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EARTHQUAKE PROTECTION OF HISTORICAL BUILDINGS ACCORDING TO PROHITECH

F.M. Mazzolani

Department of Structures for Engineering and Architecture
University of Naples "Federico II", Naples, Italy
fmm@unina.it

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Abstract.*The FP6 EC PROHITECH research project "Earthquake PROtection of HIstorical Buildings by Reversible Mixed TECHnologies" (2004-2009) developed a wide experimental and numerical activity on structures, sub-structures, elements and devices, involving 16 academic institutions of 12 Countries, mostly belonging to the South European and Mediterranean area (AL, B, EG, GR, I, P, RO, SL, TR, ISR, M, MK). The final results were presented at the International PROHITECH Conference held in Rome on 21-24 June 2009.*

The main objective of this project was to develop sustainable methodologies for the use of reversible mixed technologies in the seismic protection of existing constructions, with particular emphasis to buildings of historical interest. Reversible mixed technologies exploit the peculiarities of innovative materials and special devices, allowing ease of removal if necessary. At the same time, the combined use of different materials and techniques yields an optimisation of the global behaviour under seismic actions.

A challenging activity within the project was devoted to large scale models of monumental buildings, which were tested on shaking table for producing damage and then for evaluating the effectiveness of the proposed consolidation systems. In particular, the following monumental models were tested: the Mustafa Pasha Mosque in Skopje, the Gothic Cathedral in Fossanova, the St. Nikola Byzantine Church in Psacha and the Parthenon temple in Athens. Beside the experimental activity, appropriate numerical models were developed in order to both predict and interpret the testing results.

1 INTRODUCTION

The seismic protection of historical and monumental buildings, namely dating back from the ancient age up to the 20th Century, is faced with greater and greater interest, above all in the Euro-Mediterranean area, its cultural heritage being strongly susceptible to undergo severe damage or even collapse due to earthquake. The cultural importance of historical and monumental constructions limits, in many cases, the possibility to upgrade them from the seismic point of view, due to the potential risk of using intervention techniques, which could have detrimental effects on their cultural value. Consequently, a great interest is growing in the development of sustainable methodologies for the use of Reversible Mixed Technologies (RMTs) in the seismic protection of the existing constructions. RMTs, in fact, are conceived for exploiting the peculiarities of innovative materials and special devices, and they allow ease of removal when necessary.

This paper deals with experimental studies, framed within the FP6 EC PROHITECH research project "Earthquake PROtection of HIstorical Buildings by Reversible Mixed TECHnologies" (Mazzolani, 2009a), on the application of RMTs to the historical and monumental constructions mainly belonging to the cultural heritage of the Euro-Mediterranean area (Mazzolani, 2009b). Within the range of the experimental research activities, shaking table tests were carried out on four large scale models of the following monumental constructions: the Mustafa Pasha Mosque in Skopje, the Gothic Cathedral in Fossanova, the St. Nikola Byzantine Church in Psacha and the Parthenon temple in Athens .

The large scale models of the Mustafa Pasha Mosque (scale 1:6), of the Fossanova Gothic Cathedral (scale 1:5.5) and of the St. Nikola Byzantine Church (scale 1:3.5) were tested on the shaking table at the IZIIS Laboratory in Skopje, Macedonia.

The seismic shaking table tests on the first two models were performed through three main phases with different loading intensities: 1) Testing under low intensity level earthquakes, causing minor damage in the model; 2) Testing under intensive earthquakes, producing a near collapse limit state to the structure; 3) Testing of the strengthened model until reaching heavy damage.

In the case of the third model, the particular consolidation system was able to fully protect the model under the maximum capacity of the shaking table and, therefore it was necessary to remove it for producing damage.

The tests on the large scale model of a part of the Parthenon temple (scale 1:3) was done on the shaking table of the Earthquake Engineering Laboratory of the National Technical University of Athens (NTUA). Three different configurations have been considered: namely three freestanding columns in a row with and without architraves and three columns in corner configuration. In all cases, the influence of metal connectors have been examined.

The carried out experimental activity (Mazzolani, 2011a), together with a systematically related numerical activity (Mazzolani, 2011b), has provided an important contribution to understand the seismic behaviour of monumental constructions, as well as to validate the consolidation interventions based on RMT systems.

2 TESTING EQUIPMENTS

The shaking table of the Laboratory of the Institute for Earthquake Engineering and Engineering Seismology in Skopje (IIZIS) was used for the Prohitech experimental activity (Krstveska et al., 2009). It consists of a 5.0×5.0 m pre-stressed reinforced concrete plate, which is able to both sustain a maximum mass of 40 tons and simulate different types of dynamic/seismic load in horizontal and vertical direction, separately or simultaneously. The table is supported by four vertical hydraulic actuators located at four corners, at a distance of 3.5 m in both orthogonal directions. The table is controlled in horizontal direction by two hydraulic actuators at a distance of 3.5 m with a total force capacity of 850 kN. The four vertical actuators have a total force capacity of 888 kN.

The data acquisition and sequence generation system (DAC) for the shaking table is a computer based system, which allows simultaneous control of eight and data acquisition of 72 channels, storage of the acquired data to a computer recording device (HDD) as well as signal analysis and graphical presentation of the acquired data.

The shaking table of the Earthquake Engineering Laboratory of the National Technical University of Athens consists of a rigid platform and of a system controlling the input motion and the response of the specimen tested on the platform (Mazzolani, 2011a).

The material of the shacking table is steel and the dimensions are 4,0x4,0x0,6 m. The table can move in all six degrees of freedom (three translations and three rotations) independently or simultaneously. The maximum weight of the specimen can be up to 10 tons, if the centre of its mass is 2 m above the simulator platform. The maximum displacement, which can be achieved, is ±0,10 m in each direction and the maximum acceleration is 2,0g in each horizontal direction and 4,0g in the vertical one. The operating frequencies in each degree of freedom range from 0.1 to 50 Hz.

3 THE MUSTAFA PASHA MOSQUE MODEL

3.1 Design phase

The model of the mosque was built at the IZIIS Laboratory, in order to be tested on the biaxial seismic shaking table (Krstveska et al., 2009). Considering the base dimensions of the prototype structure (20x20 m) and its height (22.0 m), the model was built into a scale 1/6. So, the model dimensions were 3.3x3.3 m in plan and 3.6 m in elevation, whereas the minaret was 6.3 m high (Fig. 1).

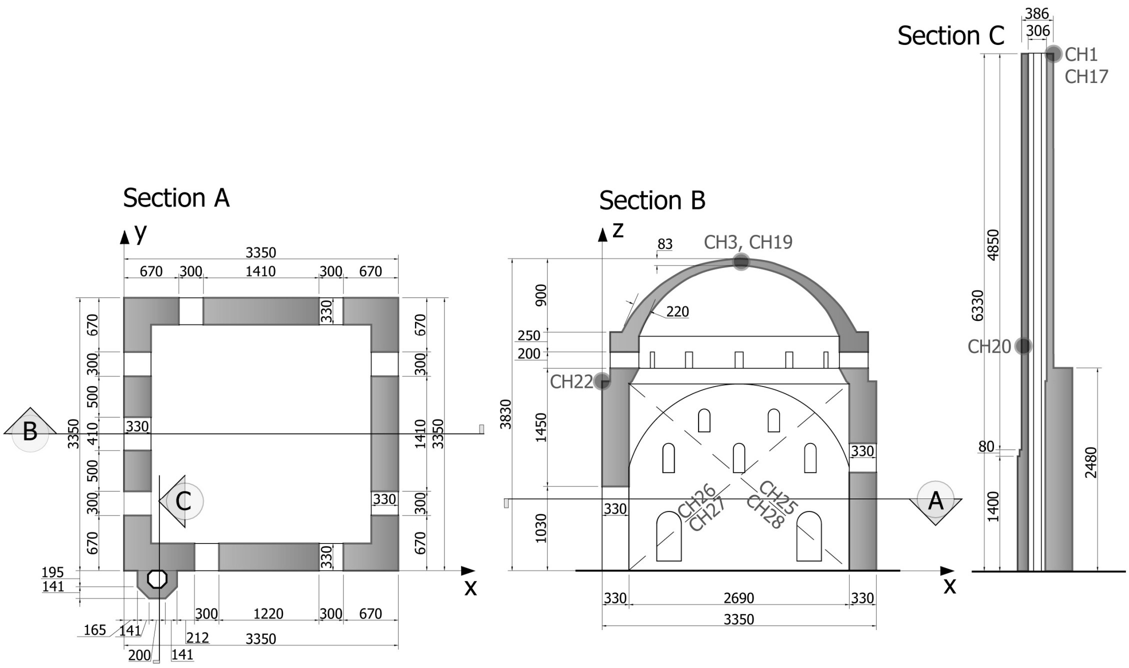


Figure 1: Dimensions of Mustafa Pasha Mosque large scale model.

The model was designed according to "*gravity forces neglected*" modeling principle, using the same materials as in the prototype structure, namely stone (travertine), bricks and lime mortar. The main mechanical properties of this masonry were achieved by experimental tests. The model was constructed on a RC foundation with strong hooks at the corners necessary to transport and lift the model on the shaking table.

The walls of the model were conceived in accordance with the typical Byzantine design: two faces of stone and brick separated by an infill of stone and brick rubble set in lime mortar. Details related to both materials and constructive techniques were provided by the experts of the Institute for Protection of Cultural Heritage in Skopje. Wooden ties – two beams connected in transverse direction –were placed in horizontal mortar joints at each second layer. The construction of the model of the mosque and the completed model fixed to the shake table and ready for testing are shown in Fig. 2.

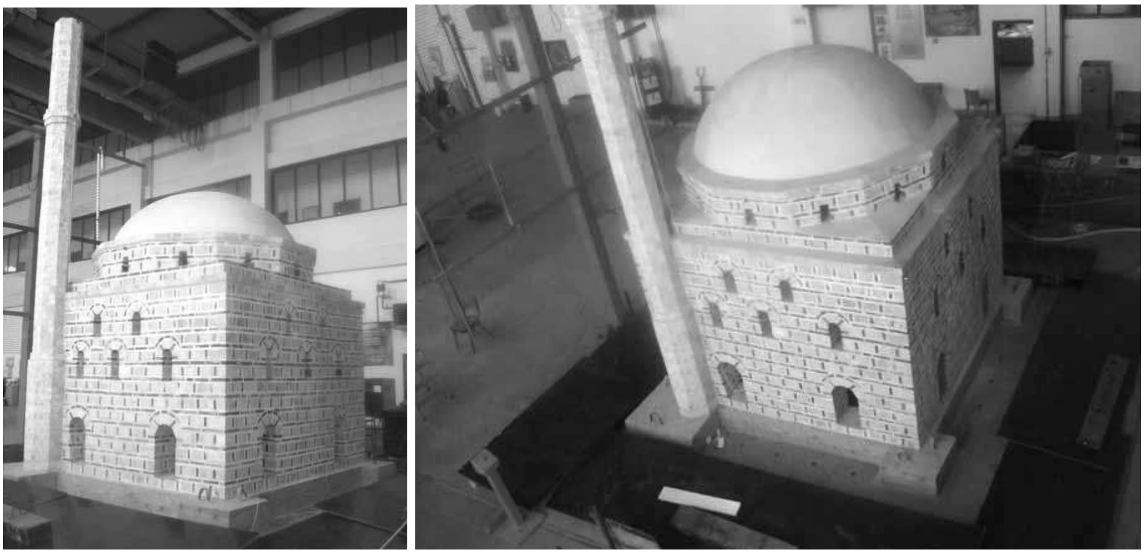


Figure 2: The prototype of the mosque ready for testing.

