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DYNAMIC FINITE ELEMENT ANALYSIS OF EARTH MASONRY STRUCTURES BASED ON EXPERIMENTAL MATERIAL DATA

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Abstract. *Despite the fact that earthen construction is a significant feature of the international architectural heritage, the application of sophisticated analysis methods such as Finite Element (FE) modeling in the appraisal and design of adobe masonry structures has received limited attention up to this day. This paper presents some of the outcomes of an ongoing research programme at the University of Cyprus which aims to investigate the structural response of adobe construction. The behaviour of adobe masonry assemblages under the action of compressive loading-unloading cycles is hereby examined both experimentally and numerically. The development of the FE model used in the numerical simulation is based on a macro-modeling strategy while a simple elastic-plastic constitutive law is adopted. All input parameters required are derived from experimental material data. The numerical results are compared with laboratory test outcomes and useful conclusions regarding the applicability of FE modeling are deduced. The use of the formulated FE model is broadened by analyzing a complete adobe structure under various levels of seismic action. The evaluated response of the adobe structure to dynamic excitations is discussed and the constraints imposed by the model used are analyzed. Finally, critical issues that future research should address in order to enable the efficient computational analysis of earthen construction are identified.*

1 INTRODUCTION

Earth masonry composed of unfired clay bricks and earth mortar joints has been traditionally used for thousands of years. Despite its broad and extensive use in the past centuries, its applications in contemporary architecture are nowadays rather limited. However, a large heritage of adobe buildings still survives and constitutes a significant part of the global building stock and an important feature of the international architectural heritage [1]. Experience has shown that unreinforced adobe masonry construction has relatively poor response to dynamic loading and is prone to seismic damage [2-4]. This implies that there is an imminent need for protecting existing earthen buildings from the destructive effects of earthquakes.

The application of detailed/formal engineering procedures in the appraisal of mud-brick structures and the assessment of retrofitting measures is currently hindered, among others, by the absence of accurate computational methods that would account for the specific characteristics of unfired earth. Although over the last few decades the numerical modeling of conventional masonry construction has been studied in depth through academic research (e.g. [5-7]), adobe masonry has generally received limited attention. This is possibly due to its intrinsic complexity that derives from the natural randomness and inhomogeneity of earthen materials. The various constitutive models that have been applied or developed for the numerical analysis of masonry have not yet been studied in the context of earthen construction. In addition, the applicability of the Finite Element (FE) method in the structural evaluation of adobe masonry structural members and buildings has not been adequately investigated and no attempts have been made so far for calibrating and validating numerical models based on the outcomes of laboratory tests and field observations. Furthermore, the formulation of FE models is, up to this date, precluded due to the lack of adequate experimental data describing the properties of earthen materials and the structural behaviour of adobe masonry assemblages.

This paper aims to examine the structural response of earth masonry construction to compressive loading and seismic action through FE analysis. Initially a model of an adobe masonry assemblage is developed and its behaviour under the implementation of compressive loading-unloading cycles is investigated. The formulation of the model is based on the concepts of homogenized material and isotropic elastic-plastic constitutive law, while all the required input parameters are derived from laboratory tests. The numerical results obtained are compared with corresponding experimental data thus enabling the deduction of useful conclusions on the validity of the simulation procedure. In addition, a FE model of a traditional earthen building is developed and used for performing non-linear dynamic analyses. The investigation conducted involves the use of real time accelerograms from a past earthquake. The predicted response of the structure to various levels of seismic acceleration is studied and the outcomes of the FE analyses are presented and discussed. Useful comments regarding the accuracy of the numerical results are made and the technical challenges encountered when dealing with the FE modeling of earthen construction are noted. Furthermore, key issues that future research should address in order to allow for the efficient use of FE modeling in the analysis of adobe structures are identified.

2 FINITE ELEMENT SIMULATION OF ADOBE MASONRY SUBJECTED TO COMPRESSIVE LOADING AND UNLOADING

2.1 Experimental investigation of the response of adobe masonry to compressive loading and unloading

The development and calibration of a reliable FE model that would predict important aspects of the behaviour of masonry construction with adequate accuracy requires the implementation of laboratory tests [8, 9]. Experimental results can provide information regarding the material properties required for the formulation of the FE model. They can also be used for correcting the inherent deficiencies within the FE model by matching numerical outputs to measured data [9]. Within this framework an adobe masonry assemblage composed of five full-size adobe bricks and earth mortar joints has been subjected to compressive loading-unloading cycles in order to investigate its elastic and post-yield structural response. Prior to this, a number of uniaxial compression tests were also conducted on mud-brick assemblages to assess the capacity of earth masonry to bear static vertical loads [10].

