The Performance of Post-Byzantine churches during the Kozani-1995 Earthquake – Numerical Investigation of their Dynamic and Earthquake Behavior.

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SUMMARY:
The earthquake performance of stone masonry Post-Byzantine churches of either the "Basilica" or the "Cruciform with Central Dome" form damaged during the Kozani 1995 earthquake, is investigated numerically. The performance of the various structural elements, located at the peripheral masonry walls, is checked by applying either an assumed Mohr- Coulomb failure envelope (in-plane shear) or limit tension (in-plane and out-of-plane flexure); these limit-state criteria are assumed to be valid for the stone-masonry walls of these churches. From the observed performance of these churches it can be deduced that the regions of the masonry walls near the foundation, the door and window openings, and roof appear to be the most vulnerable either in out-of-plane bending or in in-plane shear for the critical combination of earthquake loads and gravitational forces.

Keywords: Stone Masonry, Post-Byzantine Churches, Basilicas, Cruciform with Central Dome, Dynamic Behavior, Earthquake Performance

1. INTRODUCTION

The earthquake performance during the Kozani 1995 earthquake (Hatzfeld et.al. 1998, Papazachos 1995, Theodulidis, 1999) of Post-Byzantine churches constructed with stone masonry is examined. Both the "Basilica" as well as the "Cruciform with Central Dome" are very common structural forms utilized in a considerable number of Christian Churches with variations in plan and height. The dynamic and earthquake behavior of a number of such "Post-Byzantine" churches, located in the epicentral area (figure 1a) and damaged by this earthquake, is investigated numerically (Manos et.al. 1997, 2008, 2010, 2011). First, a three-nave 19th century typical "Post-Byzantine Basilica" with a wooden roof is studied, with dimensions 19m x 11m in plan and 6.8m high, representing in form and dimensions a number of such churches. Next, an early 19th century three-nave Basilica is examined with dimensions 19m x 12m in plan and 6.6m high, having a system of cylindrical vaults and spherical domes that form the superstructure together with the wooden roof. Finally, a church of the "Cruciform with Central Dome" type is also studied. The elastic numerical simulations, which also utilized information from the ground motion recording of the main event (figure 1b), utilize assumed limit-state criteria (in-plane shear, or limit tension for in-plane and out-of-plane flexure). For all these churches, a relatively mild intervention scheme was adopted by the cultural conservation authorities utilizing special mortar injections, aimed at increasing the stone masonry strength.

2. THE "BASILICA" STRUCTURAL SYSTEM

The "Basilica" structural system is of rectangular shape, formed by the peripheral walls; a semi-cylindrical apse is usually part of the East wall, whereas the interior is divided into a number of naves by longitudinal colonnades of various dimensions and shapes, as shown in figures 2a and 2b. The roofing system is mainly in the longitudinal direction; it usually rises at the central nave at a higher level than that of the side-naves; in this sense, it can be seen as an elevated extension of the interior colonnades. All of them are composed of stone masonry for the peripheral walls, vaults and domes and internal transverse partitions with an apse at the East wall and a wooden roofing system. The roofing system that covers the side naves is partially supported on the peripheral walls and is usually lower than the roof of the central nave (figure 2b). In some instances, naves in the transverse direction are built, extending from the middle of the peripheral walls thus forming the "cruciform" in plan. The dynamic and earthquake behavior of this "Basilica" structural form will be investigated through the following distinct structural systems.

Figure 1a. Map of the epicentral area

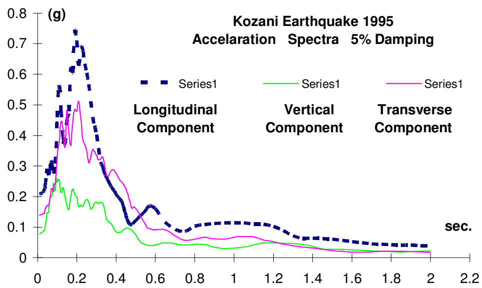


Figure 1b. Hor. acceleration response spectral curves of the main event ground motion recorded at the city of Kozani.