3.2 Testing set-up

In order to follow the dynamic response during the seismic shaking table testing, the model was instrumented at characteristic points with accelerometers and displacement transducers for measuring the absolute displacements as well as the relative diagonal deformation of the walls in the direction of the excitation (in-plane walls). 13 accelerometers (2 on the minaret, 9 on the mosque and 2 recording the input acceleration) were used. The number of displacement transducers – linear potentiometers and LVDTs – was 11 in total: 3 on the minaret and 8 on the mosque.

The main objective of the testing was to experimentally investigate the effectiveness of the reversible strengthening technology proposed for increasing the seismic resistance of such type of building. With this purpose, the seismic shaking table testing was planned in three main phases:

1) Testing of the original model under low intensity level, with the aim to produce damage to the minaret only;
2) Testing of the model with strengthened minaret under intensive earthquakes, with the aim to produce collapse of the minaret and damage to the mosque;
3) Testing of the strengthened mosque model until reaching heavy damage.

The testing procedure applied to the model consisted of several steps, consisting on the identification of the model dynamic characteristics and on seismic testing on selected earthquake records, whose period, according to the similitude requirements, was reduced 6 times. The excitation was applied in the horizontal direction only.

3.3 Testing phases

Phase 1-Testing of the original model

After the model was located on the shaking table, its dynamic characteristics were defined by means of ambient vibration method as well as by low intensity random excitation in range 0.1–50 Hz.

In this phase the shaking table tests were performed by simulation of the Montenegro-Petrovac earthquake – N-S component, as well as of the El Centro earthquake, N-S component. During this testing phase, nine tests were performed with intensity of 0.01 to 0.10 g, in order to provoke damage only in the minaret. Under input intensity of 2% g, the first horizontal crack appeared at the base of the minaret. In the next tests with intensities up to 10%g, damage in the mosque was observed as well. The reason for this damage was the frequency content of the applied excitation, which was close to the self frequencies of both the minaret and the mosque.

The damaged model is shown in Figure 3. During the last test with input intensity of 10% g, the crack in the minaret was completely developed in the horizontal mortar joint and the minaret continued to vibrate completely freely, reaching the max absolute displacement of 9 mm, while the max displacement at the top of the mosque was 2.6 mm.

Phase 2-Testing of the model with strengthened minaret

After the tests in phase 1, the model of the mosque was repaired by injection in cracks and the minaret was strengthened by application of C-FRP upon a layer of epoxy glue. The vertical strips with a width of 15 cm were placed on four sides along the length of the minaret up to the location of the balcony. They were confined by horizontal wraps, with a width of 10 cm, which were placed at four levels along the height of the minaret, while a strip of 20 cm was placed at its base. Such a strengthening enabled stiffening of the minaret and increasing of its bending resistance (Fig.4a).

Figure 3: The damaged model of the mosque after the first test.

According to the preliminary analysis of the results obtained during the testing of the original model, it was decided to continue with seismic testing applying only the accelerogram of the Montenegro-Petrovac earthquake, N-S component.

Before the seismic tests, the dominant frequencies of the model were checked by random excitation. For the minaret, the dominant frequency was 4.7 Hz, while for the mosque, two frequencies were dominating: f=7.4 Hz and f=9.6 Hz.

During this phase of seismic testing, 11 tests were performed with an input acceleration of 0.2 g to 1.5 g. The accelerogram of the Petrovac earthquake, N-S component was scaled by 6 as in the phase 1 testing.

The first cracks on the minaret were observed under an input intensity of 0.34 g, while on the mosque, the initial cracks appeared at 0.42 g input intensity. During the next tests, cracks developed and, at 0.49 g input acceleration, the upper part of the minaret totally collapsed (Fig. 4b).


a)

b)

Figure 4: The strengthened model of the mosque (a) and the collapse of the minaret (b).

During the subsequent tests, the cracks on the mosque developed and, after the test with an input acceleration of 1.5 g, the mosque model was heavily damaged and the testing was stopped. The cracking mechanism developed through diagonal cracks in the walls starting from the openings at the upper part of the model and continuing also in the dome. Permanent diagonal deformations were observed and measured due to developed cracks in the walls in the direction of the excitation (in-plane walls). The damaged model is presented in Figure 5. It is interesting to observe that one of the most damaged parts of the mosque was the one where the minaret is inter-connected to the wall of the mosque. This damage was obviously influenced by the bending moment produced by intensive vibrations of the strengthened part of the minaret which remained practically undamaged after the shaking table testing in this phase. This was a good indication of the effectiveness of the applied technique for strengthening of the minaret with C-FRP.

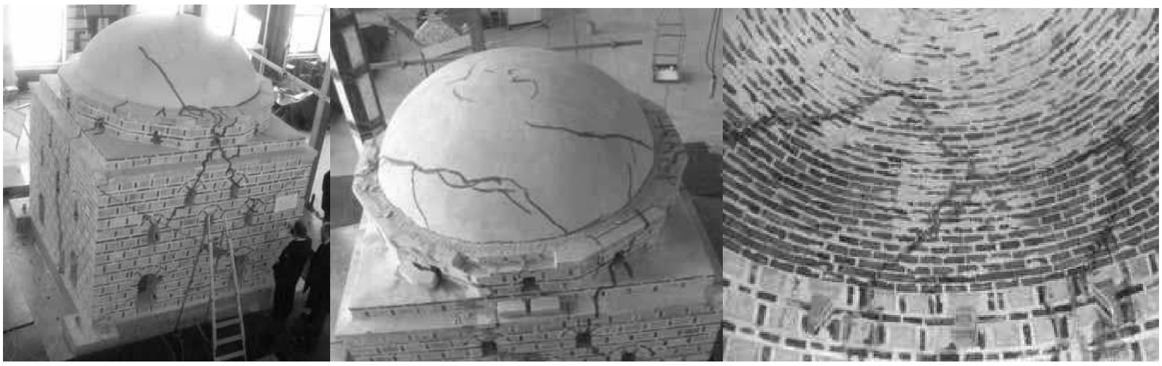


Figure 5: The final model configuration after the second test.

Strengthening of the model

After the end of seismic tests in phase 2, the dominant frequencies of the model were checked by ambient vibration measurements, the minaret was removed and the mosque model was repaired in that part. Then, the model was strengthened according to the proposed technique. The main adopted principle in strengthening the model was that the methodology to be applied be reversible and invisible. Hence, the cracks in the damaged model were not repaired by injection.

The strengthening consisted of incorporation of horizontal belt courses for the purpose of increasing the integrity of the structure at some given levels. The main operations were the following:
– Incorporation of carbon rods in two longitudinal mortar joints around the four walls at two levels: the level above the openings and at the top of the bearing walls, immediately below the tambour. With the incorporation of these carbon rods, two horizontal belt courses were formed and, as a consequence, the tensile resistance of the wall was improved and a synchronous behaviour of the bearing walls was achieved.
– Formation of a horizontal belt course around the tambour by applying a C-FRP wrap with a width of 10 cm.
– Formation of a horizontal belt course at the base of the dome by using C-FRP wraps, with a width of 50 cm.
The strengthened model ready for phase 3 seismic testing is given in Figure 6.

Phase 3-Testing of strengthened mosque model

Before the seismic tests, the dominant frequencies of the model were checked. The frequency of 9.2 Hz obtained by ambient vibrations was compared to the frequency of 8.6 Hz measured after testing in phase 2, i.e. before strengthening. The difference in the frequencies
indicated that, with the strengthening, the resonant frequency of the model was increased for about 8%, which means that the stiffness of the model was not completely recovered compared to the state before testing phase 2, f=9.6 Hz.

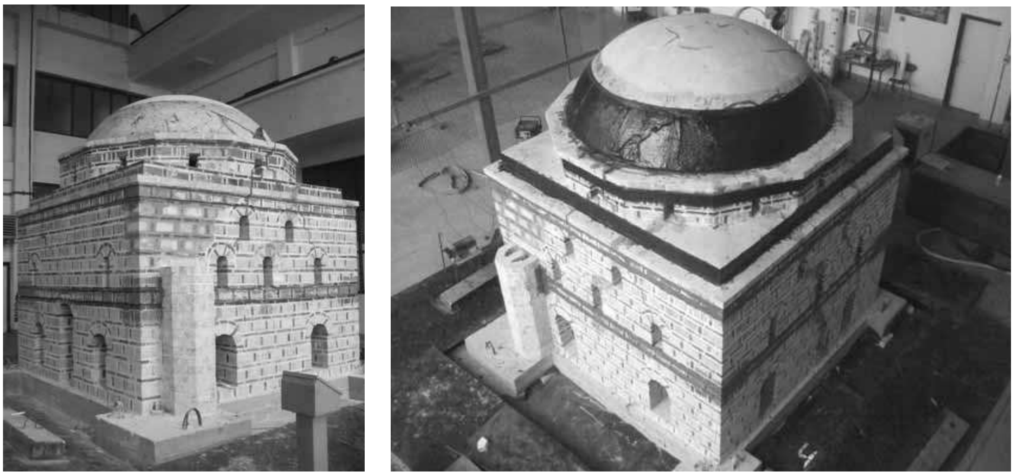
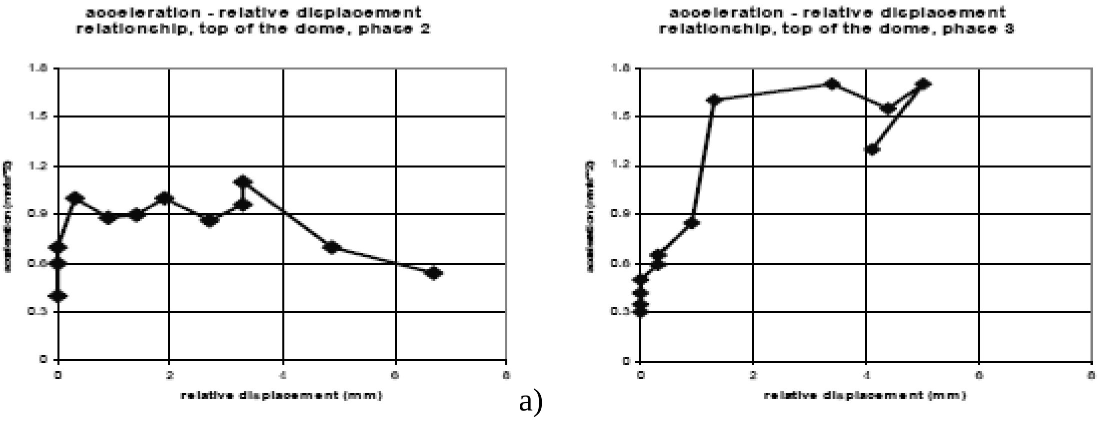


