NUMERICAL SIMULATION OF THE DYNAMIC AND EARTHQUAKE BEHAVIOUR OF GREEK POST-BYZANTINE CHURCHES WITH AND WITHOUT BASE ISOLATION

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Abstract. *The dynamic and earthquake response of Post-Byzantine stone masonry churches, which were subjected to a damaging earthquake sequence in the region of Western Macedonia in Greece, is studied. Two structural systems of the "basilica" form are first examined. Apart from these two "basilica" churches, the behavior of another much older church, which represents a much simpler orthogonal structural formation, is also examined. This is a two-story church with a simple orthogonal vault and a wooden roof that belongs to an 11th century monastic-complex. Two of these churches are also studied together with the introduction of a base isolation system. The performance of each of these three distinct structures is studied under the combination of the gravitational and seismic forces. The predicted performance of the various structural elements, located at the peripheral masonry walls is then checked by applying an assumed Mohr-Coulomb failure envelope that is believed to represent the limit-state stone-masonry behavior. The seismic damage, due to a relatively recent strong earthquake sequence, is utilized to validate the realism of the numerical predictions. The numerical introduction of a base-isolation system was quite effective in lowering the demands and thus resulting for most structural elements in Capacity/Demand ratio values larger than one (Capacity/Demand > 1), which is assumed to indicate acceptable structural performance. However, the introduction of base isolation under an existing old stone-masonry structure is also faced with considerable difficulties, despite the advantages that are shown by the results of numerical investigation. In the particular case of "The Church of The Virgin Mary of Tourniki" these difficulties must be faced because this old church has to be moved in a different location due to flooding. Thus, the placement of a base isolation scheme under this structure in its new location is a feasible solution for this particular case with no significant extra cost. Obviously, thus base isolation scheme should be considered in addition to all other traditional repair and strengthening measures for this church.*

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. One of the most demanding tasks for counteracting the consequences of all these seismic events was the effort to ensure the structural integrity of old churches, that were built in periods ranging from 400 A.D. up to today; in many cases they sustained considerable damage (ref. 1., Manos et.al. 2008). In what follows, selected results and summary observations are presented of the dynamic and seismic behavior of two specific types of structural systems that are utilized in a considerable number of churches belonging to the so-called Post-Byzantine period (16th to 19th century A.D.). The first is the structural system of the "Basilica" that takes a number of variations in plan and height whereas the second type is a simple box-type structural formation. The present study examines the seismic behavior of two churches belonging to the 1st structural system ("Typical Post-Byzantine Basilica" and "Taxiarhes") and a church belonging to the 2nd structural system (Virgin Mary at Tourniki). All these churches are located in the region of Western Macedonia – Greece and they were all subjected to the damaging strong Kozani earthquake sequence of May, 1995. The numerical results presented here are part of an attempt to numerically explain the observed behavior. The numerical investigation includes modal analysis results; no attempt was made to compare these results with in-situ measurements for dynamic identification. Next, for all examined structures the numerical analysis predicts the demands imposed on the masonry structural elements by critical load combinations which include earthquake loading as specified by the Greek Seismic Code (ref. 2). Finally, the performance of the various masonry structural elements is evaluated in section 3 by examining the ratio values of the Capacity / Demand. The demands are obtained by assumed strength values, without the incorporation of any safety factors. Space limitations allow only very limited presentation of the observed actual damage. In general, the predicted damage regions agree well with observed damage.

It is obvious that an important next step is the rehabilitation of the damaged masonry. The structural repairs of the stone-masonry churches examined here are inhibited by restrictions arising from retaining the visible stone-masonry architectural formations at the exterior and the frescos at the interior of the masonry walls. An acceptable type of repair is the application of low-pressure lime-mortar injections enriched with ceramic powder, resulting in a modest increase in the capacities of the various structural elements. The application of techniques used for the repair of reinforced concrete structural elements (jacketing or use of fiber reinforcing plastics), utilizing the resultant considerable capacity increase, is not acceptable for the stone masonry of such heritage structures. An alternative solution is lowering the demands by the use of base-isolation as the one investigated here. However, the introduction of base isolation under an existing old stone-masonry structure is also faced with considerable difficulties, despite the advantages that are shown by the results of numerical investigations as the one performed here.

1.1 The "Typical Post Byzantine Basilica"

The first case is a three-nave Post Byzantine "Basilica" structural formation which, in the overall geometry, is typical of a number of churches with similar geometry which were damaged by a strong earthquake sequence in the region of Western Macedonia, Greece during the Kozani Earthquake of 1995. 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 (figures 1a, 1b and 1c).
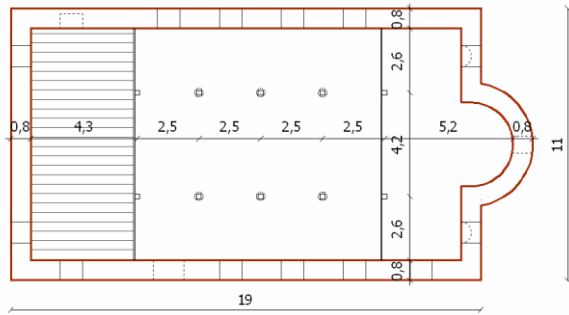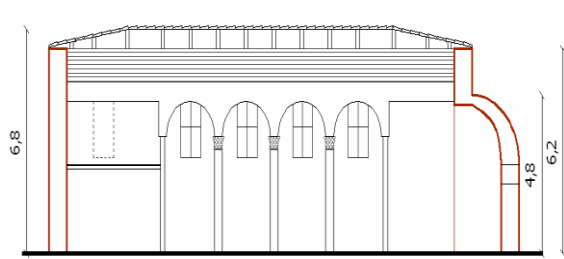
Figure 1a plan   Figure 1b Cross section.

