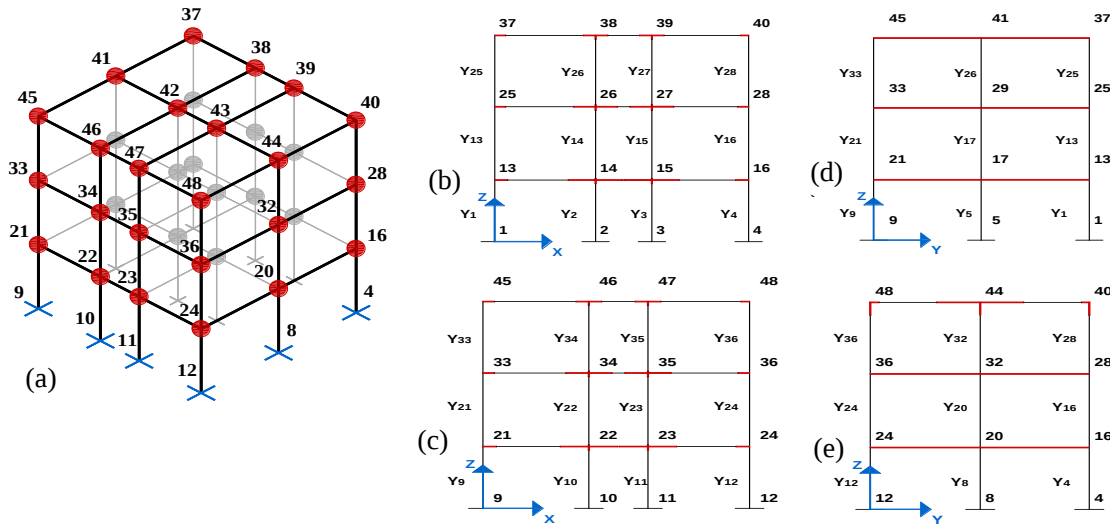


Fig. 8 (a) 3-D model, vertical elements, (b) Northern view, (c) Southern view, (d) Western view, (e) Eastern view

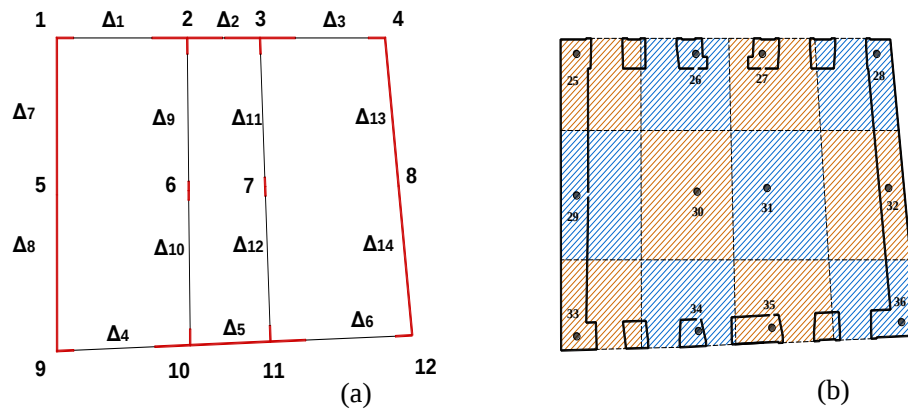


Fig. 9 (a) Horizontal members of the model at the basement level, (b) tributary areas of idealized columns for estimation of axial loads