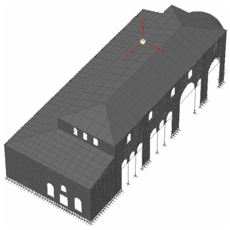


Figure 2a. The Basilica structural system with the interior colonnade of the central nave

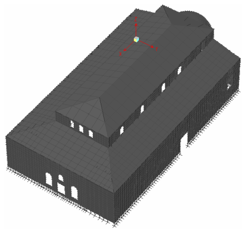


Figure 2b. The Basilica structural system with the peripheral longitudinal and transverse walls

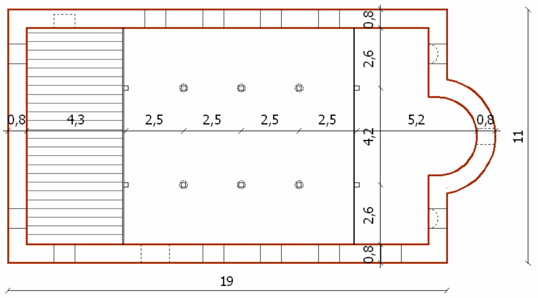


Figure 3a. 1st Post-Byzantine "Basilica"– plan

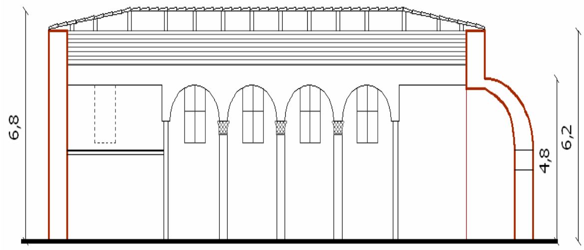


Figure 3b. 1st Post-Byzantine "Basilica" –section

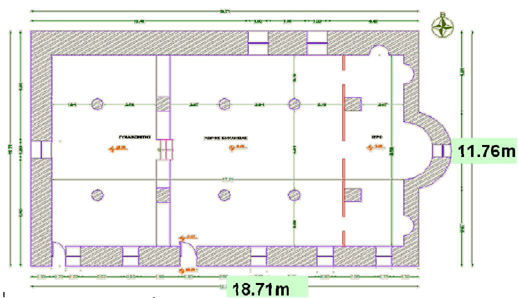


Figure 4a. 2nd Post-Byzantine "Basilica" plan

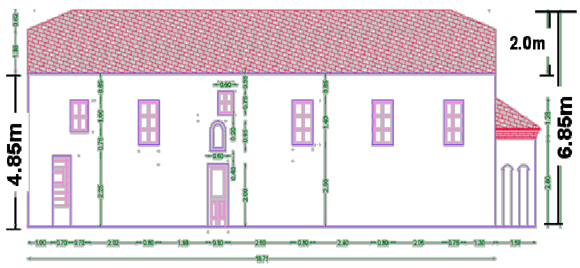


Figure 4b. 2nd Post-Byzantine "Basilica" – South Elevation

a) The first case is a three-nave Post Byzantine "Basilica" structural formation (figs. 3a, 3b) which, in the overall geometry, is typical of a number of churches, located in the villages of Lefkopigi, Knidi, Panagia, Paliochori, which were damaged by a strong earthquake sequence in the region of Western Macedonia, Greece during the Kozani Earthquake of 1995 (Manos et.al. 1997, 2008, 2010, 2011). The overall dimensions of this "typical" case are 19m length, 11m width and 6.8m height of the central nave (the level at the top of the roof). The internal colonnades are made of wood.

b) This structural formation is also a 19th century Post Byzantine church of Taxiarhes at the village of Rodiani in the prefectures of Kozani, Greece; it was also damaged by the Kozani Earthquake of 1995. The length of the longitudinal walls is 18.7m whereas that of the transverse walls 11.75m, almost similar to the plan dimensions of the 1st church. However, the height of the peripheral walls is 4.85m, lower than that of the 1st case. Moreover, an additional distinct difference from the 1st case is a system of masonry cylindrical vaults, spherical domes and arches which support the wooden roof that rises another 2.0m from the top of the peripheral masonry walls (Figures 4a and 4b). The thickness of the masonry walls varies from 700mm to 1200mm. A church of the same structural formation with a similar vaulting system, located in the village of Sarakina, was also damaged by this earthquake.

3. THE "CRUCIFORM WITH THE CENTRAL DOME" STRUCTURAL SYSTEM

The church to be investigated here is the Katholikon of the Metamorfosi tou Sotiros, in the monastic complex of Zavorda, in the prefecture of Grevena, Greece (Figures 5 & 6). Its structural system is of the "cruciform with a central dome". The central dome is formed by spherical or cylindrical shapes and is supported by the cylindrical vaulting through a system of cylindrical rings and spherical pendatives that form the basis of the central dome and the links with the vaulting. Apart from the main vaulting system that forms the cruciform, the superstructure is also composed of a transverse secondary vaulting system. Moreover, the peripheral walls are either plane or are combined with semi-cylindrical apses (figures 7a and 7b).