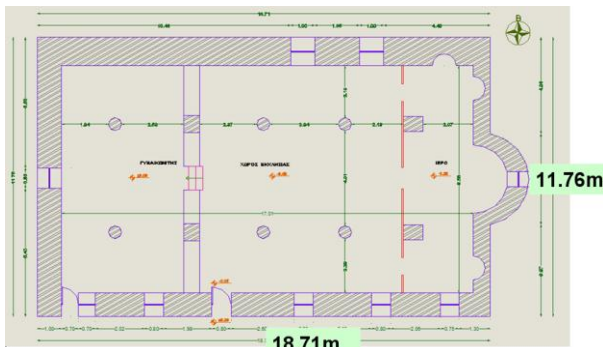Figure 1c. East wall and Apse.
Figure 1. Typical Post-Byzantine "Basilica"

Figure 2a plan
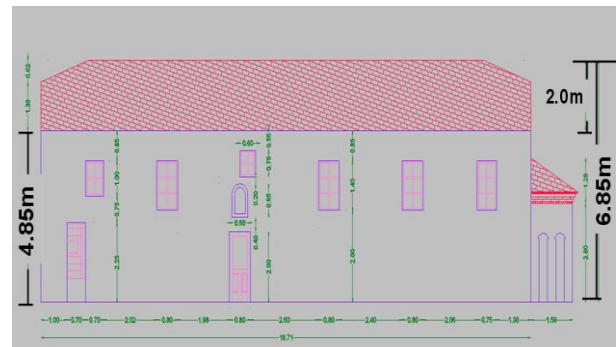Figure 2 " The church of Taxiarhes"

Figure 2b South Elevation

1.2 The "Post Byzantine Basilica church of Taxiarhes"

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 2a and 2b). The thickness of the masonry walls varies from 700mm to 1200mm.

1.3. The "church of The Virgin Mary of Tourniki"

This is a simple box-type structural system of rectangular shape, formed by the peripheral walls; a semi-cylindrical apse is part of the East wall, whereas the interior is usually divided into a number of naves by longitudinal colonnades of various dimensions and shapes. The simplest formation of a "Basilica" is that of a single-space rectangular structure without interior separations. Such a structure is also examined in this paper; this is a two-story Basilica, which is devoted to the Virgin Mary and belongs to the Monastic complex of Tourniki.

Such a two-story Basilica (Figure 3) is a rather rare structural system that can be found in the monastic complexes of Mt. Athos. The behavior of this two-story Basilica is numerically simulated by focusing on the super-structure and in particular on the influence of the cylindrical vaulting system that exists at the 1st story level as well as at the roof. The numerical investigation for this 2nd church is extended to study the introduction of a base isolation system.
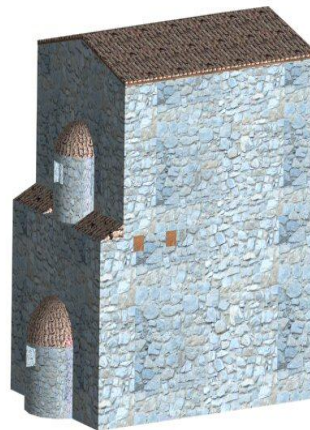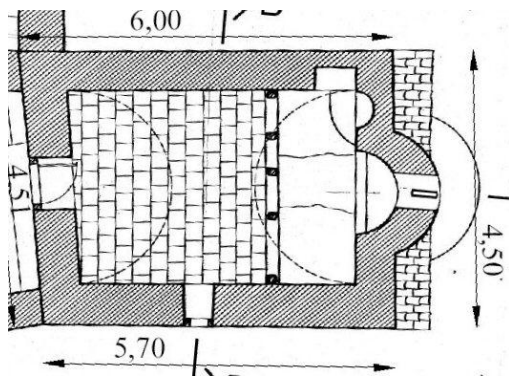

Fig. 3a.

Fig 3b.

Fig. 3c. Plan

Figure 3d. Cross section

Figure 3. of Virgin Mary of the Monastic complex of Tourniki
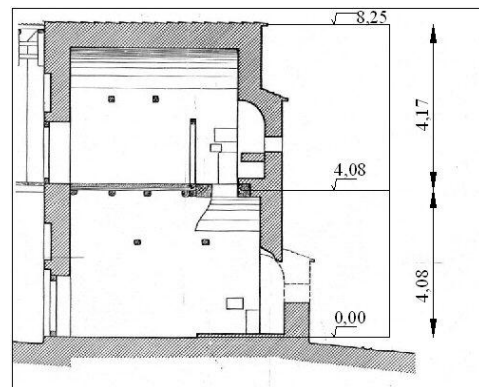
1.3.1. Main Features of The church of The Virgin Mary of Tourniki

The establishment of the monastic complex and the building of this church is set at the 11th century A.D. This church is a two-story box-type "Basilica" structural system formed by

two longitudinal walls (North and South) and two transverse walls (East and West) with a thickness of 700mm. The East transverse wall is dominated by two apses (one for each story) with a thickness of 700mm (Figs 3a and 3b). A relatively complex system is constructed at the East transverse wall with the two Apses in order to transfer the gravitational forces of the top East wall and Apse, which are offset by 700mm towards the interior of the church, when compared with the East wall and Apse of the bottom story (Figures 3a to 3d). The top story ceiling is formed by a longitudinal semi-cylindrical vault with a thickness of 350mm which supports the wooden roof together with the peripheral walls that extend appropriately for this purpose. Despite some small differences in the length of the North and South longitudinal walls, the structural system as a whole can be assumed to retain a mid-axis of symmetry with respect to the East-West longitudinal direction. Apart from the difference in the length of the longitudinal walls the exact location of the window openings deviates from this symmetry; however, these deviations are not believed to be very significant. This church was also subjected to the same 1995 strong earthquake when the monastic complex was not in use; damage visible at the cylindrical vaults cannot be linked directly to the event.

2. RESULTS FROM THE MODAL ANALYSIS OF THE POST-BYZANTINE AND THE 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.
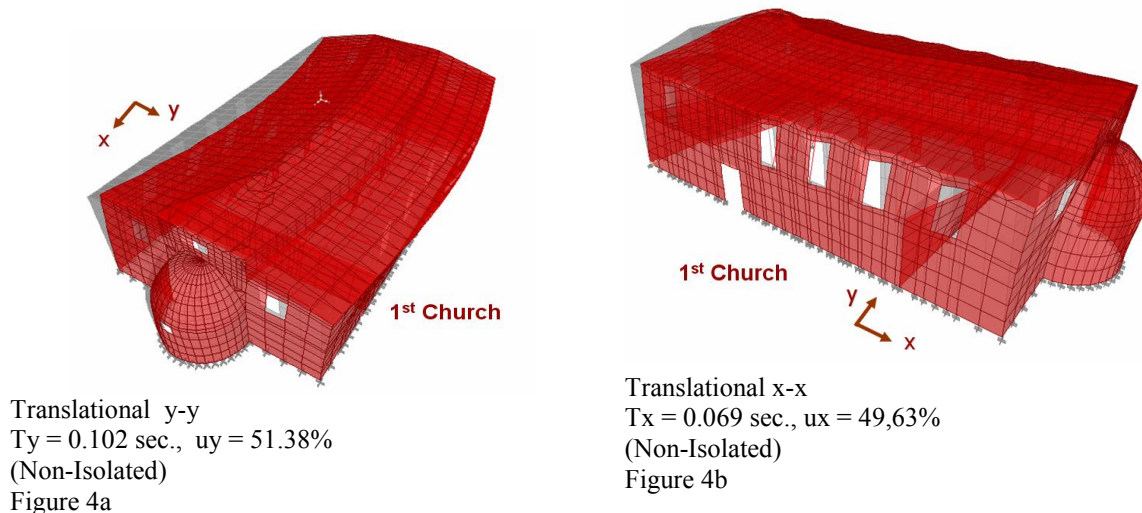

