STRUCTURAL ANALYSES OF THE KATHOLIKON OF DAPHNI MONASTERY WITH ALTERNATIVE INTERVENTIONS IMPROVING ITS OVERALL BEHAVIOUR

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ABSTRACT

The Athens 1999 earthquake provoked severe damage to the Katholikon of Daphni Monastery, a world heritage monument famous for its mosaics. Due to the importance of the monument and the complexity of problems to be faced, a “design in process” was adopted. Thus, the partition of the restoration works in two phases was decided and an earthquake monitoring system was installed. After the completion of the first phase of works, detailed structural analyses and safety checks in critical elements were undertaken, together with detailed dimensioning of optimum strengthening interventions of the second phase of works. Two solid FE models, calibrated using the monitoring data, were developed. In the first model, the dome and drum were studied employing static analysis. In the second model, simulating the whole structure, linear time history dynamic analyses were performed. These analyses confirmed the necessity of additional strengthening measures to alleviate the structure and reduce its vulnerability. In this paper, the numerical analyses for the selection of the 2nd phase optimum strengthening measures are presented. The aforementioned models were used for the qualitative evaluation of the efficiency of various strengthening interventions alone (ties, diaphragms, etc.) or in combinations. When necessary, the models were adequately modified and in some cases “cracks” were induced, to overcome shortcomings of elastic analysis. A synthesis of the results of those analyses and of all the detailed documentation led to the selection of optimum interventions, taking into account all the Values of the monument and above all the preservation of its mosaics.

Keywords: Numerical models, byzantine monuments, masonry confinement, ties, timber and steel diaphragms

1. INTRODUCTION

Daphni Monastery is one of the major middle-Byzantine monuments (Millet 1899, Stikas 1962, Bouras 1998, Delinikolas et al. 2003), inscribed in the World Heritage List of UNESCO since 1990. The Monastery is situated in a Neocene tectonic graben between the mountains Egaleo and Korydallos at the west side of Athens, 150 m away from the E-W trending marginal fault between the alpine Mesozoic limestone and the post-alpine deposits (Mariolakos et al. 2000). During the last two centuries, several earthquakes of Richter local magnitude exceeding M_L=6 have affected the monument (Delinikolas et al. 2003, Miltiadou-Fezans 2016). Due to the earthquakes, occurred since the 11th century, severe damages or partial collapses have developed in the buildings of the monastery, as well as the mural mosaics of the Katholikon (Delinikolas et al. 2003, Miltiadou-Fezans et al. 2004). Thus, when the 1999 Athens earthquake occurred (M_L=5.7), most of the buildings were in ruins, except for the Katholikon, part of the cells in the south courtyard, the cistern and the northern fortification wall (Fig. 1a).

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The aforementioned earthquake provoked severe damage to the Katholikon and its mural mosaics (Fig. 1b). The comparative examination of the historical, architectural and structural qualitative and quantitative data proved the inherent seismic vulnerability of the structural system of the monument and the pronounced deformability, especially of its upper part, through the interpretation of the observed severe damage (Miltiadou-Fezans et al. 2004). As a result, in addition to interventions necessary for the repair and strengthening of damaged masonry elements, adequate further strengthening measures had to be taken to improve the overall behaviour of the monument (in the form of tie-rods, diaphragms over the extrados of vaulted roofs to connect the vertical walls, pillars and piers, bracing of the windows of the dome, confinements of drum and piers, etc.).

![Figure 1](image1.png)  
(a)  
(b)

Figure 1. a) View of the Monastery from NE; b) damage of the NE squinch of the Katholikon.

In order to select and design the optimum strengthening interventions in a rational way, avoiding conservative assumptions that would lead to invasive and possibly unnecessary interventions, the decision was made to thoroughly investigate the seismic behavior of the monument, using reliable data both for numerical analysis and safety verifications. Thus, a program of in situ and in-laboratory investigations was set up. In parallel, the Hellenic Ministry of Culture has adopted a two-phase “design in process” (as proposed in Delinikolas et al. 2003, Miltiadou-Fezans et al. 2003). The first phase comprised the repair and strengthening of the masonry elements (stitching of cracks, local deep re-pointing, systematic hydraulic lime grouting), while the second included strengthening measures aimed at improving the overall behaviour of the structure (installation of ties-struts, confinement of structural elements, diaphragms, etc.).

