NUMERICAL SIMULATION OF THE DYNAMIC AND EARTHQUALE BEHAVIOUR OF GREEK POST-BYZANTINE CHURCHES

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ABSTRACT :

A series of linear and non-linear numerical simulations are presented of the dynamic and earthquake behaviour of post-Byzantine churches in Greece. They are made of masonry including a dome and a cruciform vaulting system. The peripheral walls include apses and a variety of door and window openings. Two such churches were selected with a large number of geometrical and structural features in common. The first is Agia Triada in Drakotrypa, in the prefecture of Karditsa, and the second is the Metamorfosi tou Sotiros, in Zavorda, in the prefecture of Grevena. The later was subjected to a damaging earthquake sequence (1995) and is undergoing repair. The linear numerical simulation focuses on the influence that certain parts have on the dynamic behaviour, such as the dome, the vaulting, the tympana, the apses and the door and window openings. The demands, in terms of normal and shear stresses, predicted by the linear numerical simulation, are utilized with a Mohr-Coulomb failure criterion to quantify the capacity of the stone masonry. The non-linear numerical simulation attempts to predict the initiation and propagation of failure at various masonry structural elements by applying the combination of gravitational and seismic forces with a modified Von-Misses failure envelope. A step-by-step incremental analysis approach is adopted with imposed patterns of horizontal displacements similar to the fundamental modal shapes. The initiation and propagation of failure together with the corresponding load-displacement curves are presented and discussed. The actual formation of seismic damage is utilized to validate the realism of the numerical predictions.

KEYWORDS: Church, Greek, Masonry, Post-Byzantine, Numerical

1. INTRODUCTION

During the last thirty years various parts of Greece have been subjected to a number of damaging earthquakes ranging from Ms=5.2 to Ms=7.2 on the Richter scale. Some of these earthquakes, not necessarily the most intense, occurred near urban areas and thus subjected various types of structures to significant earthquake forces leading to damage. One of the most demanding tasks for counteracting the consequences of all these seismic events was the effort to ensure the structural integrity of monumental structures, mainly churches that were built in periods ranging from 400 A.D. up today. For numerous churches that sustained considerable structural damage rather “mild” intervention/repair schemes were dictated that in many cases are still in progress even more than twenty years after the seismic event.

In what follows, selected results and summary observations will be presented for a specific type of the dynamic and seismic behavior of a structural system that is utilized in a considerable number of churches belonging in the so called Post-Byzantine period (16^{th} to 19^{th} century A.D.). This structural system is the cruciform plan with a central dome and a system of cylindrical vaults that also cover the central cruciform plan. Two such structures are investigated here having their basic structural system of the cruciform with central dome, as briefly described above. The first is the church of Agia Triada in Drakotrypa, in the prefecture of Karditsa, and the second is the Katholikon of the Metamorfosi tou Sotiros, in Zavorda, in the prefecture of Grevena. The later was subjected to a damaging strong earthquake sequence in 1995 and is still undergoing repair. The linear numerical simulation is focusing on the super-structure and in particular on the influence that certain parts may have on the dynamic behaviour, such as the dome, the vaulting, the tympana, the apses and the door and window openings of both the dome and the peripheral walls. Moreover, the non-linear numerical simulation is attempting to predict the initiation and propagation of failure at the various masonry structural elements. This is done by applying the combination of the gravitational and seismic forces together with a modified Von-Misses failure envelope that is believed to represent the limit-state stone-masonry behaviour. A step-by-step incremental analysis approach is adopted for this purpose with the seismic actions being replaced by imposed patterns of horizontal displacements similar to the fundamental modal shapes found from the linear analysis. The initiation and propagation of failure together with the corresponding load-displacement curves are presented and discussed. The actual formation of seismic damage is utilized to validate the realism of the numerical predictions.