Translational y-y
Ty = 0.102 sec., uy = 51.38%
(Non-Isolated)
Figure 4a

Translational x-x
Tx = 0.069 sec., ux = 49,63%
(Non-Isolated)
Figure 4b

Figure 4. Dynamic characteristics "*Typical Basilica*"

2.1 The "Typical Post Byzantine Basilica"

Figures 4a and 4b 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 4a 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 lon-

gitudinal East-West (x-x) direction is the next longest eigen-period (Figure 4b 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 modes; however; these latter modes mobilize relatively small portions of the total mass.

2.2 The "Post Byzantine Basilica church of Taxiarhes"

In comparison, figures 5a and 5b 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.
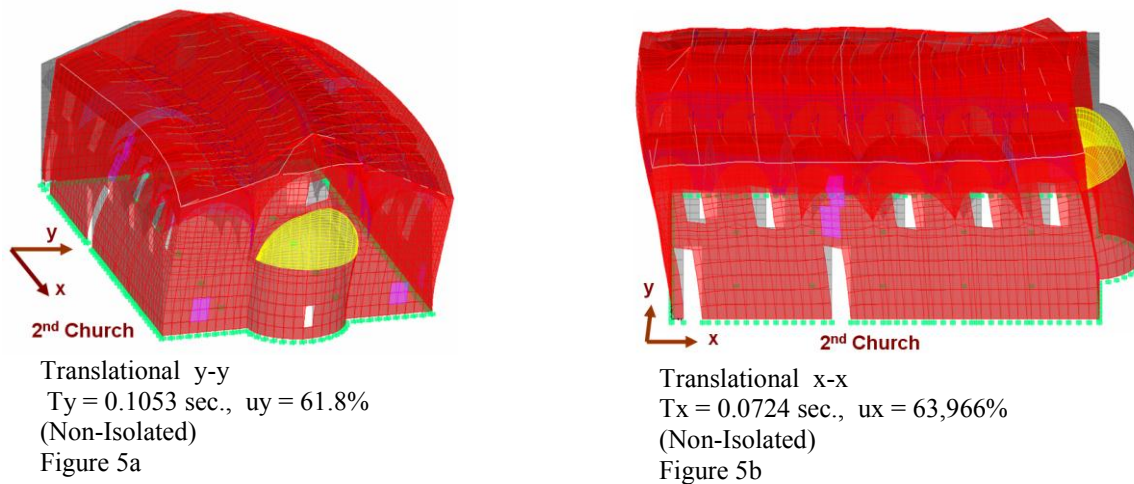

Translational y-y
Ty = 0.1053 sec., uy = 61.8%
(Non-Isolated)
Figure 5a

Translational x-x
Tx = 0.0724 sec., ux = 63,966%
(Non-Isolated)
Figure 5b

Figure 5. Dynamic characteristics *"Church of Taxiarhes"*

2.3. The "church of The Virgin Mary of Tourniki"

For this structural system the first two horizontal translational mode shapes and eigen-periods are shown in figures 6a and 6b. As can be seen, due to the two-story formation, these two eigen-periods are relatively longer than the corresponding values of the previously examined Post-Byzantine Basilicas. Moreover, these two eigen-modes also mobilize much larger modal mass ratios than the corresponding values of the previously examined Post-Byzantine Basilicas, a fact that must be attributed to the box-type form of this structure; moreover, the peripheral masonry walls have relatively small dimensions to excite them in an out-of-plane manner.
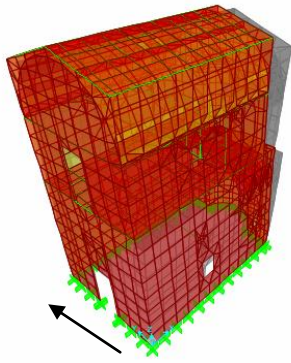
Figure 6a. 1st y-y
T=0.1375sec., Uy =73.94 %
Non -Isolated

Figure 6b 1st x-x
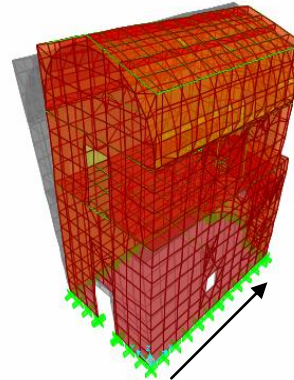T= 0.0942 sec Ux = 68.02 %
Non –Isolated

Figure 6. Dynamic characteristics Virgin Mary of Tourniki"

3.   THE DEMANDS ON THE STRUCTURE FROM THE GRAVITATIONAL AND EARTHQUAKE 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. The structures were assumed to have all three translational degrees of freedom (ux, uy, uz) restrained 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.

3.1 The "Typical Post Byzantine Basilica" and the "church of Taxiarhes"

The results for the 1st and 2nd Post-Byzantine Basilicas are depicted in figures 7 and 8, 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.
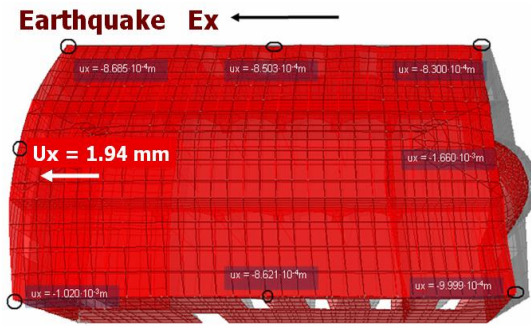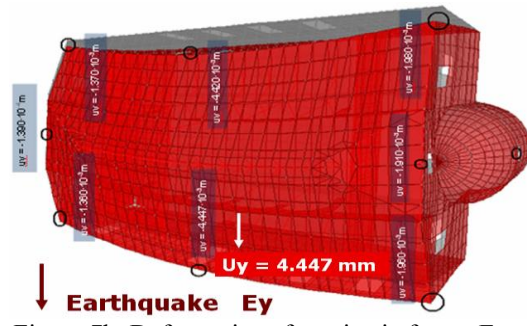
Figure 7a. Deformations for seismic forces Ex
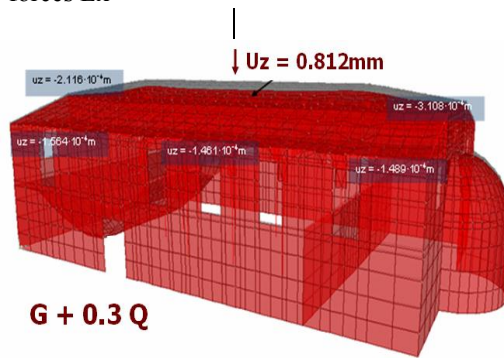
Figure 7b. Deformations for seismic forces Ey