The bricks used for the preparation of the specimen were supplied by a Cypriot manufacturer and belonged to the same production batch. The earth mortar had the same composition as the adobe bricks. It was prepared by mixing soil and straw fibers originating from crushed adobes with water (1155 g of water per 1850 g of solid constituents) to plastic consistency using a mechanical mixer. Stones and gravel with diameter exceeding 4 mm were not included in the mix. The mortar was applied at relatively thin layers (~ 10 mm) between the adobes. The composition of the mortar and the formation of thin joints were intentionally selected in order to replicate the form of adobe masonry that is encountered in local traditional earthen structures. The dimensions of the resulting masonry assemblage were (height x width x length) 28 x 30 x 45 cm^3. After being prepared, the assemblage was allowed to cure in the laboratory (22±2 ^oC and 42±5% R.H.) in order to ensure sufficient bonding between the adobe bricks and the mortar.

For the implementation of the compressive loading-unloading test a Lloyd LR300K universal testing machine with 300 kN capacity was used. The machine's platens have a swivel head to accommodate non-parallel bearing surfaces. Four vertical transducers were attached on each corner of the upper loading plate to record vertical displacements during the testing procedure. In addition, two horizontal transducers were placed on each side of the specimen to monitor its deformation in the transverse direction. The test setup is shown in Figure 1. Loads on the specimen were imposed using a displacement-controlled procedure. Displacement was applied at a constant rate of 0.1 mm/sec. Unloading cycles were programmed to take place successively when the load exerted reached a value of 10, 30, 50, 100, 150 and 200 kN. Figure 2 shows the stress-strain curve derived from the experimental data along with the corresponding numerical results obtained from the analysis of the FE model presented in subsection 2.2.

The results of the test indicate that the response of adobe masonry to compressive loading is non-linear and is characterized by intense plasticity and deformability. The parts of the stress-strain diagram that correspond to the application of loading show that the material exhibits progressive hardening. The form of the unloading branches reveals that when the exerted load is released, induced deformation is only partially removed, while considerable inelastic deformations remain present. Plastic deformations start to develop even when the specimen is subjected to low compressive stresses at the region of 0.08 MPa. This behaviour can be attributed to the fact that the sliding and displacement mechanisms, which take place between the soil grains due to the application of pressure, are non-reversible.

During the implementation of the test and after its completion it was noted that the specimen responded monolithically to vertical loading. Compression resulted in the compaction of adobe and mortar in the central axis of the masonry specimen. This led to the formation of a central zone within the body of the masonry assemblage where the visual distinction between the individual adobe bricks and the mortar joints was no longer feasible. No significant cracks or slips occurred at the joints during the loading procedure. Despite the fact that significant cracking and damage was recorded at the lateral sides of the adobe bricks, their central core remained sufficiently integer and did not lose coherence. This particular mode of failure experienced by adobe masonry under compression has been also noted during the execution of monotonic compression tests [10] and has been reported by other reserachers as well [11].

Figure 1: Pictures of the experimental setup used in the implementation of compressive loading-unloading tests on an adobe masonry assemblage composed of five full-size adobe bricks and earth mortar joints.