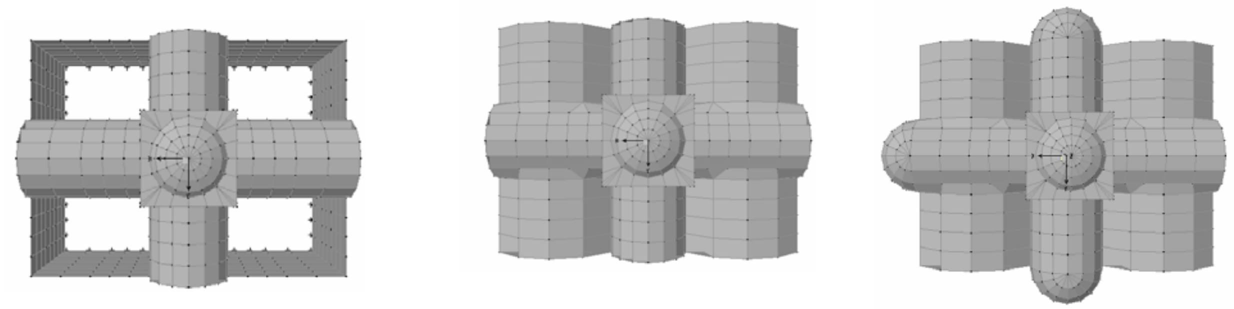


Figure 1. Plan of the Structural Formation of the superstructure of Churches of the Post Byzantine Period

1.1. Structural System of the Cruciform with Central Dome.

1.1.1 The church of Agia Triada at the prefecture of Karditsa

The church of Agia Triada (Holy Trinity) of the village of Drakotrypa dates from January 1743 A.D., built on the same site of an old monastery which was burned. It is a single-nave basilica with a central dome and a transverse nave together with three apses at the sides of the church. This church belongs to the post-Byzantine type of construction. Churches of similar dimensions, construction and architectural form can be found in the greater area of Central and Northern Greece. Figure 2 depicts an external view together with the basic plan of this church, where two distinct parts are shown. The first is defined by the peripheral stone masonry walls (ABCDEFG) and represents the main church with the altar at the eastern side. At the west side there is a separate part “the Narthex”; this part (HJED) was constructed at a later period. Because the connection of the “Narthex” with the main church is not clearly defined, this part is excluded from the current investigation.

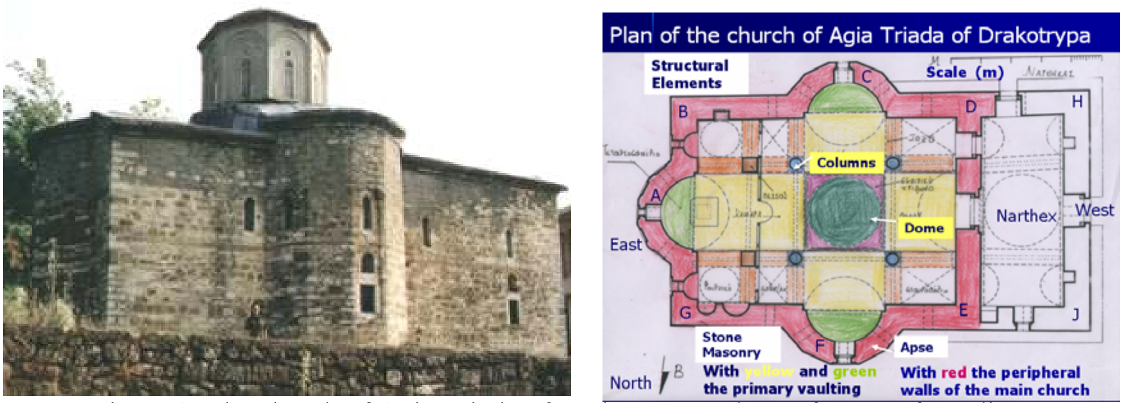


Figure 2. The church of Agia Triada of Drakotrypa at the prefecture of Karditsa.

Figure 2 also shows the main structural elements of the church. They are: a) the peripheral walls, made of stone masonry with a thickness of 0.85m, which include the three apses (at the East, North and South sides where the masonry is 0.75m thick); b) The central columns with a diameter of 0.4m; c) Internal masonry piers with a thickness of 0.53m; d) The primary and secondary vaulting system with a central dome and pendatives having a thickness of 0.25m; e) The arches of the vaulting system which are 0.5m thick. The length of the longitudinal East-West walls (without the apses) is 9.4m; the width (without the apses) is 7.26m. When the apses are included, the length becomes 11.03m and the width 10.52m. The height of the peripheral stone masonry walls