Figure 6. The strengthened model of the mosque ready for the 3rd testing phase

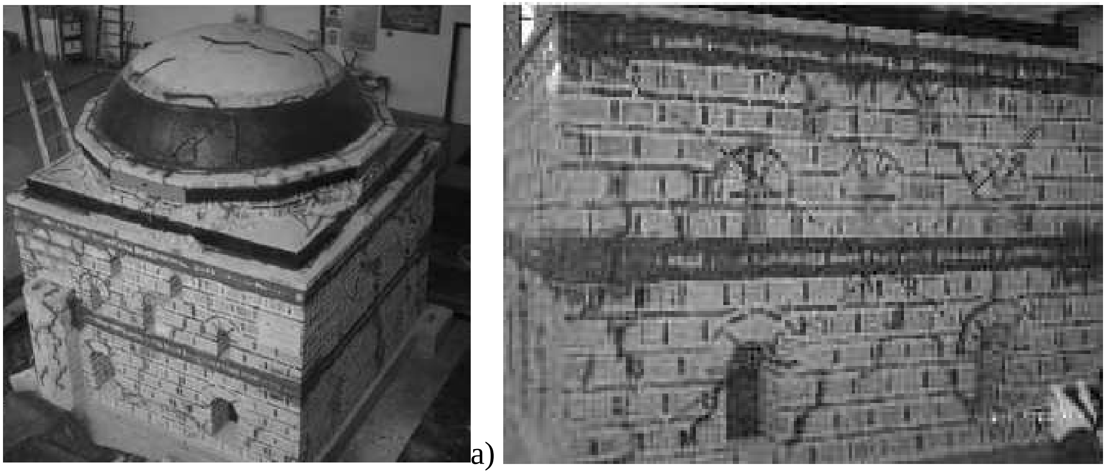
The tests were performed with an input acceleration ranging between 0.15 g and 1.5 g. The accelerogram of the Petrovac earthquake was scaled by 6 in the first 15 tests. During the tests with input intensities of 0.15 g to 0.40 g, the model behaviour was stable, without occurrence of large cracks. In the next 6 tests with an input acceleration of 0.60 to 0.80 g, sliding of the dome took place with a visible horizontal crack at its base. The increasing of the input intensities during the tests with a scaling factor of 6 induced intensive vibrations and sliding along the horizontal crack at the base of the dome as well as dislocation of the stones due to failure of the mortar in the joints. An interesting information was obtained comparing the response of the model at the top of the dome and at the top of the walls, by means of the ’push-over’ curves obtained for the model during testings in phase 2 and in phase 3 (Fig. 7). As it can be seen from these figures, in the case of the original model (phase 2), the dome lost its integrity by sliding along the base of the dome, while the walls still kept their load-bearing capacity.



a) b)

Figure 7. Pushover curves of the structure in the phases 2 (a) and 3 (b)

Considering phase 3, the curve shows that the effect of recovering and increasing of the dome strength is much bigger than the effect in the case of the walls. To provoke a more intensive response of the model, a time scaling factor of 3 and an input acceleration of 0.46 to1.5 g was used in the next tests. In this series of tests, many new cracks appeared in the walls as well as in the dome, decreasing the dominant frequency of the model to f=4.4 Hz. This frequency value was more than twice lower compared to the initially measured frequency of 9.2 Hz, thus indicating a pre-collapse state of the model. During the test with an intensity of 1.2 g, an initial crack in the epoxy resin of the second belt layer occurred. The next two intensive tests were performed by a scaling factor of 2, with an input acceleration of 0.75 to 1.0 g. Progressive cracks appeared, but still without collapse. The dome was ‘moving’ intensively, while sliding along the horizontal crack at its base and relative displacement at its top reached 8 cm.



a) b)

Figure 8. Damage into the whole structure (a) and one wall (b) of the mosque after the third testing phase

Diagonal deformation of the walls parallel to the direction of excitation reached 20mm due to the wall crack opening during vibrations.

The final test was performed by a scaling factor of 1 (real earthquake), with an input acceleration of 0.35 g. Heavy damage to many parts of the model was observed. Due to the intensive shaking, one corner of the model was inclined giving rise to damage to the C-FRP belt in that part. The damaged model is presented in Figure 8a, while the damage pattern in one of the walls is given in Figure 8b.

3.4 Numerical analysis

After experimental activity, a numerical model of the mosque based on two different schematization of the material law was implemented aiming at investigating its behaviour in the linear dynamic and non linear static fields (Fig. 9) (Mazzolani, 2011b).

The results of FE model were compared with experimental results. The distribution of first principal plastic strains predicted by the numerical model fits well the crack patterns observed on the large scale model of the Mosque during shaking table tests. As shown by the experimental investigation, in the case of the original Mosque cracks formed on the spandrels between the openings up to the tambour, while in the reinforced model cracks on the bearing walls were observed at the base of the structure, according to the predicted damage pattern. To this regard, it should be noted, however, that sliding of the dome at tambour opening level raised before the collapse of the shear walls.

The sliding mechanism was not predicted by numerical results. This is due to several factors, including the type of performed analysis and the different quality of masonry at the base of the dome, respect to other parts of the Mosque.

The comparison between experimental and numerical responses in terms of PGA and relative displacement at the structure top is shown in Figure 9b. Also in this case, the numerical and experimental results are in good agreement, even if the experimental response is higher than the numerical one, due to the cyclic loading induced by seismic excitations.

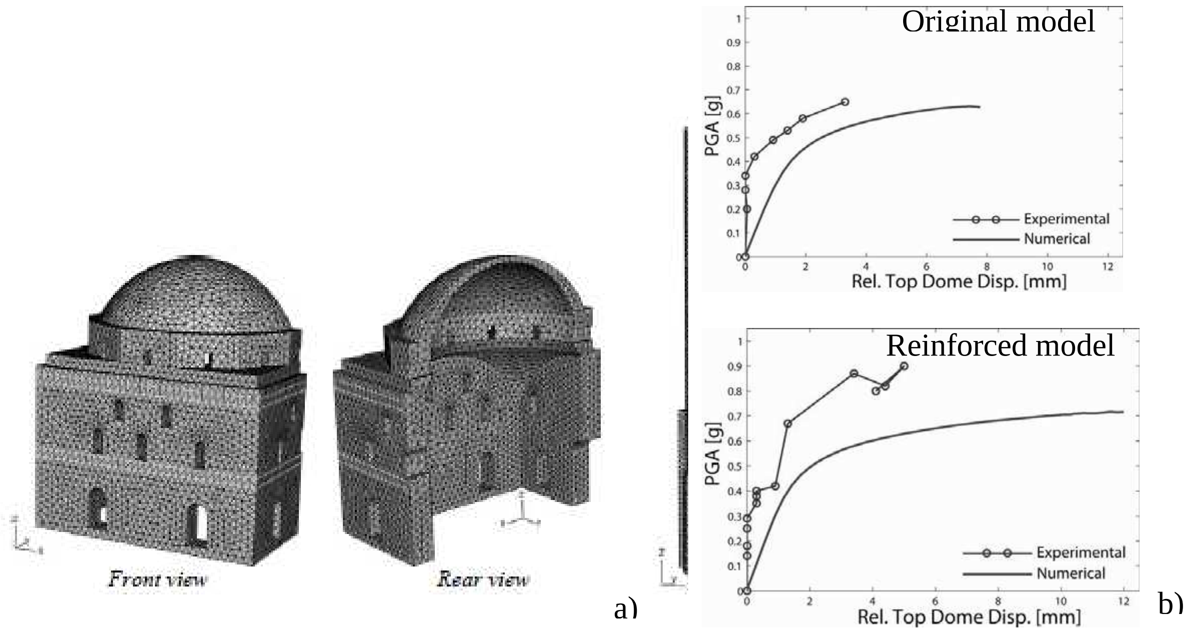


Figure 9. The FEM large scale model of the Mosque (a) and the experimental-numerical comparison in terms of PGA vs. Relative Displacement curves (b)

4 THE GOTHIC CATHEDRAL MODEL

4.1 Design phase

The model of the church was designed in a length scale of 1:5.5, according to the "*true replica*" modelling principles (Tashkov et al., 2009a), in order to investigate the dynamic behaviour of the church in the direction transversal to the main navy. For this reason a structural unit was selected and isolated from the rest of the church. This structural unit presents actual dimension of 20.71 m in the longitudinal direction and 24.00 m in the transversal direction in plane. The maximum height, at the top of the roof structure is of 23.35 m. After construction at the IZIIS Laboratory, the model was transported to the shaking table (Fig. 10).

The main objective of this test was to experimentally investigate the effectiveness of the proposed reversible technology for strengthening and increasing the seismic resistance of this type of structure.

Accordingly, the seismic shaking table testing was performed in two main phases:

– Phase 1: Testing of the original model (as-built model) until occurrence of severe damage.
– Phase 2: Testing of the strengthened model until reaching of heavy damage.

The testing procedure applied to the model consisted of several steps:

– Tests for the evaluation of the dynamic characteristics of the model (before and after the seis mic tests in each phase), in order to check the stiffness degradation of the model produced by micro- or macro-cracks developed during the tests;
– Seismic testing under selected earthquake records.

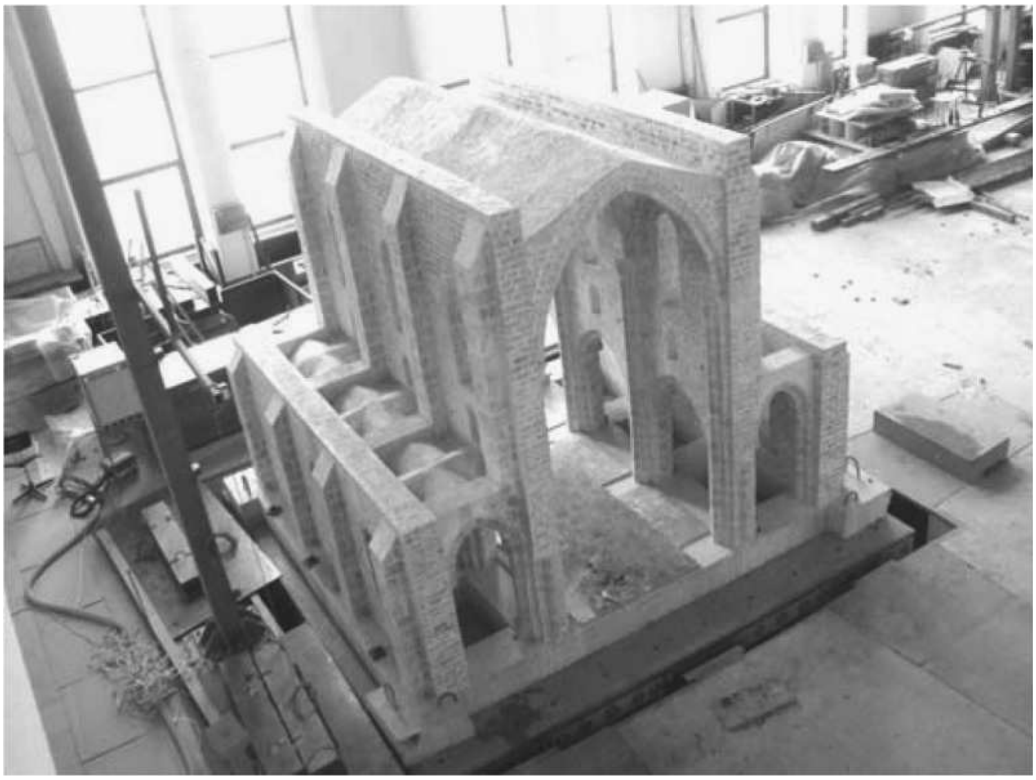
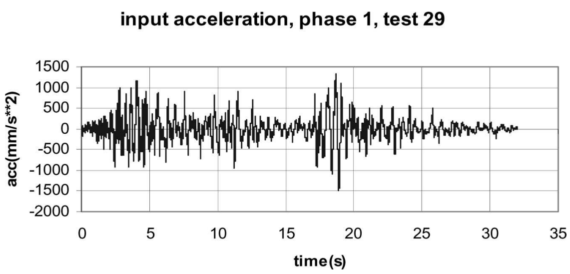


Figure 10. The model of the Fossanova church in scale 1/5.5

4.2 Testing set-up

The dynamic characteristics were evaluated by means of three methods: ambient vibration, sine-sweep and random excitation. The seismic investigation on the model was performed by simulating the Calitri Earthquake (time history – North-South component) selected as the representative one, being characteristic for the site of the monument. According to the similitude requirements for true-replica models, the original earthquake record was scaled by √5.5 in time domain (compressed).The excitation was applied in horizontal direction only. The acceleration time history and the displacement time history of the scaled earthquake are presented in Figures 11a and 11b, respectively.