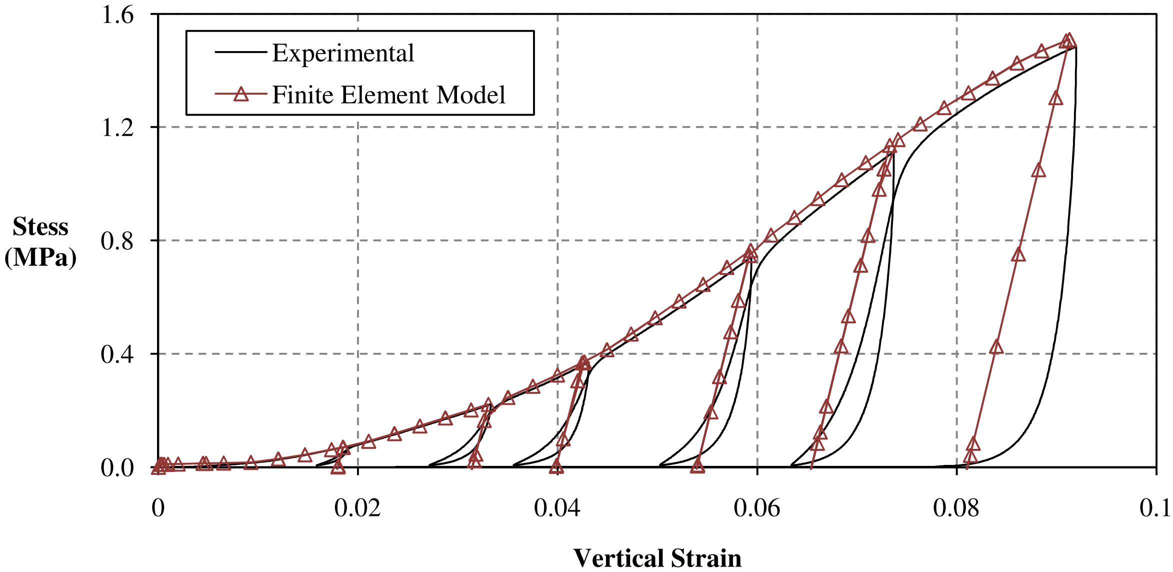


Figure 2: Experimental and numerical stress-strain curves referring to the application of compressive loading and unloading cycles on an adobe masonry specimen with dimensions (height x width x length) 28 x 30 x 45 cm^3.

2.2 Numerical simulation of compressive loading-unloading experimental test

For undertaking the numerical simulation of the experimental test on the adobe masonry assemblage, the commercial code ABAQUS CAE was employed. Both the adobe masonry specimen and the machine's steel compression plates were modeled. Since the main purpose of the current study is to examine practice-oriented analysis methods that can be applied in the appraisal and design of adobe structural members and full structures, earth masonry was numerically handled in the context of a macro-modeling strategy. The macro-modeling approach treats masonry as a fictitious homogeneous continuum and does not make any distinction between masonry units and mortar joints. Despite this rather oversimplifying assumption, macro-models have been widely used in the non-linear FE analysis of masonry structures (e.g. [12-14]) and are considered adequate for the characterization of the structural response of large-scale structural elements and full buildings [6, 7]. After all, in practice, the macroscopic structural response (estimation of forces/stresses and elastic/plastic deformations) is primarily of interest as opposed to detailed microscopic information (e.g. cracks initiation and propagation). Treating adobe masonry as a homogeneous medium is further justified by the monolithic behaviour exhibited by the tested specimen during the application of compressive loading.

All components of the FE model were discretized using 8-node 3D linear brick elements (C3D8). Each steel plate was discretized into 70 elements. A more dense FE mesh consisting of 7935 elements was used in the case of the adobe masonry specimen. It was decided to model adobe masonry using a simple isotropic Elastic-Plastic constitutive material model. The isotropic Elastic-Plastic constitutive material model available in ABAQUS CAE is based on classical metal plasticity theory and uses standard Mises yield surface with assorted plastic flow [15]. By default, when cyclic loading scenarios are examined, the aforementioned model assumes that the loading and unloading curves are parallel to the elastic loading curve (with its slope determined through the Young's modulus) [15]. The material characteristics and the elastic and inelastic properties defined in the FE model for simulating adobe masonry were assigned according to the results of laboratory tests. The density of adobe was measured following simple gravimetric methods and the Poisson's ratio was deduced from the recorded values of axial and transversal strains. The Young's modulus was derived from the stress-strain curve of the compressive loading-unloading experimental test. Its value was set to be approximately equal to the slope of the three unloading branches that were recorded after a 5% strain. In order to accommodate for the lack of elasticity that characterizes the behaviour of adobe masonry, a very low value for the yielding stress (0.01 MPa) was assigned. Post-yield behaviour was defined by providing a relation between compressive stresses and plastic strains based on the outcomes of the aforementioned laboratory test. The failure stress was computed from the results of monotonic uniaxial compression tests on mud-brick assemblages [10]. The steel plates are extremely stiff when compared to adobe masonry and were thus modeled using a Linear Elastic material model. The selected values for the properties of the adobe masonry and the steel plates are reported in Table 1. The boundary conditions provided were chosen so as to adequately simulate the test setup. The base nodes of the lower steel plate were considered to be pinned. A uniformly distributed vertical displacement was assigned to the upper steel plate. The amplitude of the displacement over time was formulated according to the mean displacement values that were computed from the data recorded by the vertically placed transducers during the experimental procedure. The contact between the adobe masonry and the steel plates was assumed to be frictionless. The FE model is shown in Figure 3. The simulated stress-strain curve is given in Figure 2.