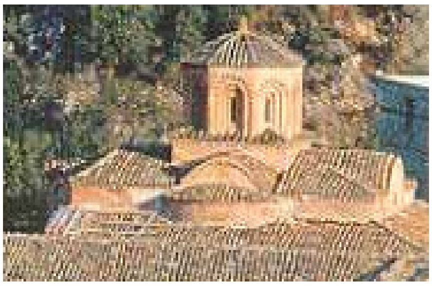


Figure 5. The Katholikon of the Metamorfosi tou Sotiros, in the monastic complex of Zavorda

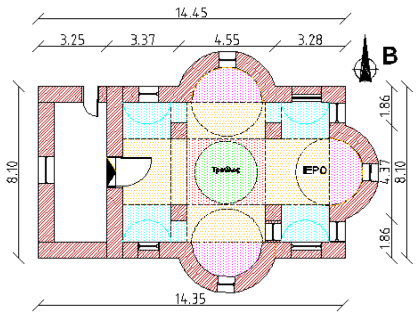
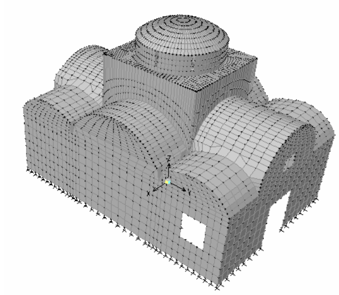


Figure 6. Plan of the Katholikon of the Metamorfosi tou Sotiros. (Dimensions in meters).

a) Case A



b) Case B

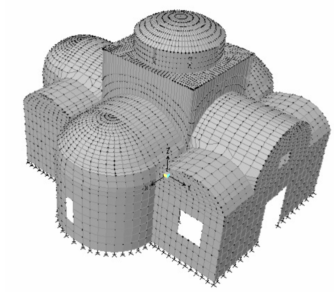


Figure 7. Central spherical dome with cruciform vaulting and cross-vaulting. Peripheral walls with Apses-case B

The numerical simulation focuses on the super-structure and in particular on the influence that certain parts may have on the dynamic behavior, such as the dome, the vaulting, the tympana, the apses and the door and window openings of both the dome and the peripheral walls. This is done by checking the numerical results from the combination of the gravitational and seismic forces by applying an assumed failure envelope that is believed to represent the limit-state stone-masonry behavior. The actual seismic damage is utilized to validate the realism of these numerical predictions (see section 6.2).

3.1. Main Features of 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 in the prefecture of Grevena, Greece. Figure 6 depicts the orthogonal shape of this church in plan. All the peripheral walls and the apses are 0.75m thick. 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. Externally, at the base of the dome, a square masonry "box" is formed with its vertical sides rising from the top of the main cylindrical vaults (Figure 8a). Externally, the vaulting system and this masonry "box" support a wooden roof system with ceramic tiles. An additional feature of this church is the narthex, which is located at the front of the church (figure 8a). The narthex is formed by peripheral walls with a height of 2.60m and a thickness of 0.70m, and in this way extends at a lower level than the West transverse peripheral wall. This church was damaged (figure 8b) during the 1995 strong earthquake sequence and its repair was completed in 2010.

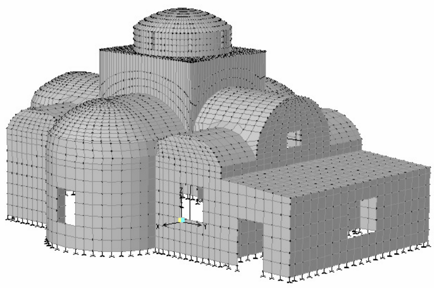


Figure 8a. 3-D representation of the Katholikon of the Metamorfosi tou Sotiros with its narthex

Figure 8b. Damage of the cruciform vaults and the pendatives supporting the central dome.

4. RESULTS FROM THE MODAL ANALYSIS OF THE POST-BYZANTINE CHURCHES

4.1. Modal analysis of the "Post-Byzantine Basilicas"

A linear-elastic modal analysis was conducted assuming a value for the Young's Modulus for the masonry walls equal to 2500Mpa. The mass of these stone masonry walls was assumed to be equal to 2.70t/m3. All the walls were numerically simulated by shell F.E. The arches on top of the internal colonnades as well as the wooden roof was also numerically simulated; the Young's Modulus of all the wooden parts was taken equal to 8400Mpa with the corresponding mass equal to 0.66t/m3.

a) Figures 9a and 9b depict the mainly horizontal translational eigen-modes for the 1st church. The translational eigen-mode in the transverse North-South (y-y) direction is the one with the longest eigen-period (figure 9a, T = 0.102seconds). The structural response in this mode displaces the longitudinal peripheral walls mainly out-of-plane; this is done with the transverse peripheral walls resisting mainly in-plane. The translational eigen-mode in the longitudinal East-West (x-x) direction is the next longest eigen-period (figure 9b, T = 0.069 seconds). The structural response in this mode displaces the longitudinal peripheral walls mainly in-plane; this is done with the transverse peripheral walls resisting mainly out-of-plane. Each one of these modes mobilizes approximately 50% of the total mass of the structure. These two modes are next followed by higher horizontal modes; however; these latter modes mobilize relatively small portions of the total mass.