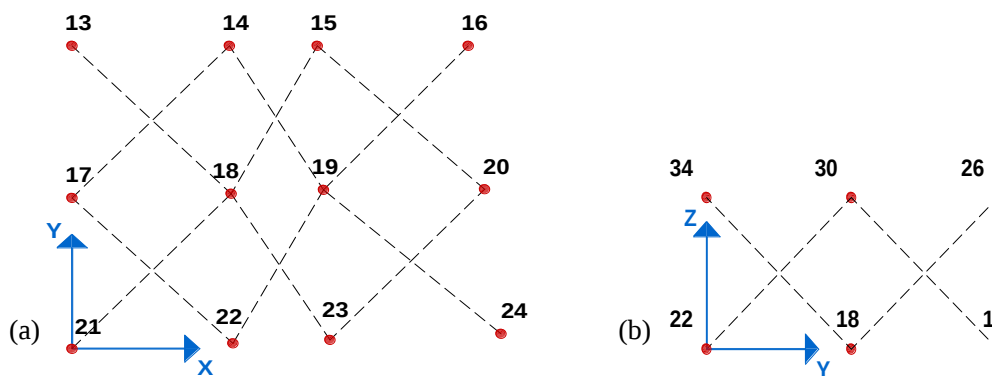


Fig. 10: (a), (b) Modelling diaphragm action through diagonal links in the horizontal plane (numbers correspond to frame nodes)

VI. DESCRIPTION AND SIMULATION OF BUILDING C

This building is located on Botsari Street in the historical center of Xanthi. Its total height is 12.5 m (measured from the basement to the top), whereas plan dimensions are 12.20 m (north-south direction) by 7.65 m (east-west direction); thus the slenderness ratio for this structure was 1.63 (in the shortest plan direction). Clear storey height is 2.50 m (basement), 3.35 m (first floor), 2.95 m (second floor), 2.5 m (third floor) and 1.20 m (roof). The building is depicted in Fig. 11. For the needs of dynamic analysis the structure was modelled using 3-dimensional finite elements; a fine discretization on the wall

piers was achieved using individual finite element dimensions in the range of 0.3m to 0.45m. Timber lacing was represented by linear beam elements (Fig. 12). A grillage analogy was used to model the floor beams at a spacing of 0.7m. Connection of the building along the vertical contact surfaces (for the embedded depth of the basement) with the ground was modeled through one-sided nonlinear gap elements (mobilized in compression only to model bearing of the wall on soil action), whereas at the base fixity conditions were assumed against horizontal sliding. Gap element stiffness in the x and y directions was estimated from

$$K_x = k_x \cdot A, \text{ or } K_y = k_y \cdot A$$

where A the tributary area of each node and

$$k_x \text{ or } k_y = 2.4E_s \cdot y/H^2,$$

where E_s the soil stiffness (estimated in the range of $E_s=40000 \text{ kN/m}^2$), y is the distance of the gap element in consideration from the ground surface, and H the embedded height of the basement.

Due to the slenderness of this structure, coupled with the relatively few openings and extensive wall piers, finite element modeling was established using shell elements to account for both in-plane and out-of-plane flexural action of each façade. Flexural stiffness of the shell elements was estimated assuming cracked section (50% reduction from the reference gross value); masses were calculated from the self-weight of the building. Masonry walls in the basement were 0.7m thick, and were reduced to a thickness of 0.25m in the upper, clay-brick walls (from first wall upwards). The seismic hazard was represented in this investigation by the nominal Eurocode 8 Type I spectrum (EC8-I, 2005) using a design peak ground acceleration for the region of Thrace of $a_g=0.16g$. Damping was assumed equal to 5% in all cases considered. Dynamic response was calculated from time history modal analysis. Figs. 13 and 14 represent the distribution of principal compression and shear stresses on three of the four building facades respectively, for the structure in its initial condition (i.e., prior to the intervention). Fig. 15 plots the deflected shape of the structure in to orthogonal axes of its plan geometry and for different instances of its time-varying response; the deflected shapes are normalized with respect the corresponding peak response of the building at the corresponding time in consideration, so that the deflected shapes may be used to illustrate the tendency for localization of the relative displacement (or interstorey drift) demand, thereby identifying the locations of maximum expected damage in the structure.

