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A NUMERICAL INVESTIGATION OF THE DYNAMIC AND EARTHQUAKE BEHAVIOR OF BYZANTINE AND POST-BYZANTINE BASILICAS

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ABSTRACT

The dynamic and earthquake behavior of structural formations representing Byzantine and Post-Byzantine churches of the "Basilica" form is investigated numerically. The "Basilica" structural system is one of the oldest structural forms and is utilized in a considerable number of Christian Churches with a number of variations in plan and height. Three specific structures are studied here numerically. The first is a three-nave 19th century typical Post-Byzantine Basilica with dimensions 19m x 11m in plan and 6.8m high that retains its original structural formation as a whole. The second structure is also an early 19th century three-nave Basilica with dimensions 19m x 12m in plan and 6.6m high. This church utilizes a system of cylindrical vaults and spherical domes that form the superstructure together with the wooden roof. The third church is a much older (13th century) and larger structure (37.8m x 17.8m in plan and 12m height). An additional feature of this 3rd structure is the fact that its South nave is completely missing today, as it collapsed well in the past. The numerical results together with assumed strength values for the various masonry elements are utilized to predict the behavior of the various masonry parts in in-plane shear and normal stress conditions as well as in out-of-plane bending for all three churches. For these three Basilicas it can be deduced that the regions of the masonry walls near the foundation and the door and window openings appear to be the most vulnerable in out-of-plane bending for the critical combination of earthquake loads and gravitational forces. For the second structure the system of vaults and domes appears to be also vulnerable. When comparing the numerically predicted regions that reach limit state conditions with actual damage patterns a reasonably good agreement in a qualitative sense can be observed.

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Introduction

During past centuries various parts of Greece have been subjected to a number of damaging earthquakes. Some of these events, not necessarily the most intense, occurred near urban areas. One of the most demanding tasks for counteracting the consequences of all these seismic events is the effort to ensure the structural integrity of old churches, that were built in periods ranging from 400 A.D. till today; in many cases they sustained considerable damage. In what follows, selected results and summary observations are presented of the dynamic and earthquake behavior of a specific type of structural system that is utilized in a considerable number of churches belonging to the so-called Byzantine and Post-Byzantine period (11^{th} to 19^{th} century A.D.). 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 in a number of naves by longitudinal colonnades of various dimensions and shapes, as shown in figure 1a. The roofing system is mainly in the longitudinal direction; this roofing system at the central nave in some cases rises 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 whereas the roofing system that covers the side naves is partially supported on the peripheral walls and usually rises at a height lower than that of the central nave (figure 1b). In some cases the wooden roof is supported on a system of cylindrical vaults, spherical domes and arches, made of masonry, that rise from the masonry peripheral walls and internal colonnades.

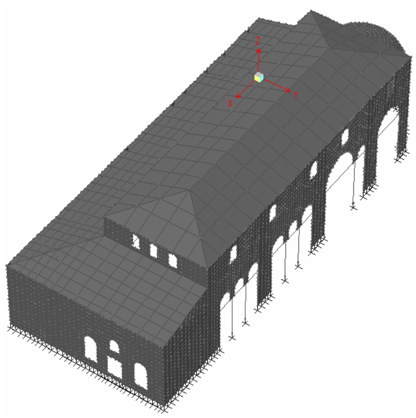


Figure 1a. The Basilica structural system with the interior colonnade of the central nave

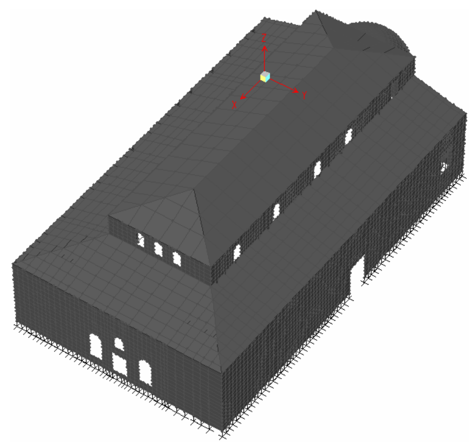


Figure 1b. The Basilica structural system with the peripheral longitudinal and transverse walls

In the present study the dynamic and earthquake behavior of this "Basilica" structural form will be investigated with the following three distinct cases.

a) The first case is a three nave structural formation which, in the overall geometry, represents five 19^{th} century Post Byzantine churches with similar geometry which were damaged by the 1995 strong Kozani- Greece earthquake sequence. 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 height of the peripheral walls is 6.2m; from this level the top of the wooden roof rises another 0.6m (Figures 2a and 2b). The internal colonnades are made of wood and the thickness of all masonry walls is assumed to be equal to 800mm.