b) Figures 10a and 10b depict the mainly horizontal translational modes for the 2nd church. The eigen-periods in both the longitudinal and the transverse directions for the 2nd church are somewhat longer than those of the 1st church. However, this time the modal mass ratios, which are mobilized by these two translational modes, are noticeably larger than those of the 1st church. Both these effects must be attributed to the mass of the system of masonry cylindrical vaults, spherical domes and arches.

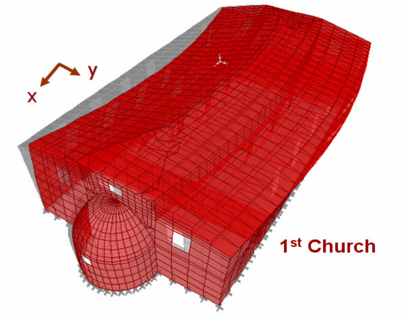


Figure 9a. Ty = 0.102 seconds, uy = 51.38%

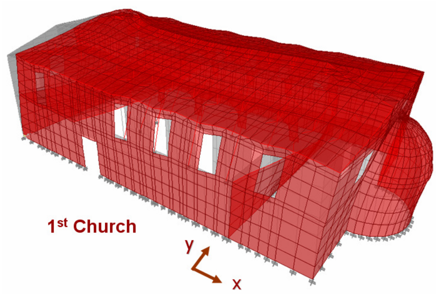


Figure 9b. Tx = 0.069 seconds ux = 49,63%

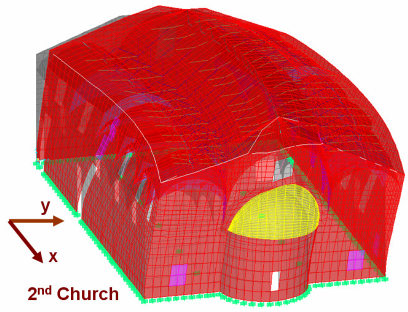


Figure 10a. Ty = 0.1053 seconds, uy = 61.8%

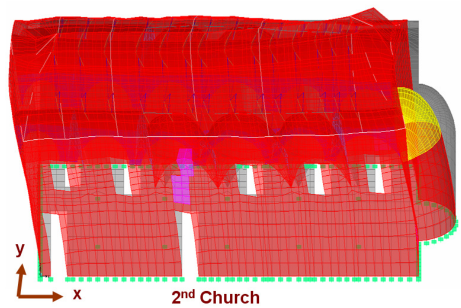
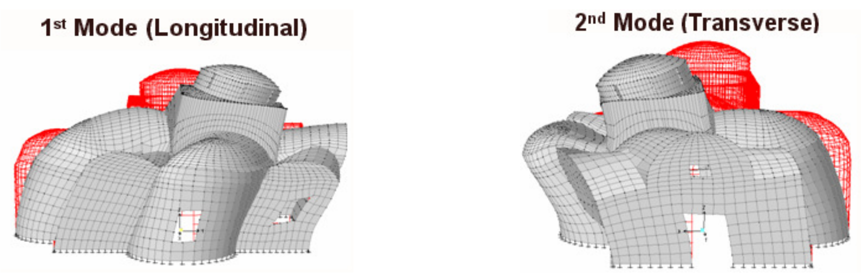


Figure 10b. Tx = 0.0724 seconds ux = 63,966%

4.2. Modal analysis of the "Cruciform with Central Dome" Post-Byzantine church

This investigation adopted values equal to 2000N/mm2 for the Young's Modulus and 0.2 for the Poisson's Ratio (Manos et al. 2008). All masonry is constructed with natural stone extracted locally and relatively weak mortar based on lime. No experimental data are available for the properties of the constituent materials or the masonry for the churches being investigated. The numerical study includes the determination of the fundamental eigen-modes and eigen-periods of the studied structural system. The eigen-periods for this church (Tx and Ty) and the corresponding modal mass participation ratios (Ux and Uy, as percentages of the total mass), were obtained from the numerical model and are shown in figure 11. 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. 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.