Furthermore, after the implementation of emergency measures and before the application of any repair/strengthening interventions, an earthquake monitoring system was designed and installed (Mouzakis et al. 2008). Finally, experimental research programs, regarding the design of adequate grout compositions, their application methodology (Miltiadou-Fezans et al. 2008, Kalagri et al. 2010) and the mechanical behaviour of the three-leaf masonry before and after grouting (Vintzileou and Miltiadou-Fezans 2008) were also carried out, together with the application of NDT’s for the investigation of non-visible parts of the monument and controlling the effect and efficiency of grouting (Côte et al. 2004, 2008, Palieraki et al. 2008).

The design of intervention measures to be applied during the second phase was undertaken by a multidisciplinary working group in 2012 (Miltiadou-Fezans et al. 2012). It included all necessary structural analyses, checks and calculations for the assessment of the efficiency and the dimensioning of optimum interventions, as well as detailed drawings after in situ checks of their applicability (existing geometry, mosaic locations, etc.). In this paper, selected results of the numerical analyses performed with the purpose of choosing adequate interventions to improve the overall behaviour of the structure are presented.
2. BRIEF PRESENTATION OF THE KATHOLIKON AND ITS PATHOLOGY

The Katholikon (church) belongs to the octagonal type and preserves large part of the original mural mosaics of exquisite art. It comprises the main church, the sanctuary, the narthex and four chapels, which complete its orthogonal plan. In the western part, only the perimeter walls of an exonarthex or portico and those of a spiral stairway tower leading to the upper floor have survived (Fig. 2). The central part of the main church is cross-shaped in plan, the hemispherical dome rising over its square core. The dome is 8,2m in diameter and 16,4m high, resting on an almost cylindrical drum with 16 piers and 16 vaulted windows (Fig. 2). The dome and its drum are carried by eight pendentives and eight arches (four semicircular and four in the squinches of the corners), forming an octagon and, in this way, achieving the transition from circle to square. Thus, twelve piers (laying out in a square plan), provide support to the dome together with the groin vaulted arms of the cross, situated at a higher level. All the other parts of the monument are covered with byzantine groin vaults.

The exterior faces of the vertical perimeter walls are built according to the cloisonné system (stones surrounded with bricks). Two distinct masonry construction types are detected, in the lower and upper part of the walls (Delinikolas et al. 2003). Both types of masonry are constructed according to the three-leaf pattern, with varying widths of their leaves (Palieraki et al. 2008). Brick masonry was used to construct the system of arches and vaults (with the exception of the west façade of the portico, where curved stones were used by the Cistercians Monks), as well as the arches around windows and doors (Fig. 2).

The foundation of the structure is based on a cohesive soil consisting in the first ~5 m of stiff clay and then of siltstone, as confirmed by geotechnical investigations (O.T.M. 2000).

The main construction phases of the Katholikon, its historic pathology and major past interventions to the monument, the pathology occurred after the 1999 earthquake, along with the qualitative interpretation of damage and the preliminary numerical verification of its pathological image were presented in previous works (Delinikolas et al. 2003, Miltiadou-Fezans et al. 2004, Miltiadou-Fezans 2008). Therefore, a brief summary of this information is provided herein.

Due to the 1999 Athens earthquake, numerous (shear and/or bending) cracks occurred in the walls and piers of the monument; pre-existing cracks (due to past earthquakes) increased in both length and width. The damage was more extensive at the higher parts of the structure, especially in the sanctuary, the arms of the cross and the arches below the dome. As shown in Figures 1b and 3, the small NE and NW arches (just below the squinches) presented severe dislocation near their crown, followed by out-of-plane deformations of the squinches themselves. The structural condition of the dome (reconstructed in 1891 and damaged soon after, in 1894), was assessed as extremely critical, due to a multitude of in-plane shear and flexural cracks in piers parallel to the direction of the motion and out-of-plane flexural cracks in piers perpendicular to the direction of the seismic event. This observed pathology (Fig. 3) confirms
seismological data regarding the predominant direction of the 1999 earthquake. The existing external upper metal tie, which confined the hemispherical dome near springing level, prevented the occurrence of severe cracking of the dome shell itself.