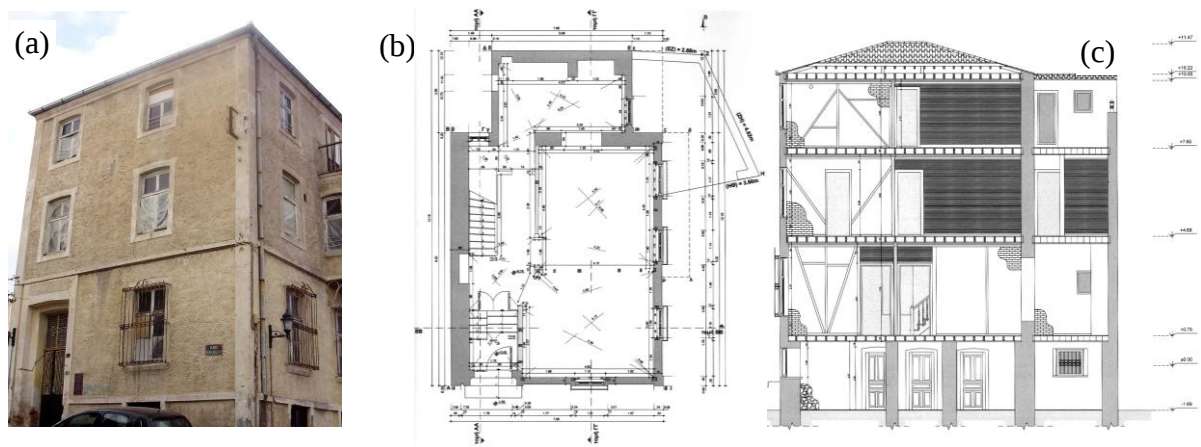


Fig. 11 Building C on M. Botsari Street: (a) South-eastern view, (b) Plan, (c) Section

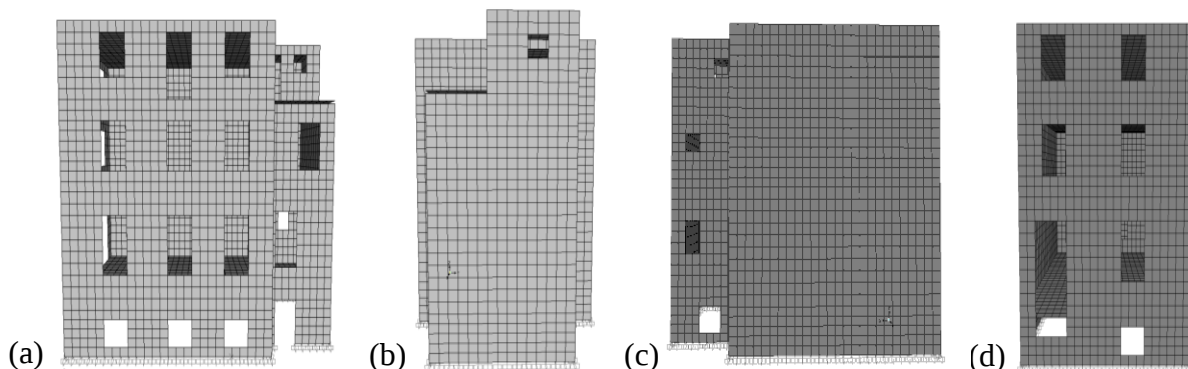


Fig. 12 Discretization of the traditional system, - Views (a) Eastern, (b) Northern, (c) Western, (d) Southern

