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SIMULATION FOR SEISMIC ASSESSMENT OF TRADITIONAL HOUSES IN THE HISTORICAL CORE OF THE CITY OF XANTHI BEFORE AND AFTER NON-INVASIVE STRUCTURAL INTERVENTIONS

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Abstract. *This paper examines the seismic behavior of three typical traditional houses in the old city of Xanthi using simulation models and the possibilities of restoration and optimization of the Local Historical Structural System used. For this purpose the function of their existing structural system is analyzed and their seismic behavior examined and evaluated using simulation models and time history analysis of the dynamic response to a series of ground motions. Simple, non-invasive structural interventions are selected for restoration and optimization. The seismic behavior of the improved structural system is subsequently re-evaluated using the same modeling procedures. A comparative evaluation of the results obtained from the initial and the post-intervention models is used to assess the global modification of the dynamic characteristics and the anticipated performance effected by the types of interventions examined. 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.*

1 INTRODUCTION

In the framework of a comprehensive strategy for preservation of historical constructions, a decisive step is the thorough identification of the internal force path implicit in their structural system 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. In this field of application, detailed simulation through Finite Element modeling, holds promise in promoting understanding of the global building function although at the local level there is great uncertainty with regards the accuracy of modeling a number of behavioral issues, such as interactions between components, individual member connectivity, and constitutive properties of the associated materials.

The present paper explores the use of computer-aided simulation for seismic assessment of a typical historical structural system used in construction of urban residences up to the early 1900's in the area of Xanthi, in Northeastern Greece. Objective of this effort is to identify the potential

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

utility of such methods of analysis in the process of restoration, rehabilitation and reuse of historical or traditional houses, by enabling a better understanding of the structural function of the system of construction, but also in providing guidance for selecting the important behavioral parameters that need to be modified through interventions for improved seismic performance. By definition, when referring to monumental structures that convey a historical value for the community, a certain type of equilibrium must be sought 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.

In the provinces of Eastern Macedonia and Thrace in the Northeastern part of Greece, and particularly in the region of the city of Xanthi the notion of heritage construction coincides with the traditional buildings in the old city quarters and generally with buildings made with the regional historical building system (RHBS), representing the most significant (from a historical perspective) inventory of the built environment in the region. With regards to the city of Xanthi these buildings comprise a vital portion of its historical fabric, identifying the city (Fig.1). The RHBS uses as primary construction materials the stone (natural blocks or man-made solid clay-bricks) and timber (such as timber structural elements, floor ties, timber lacing elements, etc.) enhances in some situations by the use of iron clamps and ties. The art of construction with these materials, with builder functioning in a complex role of observer, creator and engineer, evolved over the years in an improved system whose performance has stood the test of time in a region of active seismic activity. It is paramount to note here, that although empirical, none of the details of the RHBS was left to chance – the contribution of these techniques therefore, in shaping the art of construction through the history of engineering is most likely their most significant attribute as a human achievement. It is noteworthy that the line of development of timber-laced construction dates back to Minoan times, continues uninterrupted in classical Antiquity and the Roman times (Fig.2), and survived till the previous century when industrialization and the emergence of reinforced concrete displaced tradition in the interest of development ([1]Vitruvius 27-23 b.C. 1997, [2] Touliatos P. 2000, [5] Langenbach R 2002, [9]Müller-Wiener, 1995). Evolution of timber laced construction became prevalent in central Europe but it is also the primary mode of traditional construction in the East (Kashmir, India, [4] Langenbach R 2000) presenting several different versions of the basic structural scheme.