Figure 7c. Deformations for gravity forces

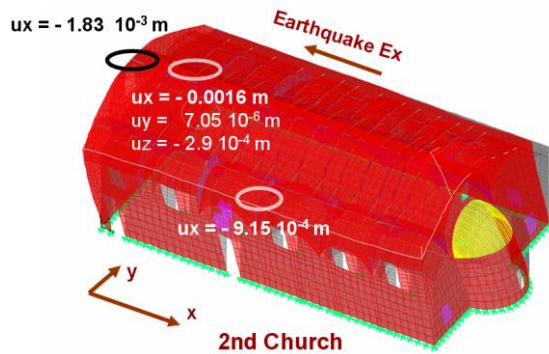Figure 7 Deformations of *"Typical Post Byzantine Basilica"*

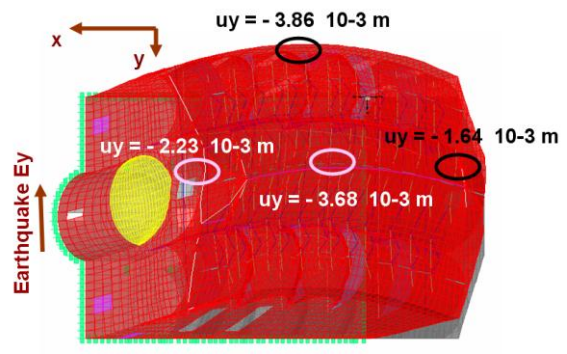Figure 8a. Deformations for seismic forces Ex

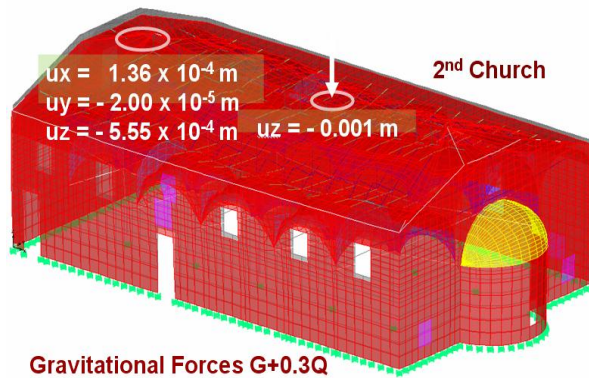Figure 8b. Deformations for seismic forces Ey

Figure 8c. Deformations for gravity forces

Figure 8. Deformations *"Church of Taxiarhes"*

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.

3.2. The "church of The Virgin Mary of Tourniki"

The small size in plan for this church and the box-type form results in relatively small horizontal displacement amplitudes at the roof level for the combination of gravity loads and seismic forces along the x-x and the y-y direction as depicted in figures 9a and 9b, respectively. The maximum value of the horizontal displacements at this level is approximately 3.1mm and develops, as expected, along the direction with the relatively smaller stiffness (North-South, y-y). This structural formation is designated in these figures as non-isolated to differentiate it with the same structure being isolated, as will be presented in section 5.
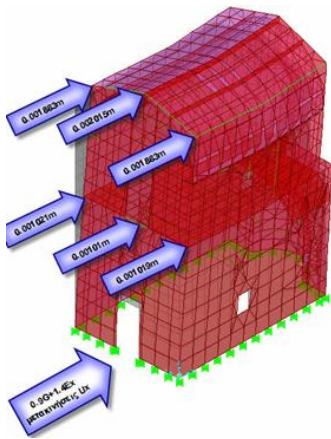

Figure 9a.
Non-Isolated, 0.9G + 1.4Ex
δmax = 2.015mm

Figure 9b
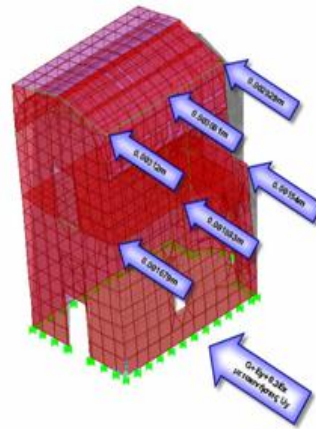Non-Isolated, 0.9G + 1.4Ey
δmax = 3.081mm

Figure 9. Displacements of The Virgin Mary of Tourniki"

4. EVALUATION OF THE STRESS RESULTS FOR THE THREE BASILICAS SUBJECTED TO EARTHQUAKE LOADING.

This time the design spectrum of the Greek Seismic Code (Greek Seismic Code 2000, ref. 2) 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 as-

sumed to be valid for the critical mechanical properties for the masonry segments (refs. [1] and [3]).

Table 1. Assumed Mechanical Characteristics of the Stone Masonry

	Stone Masonry Compressive Strength (N/mm2)	Stone Masonry Tensile Strength (N/mm^2)	shear strength f$_{vko}$ (N/mm^2)
Upper limit	3.846	0.250	0.192
Lower limit	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

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

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.
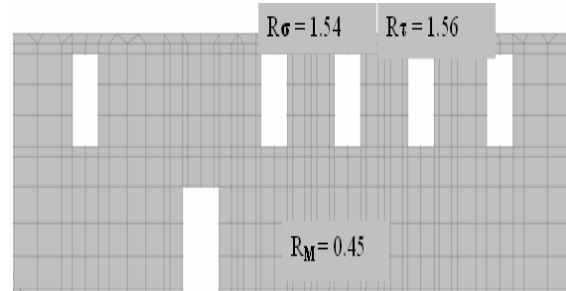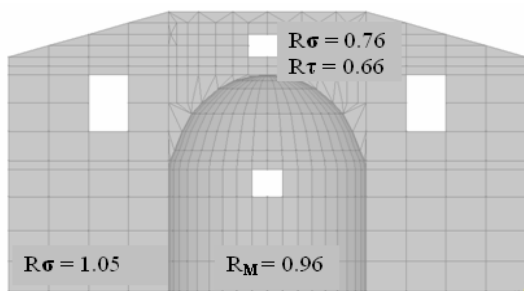
East wall – Ratio values of strength over demands   South wall – Ratio values of strength over demands
Figure 10. "Typical Post Byzantine Basilica"