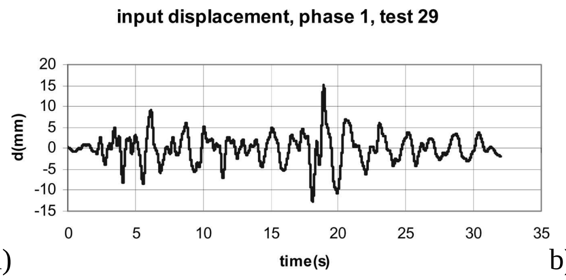


Figure 11. Compressed time history of acceleration (a) and displacement (b) of the Calitri earthquake, N-S component

To follow the dynamic response during the seismic shake-table testing, the original model was instrumented at characteristic points with accelerometers and displacement transducers, for measuring the absolute displacements (LPs) as well as relative deformations between the columns in the direction of the excitation (LVDTs).

4.3 Testing phases

Phase 1-testing of the original model

Initial dynamic characteristics of the model

The dynamic characteristics of the model were defined by means of the ambient vibration method. The mode shapes are shown in Figure 12. The model frequencies compared to the frequencies obtained by in-situ ambient vibration measurements on the Fossanova Church are given in Table 1. The achieved scaling factor model-prototype was very good, which means that the materials for the model construction (stone and mortar) fulfilled the similitude requirements for the material and its mechanical characteristics.

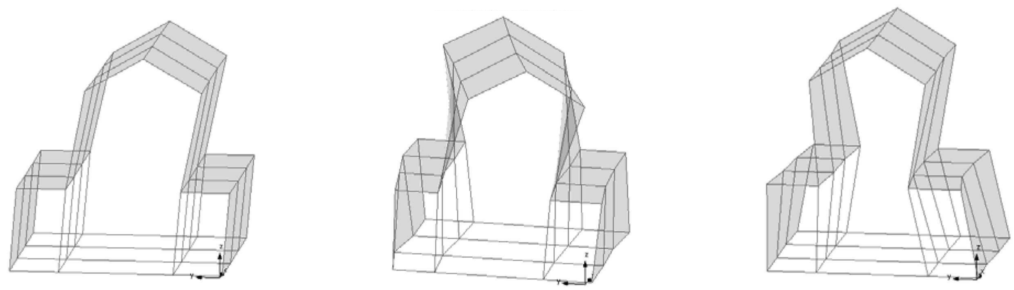


Figure 12: Mode shapes of vibration of the original model

Mode	Prototype/model Hz	Scaling factor required	achieved
Transveral	3.8/8.3	2.35	2.18
Torsional	6.8/15.7	2.35	2.31
Vertical	7.6/19	2.35	2.50

Table 1: Resonant frequencies model/prototype.

Performed seismic tests

The shaking table tests in this phase were performed by using the Calitri earthquake, N-S component, scaled in time by √5.5. During this testing phase, 22 tests were performed with an intensity of 0.004 to 0.14 g. To check the frequency decrease, random and sine-sweep tests were performed after the seismic test with intensity of 10% g. Until the final test, some micro-cracks were observed on vaults and arches at the level of the buttresses. During the final test with 14% g, a severe damage to the model was produced, with development of two main cracks along the central arches of the model and many cracks in the arches and vaults of the buttresses. The reason for this damage was the frequency content of the applied excitation, which was close to the self frequencies of the model. The photos of Figure 13 show details of damage after testing in phase 1. It is interesting to make some comments on the frequency decrease of the model during the seismic testing. It can be observed that the frequency is decreased for more than twice (from 5.8 to 2.5 Hz) after the final test, which leads to the conclusion that the model is near collapse. The push-over curve obtained during the testing of the original model at the levels of the buttresses, and of the central arches is presented in Figures 14a and 14b, respectively. It can be noted that both curves have bi-linear shapes. The cracks at the buttresses appeared at a response acceleration of 0.12 g and relative displacement of 2 mm, when stiffness degradation occurred as well as progressive deformation up to 12 mm, announcing heavy damage state. The cracks in the arches appeared at response acceleration of 0.25 g and relative displacement of about 5 mm, achieving progressive deformation of 35 mm for a small increasing of the acceleration, announcing a near collapse state.

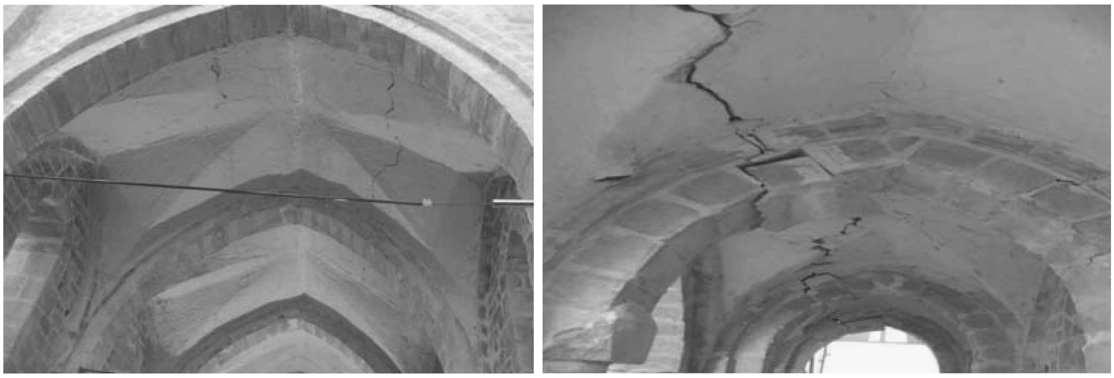
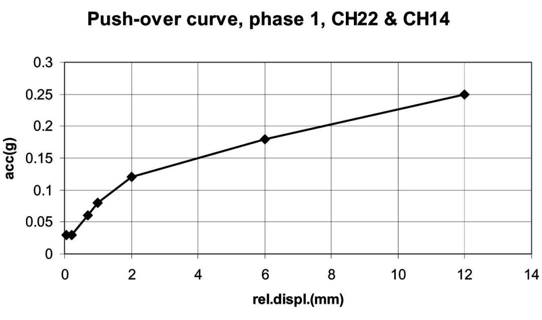
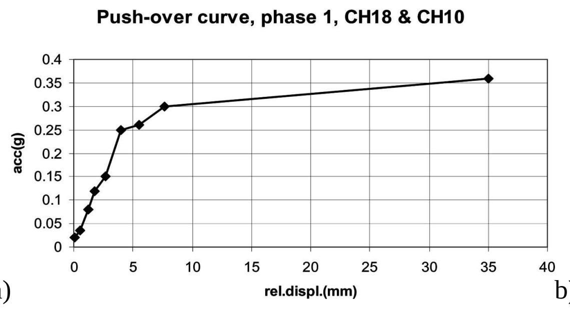


Figure 13: Damage of the model after the first test



a)



b)

Figure 14: Pushover curve at the level of buttresses (a) and arch (b)

Phase 2- Testing of the strengthened model

Repairing of the model

After the testing of the original model and the development of the failure mechanism, the cracks were repaired by an expansive aluminium-cement mortar. The intention was to recover the contact between the masonry parts without altering significantly the original material properties.

After this reparation, ambient vibration measurements were performed to check the natural frequencies of the model. The obtained value in translational direction was f=9.7 Hz, which is higher than the frequency obtained for the original model f=8.3 Hz, showing that the strength of the model was completely recovered.

Strengthening of the model

The proposed methodology for strengthening consisted in the introduction of pre-stressed vertical and horizontal carbon fiber ties at given position in the model (De Matteis et al., 2009). In the first session of phase 2 (phase 2A), only the upper horizontal ties at the external side (the main central arch and the arches of the buttresses) were introduced; while in phase 2B, horizontal ties at the internal side were added, as shown in Figure 15. In both sub-phases A and B, vertical ties in the columns were always present. The pre-stressing forces in the ties were checked several times during the shaking table testing.

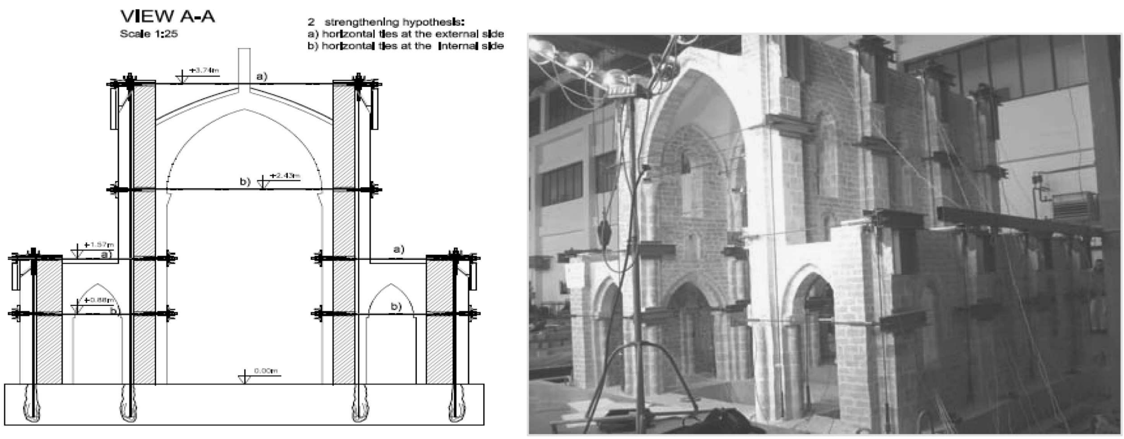


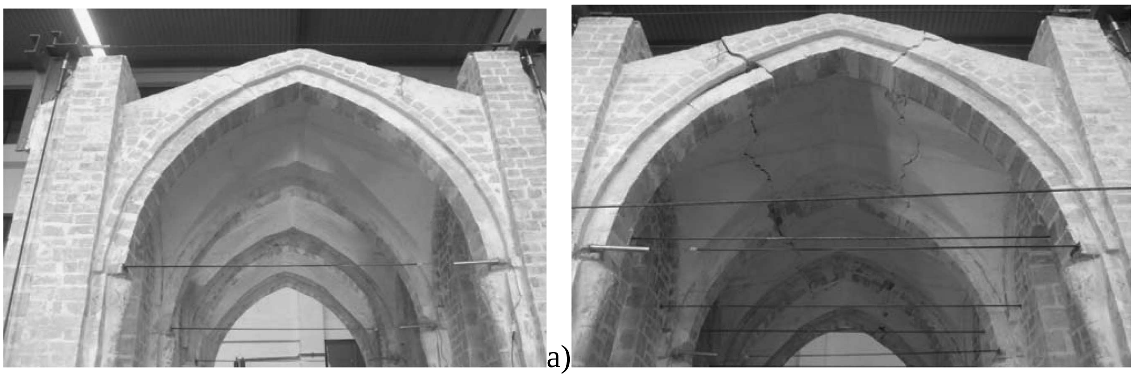
Figure 15. Strengthening of the model with pre-stressed vertical and horizontal external and internal carbon-fiber ties

Seismic tests

During testing phase 2A, 11 seismic tests were performed, with input acceleration between 0.03 g to 0.28 g, by applying the scaled time history of the Calitri Earthquake, as in phase 1. Testing in phase 2A was performed until the development of severe cracks at the level of the main arches and the vaults. In testing phase 2B, when internal ties were also applied to the model together with the external ties, 6 seismic tests were performed with input intensity 0.14 g to 0.40 g.

Actually, the same characteristic horizontal cracks as in phase 1 occurred (Fig. 16a). During the final test of phase 2B, with intensity of 0.4 g, the cracking mechanism developed completely. Severe damage in the model was observed and some of the stones of the central arch fell down. Figure 16b shows details of damage after the accomplishment of the tests.