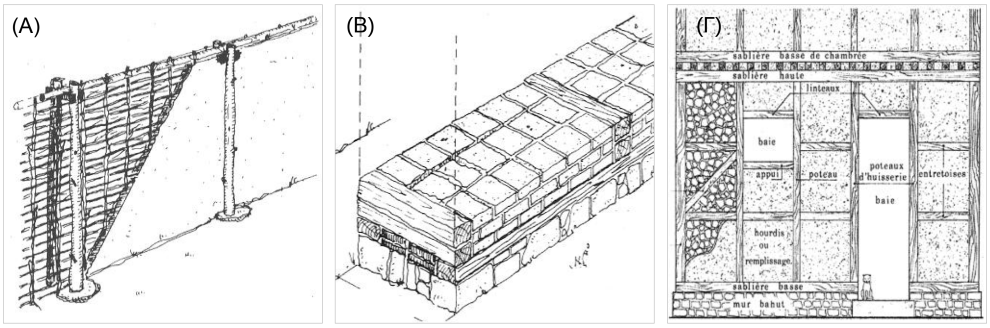

Fig. 2: (A) Early form of timber weaved walls [9], (B) Stone masonrytimber laced walls [9], (C) Timber laced walls [1].

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; these were known from antiquity as “opus craticium”. Today they are referred to as fachwerk, tsatmas, or half-timbered system. 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 ductility).

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. We may say therefore that structures of this class survive the earthquake by practically not "taking", or "attracting" significant forces; instead, they comply in 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. Thus, 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. 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 that the above observations and interpretations concern systems where the structural function has not been altered or reversed by arbitrary interventions either by the 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 that present comparative evaluations with the parallel performance of lightly reinforced – concrete construction in major seismic events which, upon collapse, have proven lethal to human life. ([8] Gülhan D, Güney I.O 2000, [7] Tobriner S. 2000, [5] Langenbach R., 2002, [3] Vintzileou E.,,2007, [4] Langenbach R., 2000, [6] Makarios T., Demosthenous M. 2004).

With regards 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 encompass both the use, accessibility, appearance of the edifice, but also the structural function of the various components (such as foundations, ability 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, etc.). A determining factor is the building itself and its condition.

To demonstrate the efficacy of the 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.

2 PROPOSED REHABILITATION WITH NON INVASIVE METHODS

Usual conditions imposed by international treaties on historical structures restrict the breadth and extent of interventions by the requirement of reversibility and compatibility with the existing substrate. For this reason, 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 each remedial plan includes 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 add intermediate beams 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.
(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 ($f_{wk}=Kf_b^{0.7}f_m^{0.3}$, short term elastic modulus $E=1000f_{wk}$, where K a parameter that depends on the type of the stone blocks (sb) and joining mortar (jm), f_b is the compressive strength of the sb's, and f_m the average compressive strength of the jm), the following properties are assumed:
-Stone masonry of lime and granite composition, f_b =20 MPa, f_m = 2MPa, K=0.45. Characteristic strength f_{wk}=4.5 MPa, E=4.5 GPa, and density γ=2200 kg/m^3.
-Solid clay bricks, f_{wk}=2.25 MPa, E=2.25 GPa, and density γ=1800 kg/m^3.
-Voided clay bricks, f_{wk}=2.25 MPa, E=2.25 GPa, and density γ=1800 kg/m^3.
-Timber lacing components, E=10 GPa, density γ=800 kg/m^3.
-Repair 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, δ=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.

3 DESCRIPTION AND SIMULATION OF BUILDING A:

The building is depicted in Fig. 3. 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.4). The earthquake record used to assess the peak dynamic response of the structure is the well known ElCentro (1940) NS component