All the masonry parts of the three 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.
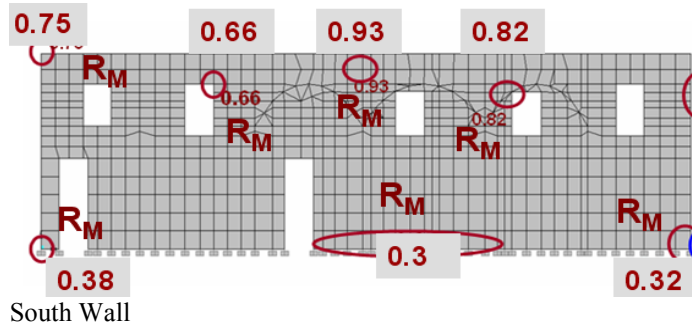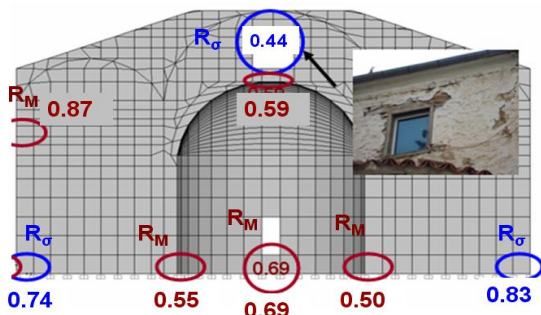
East Wall   South Wall
Figure 11. *"Post Byzantine Basilica church of Taxiarhes"*

Due to space limitations such results are not shown in detail here. Selective results obtained from this evaluation process are shown in figures 10, 11 and 12, for the three studies

churches, respectively. 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.
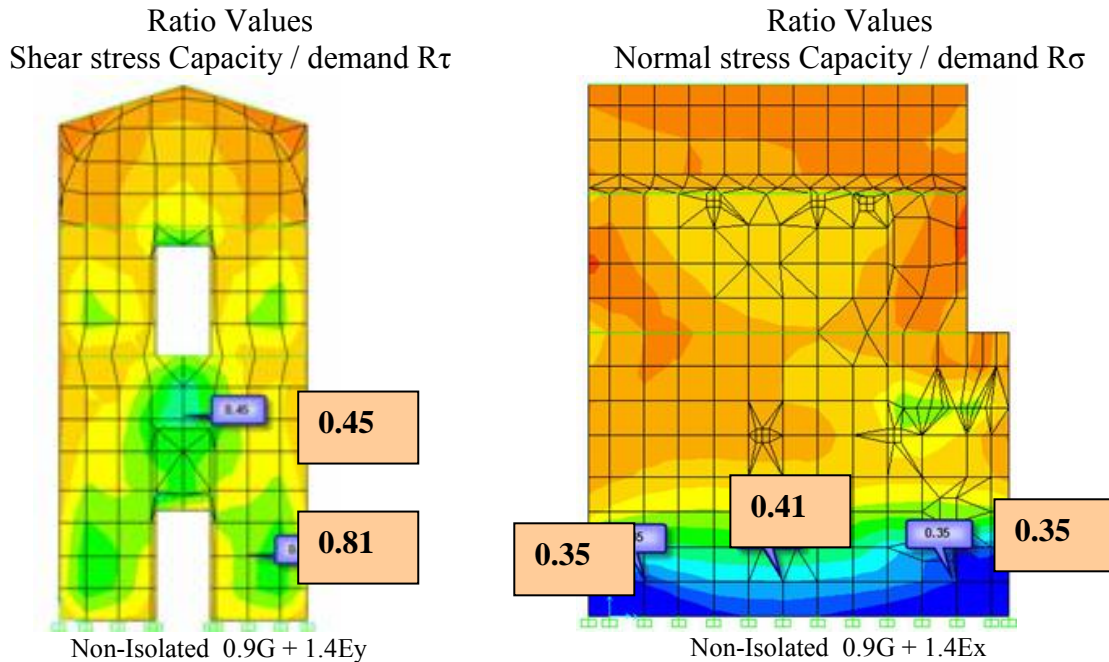
Ratio Values
Shear stress Capacity / demand Rτ

Ratio Values
Normal stress Capacity / demand Rσ

0.45

0.81

0.41

0.35

0.35

Non-Isolated 0.9G + 1.4Ey

Non-Isolated 0.9G + 1.4Ex

Figure 12. *The "church of The Virgin Mary of Tourniki"*

As can be seen from the capacity over demand ratio values presented in figures 10, 11 and 12 there are regions on the masonry walls that these ratio values are well below 1, indicating damage. All these predicted zones of potential failure are credible, as can be deduced from such damage patterns observed in the three studied structural systems after the 1995 strong earthquake event.
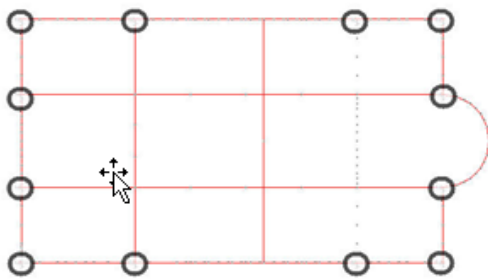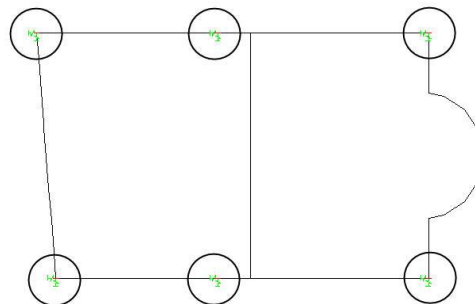

