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DYNAMIC BEHAVIOUR AND EARTHQUAKE PERFORMANCE OF GREEK BASILICA CHURCHES WITH FOUNDATION DEFORMABILITY

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Abstract. *This paper attempts to address the earthquake performance of Basilica churches in parts of Greece during the last 30 years. Towards this objective use is made of in-situ observations together with relatively simple numerical tools. The numerical study includes soil-foundation deformability. The earthquake performance of three specific stone masonry Basilica churches is investigated. The second and third of these churches are simply covered by a wooden roof whereas the first has also a stone masonry vaulted superstructure under its wooden roof. All three churches developed significant structural damage typical of churches of this Basilica typology as was observed in Greece during the last 30 years. It has been observed that such churches develop structural damage to the masonry walls and vaults, in certain cases quite serious, that is believed to arise from the amplitude of the gravitational and seismic actions combined with the deformability of the foundation which is included in the current investigation. The numerical models simulating the main structural system of these churches are supplemented at the foundation level with flexible springs in an effort to capture the influence of such soil-foundation deformability. Simple numerical analysis results together with assumed strength values are utilized to predict the behaviour of the various masonry parts that belong to the longitudinal and transverse peripheral wall in in-plane shear and normal stress as well as out-of-plane flexure. The recorded ground acceleration during the Kozani 1995 earthquake for the first two churches and the 1978 Thessaloniki earthquake for the third church, as it was recorded at a relatively close distance, is used in this numerical investigation. Towards this end, the dynamic characteristics in terms of eigen-modes and eigen-frequencies of the examined structural systems are also utilized.*

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 [1]. 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 ([1], [2], [3], [4], [5]). The earthquake behaviour of churches belonging to the so-called Post-Byzantine period (16th to 19th century A.D.) has been studied numerically in some detail ([2], [3], [4], [5], [6]) In all these cases the foundation was considered to be non-deformable. However, this is a gross approximation as in most cases these churches are founded on deformable soil. In some instances, the deformability of the soil caused considerable damage as is the case of the church of The Assumption of the Virgin Mary at Dilofo-Voio-Kozani, as will be presented in section 2 ([7], [8]). In some other cases the main cause of recent damage is the earthquake activity that is accentuated by the deformability of the soil ([9], [10], [11], [12]). This is presented in the section 3, where the damage of the church of Agia Triada at Vithos-Kozani and of Profitis Elias at Siatista-Kozani is also included, as well as in section 4 where the earthquake performance of the old Byzantine church of Achiropoiitos in Thesalloniki is examined. It must be noted that all these churches were built in a number of phases on old existing sacred sites. During the Turkish occupation the most prominent Christian churches were transform to mosques or prohibitions were imposed forbidding the construction of new churches but allowing the maintenance of existing ones, after special permit [13]. In what follows an overview of three specific cases of Greek Byzantine churches where the foundation deformability influenced the dynamic and earthquake behaviour is presented and discussed.

Figure 1. Damage to the South-East corner

Figure 2. Partial collapse of the central dome

2 THE CHURCH OF THE ASSUMPTION OF THE VIRGIN MARY AT DILOFO

The West part of this church is founded on hard soil (weathered flysch layers) whereas the East part is founded on relatively soft soil that was deposited on top of the layers to compensate for the natural slope at this location ([7], [8]). The unequal settlement of the foundation of the stone masonry walls as well as of the internal columns that supported the vaulting system, which covered this church, caused considerable damage. A very wide crack initiates at the top of the East peripheral wall near the apse and propagates towards the bottom of this wall being inclined to the South-East corner (figure 1). Furthermore, a similarly wide crack propagates through the North peripheral wall from top to bottom. It must be noted that the thickness of these masonry walls varies from 750mm to 800mm. From this extensive peripheral wall damage, the vaulting system that is supported by these peripheral walls also suffered extensive cracking that eventually caused the partial collapse of a part of the central dome, as shown in figure 2.