Tx = 0.1008 seconds ux = 53,73%

Ty = 0.0948 seconds uy = 48,84%

Figure 11. Main eigen-modes and eigen-periods of the final model for the church of Metamorfosi tou Sotiros

5. THE BEHAVIOR OF THE CHURCHES SUBJECTED TO GRAVITATIONAL AND EARTHQUAKE LOADING IMPOSED IN A STATIC MANNER

The behavior of all three structures was examined next when they were subjected to three distinct loading conditions. The forces in all these three loading conditions were applied in a static manner. Base fixity was assumed for all masonry at the foundation level. The first loading case included the dead (G) loads of all parts plus the live (Q) loads (mainly snow at the roof level plus the live load at the level used as women's quarters). During the second and third loading conditions the earthquake forces Ex and Ey were applied along the x-x and the y-y axis, respectively. This was done in a simple way assuming unit acceleration for all the parts of the structure equal to 1g (where g is the acceleration of gravity). The dynamic nature of the seismic forces was taken into account in a separate series of simulations presented in the next section.

1st Church | **2nd Church**

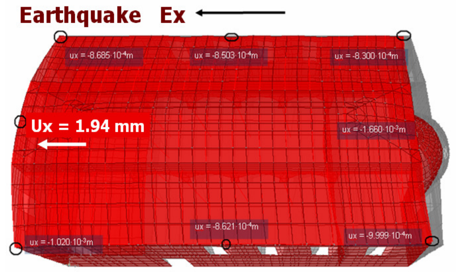


Figure 12a. Deformations for seismic forces Ex

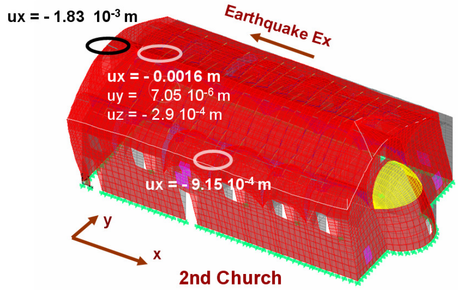


Figure 13a. Deformations for seismic forces Ex

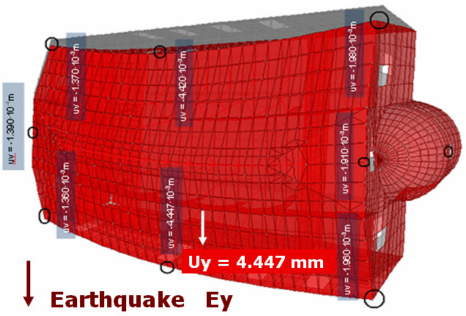


Figure 12b. Deformations for seismic forces Ey

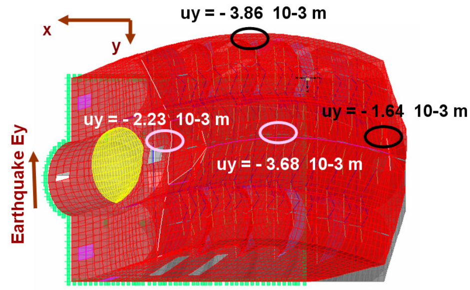


Figure 13b. Deformations for seismic forces Ey

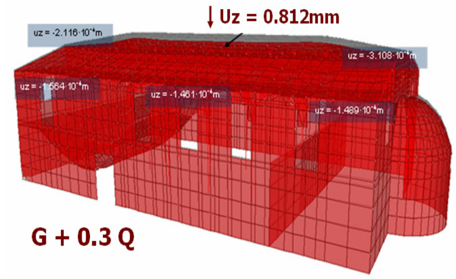


Figure 12c. Deformations for gravity forces

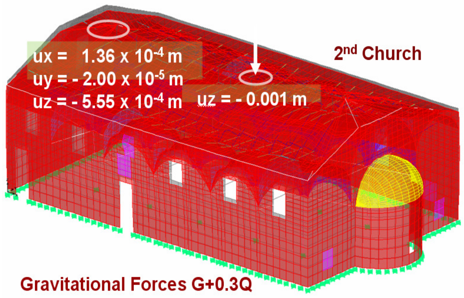


Figure 13c. Deformations for gravity forces

5.1. The "Post-Byzantine Basilicas"

The results for the 1st and 2nd Post-Byzantine Basilicas are depicted in figures 12 and 13, respectively. As can be seen in these figures, the structural system of both churches is more flexible in the transverse than in the longitudinal direction. The resistance of the internal colonnades to either the x-x or the y-y seismic forces is very small as these structural elements are quite flexible. The maximum horizontal displacement at the roof level is equal to 1.94mm for the 1st church (0.16mm for the 2nd church) for the loading case Ex whereas it attains the value of 4.447mm for the 1st church (3.86mm for the 2nd church) for the loading case Ey, more than double. The seismic forces are mainly resisted by the in-plane action of the peripheral walls parallel to the direction of these forces as well as by the out-of-plane action of the peripheral walls normal to the direction of these forces. The maximum value of deformations from the gravitational forces is equal to 0.812mm for the 1st church (1.0mm for the 2nd church); this occurs along the vertical direction at mid-span of the top of the roof. The vertical deformations at the top of the peripheral walls are of the order of 0.1mm to 0.2mm; moreover, the out-of-plane flexibility of the longitudinal walls results, at their top, in out-of-plane deformations of the order of 0.15mm when the structure is subjected to the gravitational forces.