	Adobe masonry	Steel plates
Weight per unit volume (kg/m^3)	1300	7750
Young's modulus (MPa)	135	220x10^3
Poisson's ratio	0.35	0.30
Yield stress (kPa)	10	-
Failure stress (kPa)	1650	-

Table 1: Parameter values adopted for the formulation of the FE model that was used in the analysis of adobe masonry assemblage subjected to compressive loading and unloading.

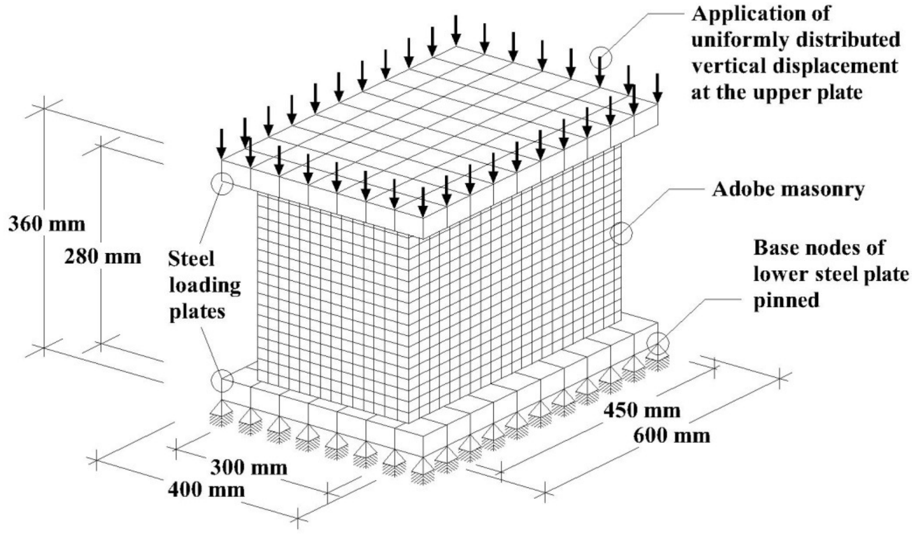


Figure 3: Finite element model used for the analysis of adobe masonry assemblage subjected to compressive loading and unloading.

The numerical results show that the FE model, although quite simplistic, reproduces very well the shape of the envelope of the experimental diagram. The difference between the ultimate stress and strain recorded during the experiment and those obtained from the numerical simulation is less than 2%. Taking into consideration the natural randomness and inhomogeneity of earth-based construction material this difference may be considered as negligible. Regarding the loading-unloading branches, these cannot be reproduced in detail by the constitutive model that has been chosen. However, despite the fact that in the FE model the loading-unloading branches coincide, they occur at approximately the same value of strain as the experimental ones. Furthermore, the stress computed by the numerical model after the end of each unloading cycle is equal to zero, as in the case of the actual masonry specimen. Therefore, the simulated response can be deemed adequate when the main aim is the macroscopic investigation of structural behaviour.

3 FINITE ELEMENT MODELING OF TRADITIONAL ADOBE STRUCTURE SUBJECTED TO SEISMIC LOADING

For examining the applicability of the formulated FE model in the assessment of the dynamic response of adobe construction, a model of a traditional earthen building was developed and analyzed under seismic excitation. The model prepared represents a “monochoro” which is the simplest and oldest typology of Cypriot earthen dwelling. A “monochoro” is a longitudinal, rectangular, single-roomed structure whose length is limited by the timber rafters of the roof. The abatements of the roof’s beams are set into the masonry and span the space between the two opposite longitudinal walls. The four load-bearing walls carry the superimposed weight of the roof.