Initially, the 3-D numerical simulation of this church, including the peripheral walls, the vaulting system of the superstructure and the wooden roof, assumed that the foundation was non-deformable. The results of the modal analysis for the church supported on the non-deformable foundation are shown in a summary form in figures 3a and 3b. In these figures the most significant eigen-modes are depicted together with the corresponding values of the eigen-periods as well as the modal mass participation ratios either in the x-x direction (Ux) or in the y-y direction (Uy). As can be seen the longest eigen-period is, as expected, for the fundamental x-x direction translational mode, with a value equal to 0.228seconds that mobilizes 72.48% of the total mass. The next longest eigen-period is for the fundamental y-y direction translational mode, with a value equal to 0.132seconds that mobilizes 59.37% of the total mass.

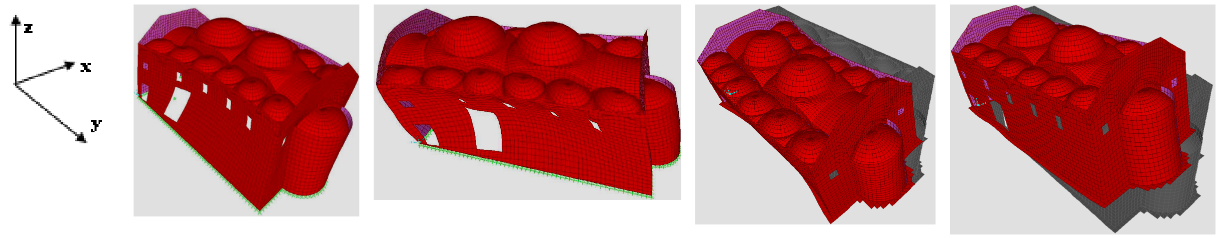


Figure 3. Eigen-modes

a) Mode 1, T1=0.228sec Ux=72.48%

b) Mode 2, T2=0.132sec Uy=59.37%

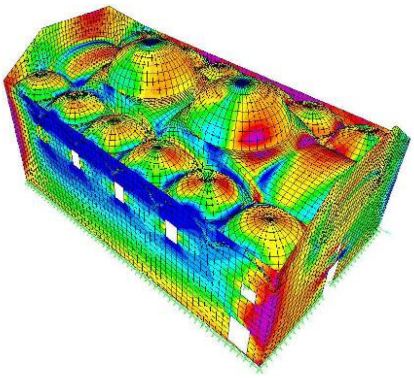
c) Mode 1, T1=0.391sec Ux=83.48%

d) Mode 2, T2=0.276sec Uy=78.37%

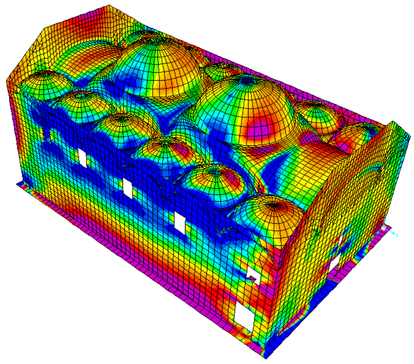
Next, layers of deformable soil were introduced beneath the foundation of all the peripheral walls and internal colonnades. The stiffness of these soil layers was varied with the depth as well as in plan in order to simulate the relatively hard soil at the West part of the church and the relatively soft soil at the North-East part of the church. The results of the modal analysis for the church (low-stiffness soil layers) supported on the deformable foundation are shown in a summary form in figures 3c and 3d. In these figures the most significant eigen-modes are depicted together with the corresponding values of the eigen-periods as well as the modal mass participation ratios either in the x-x direction (Ux) or in the y-y direction (Uy). As can be seen the longest eigen-period is, as expected, for the fundamental x-x direction translational mode, with a value equal to 0.391 seconds that mobilizes 83.48% of the total mass. The next longest eigen-period is for the fundamental y-y direction translational mode, with a value equal to 0.276seconds that mobilizes 78.37% of the total mass. As can be seen, apart from the lengthening of the fundamental eigen-period values, the deformable foundation resulted in a larger portion of the mass being mobilized for the first two translational eigen-modes in the x-x and y-y direction than for the case of non-deformable foundation.

The 3-D numerical representation of the church, including the peripheral walls, the vaulting system of the superstructure and the wooden roof were subjected to load combinations of either 0.9G±1.4Ex or 0.9G±1.4Ey, where G is the gravitational loads and Ex, Ey the earthquake forces along the x-x axis (longitudinal East-West direction) or the y-y axis (transverse North-South direction, respectively.