The push-over curves at the top of the central arch and the top of the buttresses in phases 2A and 2B, respectively, are given in Figures 17 and 18.



a) b)

Figure 16. Damage in the model after the phase 2A (a) and 2B (b)

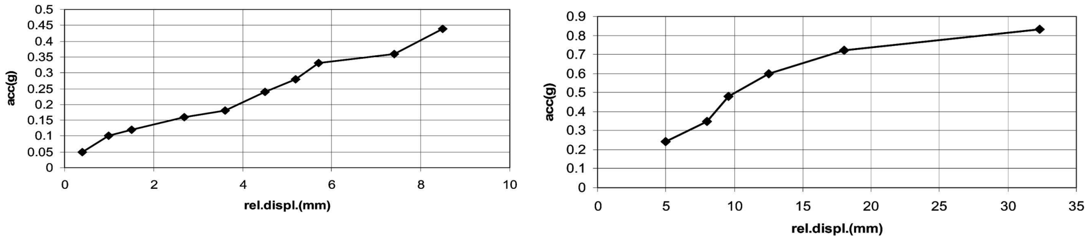


Figure 17. Push-over curves at the top of the central arch obtained in testing phases A and B

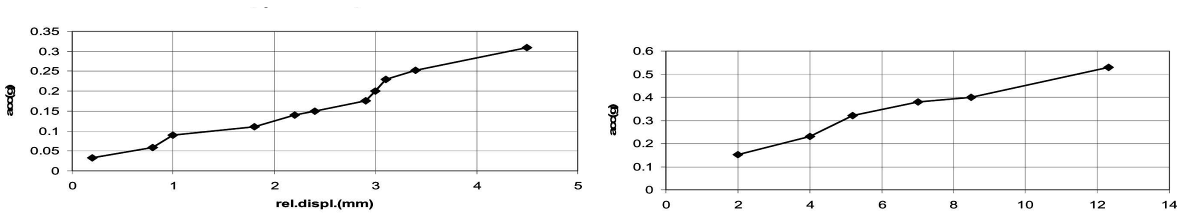
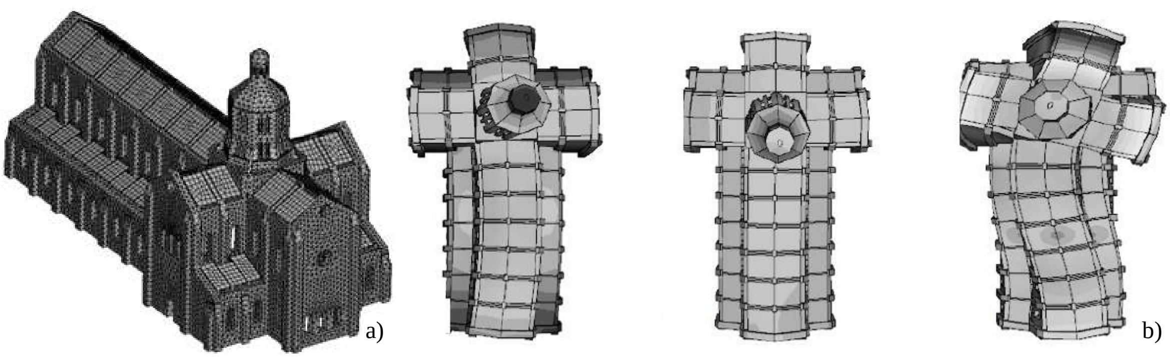


Figure 18. Push-over curves at the top of the buttresses obtained in testing phases A and B

4.4 Numerical analysis

The results achieved in the experimental tests have been numerically reproduced under numerical way (De Matteis et al., 2009). A global 3D view of the church is given in Figure 19a. The material properties were determined on the basis of experimental tests. The first three vibrational modes obtained for the numerical model are shown in Figure 19b and the corresponding eigenvalues compared with the experimental frequency values achieved from ambient vibration tests are reported in Table 2, where it is apparent that a very good agreement among results was reached.



a) b)

Figure 19. The FEM model (a) and the main modal vibration shapes (b) of the Fossanova church

Direction	Detected natural frequency value [Hz]		
	Ambient Vibration	FEM (1)*	FEM (2)**
Transversal	3.8	3.77	3.65
Longitudinal	4.6	4.63	4.30
Torsional	6.8	6.45	6.15

** with roofing wooden elements*
***without roofing wooden elements*

Table 2: Experimental-numerical comparison in terms of frequencies

As final step of the numerical activity, a non linear analysis was also performed by assuming a macro-modelling-smeared cracking approach (ABAQUS) for masonry. The modal analyses revealed that the more important structural part of the actual complex had to be recognized in the three-central bays of the main nave, as shown in Figure 20a.

This model was considered for the numerical analysis and a symmetry condition about the mid vertical transversal plane was adopted in order to reduce the computational effort. The model was fixed at the base and solid C3D4 (four nodes linear tetrahedron) elements were used with a refined mesh having an average length of 100 mm (Fig. 20b). A proportional load distribution was introduced as an external acceleration applied at the masses.

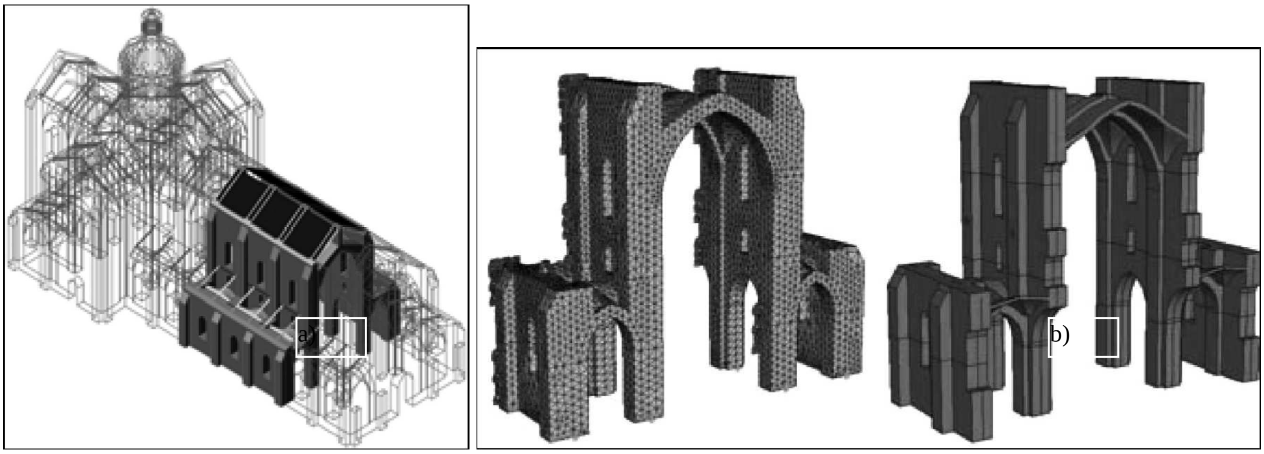


Figure 20. Selected seismic resistant unit in transversal direction (a) and the FEM model for push-over analyses (b)

The analyses on the FEM model showed that the same sequence of damage phases detected in the experimental activity were achieved also in numerical way. Finally, it was noted that when the ultimate experimental control point displacement was applied (d =34 mm), a large deformation appeared at the base of the central piers, whose sections achieved the assumed limit compressive stress.

The global response of the non linear model is synthesized in Figure 21a, whereas in Figure 21b the shear capacity curve of the model, expressed in terms of base-shear/gravity factor versus control point displacement, is plotted. It is apparent that a very good agreement between the non-linear numerical analysis and the experimental tests was achieved in terms of evaluation of the structural capacity of the system.

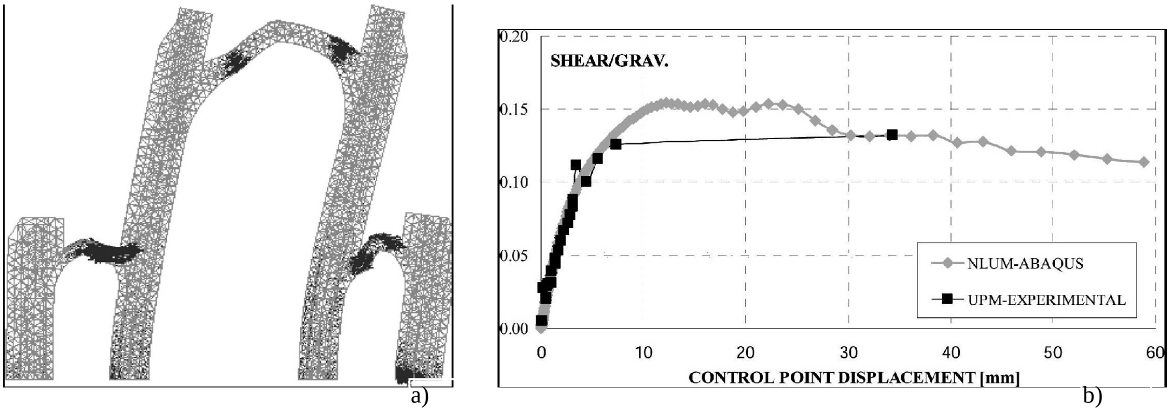


Figure 21. Crack distribution at collapse (a) and experimental-numerical comparison in terms of capacity curves (b)

5 THE GREEK TEMPLE MODEL

5.1 Design phase

Experimental shaking table tests were also carried out at the Earthquake Engineering Laboratory of the National Technical University of Athens on temple sub-assemblages with two different configurations: freestanding columns (Fig. 22a) and columns in a row (Fig. 22b) or in a corner connected by architraves (Fig. 22c) (Dasiou et al., 2009a).

Figure 22: Freestanding (a) and with architraves (in a row (b) and in a corner (c)) columns (C1, C2 and C3 from left to right)

Each column consists of a base, twelve drums and a capital. In order to evaluate the effect of various parameters, the columns were made similar, but not identical. One difference concerns the material used for each column. The capital and the drums of column C1 are made of Dionisos marble, while the base is made of white marble from Kavala. All the blocks of Column C2 and Column C3 are made of Pentelikon marble and Kavala marble respectively. Another important difference is that Column 1 has entasis. In architecture, entasis is the application of a convex curve to a surface for aesthetic purposes and, in particular for classical columns, a small bulge that appears almost at the middle of its height. In accordance with the ancient cuttings, two types of clamps were constructed (Clamp B and Clamp C).

The positioning of the connections in the architraves can be seen in Figure 23. The material of the T-shape clamps used in the experiments is different from the one used in real interventions. In the restoration process, the use of titanium is necessary to avoid corrosion, but since there was not such need in the experiments, the clamps were made of steel. To secure the connection, leafs of lead were placed between the clamps and the marble. Cement mortar, which is used in practice instead of lead, could not be used in the experiments, due to the scaling restriction (the grain size should be to small).

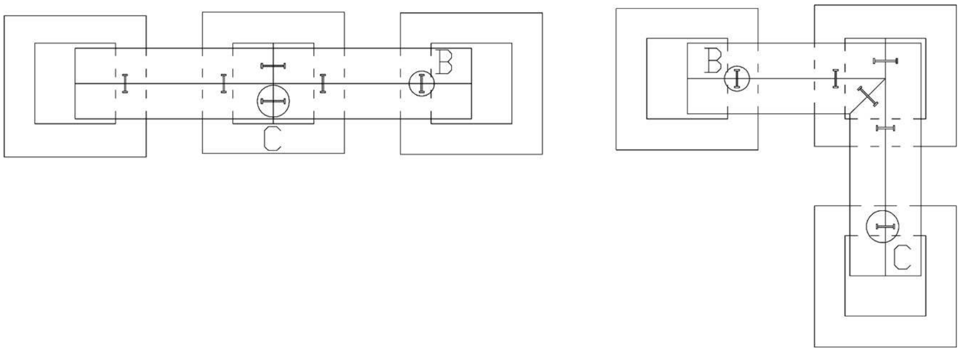


Figure 23: Plan view of the two configurations of the columns and the positioning of the clamps B and C

After these experiments were performed, a new series of tests on these systems upgraded with double T-clamps able to connect architraves was carried out (Mouzakis et al., 2002). The connections are secured in position with cement mortar. The basic criterion for the design of clamps is that, in case of a seismic event, the clamps should absorb the seismic energy and fail before the arble suffers any damage. However, the influence of these connections has not been studied experimentally.