The building that was modeled has external dimensions (height x width x length) 3.50 x 4 x 9 m^3 and its walls are 0.45 m thick. A door 2.20 m high and 0.90 m wide is located on its facade while two openings 0.80 x 0.80 m are located on the two side walls. The analysis was undertaken using the ABAQUS CAE software. The adobe walls were discretized with 3042 8-noded 3D linear brick elements (C3D8) using the same constitutive material model and input parameters as those reported in subsection 2.2. The timber elements composing the structure’s roof were not modeled because it was assumed that their limited stiffness does not enable the roof to act as a diaphragm during the application of seismic loading. In addition, no constrains were imposed on the nodes at the upper parts of the walls since it was presumed that these can oscillate freely during a seismic event. The base nodes of the walls were considered to be pinned. The unit area weight of the roof was set as 120 kg/m^2. The total weight of the roof was assigned to the upper section of the four walls in the form of uniformly distributed pressure. The FE model of the structure is shown in Figure 4. The dynamic action that was chosen for the analysis of the FE model was that of the 1978 June 20th Thessaloniki Earthquake. The duration of the seismic event was 30.59 seconds and the maximum accelerations recorded in the transverse and longitudinal directions were 0.146g and 0.139g respectively. The amplitudes of the earthquake's transverse and longitudinal components over time are presented in Figure 5. In order to investigate the response of the structure to various levels of dynamic excitation, different analyses were conducted by scaling in each case the two seismic acceleration components. The scaling factors used were: 0.25, 0.50, 1, 2 and 3.

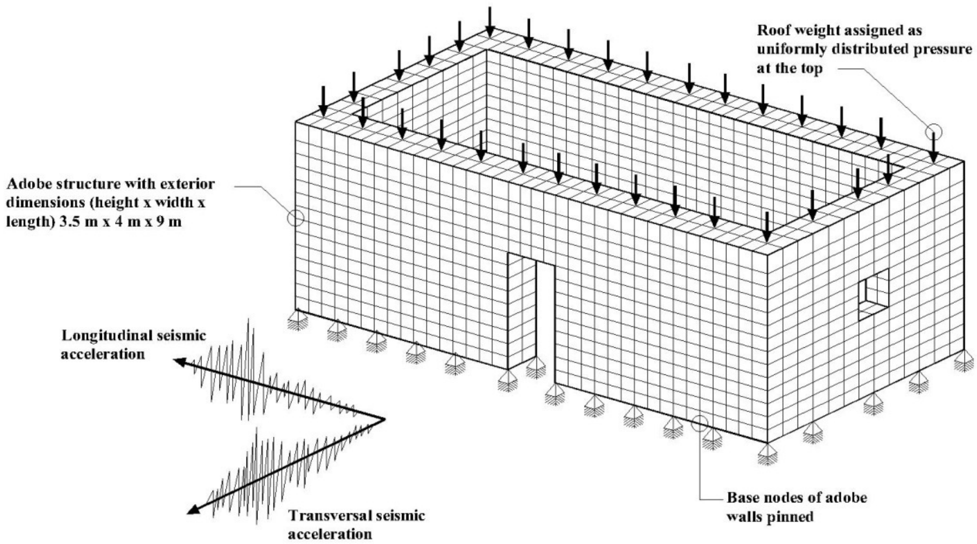


Figure 4: Finite element model used for examining the response of a traditional adobe masonry structure to dynamic loading.