is 5m. The primary and secondary vaulting system rises from this level up to a height of 6.49m from ground level. The dome has a diameter of 3.18m and rises from the level of 6.49m up to the level of 10.27m from ground level. The height of the central columns and the internal masonry piers is 3.875m. The main building materials are stone masonry and lime-based mortar. The vaulting system and dome are constructed with relatively light stone masonry (porolithe with a density 2500kg/m3). The density of the stone masonry for the peripheral walls and apses was taken equal to 2700kg/m3. The most visible signs of damage are: a) Formation of cracks at the keys of the vaults and arches, b) East-West cracks of the vaulting system parallel to the longitudinal axis of the church. Such damage at the intrados of the vaults is depicted in figure 4. This is a common form of damage to such structural elements and, as shown by previous studies, it can be generated by both earthquake forces and/or foundation settlements. For the church of Agia Triada of Drakotrypa, this specific damage cannot be attributed directly to a particular seismic event of the past.
Despite this uncertainty, which is mainly due to lack of historical records, it will be debated by this paper that the design earthquake, as defined by the current Greek Seismic Code [12], is capable of producing damage to this church. This is based on one hand on the level of demands placed on the various structural members by the gravitational loads and earthquake forces as these demands are predicted by the numerical simulations which will be presented in the following sections; on the other hand, the predicted damage to this church is also based on specific failure criteria that were adopted, and are part of the current investigation.