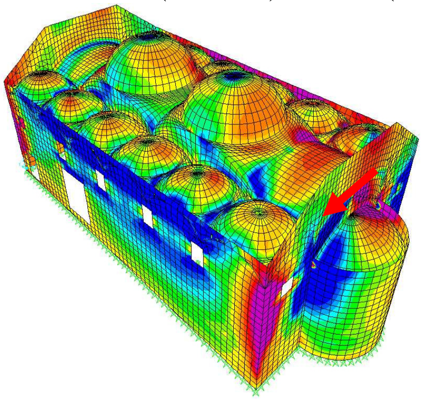
This was done for both the non-deformable and the deformable foundation. Selected results of the obtained maximum stress response are shown in figures 4a to 4d. More results are given by Manos et al. ([7], [8]). The following summarize the most significant observations of this numerical study:



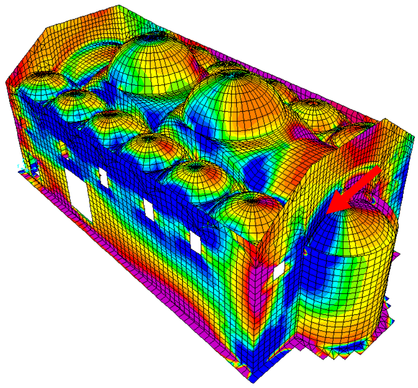
4a) **Non-deformable foundation**, $\boldsymbol{\sigma_{11}}$ top face, max $\boldsymbol{\sigma}_{11}$ = 0.38 MPa (NW view) 0.9G+1.4(+Ex)



4b) **Deformable Foundation,** $\boldsymbol{\sigma_{11}}$ top face, max σ_{11} = 0.53 MPa (NW view) 0.9G+1.4(+Ex)



4c) **Non-deformable foundation,** σ_{11} bottom face, max $\boldsymbol{\sigma}_{11}$ = 0.46 MPa (SE view)) 0.9G+1.4(-Ex)



4d) **Deformable Foundation,** σ_{11} bottom face, max $\boldsymbol{\sigma}_{11}$ = 0.64 MPa (SE view) 0.9G+1.4(-Ex)

- The foundation deformability due to layers of soft-soil deposits at the North-Eastern part of the foundation, as was approximated numerically, increased both the fundamental translational modes eigen-periods and the corresponding modal mass participation factors.

- This foundation deformability, as was approximated numerically, also increased the maximum predicted vertical and horizontal displacement values. This increase was threefold for the horizontal displacements and ten-fold for the vertical displacements.

- This foundation deformability, as was approximated numerically, also increased the maximum predicted axial stress values parallel to the bed-joint. This increase was of the order of 20% up to 40%.

- Due to this increase of the predicted maximum axial stress values parallel to the bed-joint, arising from the foundation deformability, the assumed corresponding tensile strength is exceeded in all the locations where actual damage was observed.

- Despite the simplicity of the adopted numerical modeling as well as of the assumed failure criteria in the direction parallel to the bed-joint, this simple failure criterion and the predicted relevant maximum stress values can explain the observed damaged in all the locations.

- Further research is necessary in order to establish stone masonry strength values and failure criteria that are based on tests representative of the stone masonry that is investigated.

- The capabilities of more refined numerical modeling should also be investigated.

3 THE CHURCH OF PROFITIS ILIAS AT SIATISTA – KOZANI.

This is also a Post-Byzantine three nave Basilica built in 1701 A.D. on the top of a hill in the town of Siatista of the prefecture of Kozani. It also has a wooden roof without the vaulting system of the Basilica church described in section 2. The horizontal dimensions of this church are 23.25m in length and 16.60m in width. The top of the roof lies at 7.1m from the floor level of the interior of this church. The naves are formed by 4-column colonnades built with stone masonry. The lower part of the key of each arch of these colonnades lies at 5.25m from the floor level of the interior of this church. All the exterior walls are made of stone masonry. Apart from the main church a narthex was built at the North side at a later stage; this is of a relatively lower height than the main church. The West part of the church is allocated to the women's quarters that is separated from the rest of the interior by a mid-transverse wall. The South longitudinal wall is supported by a system of wooden beams. These were installed after the Kozani-1995 earthquake sequence. Additional wooden supports are also placed at the mid-transverse wall as well as at the mid-span of a longitudinal beam that spans the women's quarters from East to West. According to past records, the structural system of this church showed signs of distress from soil-settlement sometime before the occurrence of this particular earthquake sequence. The records do not give information of any countermeasures being taken in the past up to the point of the earthquake occurrence. The main structural damage, as recorded after this earthquake sequence, is described below. Inclination of the South longitudinal wall outwards that is accompanied by extensive cracking at its joints with the East and West exterior masonry walls as well as with the mid-transverse wall. Cracking is evident at the arches of the colonnades. Recently, almost 20 years after this earthquake damage, a number of counter measures were introduced towards repairing the masonry walls as well as introducing a relatively shallow reinforced concrete beam at the foundation level from the exterior of the West, South and East peripheral walls.