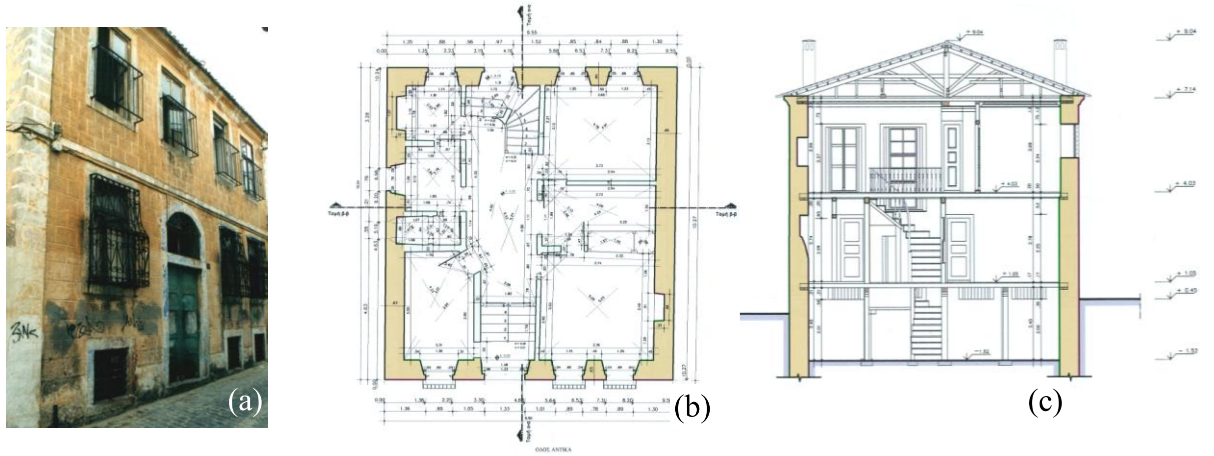

Fig. 3: Building A: (a) eastern view, (b) plan, (c) cross section

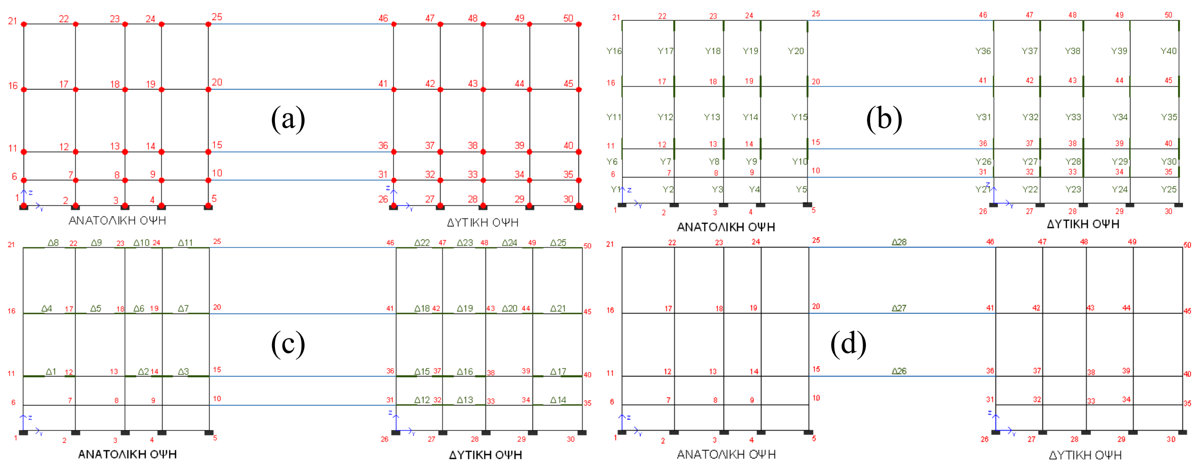

Fig. 4: Model used in simulation: (a) node numbering, (b) discretization of idealized columns, (c) idealized beams (d) connection of parallel facades using elastic links to repesent diaphragm action.

which is often used as a standard design earthquake motion (peak ground acceleration of 0.33g).

4 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.5. 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 (Fig. 6, 7). Diaphragm action was modeled using diagonal braces in the horizontal plane (Fig.8), whereas seismic hazard was represented by the Montenegro earthquake (1979).