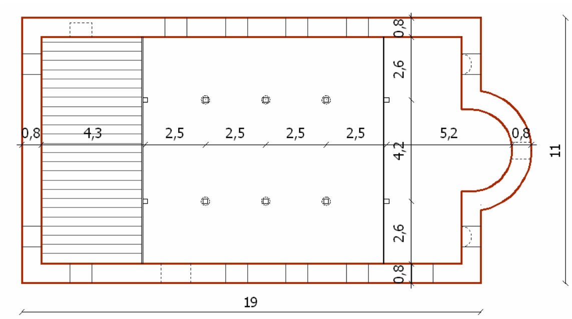


Figure 2a 1st Post-Byzantine "Basilica"– plan

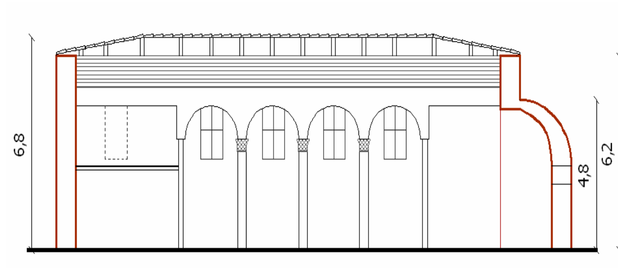


Figure 2b 1st Post-Byzantine "Basilica" – Cross section

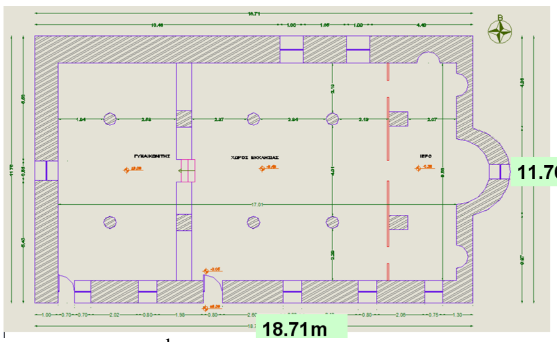


Figure 3a 2nd Post-Byzantine "Basilica" plan

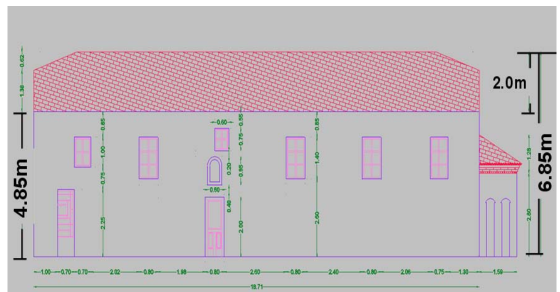


Figure 3b 2nd Post-Byzantine "Basilica" – South Elevation

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 are utilized to support the wooden roof that rises another 2.0m from the top of the peripheral masonry walls (Figures 3a and 3b). The thickness of the masonry walls varies from 700mm to 1200mm.

