NUMERICAL ANALYSIS OF UNREINFORCED THREE-LEAF STONE MASONRY BUILDINGS UNDER EARTHQUAKES

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Abstract. This paper presents a numerical investigation for 3D modelling of unreinforced three-leaf stone masonry buildings under earthquake excitations. Linear and nonlinear analyses are performed. In linear analysis, the influence of interaction between external layers and filling material in vertical stress distribution due to axial loads as well as in the global behavior of walls of building under seismic actions is investigated. In case of nonlinear analysis, masonry is considered as an equivalent composite material with inelastic behavior; the explicit dynamic procedure is used in conjunction with a smeared crack based constitutive model which simulates the main failure modes of masonry under tension, compression and shear. Finally, the numerical results of both analyses are verified with the experimental data obtained during shake table tests conducted at the Laboratory of Earthquake Engineering at NTUA, within the framework of NIKER project.
1 INTRODUCTION

Multiple-leaves masonry walls (two leaf and three leaf) are commonly met in historical monumental and residential structures in Europe. In most of the cases, the external leaves are made of stone/bricks units with horizontal and vertical mortar joints whereas the internal leaf consists of mortar and aggregate. The structural behavior of three-leaf masonry is very complex due to the non-uniform stress distribution. Moreover, three-leaf masonry was proven extremely vulnerable during earthquakes: its low tensile strength causes flexural and shear cracks, whereas the poor bond between the external and the inner leaf promotes the detachment of the leaves and the out-of-plane failure of masonry three-leaf walls.

In the last decade, experimental and analytical investigation was carried out in order to better understand the behavior of this type of walls under static and seismic loads [1-7]. Related to numerical constitutive models for three-leaf masonry, few models are available in the literature [1-7]. These models are divided into two major categories. The first one assumes that masonry behaves as isotropic composite material and masonry is still a continuum body after cracking (macro-modelling), while the second one takes into account the interaction between external and internal leaves and/or the stone-mortar joint behavior of each external leaf (micro-modelling). Most of these models predict with satisfactory accuracy experimental results obtained from monotonic loads and/or cyclic compression tests, while the correct simulation of material behavior of compression-shear test is still a challenging task.

This paper describes the use of uniaxial smeared crack model [8] for finite element modelling of three-leaf masonry. The constitutive law is subsequently validated through the experimental test results reported [9]. The experimental results refer to the seismic response of ½ scaled two-storey three-leaf timber-laced masonry building that was investigated using the shaking table facility of National Technical University of Athens at laboratory for earthquake Engineering (NTUA/LEE) under the frame of NIKER programme. The numerical analysis is performed using the commercial general-purpose finite-element Abaqus 6.11 [8]. The main experimental and analytical results are presented, while reasons for any notable discrepancies between predicted performance and experimental performance are discussed.

2 DESCRIPTION OF SPECIMEN

The geometry of the timber-laced building model is showed in Figure 1. In detail, the plan of the typical floor of each specimen is 3.65x2.30m$^2$. The height of each floor is 1.60m, whereas the total height of the specimen is equal to 3.20m. The thickness of the walls equals 0.25m. The building was constructed on a steel rigid base.

Masonry walls consist of three (approximately equal in thickness) leaves. For the construction of the exterior leaves, stones (limestone from Paramythia-Epirus) with thickness not exceeding 80-90mm were used. The mean compressive strength of the limestone is approximately equal to 100 MPa (measurements according to ELOT 408). The mortar is a lime-pozzolan one with a mixed aggregate matrix composed of siliceous river sand and limestone gravels with 3.5MPa and 0.70MPa compression and flexural strength respectively measured at 28 days.

The floors consist of timber beams (60x100 mm) placed every 340mm. Timber pavement, made of 100x10mm$^2$ timber planks and were nailed on the timber beams is provided. All timber elements were made of coniferous wood (strength class C22).

In Figure 2, the arrangement of timber ties is presented on one of the facades of the building. Timber ties are positioned (within masonry) at the top and bottom of the openings, as well as at floor levels. As shown in Figure 2, two longitudinal timber elements are arranged
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along the exterior and interior faces of masonry (dimensions: 40x40mm). The longitudinal elements are connected by means of transverse timber elements at every 0.50m approximately. The simplest connection by nails between timber ties was selected.

![Figure 1: Plan and front view of timber-laced three-leaf masonry building.](image1)

Figure 1: Plan and front view of timber-laced three-leaf masonry building.

![Figure 2: Arrangement and connection of timber-ties.](image2)

Figure 2: Arrangement and connection of timber-ties.