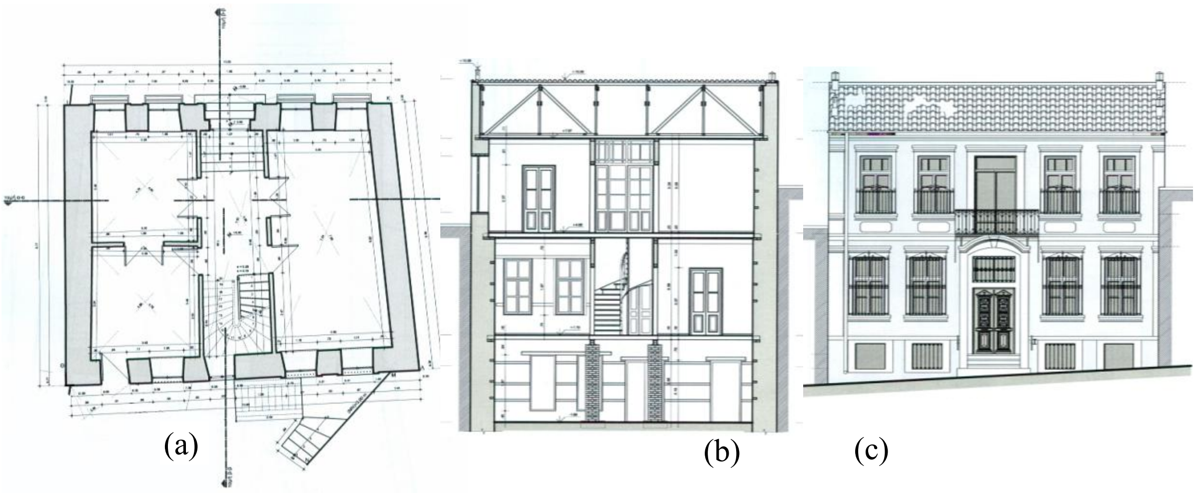

Fig. 5: Building B: (a) Plan view, (b) Cross Section, (c) View

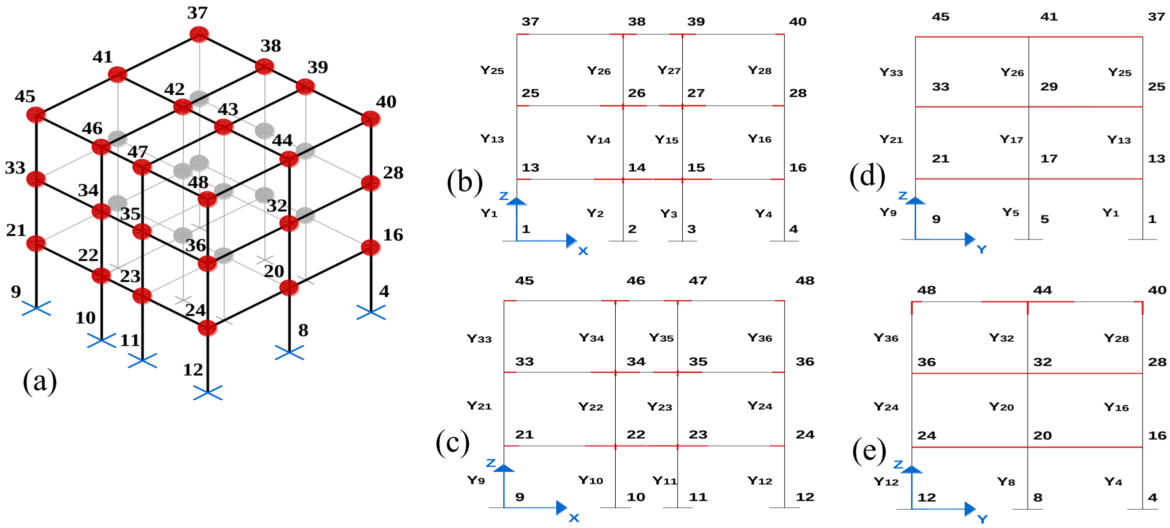

Fig. 6: (a) 3-D model, vertical elements. (b) Northern view, (c) Southern view, (d) Western view, (e) Eastern view