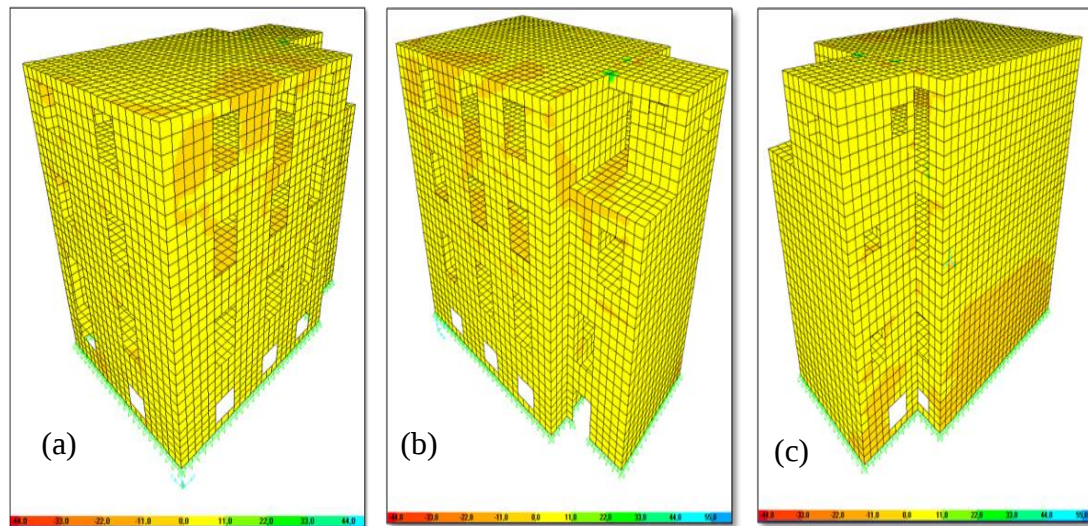
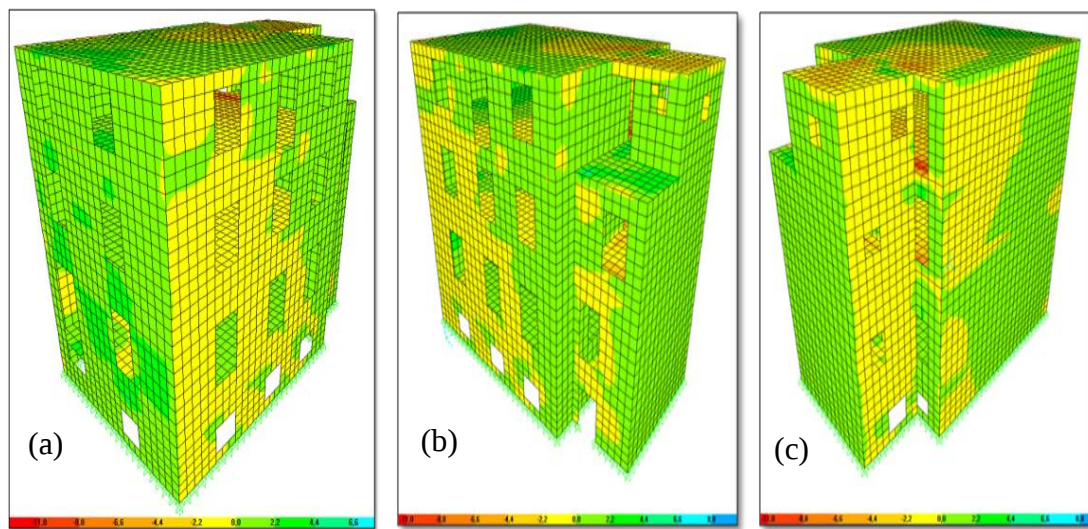
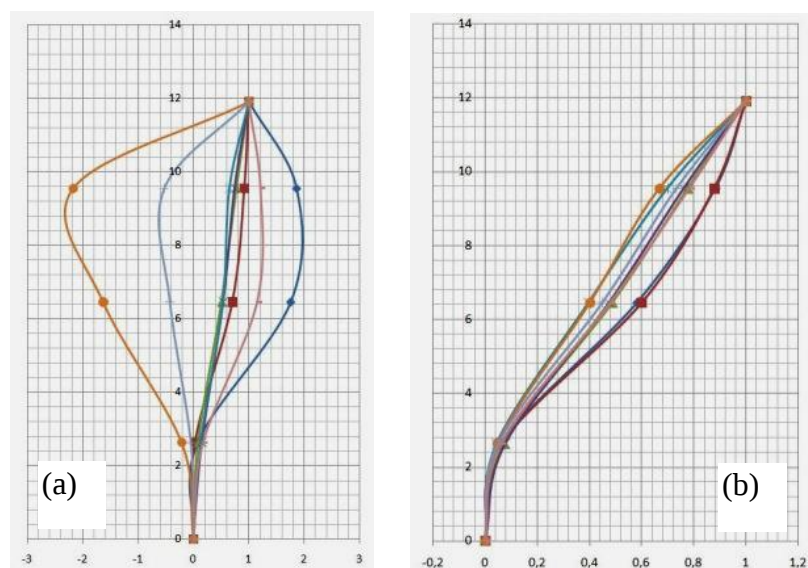
Fig. 13 Distribution of stresses $\sigma_{\max}$ in the original state of the building facadesFig. 14 Distribution of stresses σ_{12} in the original state of the building facades