5.2 Testing ser-up

For testing, the following two seismic events, with different characteristics, were chosen from destructive earthquakes in Greece:

- The Kalamata Earthquake (13 September 1986). The magnitude of this earthquake was M=6.2. The accelerogram was recorded on hard soil in a distance of about 9 km from the epicentre. The duration of the strong motion is about 6 sec and the maximum horizontal acceleration is 0.27 g.
- The Lefkada Earthquake (14 August 2003). The magnitude of the earthquake was M=6.4 and the maximum recorded accelerations were 0.42 g and 0.34 g in the two horizontal directions.

Nine displacement transducers, connected, in sets of three, to the midpoints of one side of each capital and to fixed point on the shaking table at the other end, were used in order to record the displacements at the top of the columns. With this setup, the instruments were recording the displacements along their inclined axis and, in order to calculate the displacements of the capitals in the three global axes, proper computations were needed. For each reference point, this was achieved by calculating the intersection of three spheres, centered with the fixed ends of the three transducers connected to it. In addition to the displacement transducers, accelerometers were also used, placed in various positions on the capitals or the architraves.

5.3 Experimental results

Two tests were carried out with the three columns standing free (unconnected). In the first experiment, the columns were subjected to the two horizontal components of the Lefkada record downscaled to 40% of its amplitude. Figure 24a shows the time histories of the absolute displacements of the capitals of the three columns in the longitudinal direction. During the test, each column exhibited different behaviour. The middle column (C2) practically did not move, while the other two, C1 and C3, experienced large displacements. In the case of column C1, no sliding was observed between the drums and the column was rocking like a monolithic one.

In the second experiment, the Kalamata Earthquake, scaled down to 40%, was used as the base excitation. The columns exhibited different behaviour compared to the first experiment. Column C1 experienced small displacements, while the rotations and displacements of columns C2 and C3 were quite large and similar to each other. In Figure 24b, a comparison of the time histories of the absolute displacement (in the lateral direction) of the capital of column C2 for experiments 1 and 2 is presented.

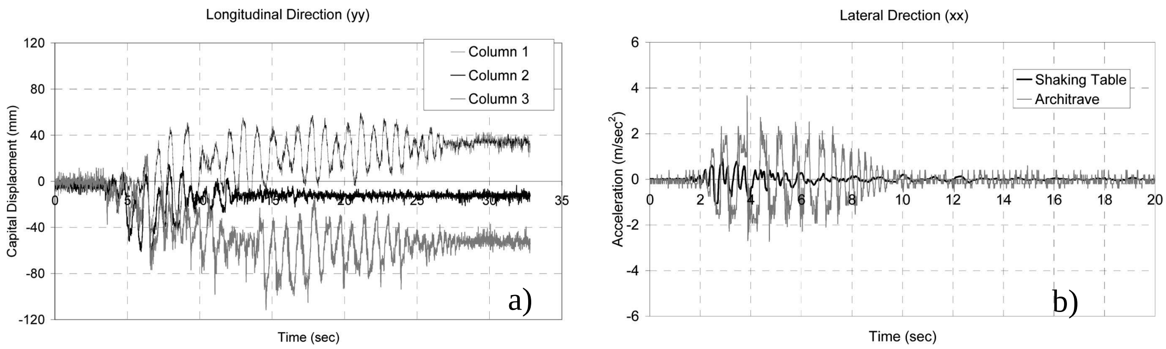


Figure 24: Time histories of the absolute displacements of the capitals of the three columns for Test No.1 (a) and comparison of the time histories of the displacement of the capital of column C2 for tests 1 and 2 (b).

40% of the Kalamata Earthquake. The columns exhibited very small displacements. Columns C1 and C2 and the architraves connecting them moved in phase. All the rocking was occurring at the in the case of temple sub-assemblages without architrave connections, the columns in a row were examined. The test was divided in two phases: in the first phase, the columns were subjected to first drum of the two columns. Column C3 behaved differently: there were many openings and closings of the joints at the middle and the upper part only. Small residual displacements were observed only at column C3 and the architraves.

Since the permanent displacements were not significant, it was decided to submit the specimen to 50% of the Kalamata motion without repositioning the blocks to their initial configuration (Phase 2). This situation corresponds to monuments with structural elements displaced from their intact position, due to previous earthquakes. Figure 25a shows the time histories of the absolute displacements of the capitals of the three columns in the lateral direction. It is noted that the displacements of all columns during the second phase of the experiment were almost twice the ones recorded in the first phase (Fig. 25b). This should not be attributed only to the amplification of the base excitation, but a significant part of the displacement increase might be owed to the initial residual displacements of the drums and the architraves after the first phase.

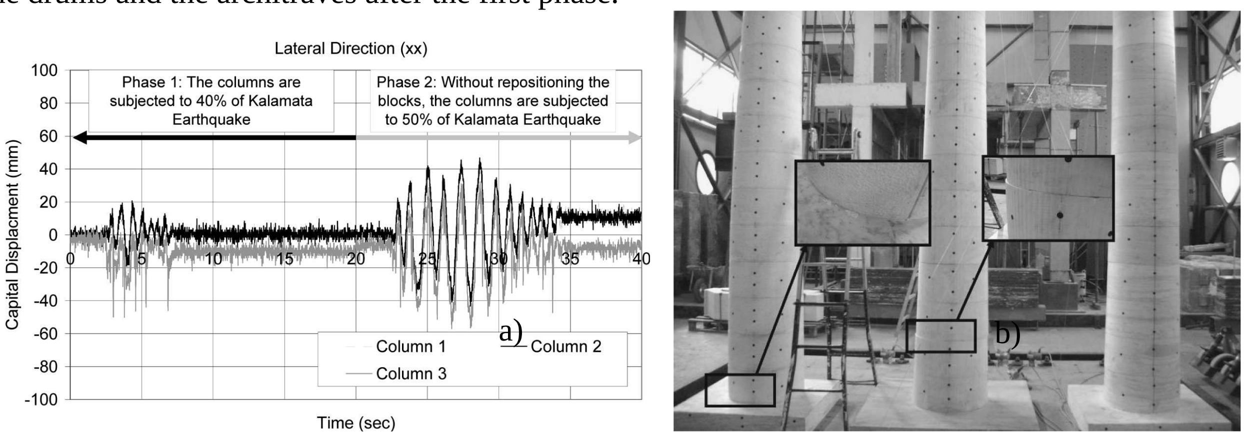


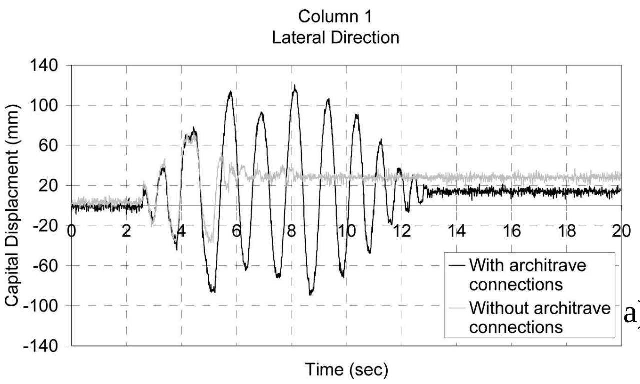
Figure 25. Time histories of the displacement of the capitals of the three columns in test No. 3 (a) and final position of the three columns after test No. 3 (b)

As verified by the experiments, the architraves are the most vulnerable part of the structure and the first to collapse. The purpose of the connections is to prevent this collapse during a seismic event. Although this intervention is used often, the influence of the clamps on the seismic response of the structure has not been studied extensively, neither experimentally nor numerically.

In a further experiment (No. 8), the specimen in a corner configuration with architrave connections was subjected to the Kalamata earthquake scaled down to 40%.

Figure 26a shows the time history of the response of the capital of column C1. On the same diagram, the corresponding displacement for the similar experiment No.7 without connections is also shown for comparison. It was observed that during the first 6 seconds, the column response is not affected by the architrave connections. However, after the strong part of the excitation, the displacement of the column is larger if the architrave beams are clamped. The residual displacement is also larger. Contrary, it was observed that the displacements of the architraves are significantly reduced if they are connected with clamps (not shown in the figure).

Comparison of these experiments with experiments No. 6 and 7 (Fig. 26b), in which the architraves were not connected and the danger of falling was obvious, leads to the conclusion that the use of clamps is in favour of the overall stability of the structure, in spite of the small increase in the response of the columns.



a)

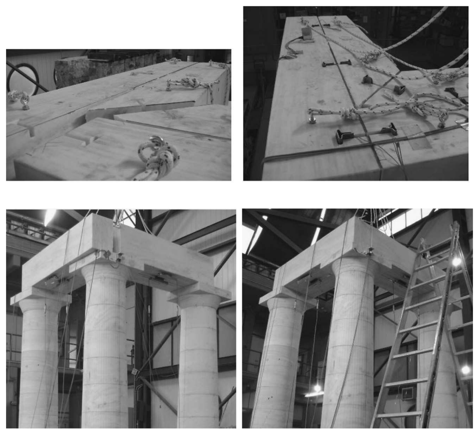

b)

Figure 26. Comparison of the time histories of the displacement of the capital of column C1 for tests No. 7 and 8 (a) and final dislocation of architrave beams after tests No. 7 (left) without connections, and 8 (right) with connections (b).

5.4 Numerical analysis

Three numerical models were created in order to evaluate their efficiency in predicting the seismic response of the considered ancient temples (Dasiou et al., 2009b). For two of them the Finite Element program ABAQUS was used. The geometry of both models was an exact representation of the test specimens. In model 1, the structural elements were assumed to behave as three-dimensional rigid blocks; the contact between the blocks was modelled by means of elastic spring connectors, including a damping coefficient. In model 2, the drums were modelled as three-dimensional elastic deformable bodies (Fig. 27 a). The contact between adjacent blocks was defined by means of mechanical contact-surface interaction elements with friction coefficient equal to 0.70. When surfaces are in contact, these elements transmit shear and normal forces across the interface. In the normal direction, a hard contact approach is considered and interpenetrating is regarded as non-physical and prevented (ABAQUS). The code allows the opening of joints, even complete detachments of the blocks and automatically detects new contacts as the calculation proceeds. Model 3 was created using the code 3DEC of Itasca consulting Group (3DEC), which is based on the distinct (or discrete) element method (Fig. 27 b). The Code was initially designed for the analysis of the behaviour of rock masses, which are modelled as assemblies of discrete rigid bodies and discontinuities are considered as boundary conditions.