Fig. 13a. *Typical Basilica*

Fig. 13b. *Church at Tourniki*

Figure 13. Arrangement of the base isolation scheme

5. THE PREDICTED PERFORMANCE OF THE EXAMINED STRUCTURAL SYSTEMS BY INTRODUCING A BASE ISOLATION

This investigation is next extended to include a base isolation system which was introduced in the structural system representing the "Typical Post Byzantine Basilica" , as it is schematically indicated in figure 13a as well as the one representing the "church of The Vir-

gin Mary of Tourniki" as it is schematically indicated in figure 13b. In figure14 the design spectrum considered for the non-isolated (fixed) and the isolated structural systems is shown together with the corresponding fundamental eigen-periods of the non-isolated (fixed) and the isolated structural systems. As can be seen in this figure, the dynamic amplification for the non-isolated and the isolated structures is approximately of the same order. This is due to the fact that for seismic isolated structures the response modification factor is given the value of q=1 instead of q=1.5 that was used for un-reinforced masonry structures. The possibility to use more flexible isolation base isolation schemes is currently investigated.
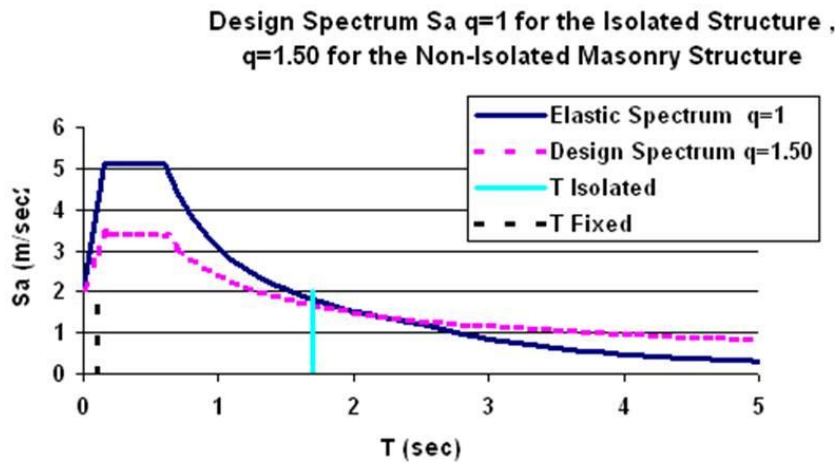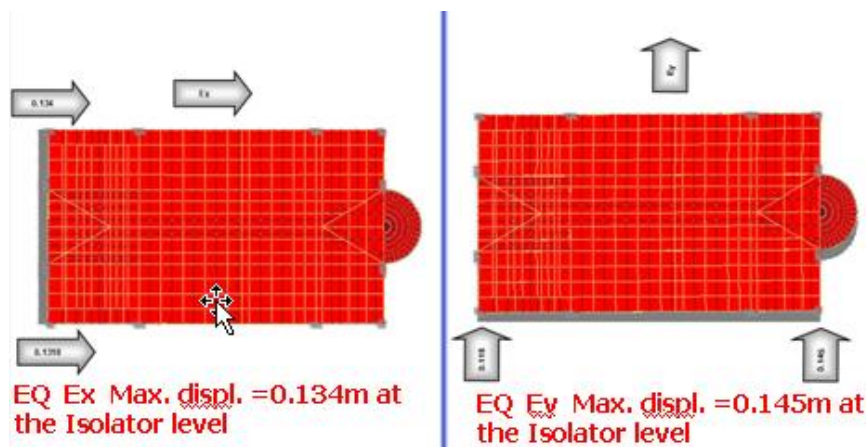

Figure 14. Design spectrum

Figure 15. Horizontal displacement of base isolated *"Typical Post Byzantine Basilica"*.

In figures 15 and 16 the deformation demands at the isolation level are depicted for the "Typical Post Byzantine Basilica" and the "church of The Virgin Mary of Tourniki", respectively. As can be seen they are of the order of 145mm maximum for the first structural system and only 163mm for the second structural system.

What is of the outmost interest, apart from the deformation demands of the base isolation devices, is the effect the introduced base isolation scheme had on the values of the capacity / demand ratio, which were used in the section 4 before to predict numerically the observed earthquake structural damage for these churches (figure 17). This is presented in the following section by comparing these ratio values for the non-isolated structure with the corresponding values of the isolated structure.
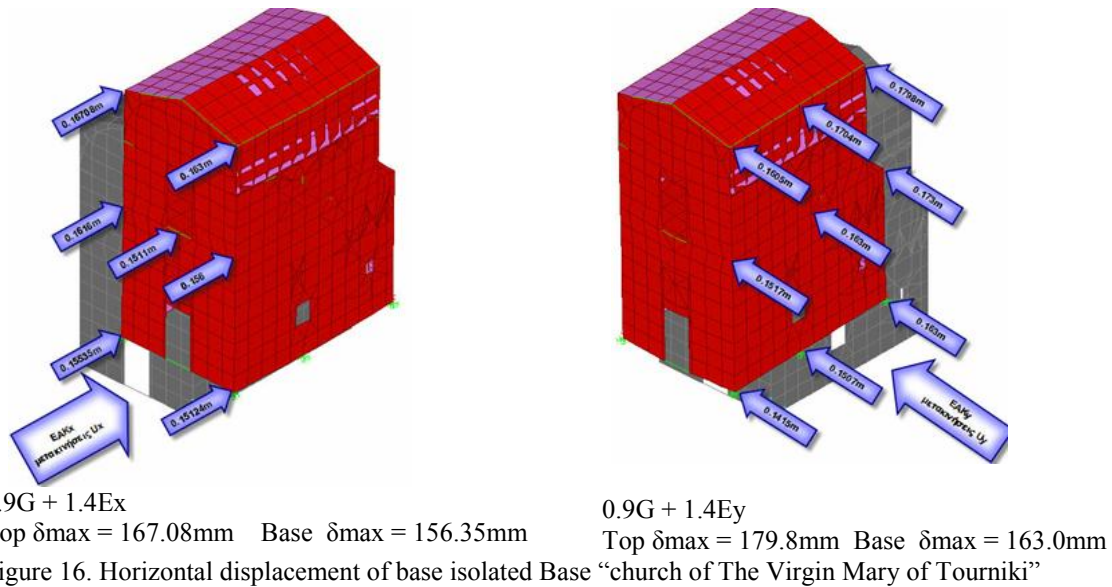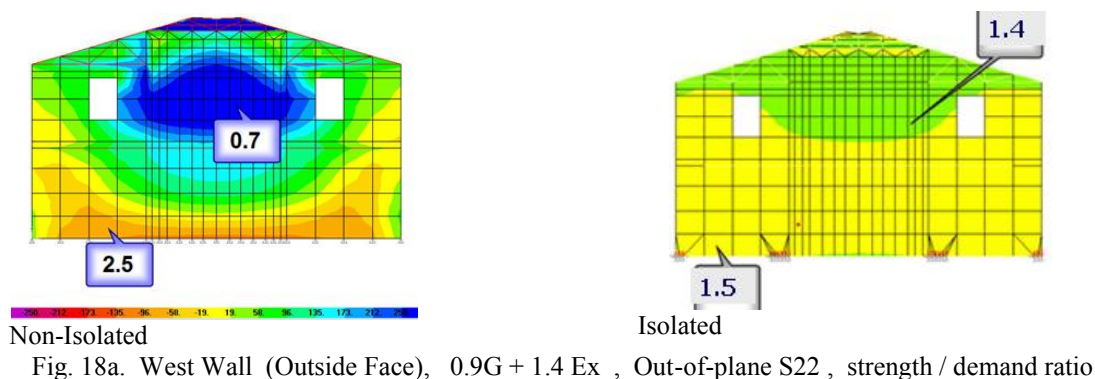
0.9G + 1.4Ex
Top δmax = 167.08mm   Base δmax = 156.35mm