Fig. 15 (a) Lateral translational pattern U1, (b) Lateral translational pattern U2

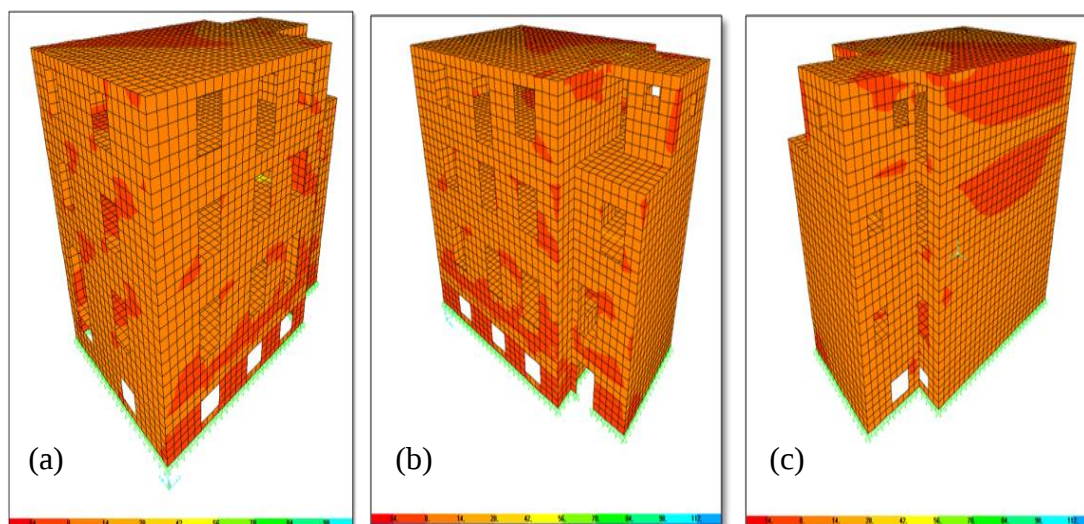


Fig. 16 Distribution of $\sigma_{\max}$ on the building facades, after implementation of intended interventions