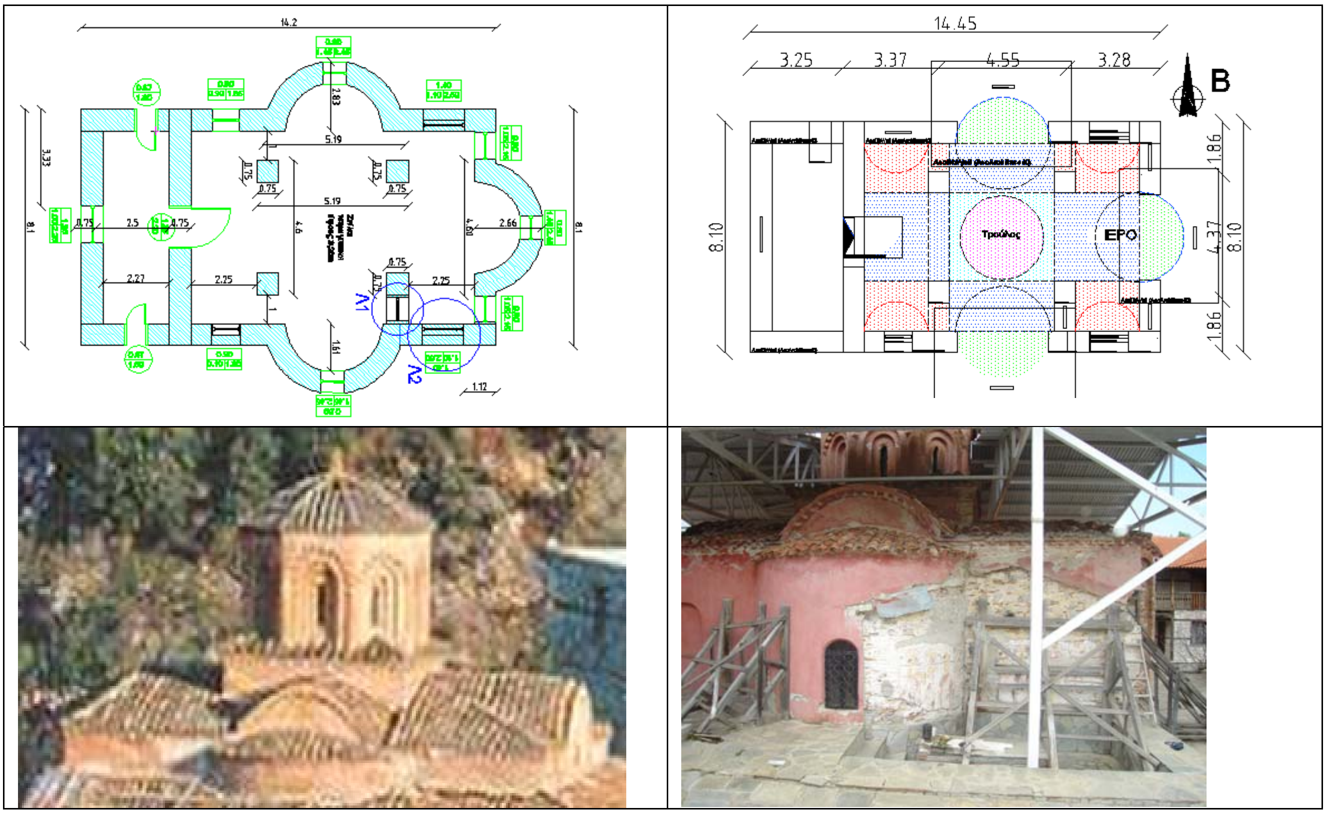


Figure 3. The Katholikon of the Metamorfosi tou Sotiros in Zavorda, prefecture of Grevena.

1.1.2 The church of Metamorfosi tou Sotiros in Zavorda

This church was built during the 16th century in the middle of a Monastery complex on the top of a rocky outcrop of Mt. Kallistrato of the prefecture of Grevena, Greece. It was built by the spiritual guidance of Osios Nikanor, who became the saviour Saint of the inhabitants of the region of Western Macedonia that relishes its memory eversinse. The structural formation (figure 3) has strong similarities with the previous church both by the fact that it is formed by similar central dome formation and cylindrical vaulting system. The peripheral North, South and East peripheral walls have again apses whereas the West wall does not have an apse; instead, the West side of the church is extended by a narthex which is in simple contact with the main structure. The

overall dimensions in plan are also similar. The orthogonal shape of the main church has 10.44m length in the E-W direction and 7.35m length in the N-S direction. The diameter of the N, S apses is 4.44m and their depth 2.0m whereas the East apse has a diameter of 3.85m and a depth of 1.90. All the peripheral walls and the apses have e thickness of 0.75m. All the apses have window openings of 0.6m width and 1.20m height. The height of the peripheral walls is 3.2m. The dome has a diameter of 3.50m, a thickness of 0.3m and a height of 1.857m. it rises from the top of a ring formed internally by the four pendatives. This ring rises 0.40m from the keys of the four cylindrical vaults which are part of the cruciform and have a thickness of 0.30m.
These main cylindrical vaults end at their supporting tympana on top of the peripheral walls; these tympana have a thickness of 0.70m The four pendatives are supported by four internal square central piers having plan dimensions 0.75m x 0.75m and a height of 3.20m. Externally, at the base of the dome a square masonry box is formed with its vertical sides ending at the top of the main cylindrical vaults. Externally, the vaulting system and the box supports a wooden roof system with ceramic tiles. The dome has eight windows symmetrically spaced around its periphery having a height of 0.60m and a width of 0.30m. The narthex is formed by peripheral walls with a height of 2.60m and a thickness of 0.70m. As main differences between the two presented churches is the fact that the height of the peripheral walls of the 1st church (5.0m) is larger than the corresponding height for the 2nd church (3.20m). Moreover, the 1st church has also a central dome of somewhat smaller diameter that is taller than that of the 2nd church; however, the dome of the 1st church is not accompanied externally by a stiffening box as is the case for the central dome of the 2nd church. The 2nd church was damaged by a specific seismic event that occurred in May, 1995. The damage patterns produced by this event extended in all the pheripheral walls as well as the main vaulting system and apses.