Regarding the effect of foundation and foundation soil on the behaviour of the structure, the observed pathology does not give signs of differential settlements, that might have contributed to the damages of the monument.

The comparative examination of all available quantitative and qualitative data proved the inherent vulnerability of the structural system of the monument, in which (a) a stiff central cupola is resting (through the drum) on four major arches parallel to the two main axes of the church, as well as on four arches oblique with respect to the longitudinal and transverse axes, (b) both the vertical and horizontal component of the self-weight of the whole system of (intersecting) groin vaults arranged around the central dome are transferred to rather flexible stone masonry piers, (c) there are no ties in the arches and vaults, or other elements to confine critical structural parts or link the various parts among themselves. Therefore, there is a tendency of the structural system to deform laterally in its upper region even under self-weight alone.

![Figure 3. Pathology of the Katholikon after the 1999 earthquake (E-W central sections)](image)

Expectedly, this behaviour is deteriorating when a seismic event occurs, leading to severe damage. Due to the aforementioned inherent vulnerability of the structural system of the Katholikon and the high seismicity of the area, testified also by its historic pathology, the deformation of the bearing elements is non-uniform. Thus, multiple cracks have occurred, practically dividing the entire structure into separate parts (Fig. 5a). This behaviour, already confirmed by preliminary numerical analyses (Miltiadou-Fezans et al. 2003, Miltiadou-Fezans et al. 2004), can explain the severe damages to the drum of the dome, as well as to its supporting arches and vaults.

It, therefore, became evident that the inherent characteristics of the monument require (in addition to interventions to repair and/or strengthen the damaged masonry elements) adequate measures to reduce its seismic vulnerability. This need is documented by the historical pathology of the monument, as well as by the fact that the extensive measures (e.g. reconstruction of the central dome and its drum, reconstruction of part of the south wall, as well as of the exonarthex, etc.), taken after the severe damage at the end of the 19th century, were unable to prevent the recent damages, which are similar to the historical ones (as described in detail in Miltiadou-Fezans et al. 2004). The interventions proposed to improve the overall behaviour of the monument aim at reducing the deformability of the system of domes, vaults and arches and to provide an improved connection among them, and include struts and ties at the base of arches, diaphragms at the extrados of vaults, bracing of the openings of the drum, etc.

The principle of these structural measures was included in the study for the first phase of restoration (Miltiadou-Fezans et al. 2003, Miltiadou-Fezans 2016) and was approved by the ad hoc Scientific Committee and the competent Services of the Ministry of Culture. The design of these measures was the object of a detailed study, briefly presented herein. It should be noted that, given the importance of
the monument and the high value of its mural mosaics (to be protected from further loss), the Authorities requested a thorough investigation of the seismic behaviour of the bearing system, based on calibrated numerical models and using values for the influencing parameters derived from in situ measurements and laboratory experiments.

3. ANALYSIS OF THE STRUCTURE AFTER GROUTING OF MASONRY

The models developed, their calibration and the numerical analyses of the structure taking into account the beneficial effect of grouting are presented in Miltiadou-Fezans et al. 2016. Some basic data and results are summarized in this paragraph to facilitate the reader, as they are also used in the analyses of the structure with strengthening interventions affecting its overall behavior, presented in this paper.

3.1 Numerical models – analysis method – base excitation

Detailed numerical models of the monument were developed to examine the effect of the structural intervention scheme. Parametric analyses were performed and their results were assessed, in order to reach an optimized combination of interventions. In addition to the numerical models of the entire structure (Fig. 4a, 4b), a detailed model of the dome and its drum was developed (Fig. 4c) to allow for the (as exhaustive as possible) investigation of the behaviour of this critical part of the structure. Three-dimensional, tetrahedral finite elements were used to model the structure, within the ABAQUS software. A detailed description of the models is presented elsewhere (Miltiadou-Fezans et al. 2016).