The numerical investigation of this church included the following ([9], [10], 11]):

a) Simulation of the behaviour assuming non-deformable supports at the foundation level.
b) Simulation of the behaviour assuming deformable supports at the foundation level, introducing at this level elastic springs with properties reflecting the actual soil deformability that was found from in-situ sampling.

The soil consists of clay in its upper layers. For quantifying the stiffness of these soil layers use was made of the data from three bore-holes drilled in the vicinity of the church relevant to the constitution of the soil deposits at a depth up to 15m. All the numerical simulations assumed elastic behaviour with relatively low-values of the modulus of elasticity for the stone masonry equal to E=1660MPa. In the case of Basilica churches damaged in Kefalonia Greece by the 2014 earthquake sequence in-situ measurements of the dynamic response of a bell-tower were utilized in order to define indirectly the soil-foundation deformability ([12], 14]). The system of the wooden roof was modeled as well as all the wooden elements that connect the longitudinal and transverse walls with the interior colonnades and the mid-transverse wall. A small number of mortar samples were taken from the church. The natural stone used in the masonry elements is a type of slate with a compressive strength equal to 24MPa, as given in the literature survey.

Next, the procedure outlined by Manos et al. ([9], [10], [11], [12]) was followed for evaluating the performance of this church. This is based:

a) On the numerical predictions of the state of stress of the masonry walls as obtained from the dynamic analysis of this elastic numerical simulation being subjected to a combination of dead load and seismic forces and
b) On the values of the strength over demand ratios at the most critical locations of the masonry walls and vaults. In doing so, strength values were assumed for in-plane shear and flexure as well as out-of-plane flexure for relatively weak unreinforced stone masonry ([9], [10], [11], [12]).

The ratio of the in-plane shear or tensile strength value over the corresponding demand is signified by $\boldsymbol{R\sigma}$ or $R\tau$ whereas $\boldsymbol{R_M}$ denotes the ratio of the out-of-plane tensile strength value over the corresponding demand. Ratio values smaller than one ($\boldsymbol{R\sigma}$, $\boldsymbol{R\tau}$,$\boldsymbol{R_M}$ <1) predict a corresponding limit state condition ([9], [10, [11], [12], [15]).

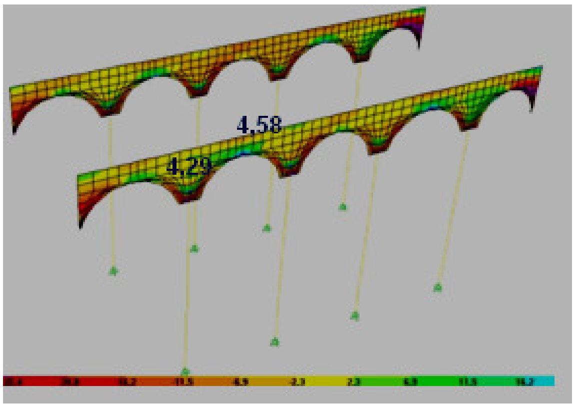


Figure 5a. G+0.3Q. Non-Deformable soil $\boldsymbol{R_\sigma}$ Ratio of in-plane strength / demand **F11** parallel to bed joint