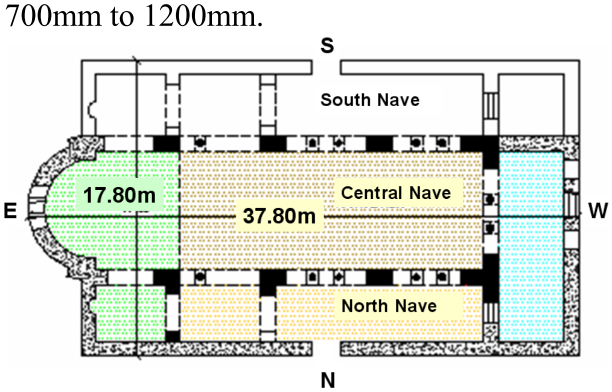


Figure 4a. Byzantine Basilica - Plan

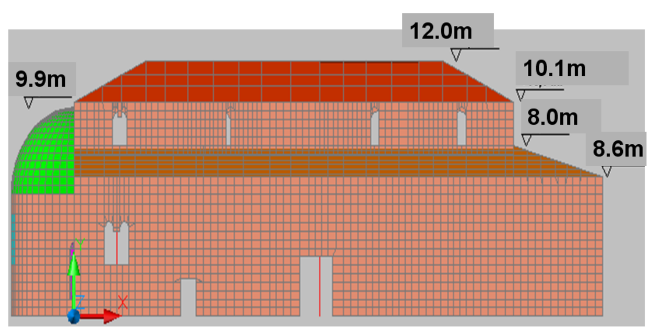


Figure 4b Byzantine Basilica - Elevation

c) The third case is again a three-nave Basilica; however, this is a much older Byzantine church and this time the overall dimensions are much larger (Figures 4a and 4b). This church is 37.8m long in the longitudinal direction and 17.8m wide in the transverse direction. The height of the central nave is 12m. The internal colonnades are made of masonry piers as well as marble columns. The thickness of the masonry walls varies from 0.85m to 1.10m.

All three structural systems are composed of stone masonry for the peripheral walls, vaults and domes and internal transverse partitions. Moreover, in all cases the East wall is characterized by an apse and a wooden roofing system is utilized.

Results from the modal analysis of the Post-Byzantine Basilica

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 $2.70t/m^3$. 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 $0.66t/m^3$.

a) Figures 5a and 5b depict the mainly horizontal translational eigen-modes for the 1st structure. The translational eigen-mode in the transverse North-South (y-y) direction is the one with the longest eigen-period (Figure 5a, 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 5b, 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.

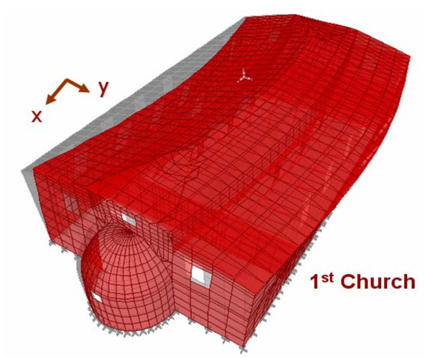


Figure 5a. Ty = 0.102 seconds, uy = 51.38%

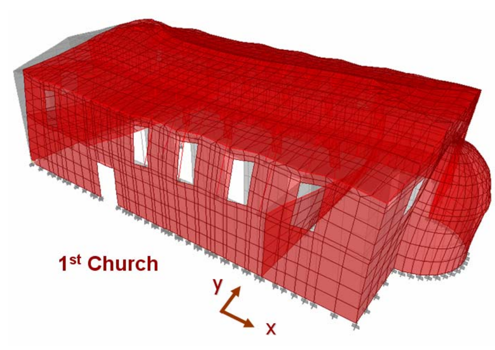


Figure 5b. Tx = 0.069 seconds ux = 49,63%

b) In comparison, figures 6a and 6b depict the mainly horizontal translational modes for the 2nd structure. It can be seen that the eigen-periods in both the longitudinal and the transverse directions for the 2nd church are somewhat longer than those of the 1st structure. However, this time the modal mass ratios, that are mobilized by these two translational modes, are noticeably larger than those of the 1st structure. Both these effects must be attributed to the mass of the system of masonry cylindrical vaults, spherical domes and arches.