![Figure 4. FE models of: a) the entire monument without and b) with interventions and c) the dome and drum](image)

Equivalent static analysis was performed for the drum-dome model, whereas linear time-history analyses were performed for the model of the entire structure. Although the behaviour of the structure under strong seismic events is clearly non-linear, the decision was made not to base the study on non-linear analyses. The reason for this choice (in addition to the enormous computational effort required for non-linear analyses of such a complex structure) was the numerous sources of pronounced uncertainties related to the inelastic behaviour of the constitutive materials and curved, planar and linear elements of the monument.

The maximum expected ground acceleration was found 0.30g, based on the seismic hazard estimation study and assuming that the expected lifetime of strengthening measures is 50 years, with 10% reference probability of exceedance (design earthquake). Based on the seismic hazard analysis the seismic scenarios were selected, namely four time histories of the Northridge earthquake recorded at distant stations. In the model for the entire structure, the base excitation was applied using the time histories of Arleta, Pacoima Kagel Can, UCLA Grounds and Hollywood Star. Regarding the model of the dome, the acceleration at the base of the drum level differs from the applied maximum ground acceleration. Based on the monitoring results, an amplification factor equal to 2.5 was applied.

3.2 Calibration of numerical models

For the calibration of the model comprising the dome, its sustaining drum and the concentric rings at the bottom (Fig. 4c), the introduction of adequate boundary conditions to represent the underlying part
of the structure was necessary. Thus, the stiffness of the underlying structure was modeled by spring elements assigned in all three directions, installed at the model’s base. Appropriately analyzed records of the monitoring system (Mouzakis et al. 2008), concerning the displacements at specific positions of the dome, were employed to define the spring constants (Miltiadou-Fezans et al. 2016). These displacements were compared to the response of the same positions after applying a static load resulted from the peak ground acceleration of 0.30g amplified by 2.5 times, corresponding to dome’s height.

The model of the entire structure was calibrated comparing the eigen periods, as derived from the records of the structural monitoring system, with those of the analytical model. In order to achieve a satisfactory convergence, the boundary conditions were examined, using various types of springs. Special attention was given to the mechanical properties of the materials assigned to the model, which were set according to the research on the mechanical behavior of the masonry type of the monument, before and after grouting (Vintzileou and Miltiadou-Fezans, 2008), and the recorded data from the monitoring system. After a parametric study, it was concluded that the accelerations developed at specific locations of the model converge with the recorded ones, when the translational degrees of freedom (dofs) of the nodes at the base of the model are restrained. This conclusion is compatible with the aforementioned soil conditions.

Additionally, the capacity of both calibrated models to reproduce the existing damages was verified. To this aim, the regions of the models in which principal tensile stresses significantly exceed the strength of masonry were compared with the surveyed crack pattern of the monument. The tensile strength of masonry before and after grouting adopted for verifications were based on experimental results (Vintzileou and Miltiadou-Fezans, 2008), as well as on tests on the final grout mix used in the monument. Thus, a value of 0.10MPa was adopted for the tensile strength of masonry before grouting, while the respective value for masonry after grouting was equal to 0.30MPa. In general, the regions of high principal tensile stresses matched the existing cracks on most of the structural members. This remarkably good agreement of the resulting stress state of the models with the surveyed pathology indicated that elastic analysis can be used successfully to determine weak regions of the structure and places where cracks will initiate during an earthquake.

The evaluation of the results of numerical calculations (in terms of principal stresses) showed that in several locations the principal tensile stresses exceed the tensile strength of masonry after grouting. Therefore, cracking of the respective elements was to be expected. However, taking into account that masonry elements can still perform in an adequate way after cracking, the results of the numerical calculations were further processed and safety verifications were performed using action-effects on relevant sections or regions of the structural members (Miltiadou – Fezans et al. 2012, 2016). The verifications revealed that several piers of the drum, as well as some vertical elements, had insufficient bearing capacity (either shear or flexural) to resist the design actions.