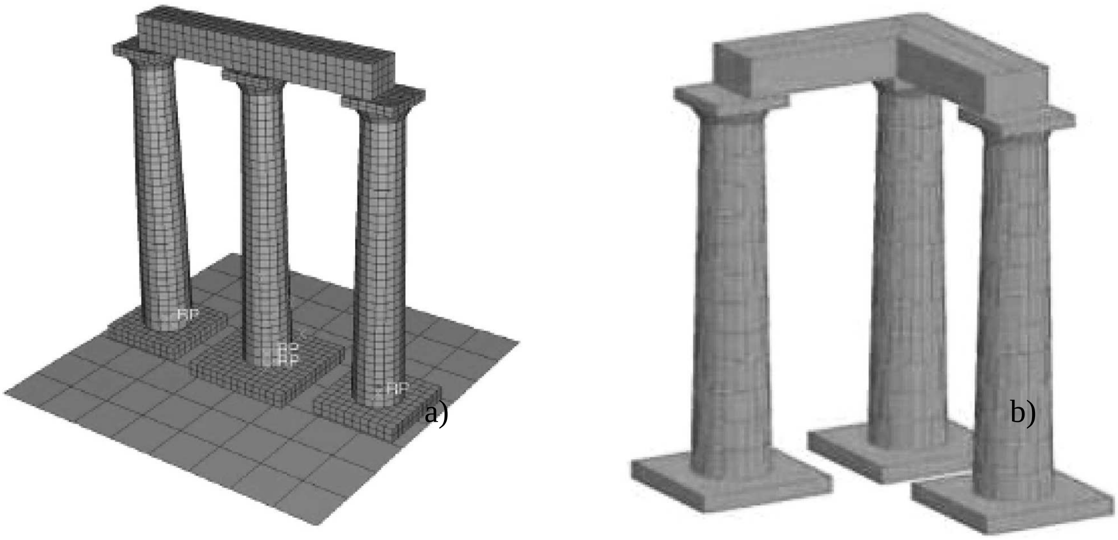


Figure 27. FEM model 2 implemented with ABAQUS (a) and FEM model 3 implemented with 3DEC (b).

Figure 28 presents the comparison of the numerical results for models 2 and 3 with the experimental data for the freestanding columns C1 and C3 obtained in 2007. The accuracy of the numerical analyses is again quite satisfactory.

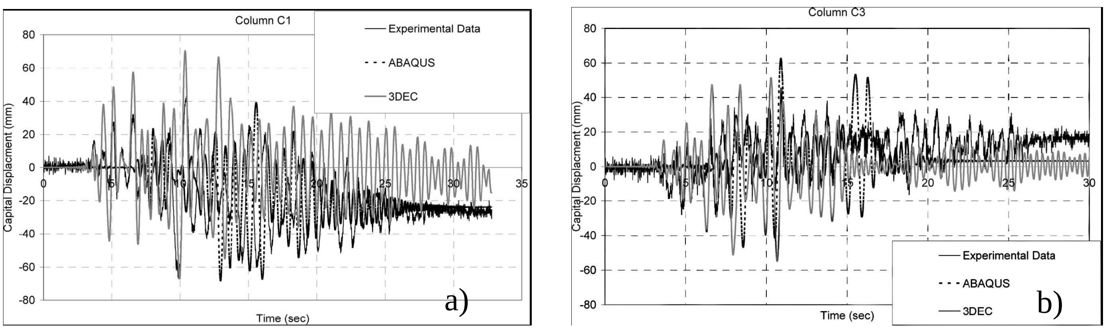


Figure 28. Time histories of the displacement of the capital of freestanding column C1 (a) and C3 (b)

The data from three experiments with specimens in a row or in a corner were reproduced numerically using model 3. It was evident that the numerical model can predict the experimental data very accurately in all cases, better than the response of freestanding columns. Even the out-of-plane slippage could be predicted. This should be attributed to the fact that, as the experiments showed, configurations of columns connected with architraves are less sensitive than freestanding columns.

Finally, it was concluded that:

- Model 1 presented the less accurate results. The results obtained with models 2 and 3 are similar and quite satisfactory.
- More accurate results were obtained for the columns connected with architraves, because the response is less sensitive in this case.
- Model 2 is significantly more time consuming compared to model 3 that makes it practically prohibitive for structure with many elements. However, the stresses induced at the blocks can be obtained. Thus, the selection of the appropriate model depends on the size of the structure and the results needed.

6 THE BYZANTINE St. NiCHOLAS CHURCH MODEL

6.1 Design phase

The shake-table test of the Byzantine church model was performed in Skopje in the period July 15–16, 2008 (Tashkov et al., 2009b). The model of the church was designed to the length scale of 1:3.5 according to the “gravity force neglected” modelling principles (Fig. 29).

The structural system of the church consists of façade walls constructed of hewn stone and brick bound with mortar. In the interior, there are two symmetrically placed rows of columns interconnected by vaults. The wall elements were constructed in the typical Byzantine style of building with two faces of brick and stone masonry and the intervening space filled with a care of rubbish set in a great quantity of mortar. The columns were constructed of the same materials. The vaults and the arches were constructed of hewn tuff, while the tambour and the domes were constructed of bricks in lime mortar. The walls were set on massive masonry foundations. The church in its full size is proportioned 12.7×8.2mat plan and is surmounted by a dome with a height of 10.6 m.

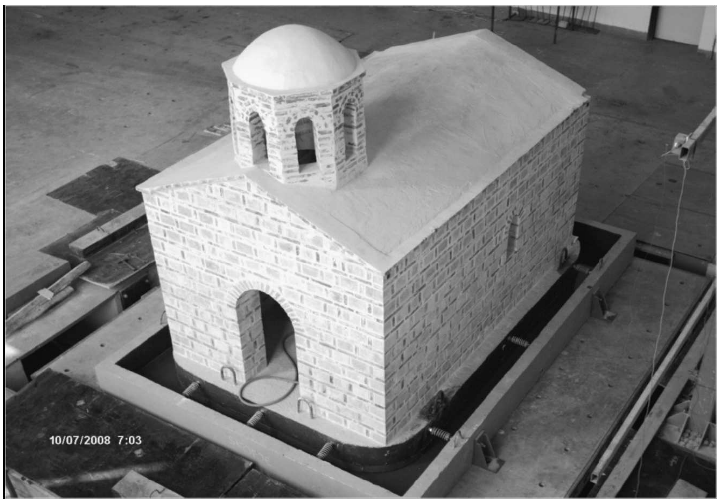


Figure 29. Byzantine St. Nicholas church

The structural system of the church consists of façade walls made of hewn stone and brick bound with mortar. In the interior, there are two symmetrically placed rows of columns interconnected by vaults. The wall elements were constructed in the typical Byzantine style of building with two faces of brick and stone masonry and the intervening space filled with a care of rubbish set in a great quantity of mortar. The columns were made of the same materials. The vaults and the arches were made of hewn tuff, while the tambour and the domes were of bricks in lime mortar. The walls were set on massive masonry foundations. The real church has the full size in plane of 12.7×8.2 m and it is surmounted by a dome with a height of 10.6 m.

6.2 Testing set-up

The model of the church was built at the IZIIS Laboratory in scale 1:3.5 and tested on the biaxial seismic shaking table. The main objective of the testing was to experimentally investigate the effectiveness of the proposed system for increasing the seismic resistance of this type of historical building, which consists on a special type of base isolation, so called ALSC floating-sliding system (Tashkov et al, 2009b).

The seismic shake-table testing was performed in two main phases:

- phase 1 : testing of the base-isolated model with the ALSC floating-sliding system:
- phase 2 : testing of the original fixed-base model.

The model with the sliding base was tested first, because it was expected that no damage will occur. That was confirmed even under the maximum capacity of the shaking table (1.5 g). The second phase was done with the fixed base model, which simulates the original structure on the site.

The automatic system for activation of the working pressure under the sliding plate and keeping it constant as long as needed plays an important role in the effectiveness of the ALSC base-isolation system. Basically, the automatic pressure control system consists of a steel reservoir filled with liquid under pressure, kept constant by means of a compressor and a servo-valve. The servo-valve is closed until the pressure of the liquid under the sliding plate is decreased for more than 10%. After that, the valve opens and the liquid from the steel reservoir comes to the sliding plate increasing the pressure up to the required level. Having this system, the structure does not need to be subjected to a permanent pressure for a long time waiting for the earthquake. It will be activated just a few seconds before the shear seismic waves attack the structure. In the case of the shake table test, the system for pressure control was activated a few seconds before the shake-table started to move and remained active during the testing time (few hours). The testing procedure applied to the model consisted of several steps:

- Preliminary tests (ambient vibration and random excitation) for definition of the dynamic characteristics of the model, in order to check the stiffness degradation of the model produced by micro- or macro-cracks developed during the tests;
- Seismic testing under selected earthquake records (Montenegro Petrovac 1979 earthquake).

To follow the dynamic response during the seismic shake-table testing, the original model was instrumented at characteristic points with accelerometers and displacement transducers, for measuring the absolute displacements (LPs) as well as the relative displacement between the fixed basin and the sliding foundation plate in the direction of the excitation (LVDTs).

6.3 Experimental results

Testing phase 1: Base-isolated model by ALSC system

The test specification is given in Table 3, together with some response parameters for the selected tests.

		Acceleration(g)			Relative displ.(mm)	
Test	Excitation	basin	plate	top	tambour	basin-plate
1	Harm 7 Hz	0.80	0.15	0.12	0.20	7.00
2	Harm 5 Hz	0.60	0.15	0.16	0.22	3.00
3	Petrovac	0.20	0.20	0.18	0.15	3.50
4	Petrovac	0.45	0.25	0.18	0.20	6.00
5	Petrovac	0.80	0.25	0.25	0.30	15.00
6	Petrovac	1.20	0.32	0.35	0.38	20.00
7	Petrovac	1.45	0.25	0.22	0.28	35.00

Table 3: Performed tests in phase 1 – base isolated model

The first two tests were performed with harmonic excitation of 7 and 5 Hz. Other tests were performed with the Petrovac, 1979 earthquake. The earthquake was scaled with a scaling factor of 3.5 according to the modeling principles of "*gravity force neglected*" model. It was compressed 3.5 times of the time scale and 3.5 times of the acceleration scale (0.43 g×3.5=1.5 g). The model was tested with 5 different levels of input acceleration: 0.45 g, 0.8 g, 1.2 g and 1.45 g. Uniform sliding of all parts of the structure was recorded (sliding plate, walls and tambour). The maximum response acceleration of the sliding plate in all cases was about 0.2–0.3 g. This was actually a limitation of the transmissibility of the forces from the basin to the sliding plate. This level of acceleration did not produce any cracks in the model, except an increased relative sliding displacement between the basin and the sliding plate. Under maximum input acceleration of 1.5 g, the relative displacement was about 3.5 cm.

Testing phase 2: Fixed base model

Before testing, the model was fixed by steel bolts and wooden beams. The pressure of the liquid was set to 0. Then, the model was transformed into a classical fixed base structure. The input acceleration was changed in several tests from 0.1 g to 0.7 g. The amplification of the response was: 1.5–2.0 for the top of the wall and 3–4 for the top of the dome. The test results are given in Table 4. The damage pattern is shown in Figure 30. The push-over curve (acceleration response versus relative displacement at the top of the wall) is presented in Figure 31.

		Acceleration (g)			Rel.displ (mm)	
Test	Excitation	basin	plate	top	tambour	basin-plate
1	Petrovac	0.10	0.18	0.25	0.20	0.50
2	Petrovac	0.15	0.25	0.38	0.25	0.70
3	Petrovac	0.20	0.30	0.45	0.40	1.00
4	Petrovac	0.27	0.42	0.60	0.70	1.50
5	Petrovac	0.32	0.50	0.70	0.80	2.00
6	Petrovac	0.38	0.55	1.00	1.00	3.00
7	Petrovac	0.40	0.60	1.50	1.10	13.00*
8	Petrovac	0.45	0.65	/	2.00	/
9	Petrovac	0.50	0.70	/	2.50	/
10	Petrovac	0.60	1.00	/	3.00	/
11	Petrovac	0.70	1.20	/	7.00	/**

*collapse of the tambour.
**heavy damage to the openings and cracks in the walls.