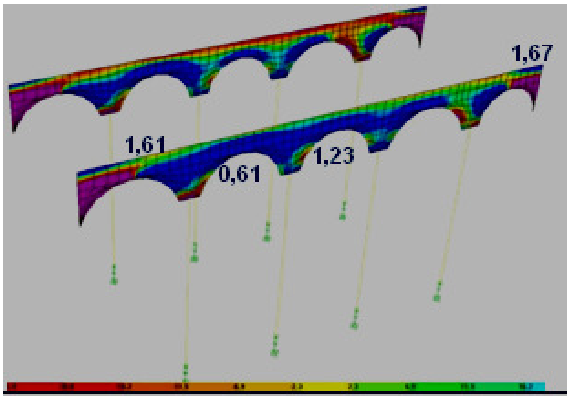


Figure 5b. G+0.3Q. Deformable soil $\boldsymbol{R_\sigma}$ Ratio of in-plane strength / demand **F11** parallel to bed joint

Figures 5a and 5b depict $\boldsymbol{R\sigma}$ ratio values for the masonry arches above the internal colonnades. These $\boldsymbol{R\sigma}$ ratio values of in-plane strength / demand were found by applying the limit tensile strength scenario parallel to bed-joint (F11). The demands in these figures were obtained for the load combination G+0.3Q. As can be seen from the above figures, the $\boldsymbol{R\sigma}$ ratio values are smaller for the deformable than the non-deformable foundation, which demonstrates the detrimental effect of the foundation deformability for this church even only for the gravitational forces. For the load combination 1.3G+1.5Q, that dictates the design for the gravitational forces under current code provisions, the above ratio values will become even smaller, which signifies that the colonnades cannot withstand the maximum gravitational forces. This is verified by the observed damage which is quite evident for these structural elements in this church having been supported for quite sometime after the earthquake event of 1995 by temporary wooden shoring internally as well as externally.

From the evaluation of the demands obtained from this numerical simulation as compared with the limit strength values adopted for the stone masonry of this church it can be concluded that the deformability of the foundation results in strength / demand ratio values smaller than the corresponding ratio values for the non-deformable foundation. This signifies critical regions of the masonry structural elements that cannot withstand the forces, as prescribed by the current code provisions. Moreover, such an evaluation can also explain, up to a point, the development of the existing current state of structural damage. Gaining such confidence in the employed methodology the designer gains also the advantage in evaluating with the same methodology the effectiveness of a potential retrofitting scheme.

In order for such a retrofitting scheme to be effective the resulting strength / demand ratios should attain values larger than the corresponding values without this retrofitting scheme and, if possible, larger than 1 (10], [11], [15]). Such an evaluation has been performed for the most vulnerable structural elements of this church; that is the internal colonnades, the South peripheral wall and the East peripheral wall with the apses. The existing or potential damage is predicted by the numerical analysis results when the strength / demand ratio values are well below 1. This is shown in figure 6 for the internal colonnade. In almost all structural elements the out-of-plane tensile behaviour results in strength / demand ratios with values well below one (R_M <1). The internal colonnade has visible signs of out-of-plane displacements and extensive cracking that are in agreement with the distress predicted by the followed methodology.

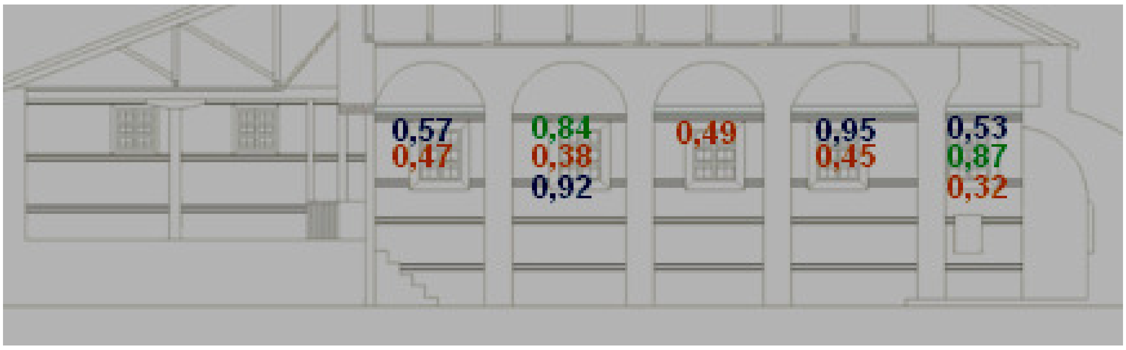


Figure 6. Internal colonnade. Regions with small values of the strength / demand $\mathbf{R_M}$ out-of-plane tensile behaviour (orange color), $\mathbf{R_\tau}$ in-plane shear behaviour (green color), $\mathbf{R_\sigma}$ in-plane tensile behaviour (blue color)