Consequently, the necessity of taking additional strengthening measures was confirmed. Indeed, as the preliminary analyses had indicated, to alleviate the inherent vulnerability of the structure and thus protect it against future extensive damage, strengthening measures affecting the global behavior of the structure should be implemented. Needless to say, the protection of the structure is a prerequisite for the preservation of the mosaics. Finally, the safety verifications confirmed the adequacy of the numerical models to simulate the behaviour of the monument, as most of the main damages were analytically reproduced. This confirmation of the observed pathology permitted the assumption that the developed numerical models were suitable for the assessment of the efficiency of the interventions to be applied.

4. ANALYSIS INCLUDING STRENGTHENING INTERVENTIONS

4.1 Methodology of analysis -Optimum scheme of interventions

In order to select an optimized combination of interventions, the calibrated models were used to qualitatively evaluate the efficiency of various structural strengthening interventions (installation of
struts and ties to the arches, confinement of masonry piers, construction of diaphragms, etc.). At a first step, the effect of each individual intervention technique was investigated. To this purpose, relevant parameters were studied, depending on each distinct measure (e.g. for ties: diameter, material (timber or steel), exact location, etc.). Once the efficiency of each individual intervention was assessed, the effect of combinations of remedial measures was studied numerically (Fig. 6a). The detailed evaluation and synthesis of the results, combined with in situ confirmation of the applicability of the examined techniques, led to the selection of the optimum scheme of interventions.

During the design of strengthening measures, in order to overcome shortcomings of the elastic analysis, the models were appropriately modified and, in some cases, “discontinuities” were introduced at the areas of the most severe cracks (Psycharis et al. 2010). For example, for the calculation of forces to be taken by ties, cracks were inserted to the model, shown in Figure 5b, in accordance with those surveyed in the monument (Fig. 5a).

![Figure 5: a) Cracking pattern top view and b) axes of “cracked pattern” inserted to the model.](image)

Given the complexity of the masonry structure and its historical pathology, the results obtained from the numerical analyses were first used to qualitatively assess the effectiveness of the proposed strengthening measures (e.g. to detect regions of high principal tensile stresses before and after the implementation of one or more measures). Subsequently, detailed verification against action-effects was performed for the elements or regions thereof that were shown to undergo principal tensile stresses exceeding the tensile strength of grouted masonry. Also, the intervention measures were dimensioned.

Due to the vulnerability of the dome, a significant part of the study was devoted to the, as detailed as possible, investigation of various alternative techniques. For this purpose, the numerical model of the dome and its drum was used. More specifically, to improve the behaviour of the drum two alternative measures were investigated, namely (i) the installation of steel shear panel dampers (Schmidt and Dorka, 2004) in the intrados of the opening of the drum to increase the hysteretic damping and (ii) the installation of stainless steel trusses in the openings to increase the stiffness and strength (Fig. 6a). The two measures were examined in combination with the replacement of the existing confining steel ring-plate, situated at the upper level of the drum, with a stainless steel one, and the installation of a second confining stainless-steel ring in the extrados of the hemispherical dome (Fig. 6b). Since the use of dampers was proven to be of limited efficiency, the solution of stainless steel trusses was selected, along with the confining stainless-steel rings. The efficiency of these measures was further investigated through linear time history analyses of the model of the entire structure.

![Figure 6: The model of the dome with: a) windows’ steel trusses and b) confining steel plates](image)
The action-effects derived from the analyses were used as input to two- or three-dimensional individual models developed to assist the dimensioning and verification of anchor plates (for struts and ties), of timber and steel diaphragms, of anchors connecting the diaphragms to masonry, etc., using linear, shell or solid elements (Fig. 7).

![Figure 7](image1.png)

Figure 7: Examples of complementary numerical models: a) anchor plate of the ties; b) steel diaphragm structures over the extrados of the groin vaults.