0.9G + 1.4Ey
Top δmax = 179.8mm Base δmax = 163.0mm

Figure 16. Horizontal displacement of base isolated Base "church of The Virgin Mary of Tourniki"

Figure 17. Representative damage patterns of the "Typical Basilica"

5.1. The predicted capacity/demand performance of the isolated structures.

In this section, the results that were obtained for the isolated structures are processed in the same way as presented in section 4 by applying the same failure criteria that were employed for the non-isolated structures. The results of this evaluation must be viewed with the consideration that there is no reduction in the earthquake loads of the isolated structures, due to the moderate flexibility of the used isolation schemes and the employed design spectrum of figure 14 combined with the response modification factor value q=1.0 for the isolated structure, as already mentioned. Such a reduction of the earthquake loads will result from the use of a more flexible base isolation scheme than the one employed here.
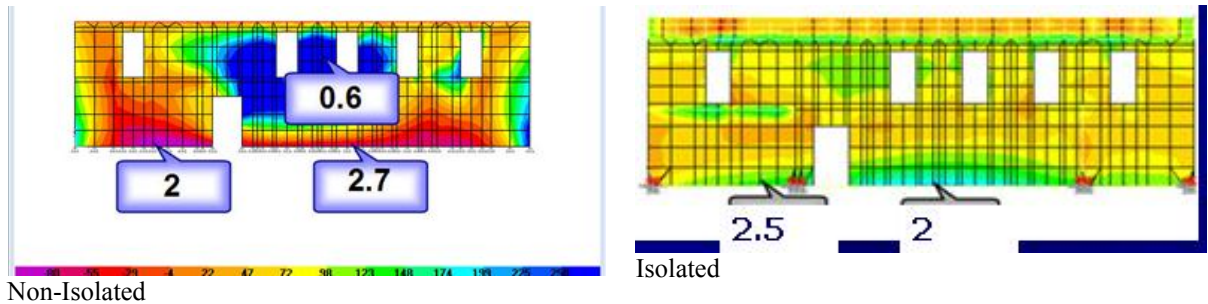

Non-Isolated

Isolated

Fig. 18a. West Wall (Outside Face), 0.9G + 1.4 Ex , Out-of-plane S22 , strength / demand ratio

13

Non-Isolated

Isolated

Fig. 18b. South Wall (Outside Face) 0.9G + 1.4 Ey , Out-of-plane S22 strength / demand ratio
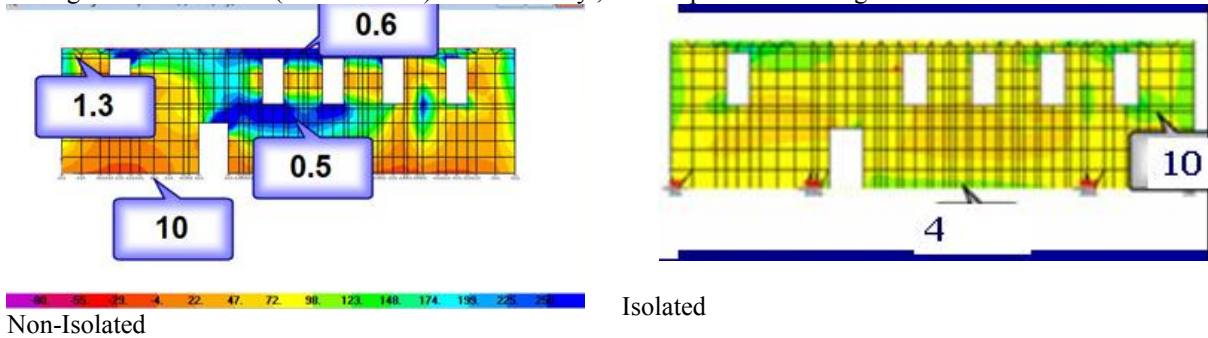

Non-Isolated

Isolated

Fig. 18c. South Wall (Inside Face), 0.9G + 1.4 Ey , Out-of-plane S11 strength / demand ratio

Figure 18. Strength over demand ratio values

5.1.1. The "Typical Post Byzantine Basilica"

In figure 18 the strength over demand ratio values for the "Typical Post Byzantine Basilica" structural system are shown for both the non-isolated as well as the isolated case. These selected results for the non-isolated structure correspond to ratio values well smaller than 1, which as explained in section 4 represents structural damage. As can be seen, for the case of the isolated structure the corresponding strength over demand ratio values at the same locations are well above 1, which indicates that with the introduction of the base isolation scheme structural damage from earthquake loads should be minimized.
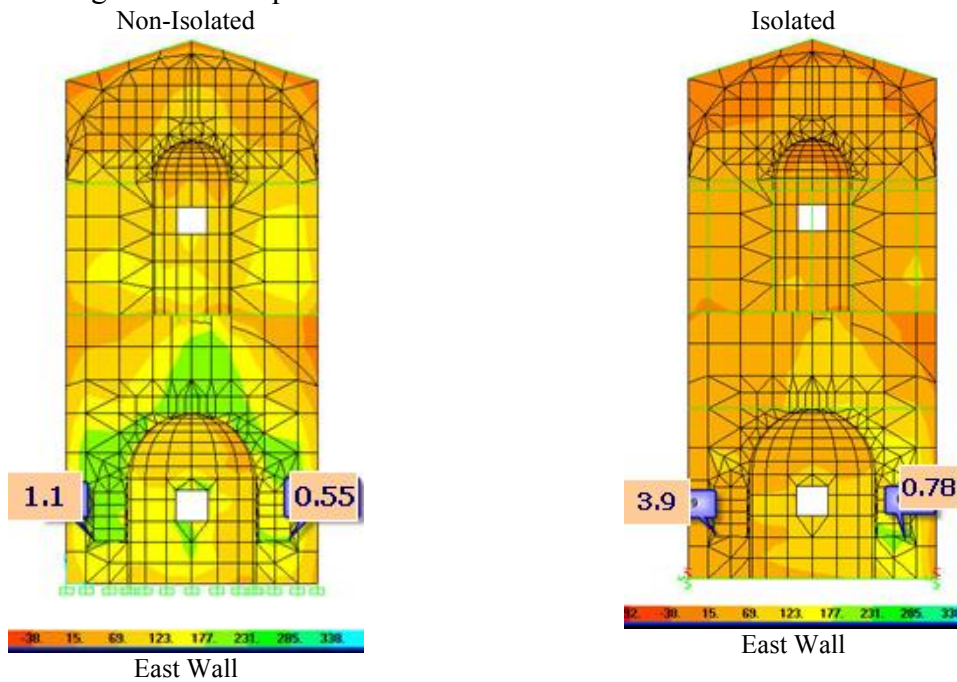
Non-Isolated   Isolated