Table 4: Performed tests in phase 2 – fixed base model

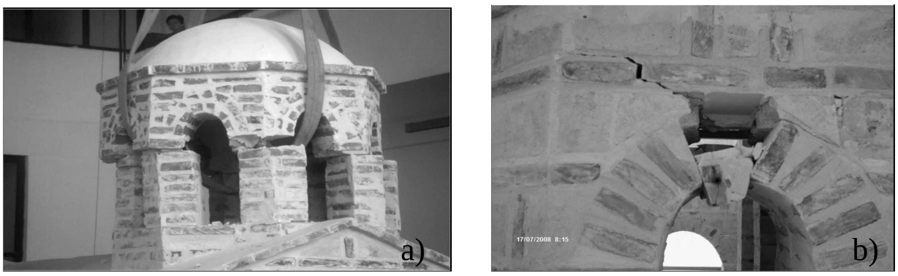


Figure 30. Collapse of the tambour (0.45g) (a) and damages to wall and openings (0.70 g) (b)

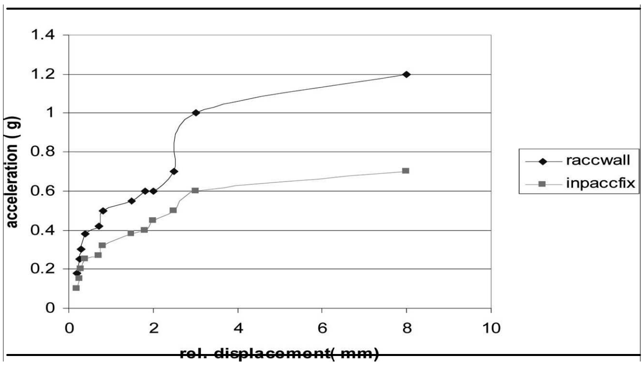


Figure 31. Pushover curve of the church

6.4 Numerical analyses

Two 3D numerical models of the scaled church were set up (Fig. 32) (Kokalanov et al., 2009). The experimental investigations were used for the verification of the mathematical model. The elastic parameters are referred to initial values which were calibrated on the basis of first random vibration tests performed on the original undamaged structure. The non-linear properties were determined on the basis of both compression and shear experimental tests carried out on masonry wall specimens (Gramatikov et al., 2005).

Linear static and dynamic analyses of the models were performed. The results obtained by modal analysis of the model were compared to the experimental results obtained by ambient vibration tests on the actual structure (Table 5).

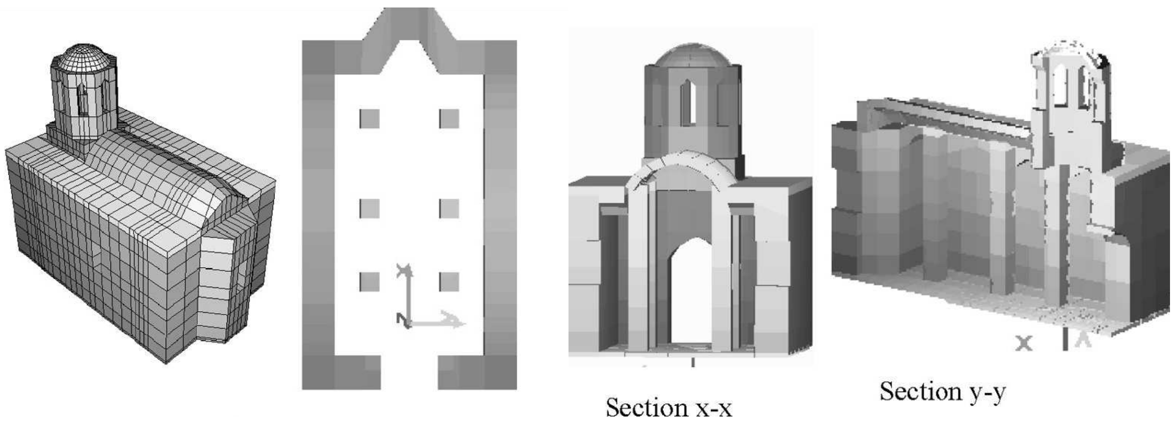


Figure 32. FEM model of the St. Nicholas church

	Module of elasticity MPa	Frequency [Hz]	
		x-x	y-y
Ambient vibration tests		2.8	2.8
Numerical model	290	2.85	2.86

Table 5. Comparison among experimental and numerical frequencies

A linear dynamic analysis was performed by using an actual earthquake record – time history analysis. The record of Petrovac (Montenegro) earthquake was used for simulation. The results from the dynamic analysis confirmed that the most vulnerable part of the church is the lower part of the tambour.

The seismic capacity of the church was obtained by a nonlinear analysis. It is common to use two different loading schemes: a constant acceleration along the total height and the loading defined using the first eigenform in the load direction. In the implemented pushover analyses, a uniform acceleration along the horizontal direction was applied. The numerical results showed that, when collapse of the tambour happened, the rest of the structure was still in good condition. A new model of the church without the tambour was created. This model was used to determinate the seismic capacity of the main structure.

The collapse load on the investigated prototype is 0.68 g, which is very closed to the experimentally determinate collapse of the structure, which happened at seismically exaltation of 0.7 g.
In general, it should be noted that the crack patterns obtained from numerical analyses have always to be ascribed to the attainment of tensile or shear strength, while the compressive resistance is never exceeded. According to the numerical model, the first diagonal tension cracks occur in the shear walls, just over the door opening.

After the analysis on the original church, a new FEM model of the retrofitted church by ALSC system was created (Figure 33).

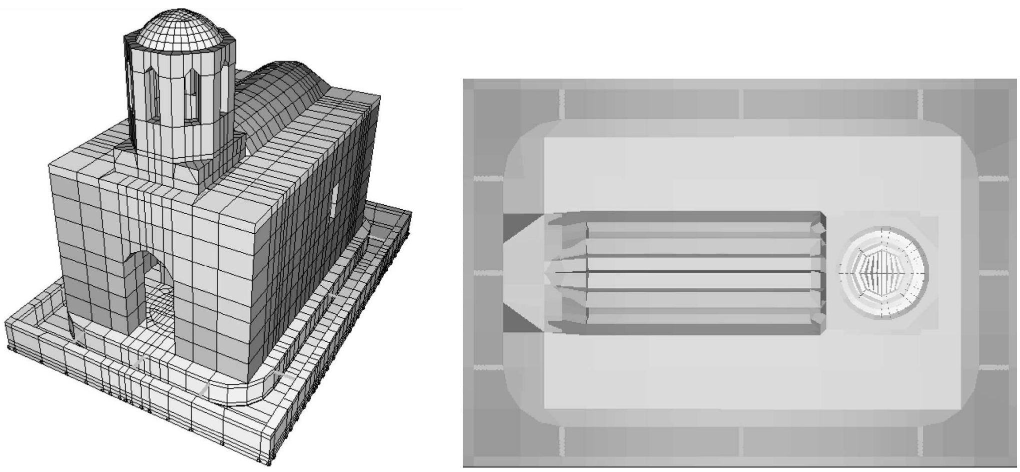


Figure 33. FEM model of the St. Nicholas church retrofitted by ALSC system

The comparison between the original church and the upgraded one by the ALSC system shows frequency shifting from 4.0 Hz (original church) to 1.0 Hz (upgraded church). The ALSC system consequently provides a reduction of relative displacement between top level and the base, from 74 mm to 1 mm, which means that the church translate horizontally over the smooth base in the range of +/−100mm, under an uplifting working pressure of 0.65 bars.

Therefore, the comparison among results shows successful upgrading of the structure by ALSC system, since the acceleration and shear force are reduced about 4.5 times comparing to original structure.

7 CONCLUSIONS

Some experimental and numerical activities, framed within the FP6 EC PROHITECH research project “Earthquake PROtection of HIstorical Buildings by Reversible Mixed TECHnologies”, on the application of RMTs to the historical and monumental constructions of the Euro-Mediterranean area, are presented and discussed in this paper. Within this range, shaking table tests have been carried out on four large scale models of the following monumental constructions: the Mustafa Pasha Mosque in Skopje, the Gothic Cathedral in Fossanova, the Parthenon temple in Athens and the St. Nikola Byzantine Church in Psacha.

Experimental testing on the Mustafa Pasha large scale model was performed to investigate the seismic capacity of the monument after applying a reversible strengthening methodology, for both the minaret and the mosque. Shaking table testing was performed for the original model and for model strengthened by C-FRP. The strengthening of the minaret by application of a C-FRP strips and wraps enabled stiffening and increasing of its bending resistance. The mosque model behaviour, after strengthening by incorporation of carbon rods in longitudinal mortar joints and horizontal belt courses at the base of the tambour as well as at the base of the dome by using C-FRP wraps, was evidently different in respect to that of the original model. Under tests of moderate intensity, the existing cracks were activated, but during the subsequent more intensive tests, the failure mechanism was transferred to the lower zone of the bearing walls, in the direction of the excitation, where typical diagonal cracks occurred due to shear stress. Considering the obtained experimental results for the original model and for the strengthened one, it can be concluded that the applied strengthening technique has significantly improved the seismic resistance of the monument. The results of this experimental investigation were the starting point for the Cultural Heritage authorities of Macedonia to adopt the same consolidation system for the real mosque (Mazzolani et al, 2009).

The experimental study performed on the Fossanova church demonstrated that the seismic performance of the model was significantly improved by the applied strengthening methodology, which represent a modern interpretation of a traditional technology based on the use of ties, which are able to confine the structure. In particular, the ties are mainly effective in preventing and controlling the relative displacements of the columns. This was manifested by controlled “opening” along the two main cracks developed at the level of the main arch during the intensive seismic shaking. In order to make an effective comparison between the original model and the strengthened one, it is important to compare the input intensity which provoked serious damage to both models. For the original model the critical input intensity was 0.14 g, while it was 0.28 g for the strengthened model in phase 2A, and 0.40 g in phase 2B. These values clearly show the effectiveness of the applied strengthening technique.

From the performed experimental activity on the Greek temples the following conclusions were drawn:

- Columns of ancient temples are more stable and their behaviour is less sensitive to small changes of the geometry or the excitation, when connected with architraves than when they are standing free.
- The response is larger for corner configurations than for columns in a row. Especially the architraves of corner columns are the most vulnerable parts of the structure with high danger of collapse during strong earthquakes.
- The nonlinearity of the response was verified during the experiments. Thus, an increase in the base motion does not necessarily result in an increase of the response. In some cases, the residual displacements were reduced by amplifying the excitation.
- The connection of the architrave beams by clamps leads, in general, to a significant decrease of the beams sliding on the capitals and reduces the danger of their collapse. However, the response of the columns might increase. Due to the ambiguous influence of the clamps, numerical investigation is recommended, before they are implemented in the restoration process.

Finally, the comparative test between the base-isolated model by ALSC system and the classical fixed base model of the byzantine church clearly shows the superior behaviour of the ALSC floating-sliding base-isolation system. The excitation of 1.5 g (maximum capacity of the shaking table and maximum peak acceleration of the Montenegro earthquake-scaled by a factor of 3.5) was not enough to produce any damage to the dome and to the walls. The tests show that the system can protect the structure in any frequency and/or amplitude range as well as against the strongest earthquake. Contrary, the fixed base model is very vulnerable. The amplification factor ranges from 1.5 for the walls to 3.0 for the dome. The consequence is an early damage to the dome (0.45 g) and severe damage to the walls (0.7 g) in the case of the model, whereas 0.12 g for the dome and 0.2 g for the walls in the case of the real structure.

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