Finally, to estimate the level of seismic actions expected to initiate cracking of the structure, further numerical calculations were performed. In the case of the structure without the strengthening measures, the initiation of cracking appears for a PGA>0.10g, i.e., 33% of the design earthquake, while for the structure with the strengthening interventions cracking appears for a PGA>0.15g, i.e., 50% of the design earthquake. Regarding the behavior of the structure with the strengthening interventions for the design earthquake (PGA<0.3g), the upper part of the structure (higher than approximately 6m) remains elastic, while at the lower part the tensile strength is locally exceeded.

![Figure 8](image2.png)

Figure 8. Presentation of interventions using the architectural 3D model: a) view of all added wooden and metallic elements from SW and b) view of the model with diaphragms from SE.

The minimum necessary interventions, selected as optimum, considering all the values of the monument, as shown in Figure 8, were: (i) In the dome and its drum: Replacement of the existing ring-plate at the upper level of the drum by a stainless steel one. Installation of a second non-visible confining stainless-steel ring in the extrados of the hemispherical dome, underneath the ceramic tiles. Insertion of stainless steel stiffening frames in the windows of the drum. (ii) Over the extrados of the groin-vaults of the main church: Installation of stainless steel “diaphragms”; their final geometry was adjusted to the available space of each area. (iii) Over the extrados of the groin vaults of the narthex and western chapels: Installation of a diaphragm composed of 45 mm LVL timber plates (kerto Q) supported by stainless-steel HEB 100 beams (for the vertical loads) and of stainless-steel U100 beams at the perimeter, connecting the entire system with the walls by means of dowels and anchors (stainless steel threaded bars of d=12mm) (iv) In the exonarthex: Installation of a diaphragm composed of 45mm LVL wooden
plates (kerto Q) supported by 17/25 timber beams (for vertical loads), and of stainless steel L250 perimeter beams, connecting the entire system with the walls by means of dowels and anchors. (v) In the exonarthex: Installation of stainless steel ties and timber struts in the arches and replacement of the masonry piers, with replicas of Ionic marble columns. (vi) In the interior of the main church, the narthex and the crypt: Installation of stainless steel ties and struts in most of the arches. Confinement of west piers of the church and the piers of the crypt, with stainless steel plates. (vii) In the staircase tower: Confinement of the masonry walls at three levels, by insertion of threaded titanium tie-bars, bonded to masonry by means of hydraulic lime injections and anchored externally on titanium anchorage plates. Installation in the tower’s interior of a round steel staircase, acting independently from the old masonry structure.

4.2 Investigation of the efficiency of diaphragms

Preliminary analyses were performed to obtain a rough approximation of the in-plane stiffness of diaphragms that would lead to significant reduction of the deformations in the higher part of the monument. A parametric analysis, based on a simplified modeling of the diaphragms, inserted to the numerical model as shells, was performed. The results of time-history analyses showed a remarkable improvement of the behaviour (in terms of both stresses and deformations) for a value of the modulus of elasticity equal to $E = 10\text{GPa}$ (Fig. 9). Based on these results and taking into account the geometric restrictions of the existing structure regarding the feasible geometry of the diaphragms, the most appropriate arrangement was identified. As shown in Figure 8, the in-situ conditions led to a tailored application of diaphragms. Subsequently, each individual member was dimensioned, along with the anchorage of the diaphragms to the masonry.

![Figure 9: Maximum principal stresses of the west wall of exonarthex: a) without and b) with diaphragm.](image)

4.3 Location and efficiency of steel ties

It is obviously impossible to derive action-effects for the dimensioning of ties through elastic analysis, as ties are expected to be mobilized after cracking of bearing elements. To overcome this obstacle, the structure was considered to be already cracked, following the pattern surveyed in-situ (Fig. 5a). To reach the maximum possible magnitude of axial force for the design of ties, as well as a (smaller, but) reasonably expected magnitude, two analyses were performed, namely (a) the cracked model was strengthened only with ties, and (b) the model was strengthened by the entire selected scheme of interventions. The results of this investigation were sensible and encouraging. For the first alternative, a maximum tie axial force equal to 350kN was calculated, whereas for the second alternative the maximum axial force for the same tie was reduced to 250kN. Although such a reduction is realistic and attributed to the reduced deformability of the upper part of the monument (thanks to the diaphragms), the decision was made to, finally, dimension the ties based on the most adverse condition. This option was adopted both because the resulting dimensions of ties and anchorage plates are acceptable and because some redistribution of forces (from the most stressed tie to less stressed ones) was detected. The implemented set-up of steel ties in the central part of the monument is presented in Figure 10.
4.4 Analysis of the structure after application of the selected intervention scheme