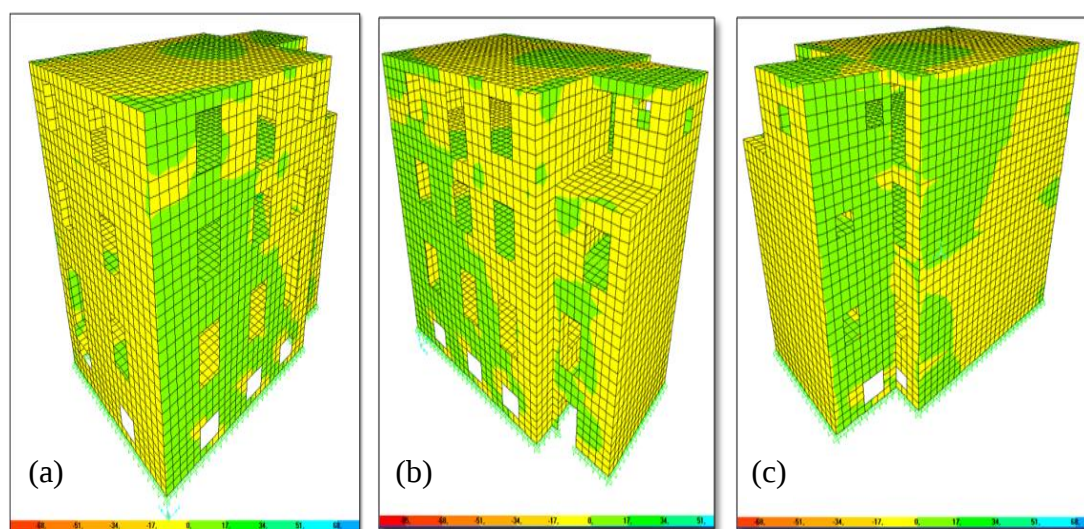


Fig. 17 Distribution of σ_{12} on the building facades, after implementation of intended interventions

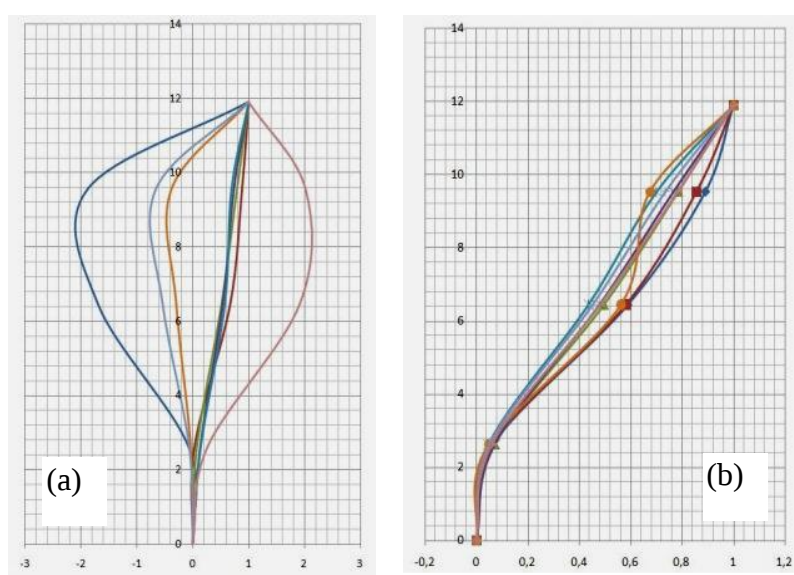


Fig. 18 (a) Lateral translational pattern U1, (b) Lateral translational pattern U2

VII. REDUCTION OF DISPLACEMENT DEMAND THROUGH INTERVENTION AND DISCUSSION

The primary results of this analysis are summarized in Table 1. Values of interest are the percent reduction in the average drift for each building estimated in the as-is, as compared with the post-retrofit condition. Note that regardless of the method of modeling and simulation, the proposed interventions effectively reduce the magnitude of the deformation demands, although they appear more effective in buildings with a frame-type tendency where stiffness enhancement was the reason for better control of displacement demand. In the case of Building C, which possessed from the beginning a flexural type behavior as marked by the rather low initial period, the intervention is found much less effective in controlling the magnitude of the already low displacement. This characteristic is more prevalent in Building C as compared to B, despite the similar overall dimensions, owing to the fact that the latter is stiffened by one more diaphragm level (5 in C as compared to 4 in B) thereby rendering the upper floors at a greater risk due to the tendency for concentration of higher drift demands in the upper floors of flexural-type structures.