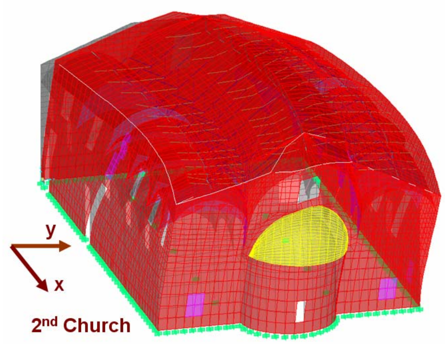


Figure 6a. Ty = 0.1053 seconds, uy = 61.8%

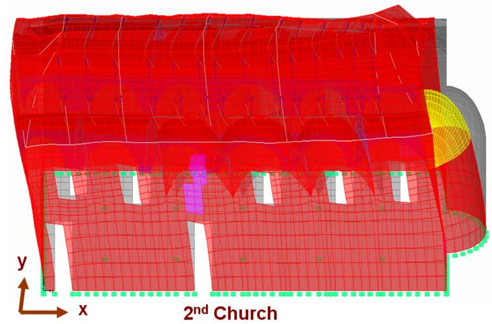
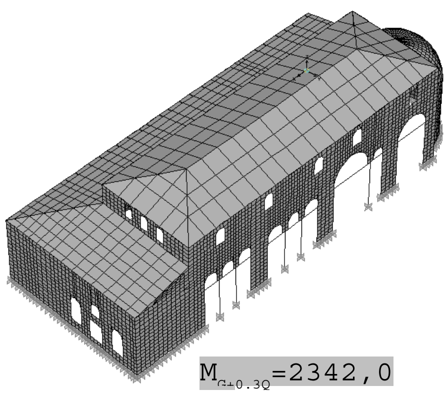


Figure 6b. Tx = 0.0724 seconds ux = 63,966%

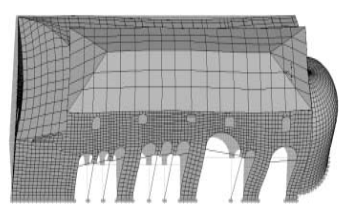
c) the modal analysis of the third structure (3rd church) which represents the old "Byzantine Basilica" structural formation includes the study of the existing structural formation, with one of the side naves missing (Model 1, figure 7a) and the original structural formation, which does not survive today (Model 2, figure 7b). By comparing the eigen-period values of the fundamental translational modes resulting from the two structural formations of this 3rd church the following observations can be made. As expected, the main influence of the missing nave is in the transverse rather than in the longitudinal direction. It must be noted that the masonry piers of the remaining West part that are in line with the internal colonnades provide sufficient stiffness in the longitudinal direction to compensate for the missing longitudinal wall of the missing nave.

Figure 7a. Model 1



Longitudinal Transl.

T_x = 0.1334sec

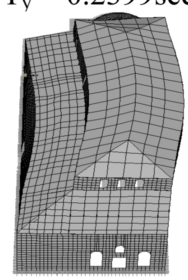


Mass Partic. Ratio %

U_x	U_y	U_z
56.32	0.00	0.00

Transverse Translational

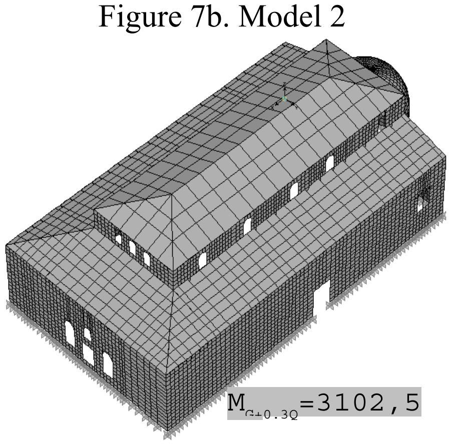
T_y = 0.2399sec



Mass Partic. Ratio %

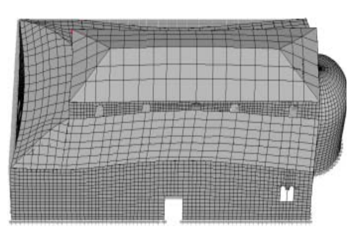
U_x	U_y	U_z
0.00	**43.38**	0.00

Figure 7b. Model 2



Longitudinal Transl.