The frequency analysis of the model showed that, after the proposed interventions, the stiffness of the entire structure will be increased and the independent response of several parts of the monument will be significantly reduced. As shown in Figure 11, after the implementation of the diaphragms, the independent movement of the exonarthex is restricted. The section of the model that comprises the drum and the dome exhibit a more uniform behavior, responding with reduced relative deformations. The strengthened structure is shown to have an improved response, moving as a single body. This indicates that acting loads will be distributed more appropriately than in the existing structure. Therefore, the objectives of the interventions, set in the beginning of the study, when the weaknesses of the structure were recognized, are satisfied.

To compare the response of the structure and the stress state before and after interventions the computational models representing the existing and the strengthened structure were subjected to four different time histories of the Northridge earthquake. The results were elaborated and the time steps providing the highest stress values across the models were selected. In Figure 12, some of the selected time steps for the Arleta record are shown.

Figure 10. Stainless steel ties and struts: a) model central part top view and b) photo of NE squinch restored area.

Figure 11. Predominant eigenmodes a) before and b) after implementation of diaphragms

Figure 12. Principal tensile stresses before and after interventions (exceedance of 0.3 MPa shown in gray).
The stress state of the strengthened model was significantly improved, exhibiting a more uniform distribution of stresses with overall lower values for the tensile ones. Although the tensile stresses developed in lower zones of the model were not greatly reduced, the higher parts of the structure exhibit significant improvement in their response. In total, the independent response of several structural parts disappeared, especially of those of the central square of the monument, bearing the dome and the drum. The deformed form of the dome itself was improved, which was an a priori aim of the interventions, so that the mosaics of the Prophets and the Pantocrator will be protected from severe future damages.

5. CONCLUSIONS

The numerical results briefly presented in this paper were subject to limitations imposed by the complexity of the structure (geometrical, variety of materials, construction phases, etc.), as well as by the unavailability of reliable constitutive models for materials, structural elements of various forms, their connections, etc.

The assessment of the structure and the selection and design of intervention measures were assisted by a systematic program of in-situ and in-laboratory investigations (including monitoring), as well as by a thorough knowledge of the physical object, even in its minute details. Such knowledge, acquired through numerous inspections (of geometry, pathology, state of materials, etc.) allowed for sensible assumptions to be made (for analysis and verifications), sound engineering interpretation of the numerical results, and the feasibility of each individual measure to be checked and ensured.

Based on this systematic and extensive work, a scheme of interventions was proposed, aiming at reducing the inherent seismic vulnerability of the monument. The proposed interventions were selected taking into account the Values of the monument, particularly the need to protect its mosaics, most of which are located at the most vulnerable regions of the church. The efficiency of each intervention technique, separately or combined with others, was investigated. The added structural elements (ties, diaphragms, external confinement, etc.) were designed and dimensioned, whereas structural details were provided for each distinct measure.

The proposed and approved scheme of interventions is considered to be the maximum acceptable for a major monument. It provides a significant improvement of its overall behaviour, without altering its appearance from the exterior or obstructing the view of the mosaics in the interior. It should be noted that, based on the analysis, the adopted intervention scheme does not exclude the occurrence of damage in case of a future strong earthquake. However, the expected damage is significantly limited in extent and severity, compared to that already and repeatedly occurred to the monument.

Thus, it is believed that the entire work performed since 1999 reached an optimized intervention, ensuring an enhanced seismic protection of the structure and, by consequence, of its precious mosaics, while simultaneously respecting the Values of the monument.

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