3 TEST PROCEDURE-RESULTS

3.1 Test and instrumentation set-up

The specimen was securely fastened on the shaking table through a rigid steel base. During each test, accelerations and absolute displacements along X and Y directions at both levels were recorded. Additional masses of 7.5Mgr were placed on the two floors, namely, 4.5Mgr and 3Mgr on the floor of the 1st and 2nd level respectively. The total mass of the specimen was approximately 22Mgr.

3.2 Test procedure

The specimen was tested on the shaking table under excitation along the two horizontal axes (X and Y directions, long and short side of the model, respectively) with increased step-wise input motion until significantly damages were observed. The E-W and N-S components of Kalamata earthquake was used (Figure 3). Before the application of the selected seismic inputs, the dynamic properties of each specimen were measured through sine logarithmic sweep excitation of low amplitude (0.02g). These tests were performed separately along X and Y directions. In Table 1, the testing procedure is shown.
3.3 Observed damages

Figure 4 shows the damage pattern of the model occurred after completion of Test 10. Vertical cracks and some minor shear cracks are formed in the long walls with cracks width of the order of 1.0-2.5mm. The presence of the timber ties prevents the opening of cracks in the corners of doors and windows. Additional, a separation of the leaves of the three-leaf masonry was apparent during this test, in the upper part of the structure along both long and short walls.

Table 1: Test procedure.

<table>
<thead>
<tr>
<th>Test No.</th>
<th>Excitation</th>
<th>Base Acceleration (g)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sine-sweep</td>
<td>0.02</td>
</tr>
<tr>
<td>2</td>
<td>Sine-sweep</td>
<td>0.02</td>
</tr>
<tr>
<td>3</td>
<td>XY time history test</td>
<td>0.04/0.037</td>
</tr>
<tr>
<td>4</td>
<td>XY time history test</td>
<td>0.10/0.09</td>
</tr>
<tr>
<td>5</td>
<td>XY time history test</td>
<td>0.14/0.13</td>
</tr>
<tr>
<td>6</td>
<td>XY time history test</td>
<td>0.18/0.16</td>
</tr>
<tr>
<td>7</td>
<td>XY time history test</td>
<td>0.22/0.21</td>
</tr>
<tr>
<td>8</td>
<td>XY time history test</td>
<td>0.29/0.24</td>
</tr>
<tr>
<td>9</td>
<td>XY time history test</td>
<td>0.30/0.30</td>
</tr>
<tr>
<td>10</td>
<td>XY time history test</td>
<td>0.35/0.35</td>
</tr>
<tr>
<td>11</td>
<td>XY time history test</td>
<td>0.37/0.34</td>
</tr>
</tbody>
</table>

Figure 4: Observed damages at the end of Test No. 10.
4 NUMERICAL INVESTIGATION- 3D VOLUMETRIC ELEMENTS

For the analysis of timber-laced specimen, a detailed geometry model was developed. Walls, timber joists and timber ties were modeled as 3D volumetric elements whereas shell elements were used for the simulation of timber planks. The geometry of the specimen and the corresponding finite element mesh are presented in Figure 5. The model was assumed fixed to the base. Due to the complex geometry, the free meshing technique was selected for discretization. The size of the elements was approximately 0.15m in order to describe the composite material ‘masonry’ and not the individual components of it (unit, mortar). The timber-laced model resulted in total 158310 nodes, 98915 elements and 482706 dof’s.

The Modulus of Elasticity was taken 0.8GPa. This value was obtained from standard compression tests performed on the same walls (scale and material). The Poisson ratio was set equal to 0.20. The density of masonry was 1.9Mg/m$^3$. The modulus of Elasticity of timber was taken equal to 9GPa; its shear modulus was taken equal to 0.560GPa and its density was 0.40Mg/m$^3$. As mentioned above, during experimental phase, the specimen was loaded with additional masses on both levels. These additional masses were taken into account by increasing the density of planks on which masses were fixed.

![Figure 5: Geometry and finite element model.](image)

4.1 Eigenvalue analysis

An eigenvalue analysis was first performed. The main aim of this analysis was to check the stiffness and the mass of the numerical model in the elastic state and to calibrate the elastic properties of materials. The results of this analysis are compared with the experimental values of natural frequencies, derived from sine-sweep tests along each X (8.58Hz) and Y (6.67Hz) respectively. The results from this analysis differ from the experimental ones, as the model appeared too stiff in both directions with the numerical fundamental frequencies in longitudinal and transversal direction greater than 10Hz. These findings could be attributed to the followings:
1) The specimen was constructed on a steel base. This base was secured to the shaking table through M30 bolts every 0.30m. The choice of restraining all the nodes of base of model maybe is not an appropriate assumption.