T_x = 0.1374sec

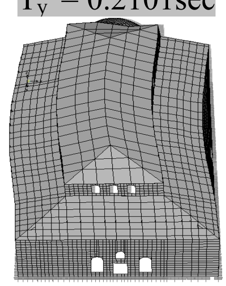


Mass Partic. Ratio %

U_x	U_y	U_z
40.56	0.00	0.00

Transverse Translational

T_y = 0.2101sec



Mass Partic. Ratio %

U_x	U_y	U_z
0.00	**42.67**	0.00

The missing portion of the transverse peripheral East and West walls as well as that of the Narthex results in a noticeably more flexible system than the original structural formation. Apart from the stiffness variation resulting from the missing nave one should also take into account the absence of the mass of the missing masonry part.

When comparing the obtained dynamic characteristics of the above two models of the Byzantine Basilica with the ones presented before for the two Post-Byzantine Basilicas the following observations can be made.

a1. Again, the translational eigen-mode in the transverse North-South (y-y) direction for the third church is the one with the longest eigen-period (see Figures 7a and 5a, 6a). 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. Because of the relatively larger size of the Byzantine Basilica the values for this eigen-period are much longer (T_y = 0.2101sec. – T_y =0.239sec) than the values obtained for this eigen-mode for the two Post-Byzantine Basilicas (of the order of T = 0.10 seconds).

a2. Again, the translational eigen-mode in the longitudinal East-West (x-x) direction for the third church is the next longest eigen-period (see Figures 7b and 5b, 6b). 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. Because of the relatively larger size of this Byzantine Basilica the values for this eigen-period are much longer (T_y = 0.131sec. – T_y =0.137sec) than the values obtained for this eigen-mode for the two Post-Byzantine Basilicas (of the order of T = 0.07 seconds). However; because of the increased stiffness in the longitudinal direction relative to the transverse direction the difference in the eigen-period values is comparatively smaller than that in the longitudinal direction, as noted in observation a1 above.

a3. Again, 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.

The two Post-Byzantine and the Byzantine Basilicas subjected to static loading

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) 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 .

The results for the 1st and 2nd Post-Byzantine Basilicas are depicted in figures 8 and 9, 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.