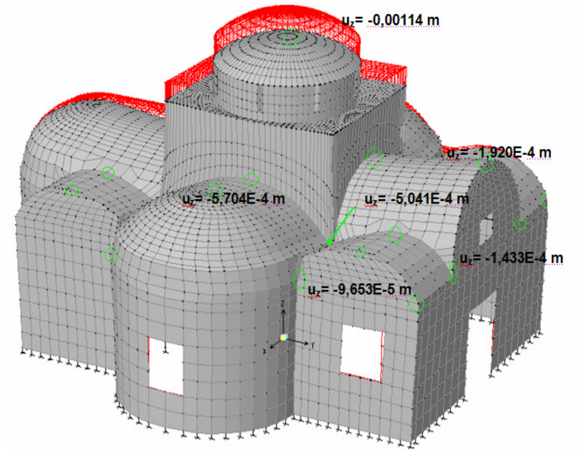


Figure 14a. Deformations for the Gravitational Loads δmax = 1.14mm at the top of the central dome.

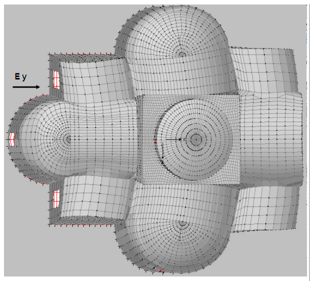


Figure 14b. **Ey** δmax = 4.49mm at the top of the central dome.

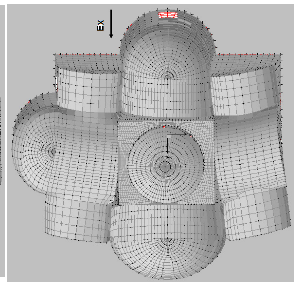


Figure 14c. **Ex** δmax = 3.63mm at the top of the central dome

5.2. The "Cruciform with Central Dome" Post-Byzantine church

The deformation patterns for the gravitational forces are shown in figure 14a whereas in figures 14b and 14c the deformation patterns for the seismic actions in the longitudinal (Ey) and in the transverse (Ex) directions are also depicted. The maximum horizontal displacements at the top of the dome level for the seismic actions Ey and Ex are 4.49mm

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 deformations (figure 14a). The cylindrical main cylindrical vaulting system is significantly distorted in the absence of tympana or apses.

b). 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. This 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.

c). 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 (figures 14b & 14c). The stress fields that develop correspond to this type of behavior.

6. EVALUATION OF THE STRESS RESULTS FOR THE THREE CHURCHES SUBJECTED TO EARTHQUAKE LOADING.

This time the design spectrum of the Greek Seismic Code (Provisions of Greek Seismic Code 2000) was utilized for seismic zone I (ground design acceleration 0.16g), soil category B, response modification factor q = 1.5 and importance factor 1.3. In the spectral dynamic analyses that were conducted, the resultant seismic forces were obtained from the Greek Seismic Code response spectrum and the following loading combinations (G the dead loads, Ex and Ey the earthquake action in the x and y directions).

0.9G+1.4Ey / 0.9G+1.4Ex / G+Ey+0.3Ex / G+Ex+0.3Ey.

From all the load combinations, the most critical in-plane demand values, either in normal or shear stresses, can be identified for all four peripheral walls. This can also be done for the most critical out-of-plane normal stress demand values for all four peripheral walls. For the 2nd and 3rd church this study was also extended to the masonry vaults and domes of the superstructure. Next, certain commonly used masonry failure criteria were adopted for either in-plane tension-compression or

shear or out-of-plane tension. The demands, in terms of normal and shear stress response, which were predicted by the linear elastic numerical simulation, are next utilized together with an assumed failure criterion with tension cut off. Initiation of failure is assumed when these demands exceed the following adopted capacities: **a)** The normal stress response exceeds the allowable compressive strength (assumed equal to 3.846 N/mm2, Edgell et al. 2002). **b)** The normal stress response exceeds the allowable tensile strength of stone masonry (assumed equal to 0.192 N/mm2) **c)** The shear stress response exceeds the shear strength of stone masonry. Following the proposed revisions to Eurocode-6 (Eurocode 6, Nov 2002, Chiostrini et al. 2003) the shear failure criterion that was adopted is:
$f_{vk} = f_{vko} + 0.4\ \sigma_n$ (1) where: f_{vko} is the shear stress when the normal stress is zero; f_{vko} was assumed to be equal to 0.192 N/mm2. σ_n is the normal stress.