2) Between specimen and steel base, a thin layer of mortar was placed. This layer was ignored during the finite element modelling.

3) As mention in Section 1, timber joists and timber planks connected through nails. Nails correspond to nodal to nodal connection between joist and plank. In the model, the timber planks supporting by timber joists and the common area assumed tied. This assumption may give large deviation between experimental and numerical results.

4) The behavior of the connection between floors and the walls is not known. The hypothesis of full wall-timber beams connections that assumed in this analysis may lead to unrealistic results.

Related to modeling of timber floor, further investigation is needed. Taking into account the floor non-linearities covered by nail connections, simple and more sophisticated models are available in the literature [11] for modeling timber members and the mechanical interaction between them. Also the joist-collector beam connection and timber tie-wall interaction is still a challenging task for numerical modeling.

5 NUMERICAL INVESTIGATION- SHELL ELEMENTS

Taking into accounts the numerical results of the previous paragraph, a new model was developed. In this case, for the simulation of walls and timber ties, four node shell elements with reduced integration scheme were used, whereas for modeling of timber beams and planks three dimensional beam elements were selected. The size of finite elements was as in the first case, approximately equal to 0.15m. For the purpose of this numerical investigation, full wall-timber beams connections were assumed as well as full joists to planks connections (nodal connection). The numerical model of the building is shown in Figure 6.
As is shown in Figure 6, in this numerical model, the steel base and the thin layer between specimen and steel base were simulated. Additional, the building was assumed fixed to the base only at the positioned (nodes) where the steel base mounted to the shaking table. The model resulted in total 5271 nodes, 3462 elements and 19290 dofs.

5.1 Material constitutive law—Calibration

It is well known that the calibration procedure of nonlinear constitutive model is an important phase of numerical modeling. Generally speaking, nonlinear material models contains two types of parameters; those that their values quantified through standard experimental tests such as Young Modulus (E), Poisson ratio \((\nu)\), compression strength \((f_c)\), peak compression strain \((\varepsilon_c)\) and some others that can be estimated from literature and their values are selected considering the direct fitting between numerical and experimental global results. The second group contains mainly parameters relating to post elastic response of material such as fracture energy in tension \((G_t)\) and shear \((G_s)\).

The uniaxial total strain smeared crack model was used to simulate the nonlinear behavior of masonry [8]. It is based on smeared crack approach in conjunction with explicit dynamic procedure. The material system is assumed to be identical to the global X-Y system, with the bed joints along the x-axis and the head joints along the y-axis and its takes into account the anisotropic behavior of masonry. Three fundamental in-plane failure modes of unreinforced masonry are considered: cracking normal and parallel to bed joints, crushing normal and parallel to bed joints and shear under compressive vertical stress and three separate total strain based criteria are used for failure initiation. Cracking and crushing are controlled through normal strains, while shear strain controls failure in shear. The equivalent strain and fracture energy concepts [12] are also adopted. The latter is used in order to overcome the problem of mesh-dependent results.

In this study, the masonry is assumed initially, isotropic and homogenous. Consequently, the mechanical characteristics of material are set equal in parallel (X-direction) and in perpendicular (Y-direction) to bed joints. In Table 2, the selected parameters are presented. Young modulus, Poisson ratio, compression strength are derived directly from standard compression tests according to EN-1052-1. Tensile and shear strength, fracture energy in tension and shear as well as unloading parameters are selected from previous work [1] which was related to three-leaf masonry walls with the same quality of stones, mortar and scale. The timber parts of the structure were modeled as elastic, since no visible damage was observed during testing.

The uniaxial smeared crack model is implemented in the general-purpose finite-element code Abaqus Explicit [10], using the Vumat user subroutine. The explicit procedure requires no iterations and no tangent stiffness matrix and is based on an explicit central difference integration rule together with the use of a diagonal lumped-mass matrix. The integration through time is performed by using many small increments.

5.2 Eigenvalue analysis

An eigenvalue analysis using the elastic properties of materials was first performed. The resonance frequency in transversal and longitudinal directions was calculated to be 6.68Hz and 8.74Hz respectively. Those values are quit close to 6.67Hz and 8.58Hz, values derived from sine- transversal and longitudinal sweep tests. In Figure 7 the mode shapes corresponding to the calculated fundamental frequencies along the two main axes are presented.
Table 2: Material property.