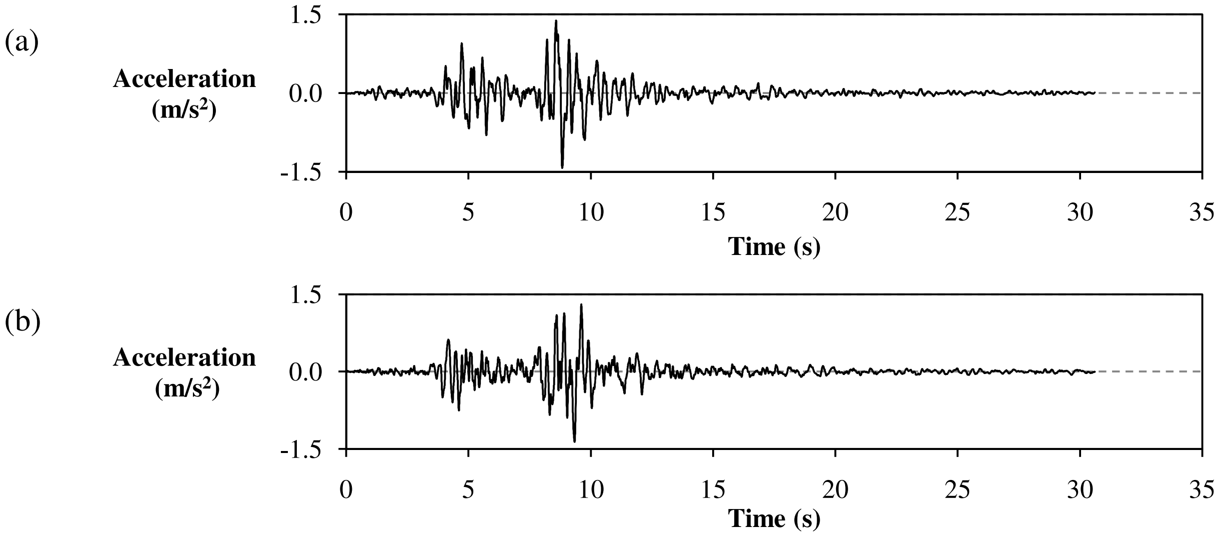
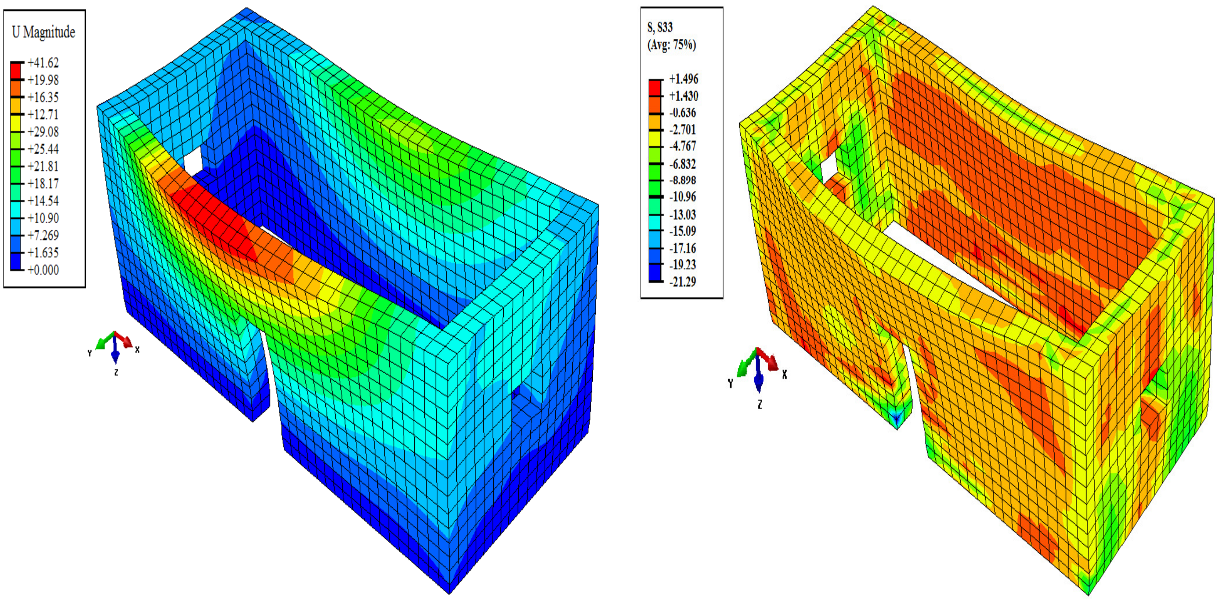


Figure 5: Transverse (a) and longitudinal (b) acceleration components of the 1978 June 20th Thessaloniki Earthquake.

The outcomes of the numerical analysis indicate that the adobe structure examined sustains considerable inelastic deformations when subjected to dynamic loading. The boundary conditions assigned at the corners and at the bottom of the model cause the walls to act as elements fully constrained on their lower part and side edges. Consequently, large displacements and deformations are encountered mainly on the central parts of the upper sections of the façade and the opposite wall (Figure 6). According to the numerical results obtained, the compressive stresses generated by the action of the 1978 Thessaloniki Earthquake do not exceed 50 kPa. However, tensile stresses up to 60 kPa develop at certain parts of the structure. After the dynamic excitation seizes, the compressive stress values encountered are below 15 kPa, but parts of the structure remain in tension (Figure 6). Figure 7 presents the transversal displacement time history of the central node at the upper part of the façade when the 1978 Thessaloniki Earthquake is applied and when the exerted accelerations are amplified by a factor of 3. The maximum transversal displacement of the same node and the out-of-plane deformation of the façade at each amplification level of the seismic action examined are reported in Figure 8. The displacementtime history results show that after the application of the maximum acceleration, the out-of-plane deformation of the façade in non reversible. An examination of the FE model during the time period of seismic loading indicates that the deformation induced does not allow the effective redistribution of loads to all parts of the structure. As a result, the loads are mainly accumulated on the opposite wall and on the two side walls. A comparison of the computed deformations at different levels of dynamic action reveals that when the maximum acceleration of the imposed excitation exceeds 0.07g, severe out-of-plane bulging develops on the façade. Between 0.14g and 0.43g non-reversible transversal displacements can reach values which range from 40 to 150 mm. It is worth noting that, according to the results yielded, the relation between the maximum applied acceleration and the maximum induced transversal deformation is not linear. The results stress out how significantly the response of masonry construction is affected by the existence of sufficient bonding between the load-bearing structural members and the diaphragmatic function at the levels of the roofs and floors.