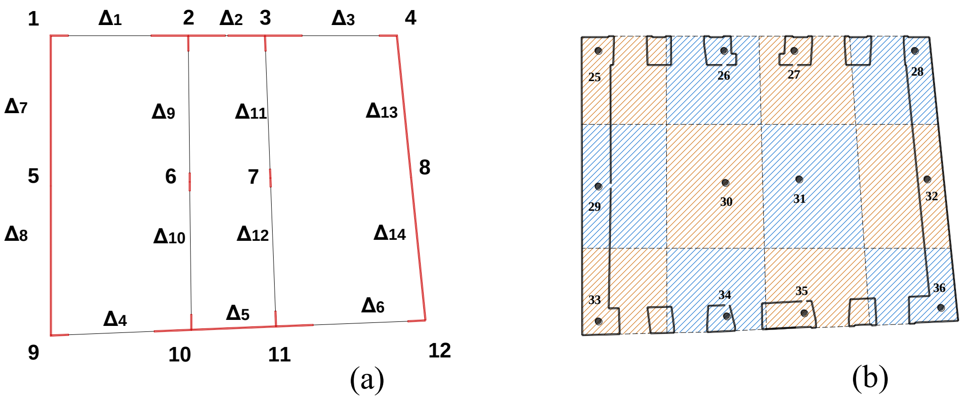

Fig. 7: (a) Horizontal members of the model at the basement level, (b) tributary areas of idealized columns for estimation of axial loads

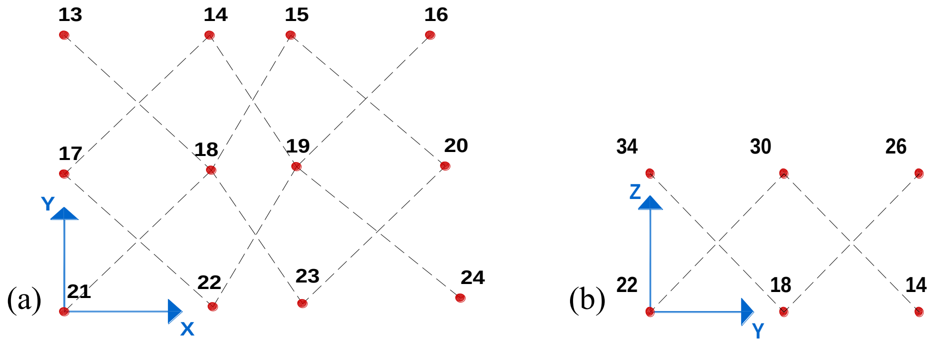

Fig. 8: (a), (b) Modelling diaphragm action through diagonal links in the horizontal plane (numbers correspond to frame nodes)

5 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 build-

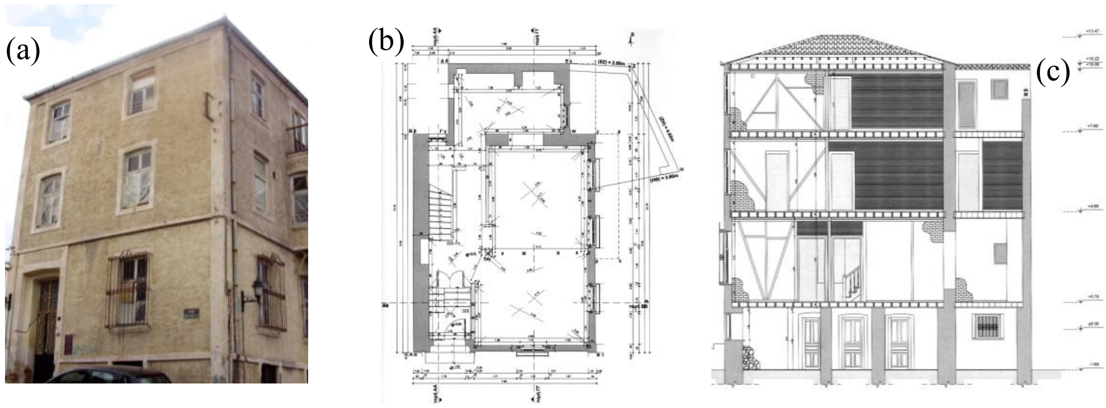

Fig. 9: Building C on M. Botsari Street: (a) South-eastern view, (b) Plan (c) Section