2. LINEAR NUMERICAL ANALYSES

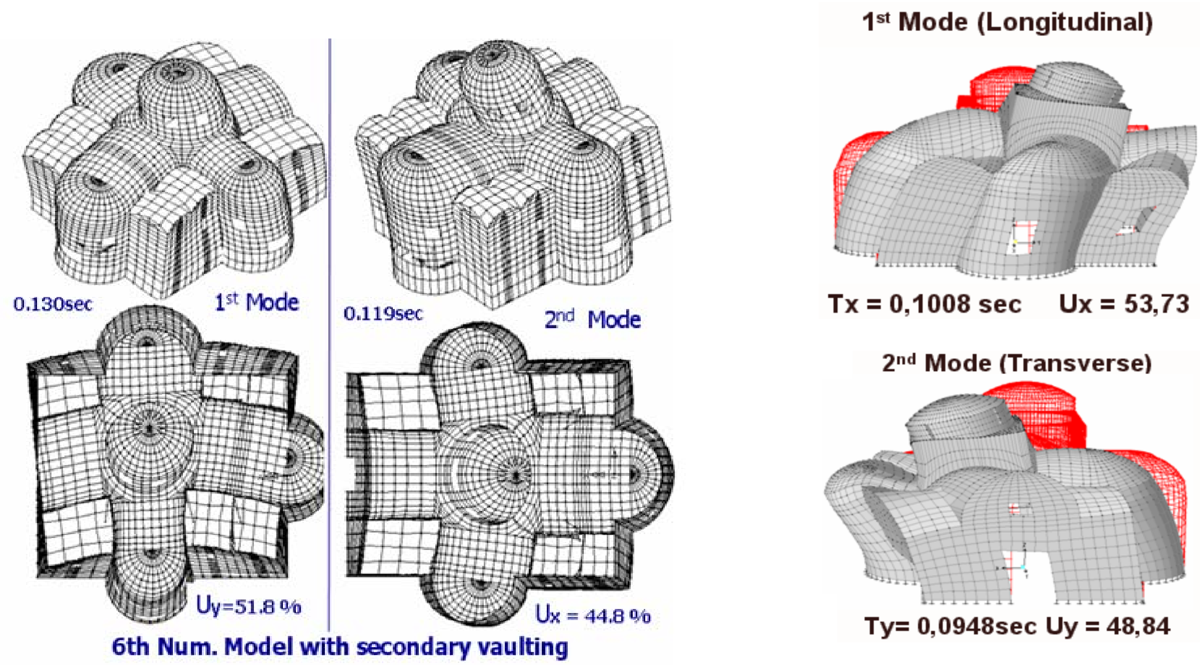
In the numerical study of the behaviour of these two churches mechanical property values were adopted which were partly based on formulas or values recommended by several references [1, 2, 3, 4, 13] for either new or old stone masonry. The authors are aware of the uncertainty that is inherent in these values; the parametric inelastic analysis, performed, can be seen as an effort to compensate this uncertainty in the assumed stone masonry strength values. Table 2.1 lists values which were assumed to be valid for the critical mechanical properties for the masonry segments and will be utilized in the following. A lower and an upper limit for the compressive and tensile strength were utilized leaving the Young's Modulus and Poisson's ratio values unaltered.

Table 2.1. Assumed Mechanical Characteristics of the Stone Masonry

	Young's Modulus (N/mm^2)	Poisson's Ratio	Stone Masonry Compressive Strength (N/mm^2)	Stone Masonry Tensile Strength (N/mm^2)
Upper limit	2500	0.2	3.846	0.250
Lower limit	2000	0.2	0.96	0.115

The current examination did not include any experimental investigation, either at the laboratory or in-situ, for experimentally validating such properties. Values for such properties, resulting from a recent experimental sequence performed by Vintzileou [11] using stone masonry walls, are in agreement with the adopted in this study limits for the strength values (Table 2.1). It is recognized by the authors that verification of the values listed in table 1 is highly desirable, because the results and conclusions of the present study are based on these assumed values. Such verification can be provided by in-situ and laboratory experimental means. This has been proposed but has not been possible so far due to lack of sufficient resources. Despite the uncertainty of the values of Table 1, certain failure envelope criteria are introduced and discussed in section 6 which are based on these values and it is hoped that the qualitative conclusions reached by this study remain, in general, valid. Base fixity was assumed for all masonry at the foundation level. The numerical study included the determination of the fundamental eigen-modes and frequencies, as presented next. The two main eigen-modes for the two churches are shown in figure 4.

2.1. Main Observations. The two dominant eigen-modes are the East-West (longitudinal) and North-South (transverse) translational modes. They mainly mobilize the mass of the central dome and vaulting system and only a small part of the peripheral walls. When there is strong connection of the vaulting system with the peripheral walls (through either tympana or apses) the modal mass, as a percentage of the total mass, does not change, as any possible increase in the modal mass is compensated for by the corresponding part of the added mass with small modal contribution. This is more evident in the longitudinal than in the transverse direction. The addition of the tympana increases the stiffness noticeably. The same is true, but to a lesser extent, with the addition of the apses, which is partially compensated for by the interruption of the planar peripheral walls by the curved elements of the apses with less total stiffness as well as with the additional mass that the apses introduce. The inclusion of the openings in the dome and peripheral walls introduces small, almost negligible, changes. The inclusion of the secondary vaulting system, although it does not alter the total stiffness, introduces considerable changes because of the additional extra mass. The central dome represents a relatively flexible subsystem with considerable mass. The external masonry confining box increases the stiffness of this subsystem and results in mobilizing its mass within the main two eigen-modes. The eigen-periods for these two eigen-modes for both churches, as they were obtained for the final models, are in the range of 0.10 to 0.13 seconds.