1st Church | **2nd Church**

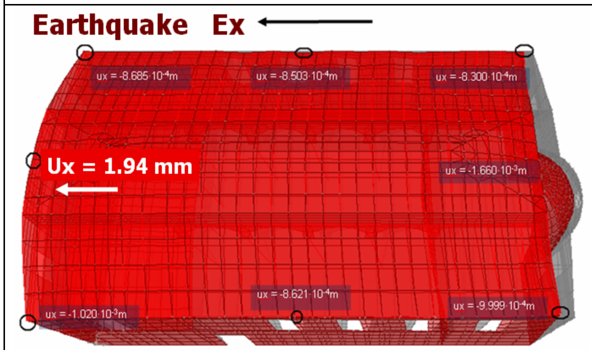


Figure 8a. Deformations for seismic forces Ex

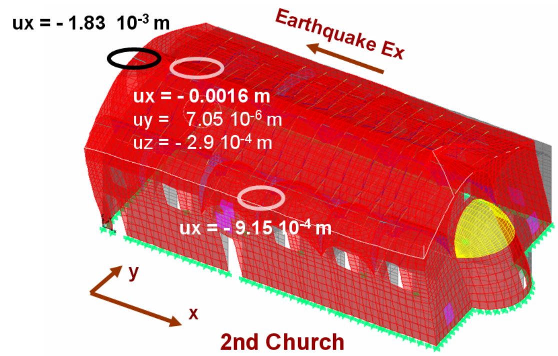


Figure 9a. Deformations for seismic forces Ex

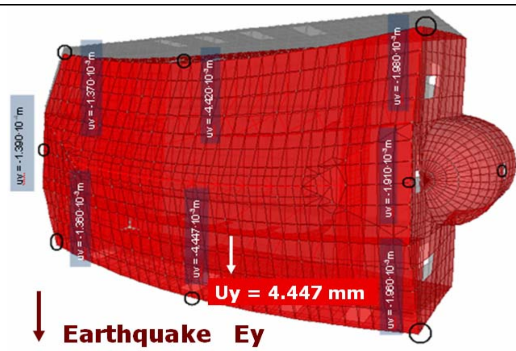


Figure 8b. Deformations for seismic forces Ey

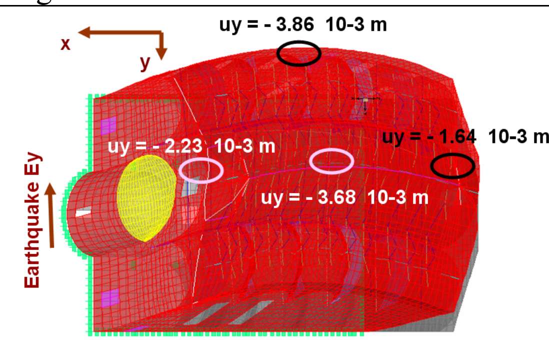


Figure 9b. Deformations for seismic forces Ey

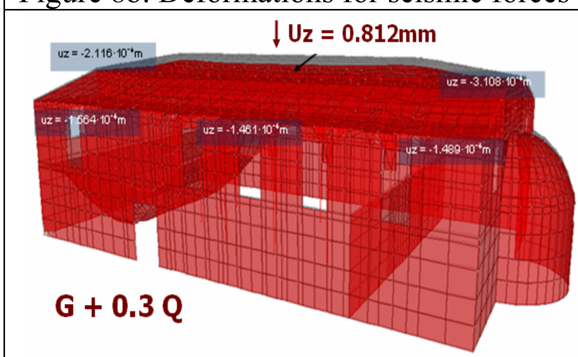


Figure 8c. Deformations for gravity forces

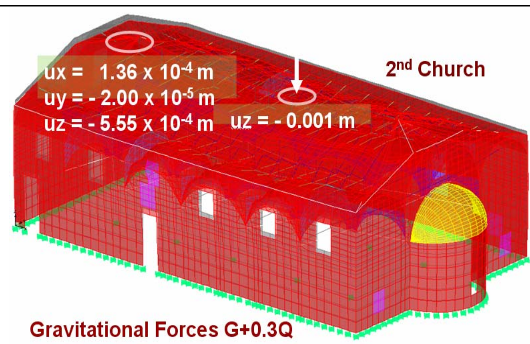


Figure 9c. Deformations for gravity forces

The results for the 3rd church (Byzantine Basilica) are depicted in figures 10 and 11 for model 1 (existing structure) and model 2 (original structure), respectively. Due to the larger size of this structure the horizontal displacements at the top of the peripheral walls and at the top of the roof levels are much larger, as expected, than those of the relatively smaller Post-Byzantine Basilicas. The observations made before for the Post- Byzantine (1st and 2nd) Basilicas with regard to the flexibility of this structural system in the y-y direction for the Ey earthquake forces are much more evident here. This effect is further amplified by the missing nave, as can be seen by comparing the maximum horizontal displacement due to the earthquake forces Ey for model 1 and model 2 in figures 10b and 11b, respectively