4 THE CHURCH OF ACHIROPOIITOS, AT THESSALONIKI

This church is a three nave Basilica located at the center of Thessaloniki, Greece. It is one of the oldest Greek Christian churches dating from the middle of the 5th century A.D. The interior is shaped by the peripheral (longitudinal and transverse) masonry walls and two series of colonnades formed by twelve columns each. The masonry is built with the old Christian construction technique whereby horizontal zones of brick masonry are succeeded by horizontal zones of stone masonry along the height of the peripheral walls.

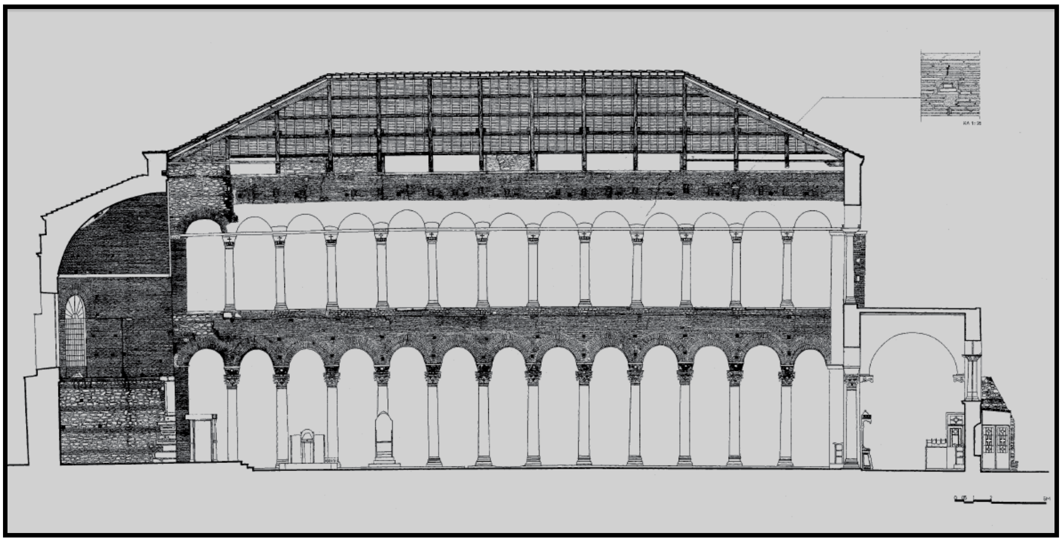
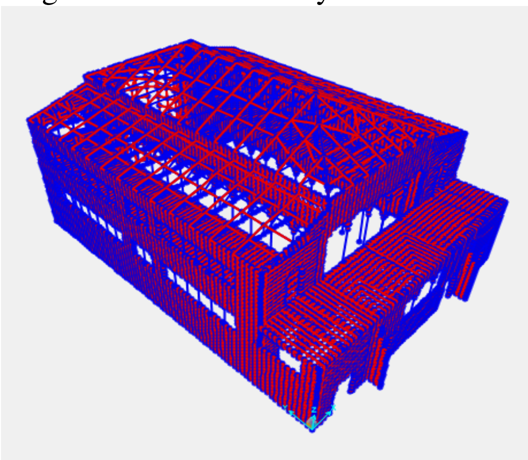
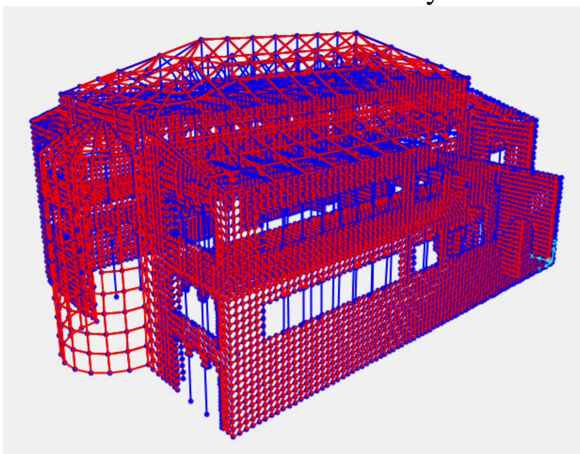