Final Model 1st Church – Agia Triada Final Model 2nd Church – Metamorfosi Sotiros

Figure 4. Main longitudinal and transverse eigen-modes of the final models for the two churches

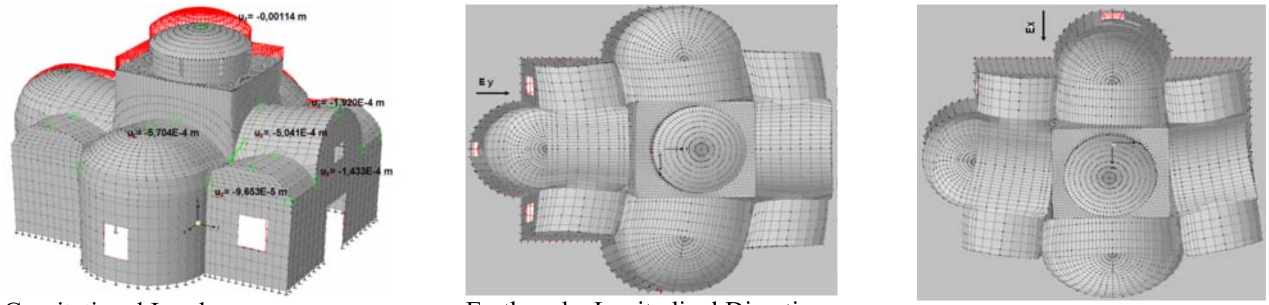
2.2.Discussion of the deformation patterns.

The objective here was to obtain the deformation and stress patterns when each structural system was subjected to the corresponding gravitational loads and earthquake forces. The load combinations for these series of analyses included:

1. G the gravitational vertical loads.
2. Ex the horizontal earthquake forces in the longitudinal direction, found as a product of the masses with an assumed normalized acceleration equal to 1g (acceleration of gravity)
3. Ey the horizontal earthquake forces in the transverse direction, found as a product of the masses with an assumed normalized acceleration equal to 1g (acceleration of gravity) 4. G + Ex. 5. G + Ey.

From a large volume of results only a selective summary is presented here with the main observations.

a). The gravitational loads from the vaulting system and dome displace the peripheral walls outwards (out-of-plane bending) generating in this way horizontal reactions that partially support the vaulting system together with the central columns and internal piers. The presence of apses significantly reduces the out-of-plane. The cylindrical cruciform main vaulting system is significantly distorted in the absence of tympana or apses.



Gravitational Loads | Earthquake Logitudinal Direction | Earthquake Transversel Direction

Figure 5. Deformation patterns

b). The out-of-plane displacement behaviour for the peripheral walls, observed for the gravitational forces when there are no apses, is further amplified by the earthquake forces. Moreover, the distortion of the cylindrical cruciform main vaulting system is also significantly amplified in this case, suggesting the development of the corresponding damage patterns in the vaulting system. The presence of tympana is effective in offsetting the vault distortion but not the dominant out-of-plane behaviour.

c). This out-of-plane peripheral wall flexibility concentrates the gravitational loads at the central columns and piers and introduces noticeable vertical deformations at the central part of the dome and vaulting system, even assuming fixity at the base of the central columns and piers. This will be further amplified if conditions at the foundation of the columns and piers are assumed to be deformable, which is closer to reality.

d). Whereas the out-of-plane displacements are significantly reduced by the presence of the apses for both the gravitational loads and the earthquake forces, the presence of the apses decreases the in-plane stiffness of the peripheral walls.