(a) (b)

Figure 6: Contour diagrams showing the distribution of transversal displacements (a) and vertical stress (b) on the adobe model structure after the implementation of non-linear dynamic analysis using the acceleration data of the 1978 June 20th Thessaloniki Earthquake.

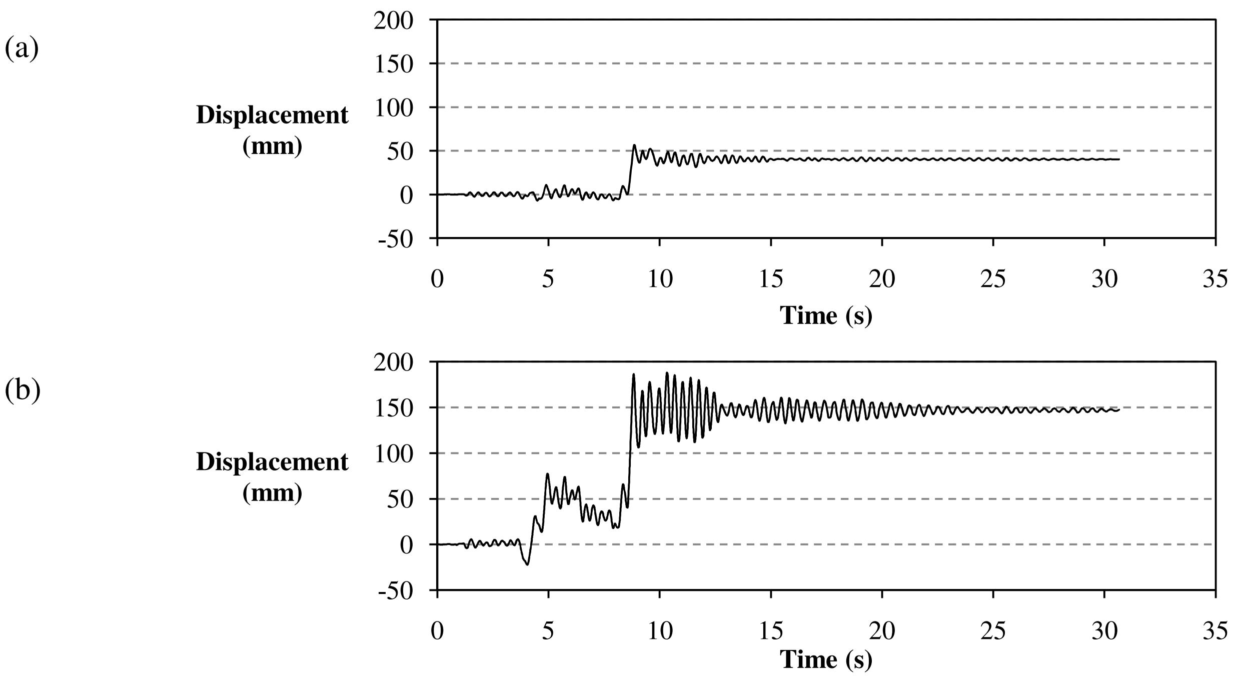


Figure 7: Transverse displacement time history for the central node at the upper part of the model structure's façade when the analysis is conducted using the acceleration data of the 1978 Thessaloniki Earthquake (a) and when the exerted accelerations are amplified by a factor of 3 (b).