<table>
<thead>
<tr>
<th>Properties</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Modulus of Elasticity</td>
<td>0.80 GPa</td>
</tr>
<tr>
<td>Poisson’s ratio</td>
<td>0.20</td>
</tr>
<tr>
<td>Tensile strength</td>
<td>0.50 MPa</td>
</tr>
<tr>
<td>Mode-I fracture energy</td>
<td>0.0005 MNm(^{-1})</td>
</tr>
<tr>
<td>Peak compressive strength</td>
<td>-1.50 MPa</td>
</tr>
<tr>
<td>Peak compressive strain</td>
<td>-0.0025</td>
</tr>
<tr>
<td>Unloading from tension to compression parameter</td>
<td>0.85</td>
</tr>
<tr>
<td>Crack closing strength</td>
<td>-0.50 MPa</td>
</tr>
<tr>
<td>Unloading from compression to tension parameter</td>
<td>0.85</td>
</tr>
<tr>
<td>Shear strength</td>
<td>0.10 MPa</td>
</tr>
<tr>
<td>Mode-II fracture energy</td>
<td>0.0001 MNm(^{-1})</td>
</tr>
<tr>
<td>Residual shear strength</td>
<td>0.05 MPa</td>
</tr>
<tr>
<td>Unloading shear parameter</td>
<td>0.90</td>
</tr>
</tbody>
</table>

5.3 Non linear time history analysis

It is well known that masonry buildings present nonlinear behavior under acceleration with very low amplitude, or even under self weight and static loads, due to the low tensile strength of masonry. Thus the adopted procedure for the comparison between experimental and numerical response is to apply to the base of examined model, the sequence of acceleration time histories with increasing amplitude that imposed to the model during the experimental testing. As nonlinear dynamic analyses are time consuming, at present, only the test at which the first damages were recorded was selected (Test No. 10). The time histories corresponding to input motion of Test 10 are given in Figure 8. To further reduce the computing time, in both cases, only the first 10 sec of the recorded signals were imposed.

Figure 7: Calculated mode shapes in transversal and longitudinal direction.

Figure 8: Base acceleration for Test 10 along longitudinal (X) and transversal (Y) direction.
A constant damping ratio of 4% was used. This value was determined during the experimental procedure through sine-sweep tests. Only mass proportional Rayleigh damping was used. In nonlinear analysis both material and geometrical nonlinearities are taken into account. Initially, the model was loaded by self-weight. Then, the input base motion recorded during Test No 10 was imposed.

The comparison between the acceleration time histories obtained by the FE model and the experiment is shown in Figure 9. These time histories refer to the top acceleration at the corners (A3X) of one of the short wall and top acceleration at the middle of long wall (A7Y). It is evident that the numerical result for accelerometer on point A3X is in good agreement with the experimental response. In contrast, the acceleration time history at the middle of long wall at point A7Y obtained by numerical analysis is not comparable to the experimental one. A shift between the signals is observed while the amplitude of acceleration obtained by numerical analysis is comparable to experimental one. This deviation is probably due to the fact that the numerical results are affected by the results of previous tests that were not analyzed. In addition, during testing, long walls suffered significant out-of-plane movements and separation of the external leaves. This mechanism is not simulated with the adopted constitutive model. Moreover, the assumption of full connection between the timber elements of floor is the appropriate one for such high input acceleration which damages the masonry.

Figure 9: Comparison between experimental and numerical results at the corners and mid-length of short wall.

The results obtained from numerical analysis are quite different compared to those presented in [4] in which the same three-leaf timber-laced masonry building is examined. In both investigations, the nonlinear material model was the same. In material properties, the value of each parameter is the same except the Modulus of Elasticity which was assumed 0.50GPa as the experimental value was not available at that time. Additional, in this study, the simulation is more close to the experimental set-up.
6 CONCLUSIONS

In this paper, the capability of the uniaxial total strain smeared crack model to describe the hysteretic response of timber-laced three-leaf masonry structures is examined. In particular, a 1:2 scaled two-storey masonry structures which have been tested under seismic actions at LEE/NTUA are analysed. Two different finite element models were developed: 3D volumetric elements and shell elements. The comparison of numerical and experimental data demonstrates that the numerical model successfully estimates the global response of three-leaf masonry structure with timber ties under seismic loads. The constitutive model was calibrated using results from standard compression tests. Further studies and parametric analyses will be performed to improve the applied model, to introduce the nonlinear behaviour of connection between timber elements of floors and to check its validity also in the case of buildings after the application of interventions.

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