East Wall   East Wall

Figure 19. Shear Strength / Demand ratio Rτ, Loading G+Ey

5.1.2 "The Church of The Virgin Mary of Tourniki"

In figures 19 and 20 the strength over demand ratio values for the "Virgin Mary of Tourniki" are shown for both the non-isolated and the isolated case. The selected results for the non-isolated structure correspond to ratio values well smaller than 1, which, as explained in section 4, represents structural damage. As was the case before, the corresponding strength over demand ratio values at the same locations are well above 1 in almost all cases for the isolated structure. Thus, as expected, structural damage from earthquake loads should be minimized with a base isolation scheme. This improvement in the expected earthquake performance of the studied structural systems will become even better if a more flexible isolation scheme than the one employed here is used.

Obviously, the increased flexibility of the base isolation scheme will result in larger deformation demands of the base isolation devices than the ones predicted here (figures 15 and 16); the selected devices must then pass such relevant checks.
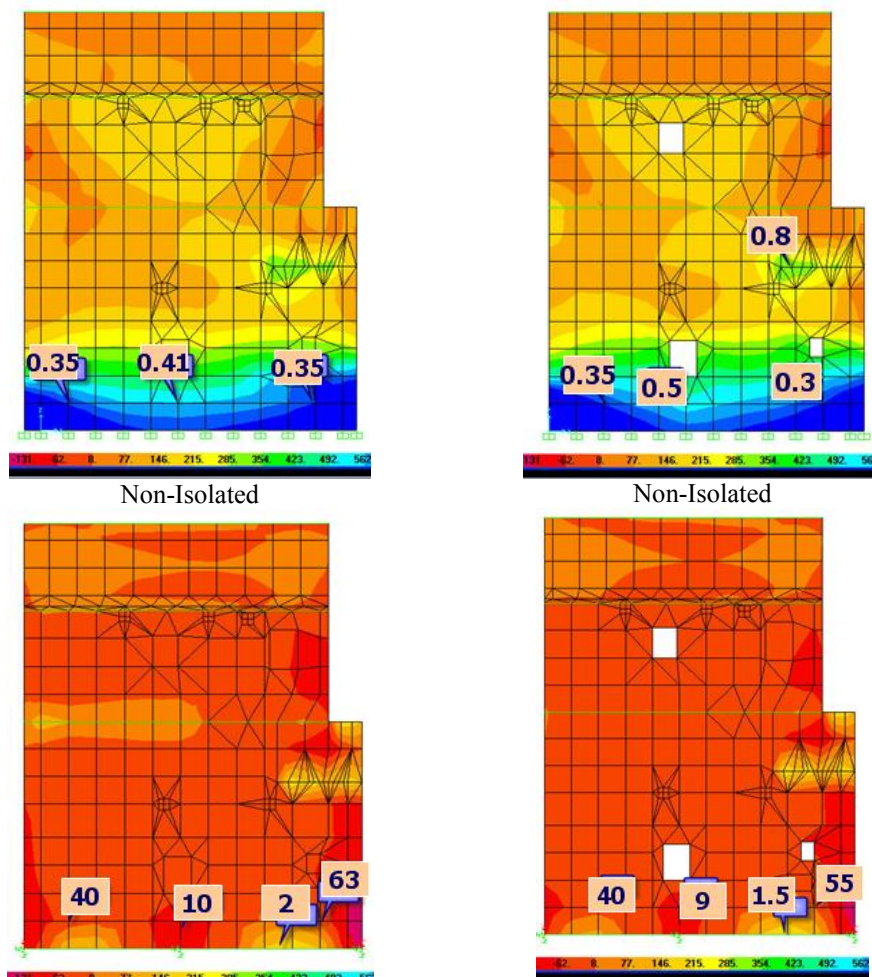

Figure 20. Normal stress Strength / Demand ratio $R\sigma$, Loading G+Ey

At this point it is important to stress the following difficulties in applying a base isolation scheme on an existing structural system especially in the case that the structural system is unreinforced masonry, as is the case of both structural systems that were investigated here. In order to place the base isolation devices under the existing structure a system of additional foundation must be constructed under the masonry walls that will enable such an operation. Such systems have been tried in the past; however, apart from the difficulty in construction

this additional foundation one should also consider the extra cost. Another difficulty is the fact that many times below the existing churches lie older structures; thus the excavation for constructing the additional foundation will unveil such old remains. This is not the case for the structures that were investigated here. On the contrary, "The Church of The Virgin Mary of Tourniki" has to be moved in a different location due to the flooding of the current location, where it was constructed almost eight hundred (800) years ago, from the construction of a dam that already starts filling the valley. In this case, the operation of construction an additional foundation for lifting and sliding this church cannot be avoided. The authors of this paper argue in favor of the placement of a base isolation scheme under this structure in its new location is a feasible solution for this particular case with no significant extra cost. Obviously, thus base isolation scheme should be considered in addition to all other traditional repair and strengthening measures for this church.

6. CONCLUSIONS

1. The numerical results presented here are part of 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 and the bases of the peripheral walls. Further extensive verification is needed for the assumed strength values utilized by the failure criteria. A number of tests are currently under way.

2. The numerical introduction of a base-isolation system for the third structural formation, was quite effective in lowering the demands and thus resulting for most structural elements in Capacity/Demand ratio values larger than one (Capacity/Demand > 1), which is assumed to indicate acceptable structural performance. However, the introduction of base isolation under an existing old stone-masonry structure is also faced with considerable difficulties, despite the advantages that are shown by the results of numerical investigation.

3. In the particular case of "The Church of The Virgin Mary of Tourniki" these difficulties must be faced because this old church has to be moved in a different location due to flooding. Thus, the placement of a base isolation scheme under this structure in its new location is a feasible solution for this particular case with no significant extra cost. Obviously, thus base isolation scheme should be considered in addition to all other traditional repair and strengthening measures for this church.

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