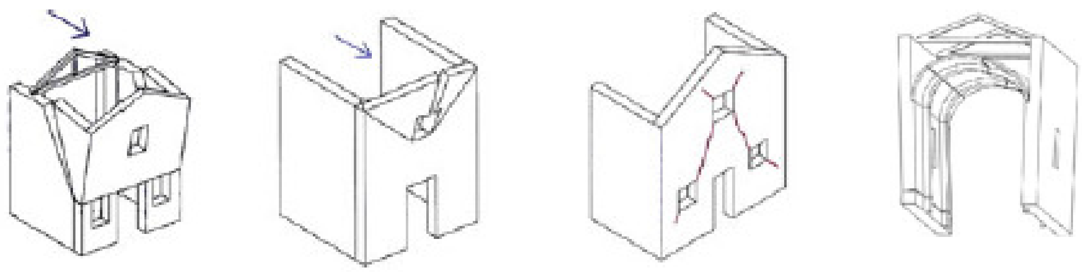
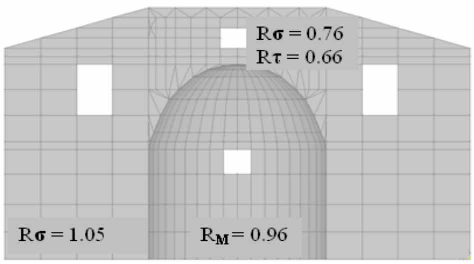


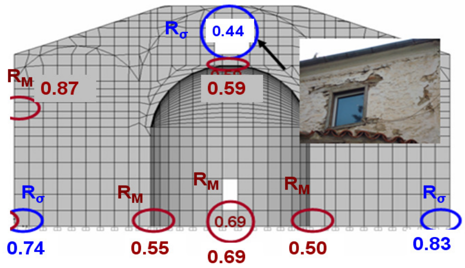
Figure 15. Some of the most common types of observed damage for the "Post-Byzantine Basilicas"

6.1. The "Post-Byzantine Basilicas"

All the masonry parts of the studied structures were examined in terms of in-plane and out-of-plane stress demands posed by the considered load combinations against the corresponding capacities, as these capacities being obtained by applying the Mohr-Coulomb criterion of equation 1 and the stone masonry compressive and tensile strength limits mentioned in 6.1. Due to space limitations such results are not shown in detail here. Typical forms of damage are depicted in figure 15. This type of damage is also reported in the recent L'Aquila earthquake (D'Ayala et al. 2011, Di Giulio G. et al. 2011, Lagomarsino 2011, Modena et al. 2010) Selective results obtained from this evaluation process are shown in figures 16 and 17. With Rσ or with Rτ the ratio of the in-plane tensile or shear strength value over the corresponding demand is signified whereas with RM the ratio of the out-of-plane tensile strength value over the corresponding demand is denoted. Ratio values smaller than 1.0 predict a corresponding limit state condition has been reached thus resulting in damage.

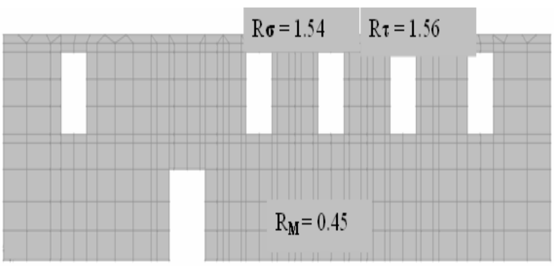


a) 1st Church

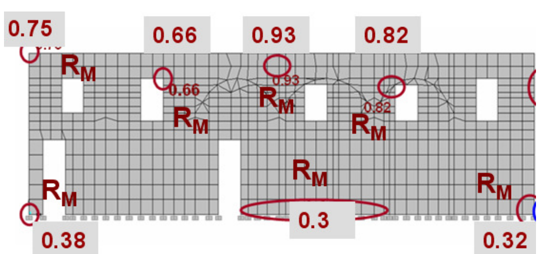


b) 2nd Church

Figure 16. East wall – Ratio values of strength over demands



a) 1st Church



b) 2nd Church

Figure 17. South wall – Ratio values of strength over demands

6.2. The "Cruciform with Central Dome" Post-Byzantine church

Figure 18 is the summary of the predicted performance for the church of Metamorfosis tou Sotiros whereas figure 19 depicts the predicted performance of the vaulting system and the South peripheral wall for the church of Holy Trinity. This is done on the basis of comparing capacity / demand ratio values for the various masonry structural elements.