Figure 7. Longitudinal (East-West) cross section of the church of Achiropoiitos

The overall external dimensions are 51.9m in the longitudinal East-West direction and 30.80m in the transverse North-South direction and a height of 14.0m for the peripheral masonry walls. The clear width of the central nave is 14.20m whereas the North and South nave width is 6.20m and 6.30m, respectively. At the East end the central nave extends further than the North and South naves by an extension that is partly formed with an apse. The whole structure is covered by a wooden roof that rises from the ground level 22.0m at the central longitudinal axis. Initially the church was larger than the one which survives today.



a) View from the North-West



b) View from the North-East

Figure 8. Numerical simulation of the church of Achiropoiitos

Figure 7 depicts a longitudinal East-West cross section whereas figures 8a and 8b depict the numerical simulation of this church. As was done for the churches of the Assumption of Virgin Mary and for Profitis Ilias the foundation deformability was also examined for this church. This was done by assuming a) rigid soil-foundation conditions (pinned), b) deformable rather stiff soil-foundation and c) deformable rather flexible foundation condition. The soil-foundation deformability was introduced in the numerical simulation with a series of link elements at the soil-foundation interface.

Cumulative Modal participation mass ratios						
Eigen-Modes	Rigid Foundation (Pinned)		Stiff - Deformable Foundation		Flexible Deformable Foundation	
	$SumU_X$	$SumU_Y$	$SumU_X$	$SumU_Y$	$SumU_X$	$SumU_Y$
1st main translational transverse x-x (North-South, Transverse)	**30.5%**	0 %	**51. %1**	0 %	**68.8%**	0.1%
2nd	30.7%	0.6%	51.2%	1.7%	68.9%	16.7%
3rd	67.9%	0.7%	67.9%	14.1%	68.9%	70.6%
4th main translational longitudinal y-y (West – East, Longitudinal)	68.0%	**70.2%**	72.1%	**72.6%**	73.2%	**71.5%**
5th	70.2%	70.5%	73.8%	72.6%	73.3%	71.8%

Table 1. Cumulative Modal participation mass ratios of the church of Achiropoiitos

Table 1 lists the values of the cumulative mass participation ratios of the most significant eigen-modes (1st to 5th), whereas in table 2 the values of the corresponding eigen-periods are listed. As can be seen from table 1 the most significant eigen-modes are the 1st x-x translational eigen-mode in the transverse (North-South) direction and the 4th y-y translational longitudinal (East-West) direction. As can be seen in table 1, the increase in the flexibility of the soil-foundation results in a 1st x-x translational eigen-mode that mobilizes a larger modal mass than for the case of the rigid foundation. Moreover, this increase in the flexibility of the foundation results, as expected, in the lengthening of the corresponding eigen-period values, as was also discussed for the previously examined churches of the Assumption of the Virgin Mary and for Profitis Ilias.

Eigen-period values (sec)			
Eigen-Modes	Rigid Foundation (Pinned)	Stiff - Deformable Foundation	Flexible Deformable Foundation
1st main translational transverse x-x (North-South, Transverse)	**0.5877**	**0.6111**	**0.6704**
2nd	0.5724	0.5740	0.5781
3rd	0.4974	0.5241	0.5617
4th main translational longitudinal y-y (West – East, Longitudinal)	**0.4734**	**0.5189**	**0.5439**
5th	0.3663	0.3688	0.3801

Table 2. Eigen-period values (sec) of the church of Achiropoiitos

As was done before, the loading conditions examined were a) G+E_x, b) G-E_x, c) G+E_y, and d) G-E_y, where G is the gravitational loads and Ex, Ey the earthquake forces ([1], [16], [17], [18]) along the x-x axis (transverse North-South direction) or the y-y axis (longitudinal East-West direction), respectively. The amplitude of these earthquake forces was obtained by utilizing the recorded ground acceleration during 1978 Thessaloniki earthquake at a close distance from the site of this church. In figures 9a to 9d the values of the ratio of shear strength value over the corresponding shear demand signified $R\tau$ is plotted for the case of rigid foundation and for the case of flexible foundation. In figures 9a and 9b this is done for the North peripheral wall and for the load combination G+Ey for the case of rigid and flexible foundation, respectively. In figures 9c and 9d this is done for the West peripheral wall and for the load combination G+Ex for the case of rigid and flexible foundation, respectively.