TABLE 1 SUMMARY OF THE PRIMARY RESULTS OF THIS ANALYSIS

Building Code ID & Slenderness ratio R (Height to smaller plan dim.)	Displ. Demand before Intervention	Displ. Demand after Intervention	Estim. Peak Drift demand before & after Intervention	Initial & Final Rotational Ductility demand
Building A, R=1.0	57 mm	9.5 mm	0.62%→0.105%	4.3→0.7
Building B, R=1.4	35 mm	14 mm	0.29%→0.11%	1.93→0.73
Building C, R=1.63	90 mm	60 mm	0.72%→0.48%	4.8→3.2

Here, the objective was to achieve an optimized demand distribution as evidenced by the deflected shape of the fundamental mode of the structure prior to and after intervention. The relative translation between any two points along the same vertical line in the structure, normalized by the linear distance between the two points in consideration defines the relative drift in that part of the structure. Deviation of the relative drift from the average value (i.e. the total relative translation of the top with respect to the base, normalized by the free height of the building), identifies the degree of damage concentration anticipated in the part of the structure considered. Significant reduction of this parameter from the as-is state of the structure to its post-repair condition quantifies the effectiveness of the intervention in mitigating the anticipated damage; this comparison is made with reference to the average drift values in the fourth column of Table 1; shorter, stockier structures (Buildings A and B with a low slenderness ratio are more sensitive to the strength and stiffness increase of the masonry material through repointing).

The last column of Table 1 lists estimated rotational ductility demands for the three structural systems, when considering apparent yielding of masonry piers and when they undergo a shear angle in the range of 0.15% as recommended by the EC-8-III in its appendix, for masonry structures. Again, ductility demand is much more effectively moderated in Buildings A and B, whereas the ductility demand in C is prohibitive even after the proposed rehabilitation.

The difficulty in moderating effectively the demands in Building C is owing to the combination of rather unfavorable characteristics in this particular type of structure: high slenderness in the building encourages a pronounced flexural component in the response, with higher drift demands in the weaker, upper levels of the structure; the free standing clay-brick walls in this case possess a thickness of only 0.25 m above the basement to the top of the structure without other forms of reinforcement. A primary conclusion of this simulation study is that non-invasive measures in this problem are inadequate to moderate the demands. More effective strengthening measures are called for, in order to moderate the out of plane flexural behavior of the vertical, cantilevering walls, through increased diaphragm action at the floor levels and at the perimeter of the roof. Increasing the in-plane stiffness of the diaphragms may prove inadequate unless improved connectivity with the vertical walls is secured through mechanical means (such as clamps and metal ties). Strengthening of the walls in shear through post-installed horizontally oriented bars placed within the horizontal mortar beds between brick layers, or textile reinforced mortars adhered on the surface of the masonry piers, or post-installed horizontal timber lacing for compatibility with traditional techniques, may be a necessary complement to the intervention scheme in order to control diagonal tension cracking in the piers due to the high shear demands.

VIII. CONCLUSIONS

Traditional building construction comprising timber-laced stone or solid brick masonry has been used in the Balkan region throughout the ages until the past century when it was phased out in favor of reinforced concrete – one such example is the old city of Xanthi, in Northern Greece, which is well known for the well maintained and densely populated traditional building stock. Performance of surviving structures to recent strong ground motions has illustrated that these structures are rather resilient to earthquake damage, as they generally combine favorable geometrical layouts with the inherent deformability imparted by timber lacing to masonry through a passive, confining-like action. Due to accumulated damage caused by aging and prolonged use, seismic upgrading of traditional, masonry structures are a societal priority, with due sensitivity to issues of heritage preservation through the requirement that any interventions for structural rehabilitation must be non-invasive. In this study, a representative group of three different traditional buildings from the city of Xanthi is examined with an emphasis on

the modification and moderation of seismic displacement demands and the pattern of drift distribution that can be effected by acceptable, established non-invasive procedures. It is shown that these methods are very effective when used on stockier structures that have a natural tendency for a shear-dominated response. Slender structures that have a dominant flexural component, which is highlighted by increased drift demands at the upper levels of the structure, are less sensitive to such non-invasive options which effectively only increase the in plane strength and stiffness of the pier; in such cases, the flexural component may only be reduced if diaphragm stiffness is enhanced at all levels including the roof base.

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