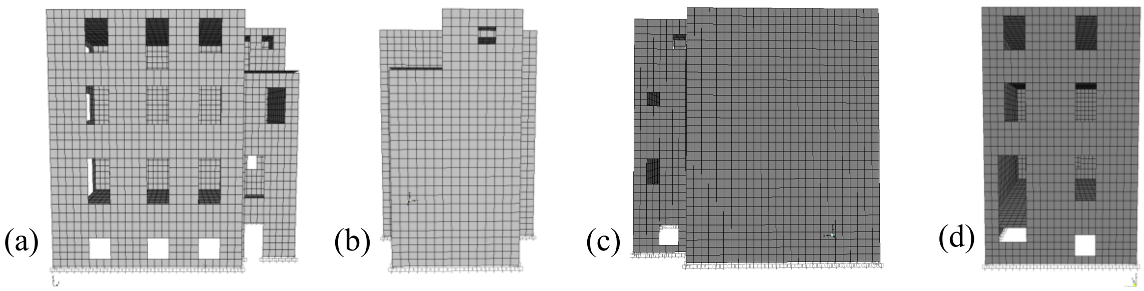

Fig. 10: Discretization of the traditional system, - Views (a) Eastern, (b) Northern, (c) Western, (d) Southern

ing is depicted in Fig. 9. This structure was modeled using 3-dimensional shell elements; finite element dimensions are in the range of 0.3 m to 0.45 m whereas timber lacing is represented by linear beam elements (Fig.10). A grillage analogy is 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

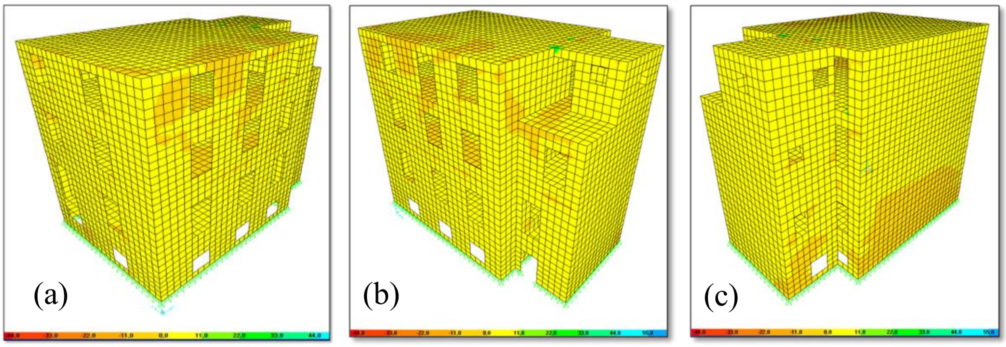

Fig. 11: Distribution of stresses σmax in the original state of the building facades