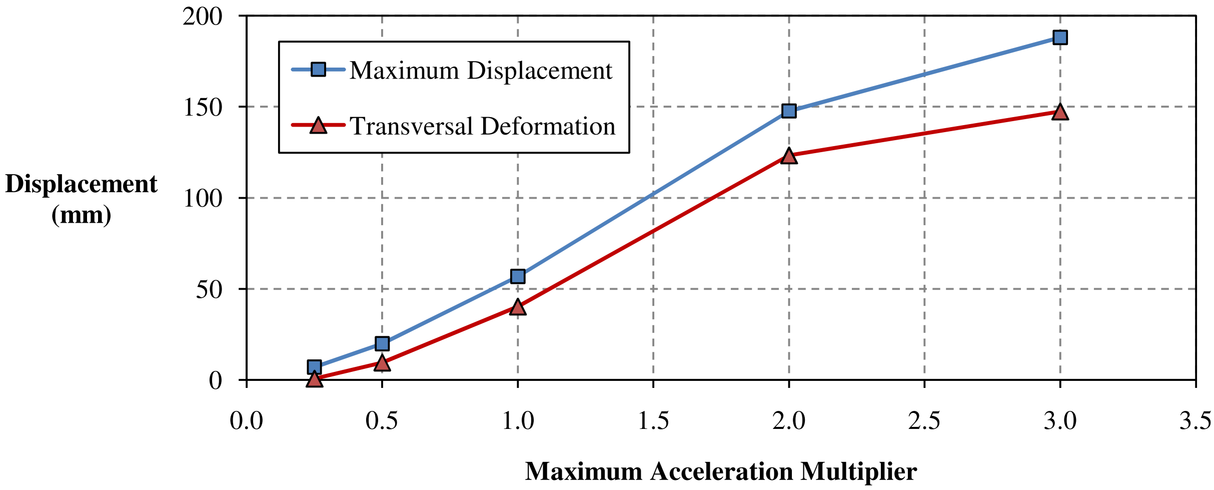


Figure 8: Maximum transversal displacement of the central node at the upper part of the model structure's façade and out-of-plane permanent deformation of the façade at different levels of seismic action. The acceleration data used are those of the 1978 Thessaloniki Earthquake while various factors have been applied in order to scale the amplitude of the dynamic excitation.

4 CONCLUSIONS AND FUTURE RESEARCH AIMS

The numerical results produced by the computational analysis of the adobe masonry assemblage specimen in ABAQUS CAE indicate that the material constitutive model and input parameters hereby suggested may correctly simulate the general behaviour of earthen construction under compressive loading. A comparison among the numerical and experimental data shows that the FE model can provide sufficiently accurate estimates for stress and deformation, especially when applied to monotonic vertical axial loading scenarios. Of particular importance is the ability of the model to predict the material's actual ultimate strength. The good agreement with experimental results and the low computational cost that results from the use of simple constitutive hypotheses appear to be very encouraging for future applications. However, concerns derive from the inability of the model to predict cyclic behaviour involving cycles of both compression and tension loading. This limitation is a product of the constraints imposed by the selected constitutive model.

Regarding the validity of the numerical results obtained from the analysis of a complete model structure, these cannot be deemed as a sufficiently accurate representation of the actual response of earthen construction, despite the fact that they appear to be in context with the general behaviour of unreinforced masonry. The computed deformations may be considered to be rather excessive with respect to the levels of seismic action examined. In an actual structure the evaluated inelastic displacements would have led to the development of significant cracking at the walls and would have probably caused the out-of-plane collapse of the façade. Although such damage mechanisms have been observed in adobe buildings [2, 3], these have occurred at seismic events where the maximum imposed acceleration was above 0.50g (e.g. Northbridge Earthquake, Iran Bam Earthquake). The overestimated displacement values can be attributed to the exclusion of the timber roof structure from the model. The latter, despite its limited stiffness, is anticipated that would have constrained, to some extent, the out-of-plane movement of the walls and would have assisted in a more effective distribution of the loads. Moreover, the boundary conditions assigned to the structure's corners cannot reproduce the actual interaction that takes place at the conjunction of perpendicular walls and force deformations to concentrate at the central part of the walls. Consequently, failure mechanisms such as the formation of detachment cracks at the interconnection of adjacent walls cannot be identified by the model. Uncertainties are also introduced by the fact that the constitutive material model selected for simulating the behaviour of adobe masonry uses a Mises yield surface, thus making no distinction between the response to compression and tension. Although adobe masonry has some considerable resistance to tensile loads, this is much lower than its compressive strength and the constitutive model should be modified in order to account for this behaviour. On the overall, it may be argued that the application of the model hereby presented in the dynamic analysis of complete structures is useful for providing qualitative information regarding the general response of adobe construction rather than accurate quantitative results.

Future work will focus on the modification of existing constitutive models and/or the development of a fully adobe masonry-oriented model that will be able to account for the specific characteristics of earthen construction. Furthermore, the construction and laboratory testing of a scaled model adobe structure in order to study its behaviour under horizontal loading has been scheduled. The experimental results that will be obtained will be used for examining the validity of the numerical data available and will be utilized in the calibration of FE models.

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