3rd Church – Model 1

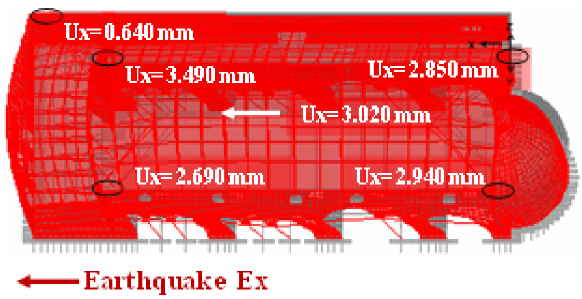


Figure 10a. Deformations for seismic forces Ex

3rd Church – Model 2

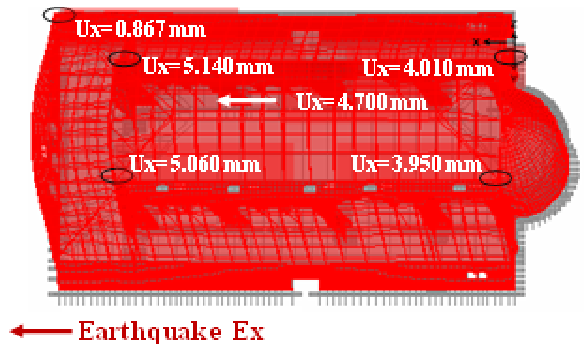


Figure 11a. Deformations for seismic forces Ex

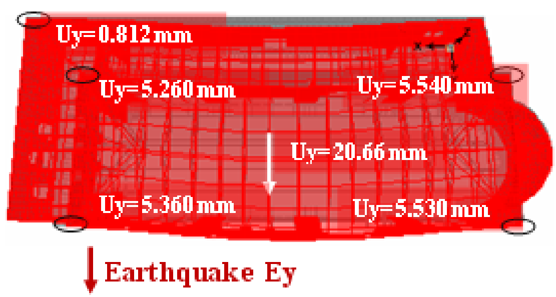


Figure 10b. Deformations for seismic forces Ey

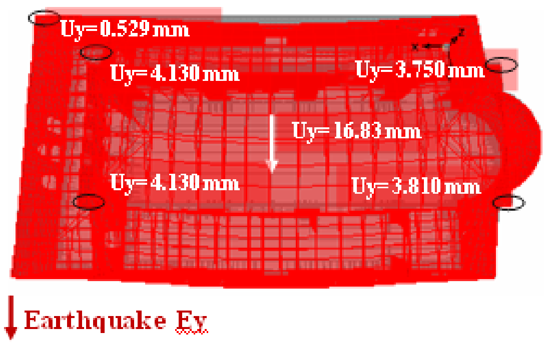


Figure 11b. Deformations for seismic forces Ey

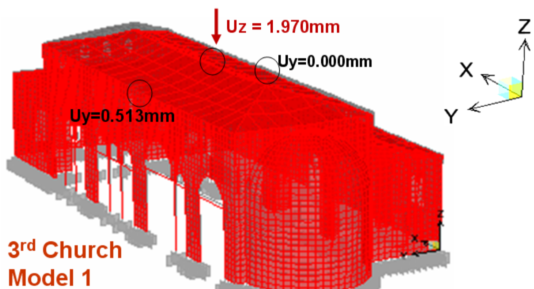


Figure 10c. Deformations for gravity forces

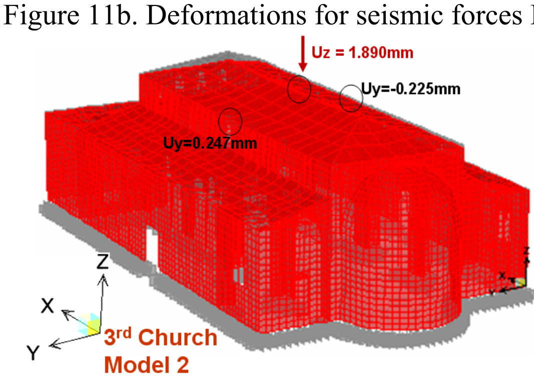


Figure 11c. Deformations for gravity forces

Evaluation of Stress Results for the three Basilicas subjected to earthquake loading.

This time the design spectrum of the Greek Seismic Code (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 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. Table 1 lists values which were assumed to be valid for the critical mechanical properties for the masonry segments (Euro code 6 and Manos et.al. 2008).

Table 1. Assumed Mechanical Characteristics of the Stone Masonry

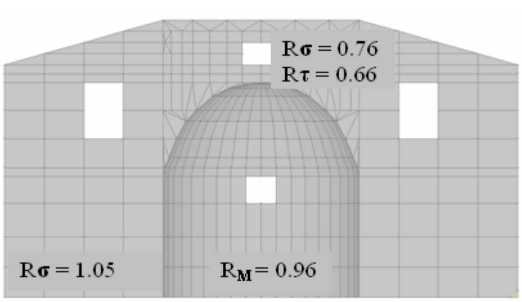
	Young's Modulus (N/mm^2)	Poisson's Ratio	Stone Masonry Compressive Strength (N/mm^2)	Stone Masonry Tensile Strength (N/mm^2)	shear strength f_{vko} (N/mm^2)
Upper limit	2500	0.2	3.846	0.250	0.192
Lower limit	2500	0.2	1.00	0.192	0.192

Moreover, a Mohr-Coulomb failure envelope was adopted for the in-plane shear limit state of the stone masonry, when a σ_n normal stress is acting simultaneously, that is defined through the relationship