a1. It can be stated that, apart from the shear damage that is predicted at the peripheral walls near the openings (figure 17, 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.
a2. The above conclusion is also valid for the peripheral walls for the church of Holy Trinity (figure 18). As expected, due to the height of the central dome, an increased seismic vulnerability of the vaulting and dome system is indicated by the predicted performance.
a2. All the above predicted zones of potential failure are credible, as can be deduced from such damage patterns observed in this structure after the 1995 strong earthquake event. An additional extensive non-linear analysis was performed (Manos et al. 2008) in order to verify the observed damage with numerical predictions. Due to lack of space these results are not included.

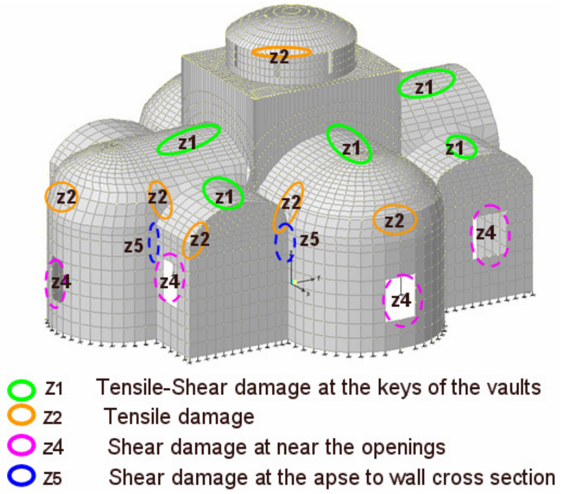


Figure 18. Predicted damage regions for the church of Metamorfosi tou Sotiros

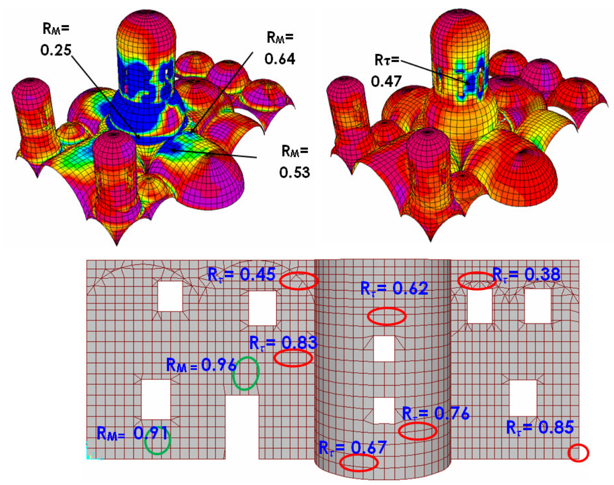


Figure 19. Predicted damage regions for the church of Holy Trinity

CONCLUSIONS

1. The numerical results presented are an attempt to numerically explain the observed seismic behavior of Post-Byzantine stone masonry churches. The relatively simplified linear analysis approach together with the adopted failure criteria seems to verify the development of the actual damage in all three examined structural formations. Both predicted and actual damage is mainly concentrated at the keys and supports of the arches and vaults, at the supports of the roofing system as well as at the door and window openings of the peripheral walls. Further extensive verification is needed for the assumed strength values utilized by the failure criteria. A number of tests is currently under way.
2. The eigen-periods, eigen-modes and the deformation patterns to horizontal earthquake actions of the examined Basilicas demonstrate that these structural formations develop much larger displacements at the top of their longitudinal peripheral walls in a direction normal to the plane of these walls than in a direction parallel to that plane.
3. The numerical stress results together with assumed strength values for the various masonry elements of the examined Basilicas predict that the regions most vulnerable to damage are near the door and window openings for the in-plane behavior. These regions together with the regions near the foundation appear to be the most vulnerable in out-of-plane bending, particularly for the longitudinal masonry walls. These regions that are shown to be vulnerable to damage are in reasonably good

agreement, in a qualitative sense, with actual observed damage. The masonry superstructure for the 2nd church is also shown to be vulnerable.

4. For the “Cruciform with Central Dome” 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 this church from the main 1995 event presents features that are in reasonable agreement with the above predictions. However, there is need for validating the employed failure criteria by relevant laboratory tests.

5. A relatively mild intervention scheme was adopted by the cultural conservation authorities in all the studied cases, utilizing special mortar injections, aimed at increasing the stone masonry strength. The repair scheme included the use of metal ties together with the reconstruction of the wooden roof.

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