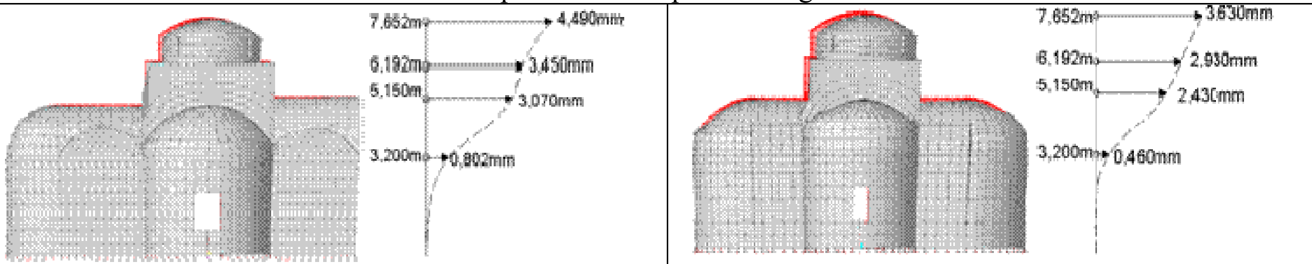
e). The decrease in the in-plane stiffness due to the presence of the apses increases the shear stresses generated by the earthquake forces at the spherical pendatives supporting the dome.

f). The earthquake forces are resisted mainly by the in-plane action of the peripheral walls, when the apses are not present. With the increase of out-of-plane stiffness through the construction of the apses the earthquake forces are almost equally resisted by the in-plane and out-of-plane actions of the peripheral walls. The stress fields that develop correspond to this type of behaviour.

g). The tympanum (with or without the external confining box) at the base of the dome has sufficient in-plane stiffness to prevent significant distortions of the horizontal cross-section of the dome and is able to transfer the earthquake forces generated by the mass of the dome through the pendatives to the vaulting system and through that to the peripheral walls, as described above.

2.3. Spectral Dynamic Analysis

Apart from the previous load combinations the 2nd church was subjected to a spectral dynamic analysis with the inclusion of the first 10 significant eigen-modes mobilizing 67.67% and 72.53% of the total mass in the longitudinal and transverse direction, respectively. The acceleration design spectrum of the Greek Seismic Code was adopted for seismic zone I (PGA=0.16g) importance factor 1.3 and response reduction coefficient 1.5. An amplification factor was also included to account for the mobilization of less than 100% of the total mass. The obtained maximum deformation patterns are depicted in figure 6.



Deformation patterns for 0.9G + 1.4 Longitudinal EQ | Deformation patterns for 0.9G + 1.4 Transverse EQ

Figure 6. Deformation patterns Spectral Dynamic Analysis, 2nd Church

The main observations from this spectral dynamic analysis agree well with the ones stated before in section 2.2. In addition, it can be observed that the maximum horizontal displacements at the top of the dome (at 7.652m from the ground level) is more than five times larger than the maximum horizontal displacement at the top of the peripheral walls (3.200m from the ground level). This was to be expected from the observations already made regarding the flexibility of the central dome and cruciform vaults superstructure. It is obvious that the main resistance and stability of this structural formation will depend upon the ability of the connections of the superstructure with the peripheral walls to successfully transfer the generated forces as well as on the capacity of the peripheral walls and apses to successfully resist them. In what follows a first attempt is made to investigate these abilities.

3. EVALUATION OF THE PERFORMANCE

3.1. Evaluation of the performance based on the results of the linear elastic analysis.

In what follows the demands, in terms of normal and shear stresses, which were predicted by the linear elastic numerical simulation presented in section 2, will be utilized together with a Mohr-Coulomb failure criterion with tension cut off (figure 12). This criterion, was adopted at this stage in order to quantify the capacity of the stone masonry of this church. The values, which were used to define this criterion are listed in Table 2.1. A verification of these values is highly desirable, as stated in section 3. The failure envelope that was assumed to be valid is schematically presented in figure 12. Failure is introduced when: a) stresses exceed the allowable compressive strength, 3.846 Nt/mm2 b) stresses exceed the allowable tensile strength of Masonry, 0.192 Nt/mm2 c) stresses exceed the shear strength of stone masonry. Following the proposed revisions to Eurocode-6 Nov 2002 [5] the shear failure criterion that was adopted is:

$$f_{vk} = f_{vko} + 0.4\,\sigma_n \qquad (1)$$

where: f_{vko} is the shear stress of the stone masonry when the normal stress is zero; f_{vko} was assumed to be equal to 0.192 Nt/mm2. σ_n ís the normal stress