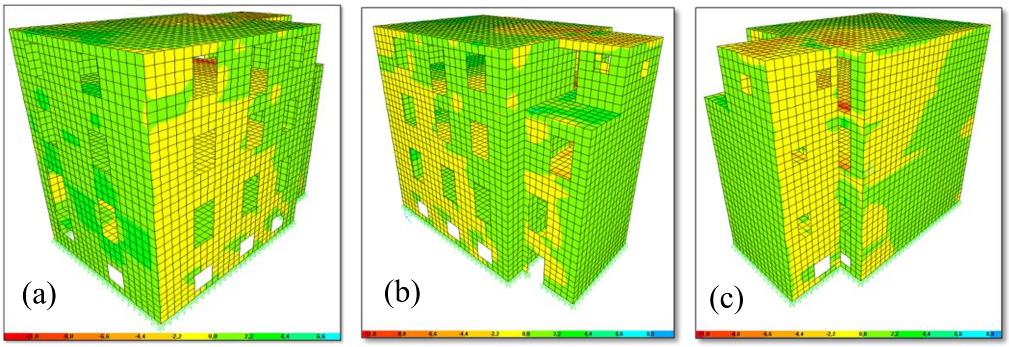

Fig. 12: Distribution of stresses σ12 in the original state of the building facades

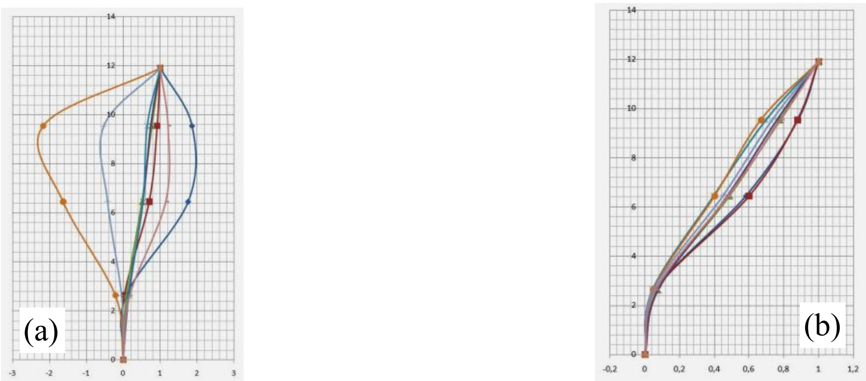

Fig. 13: (a) Lateral translational pattern U1, (b) Lateral translational pattern U2

action), whereas at the base fixity conditions were assumed against horizontal sliding. Gap element stiffness in the x and y directions are estimated from

$\mathbf{K_x = k_x \cdot A}$**, or** $\mathbf{K_y = k_y \cdot A}$ **where A the tributary area of each node**

and k_x or k_y = $2.4E_s{\cdot}y/H^2$, where E_s the soil stiffness (estimated in the range of E_s=40000 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.

Flexural stiffness of the shell elements was estimated assuming cracked section (50% reduction from the reference gross value); masses are calculated from the self weight of the building, where masonry walls in the basement are 0.7m thick, reduced to a thickness of 0.25 m in the upper, clay-brick walls (from first wall upwards). The seismic hazard is represented in

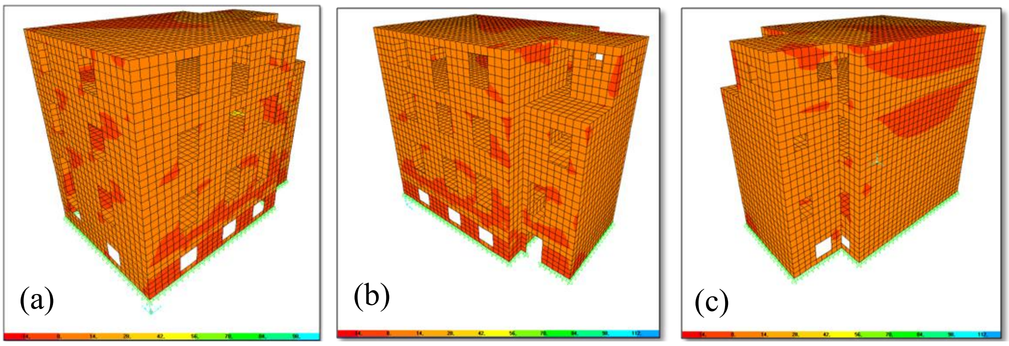

Fig. 14: Distribution of σmax on the building facades, after implementation of intended interventions