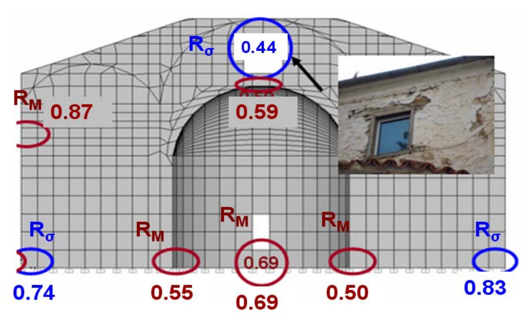
$$f_{vk} = f_{vko} + 0.4\,\sigma_n \qquad (1)$$

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

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 were obtained by applying the Mohr-Coulomb criterion of equation 1 or the upper stone masonry compressive and tensile strength limits listed in Table 1. Due to space limitations such results are not shown in detail here. Selective results obtained from this evaluation process are shown in figures 12, 13 and 14. With $R\boldsymbol{\sigma}$ or with $R\boldsymbol{\tau}$ is signified the ratio of the in-plane tensile or shear strength value over the corresponding demand whereas with $R_{\mathbf{M}}$ 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.

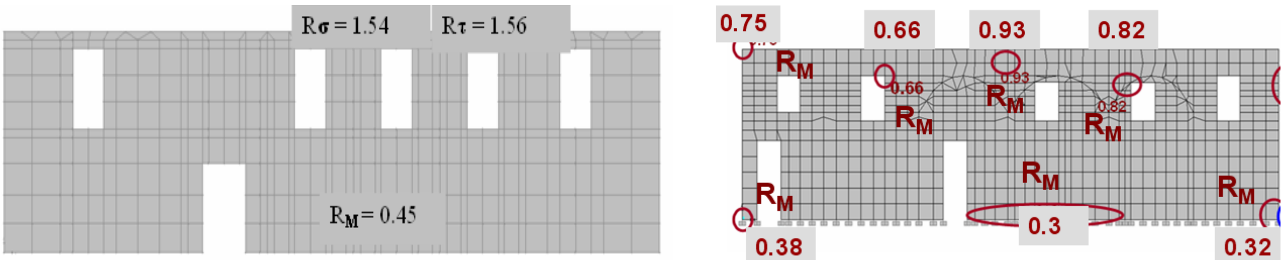


1st Church



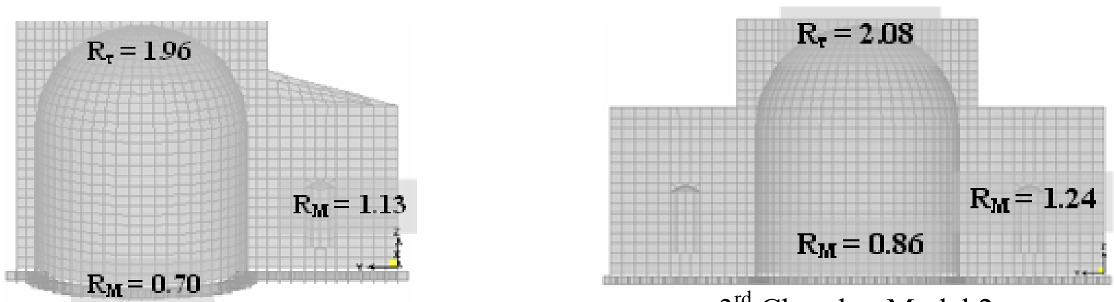
2nd Church

Figure 12. East wall – Ratio values of strength over demands



1st Church 2nd Church

Figure 13. South wall – Ratio values of strength over demands



3rd Church – Model 1 3rd Church – Model 2

14. East wall – Ratio values of strength over demands

Conclusions

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 at a direction parallel to that plane.

The numerical stress results together with assumed strength values for the various masonry elements of the examined Basilicas predict that the most vulnerable regions to be damaged 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.

One of the examined structural formations has a missing nave as it exists today. The current investigation demonstrated that the structure with the missing nave is more vulnerable than this structure in its original completed form with all three naves in place

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