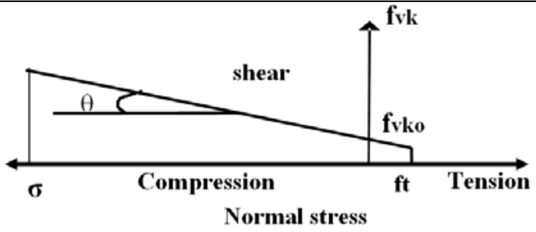


Figure 7. Mohr-Coulomb failure criterion

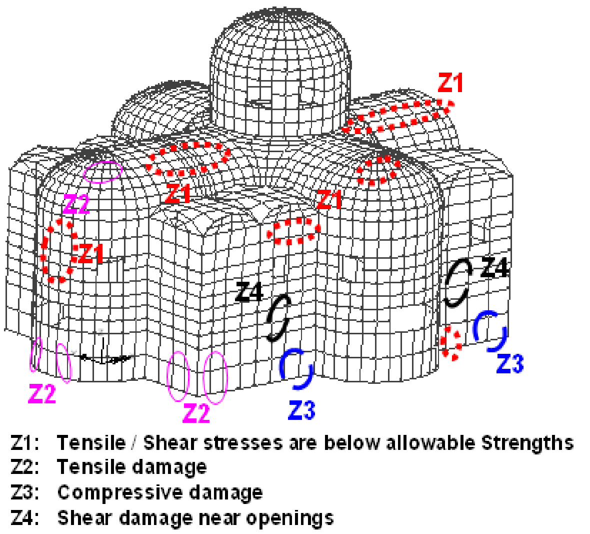


Figure 8a. Predicted damage regions 1st Church

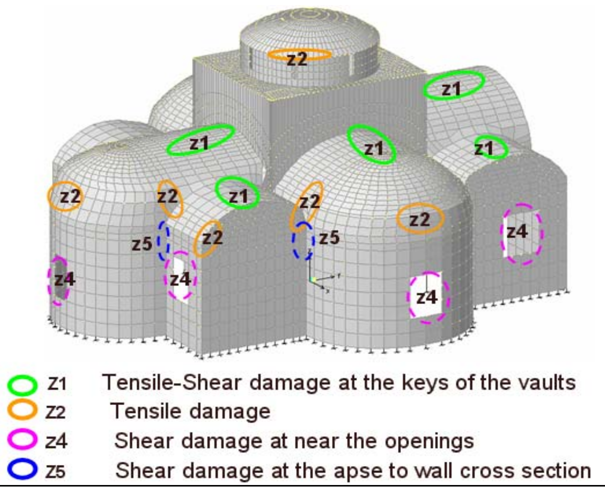


Figure 8b. Predicted damage regions 2nd Church

Figure 8a and 8b presents the effect on the numerical results of the elastic analysis for the 1st and 2nd churches when the load combination was gravitational loads plus earthquake forces after applying the previously described Mohr-Coulomb failure criterion It must be pointed out that the application of this criterion excluded the areas of bending stress concentration at the base of the dome. The following observations can be made from the outcome of applying this Mohr-Coulomb failure criterion.

For the 1st church three areas are identified as being beyond the capacity, as defined from the adopted failure envelope. Z2 at the longitudinal walls and is due to excessive shear stress demand, Z3, again at the longitudinal walls, and is due to excessive compressive stress demands and Z4 also at the longitudinal walls and is due to excessive tension compared with the adopted tension cut-off limit.

For the 2nd church, apart from the shear damage that is predicted at the peripheral walls near the openings (z4) widespread regions of tensile-shear damage (z1) is predicted at the keys of the vaults and tensile (z2) or shear (z3) damage is predicted at the connections of the vaulting system or the apses with the peripheral walls.

All the above predicted zones of potential failure are credible, as can be deduced from such damage patterns observed in this type of structures after strong earthquake events. The damage observed at the 2nd church from the main 1995 event presents features that are in reasonable agreement with the above predictions. An additional extensive non-linear analysis was performed for both churches in order to verify the observed damage with numerical predictions. Due to lack of space these results cannot be included here.

4. CONCLUSIONS

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