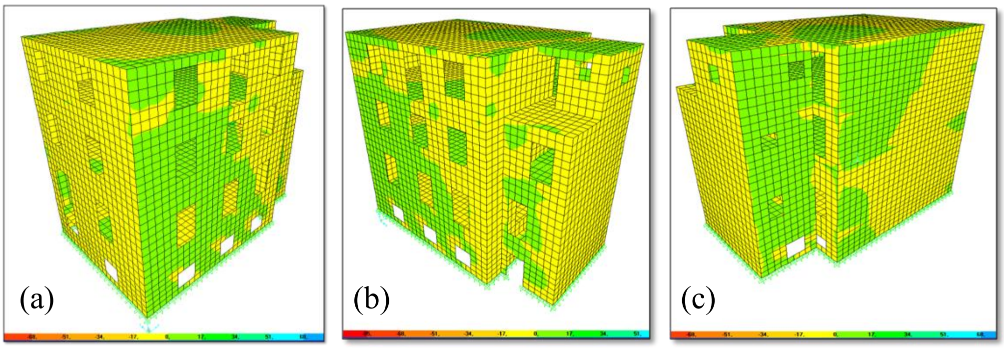

Fig. 15: Distribution of σ12 on the building facades, after implementation of intended interventions

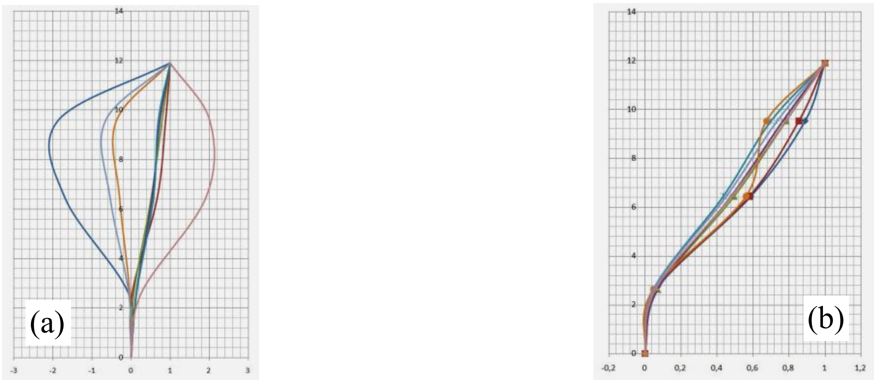

Fig. 16: (a) Lateral translational pattern U1, (b) Lateral translational pattern U2

this investigation by the nominal Greek Seismic design code spectrum (EAK 2000) using a design peak ground acceleration for the region of Thrace in the range of a_g=0.16g.

6 REDUCTION OF DISPLACEMENT DEMAND THROUGH INTERVENTION - CONCLUSIONS

The primary results of this analysis are summarized in Table 1. Values of interest is 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 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, much lower effectiveness of intervention in controlling the magnitude of the already low displacement is found. This characteristic in more prevalent in building C as compared to B

Building Code ID & slenderness ratio R (Height to smaller plan dim.)	Displ. Demand before Inter-vention	Displ. Demand after Interven-tion	Estim. Peak Drift demand before & after Interv.	Initial & Final Rotational Ductility de-mand
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

Table 1: Summary of the primary results of this analysis

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. Here, the objective is better control of demand distribution as evidenced by the deflected shape of the fundamental mode of the structure prior to and after intervention, which effectively suggests great effectiveness in the pattern of anticipated distribution of damage. Note the magnitude of estimated ductilities obtained from the post-repair drift demand assuming a drift at "yield" of the masonry structure elements in the range of 0.15% as recommended by the Greek Code for Seismic Intervention (KANEPE 2010). Note that the 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. These simulation findings suggest that more effective strengthening measures are required in the case of building C, its primary weakness apparently being the rather low thickness (and commensurate stiffness and strength) of its free standing clay-brick walls which possess a thickness of only 0.25 m above the basement to the top of the structure without other forms of reinforcement.

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