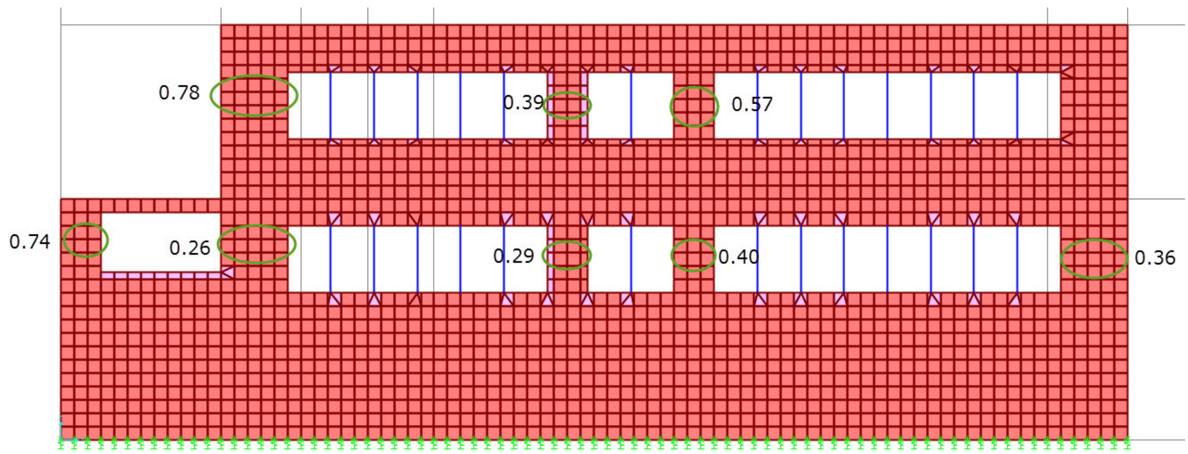


Figure 9a. Values of the ratio **Rτ** towards assessing the performance of the North peripheral wall against in-plane shear for load condition G+Ey . Case of rigid foundation

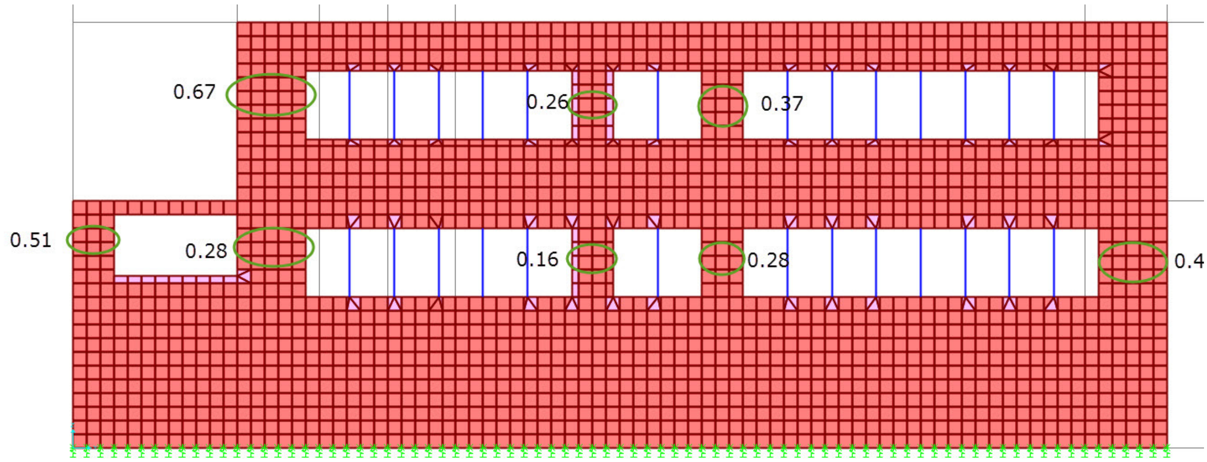


Figure 9b. Values of the ratio **Rτ** towards assessing the performance of the North peripheral wall against in-plane shear for load condition G+Ey . Case of flexible foundation

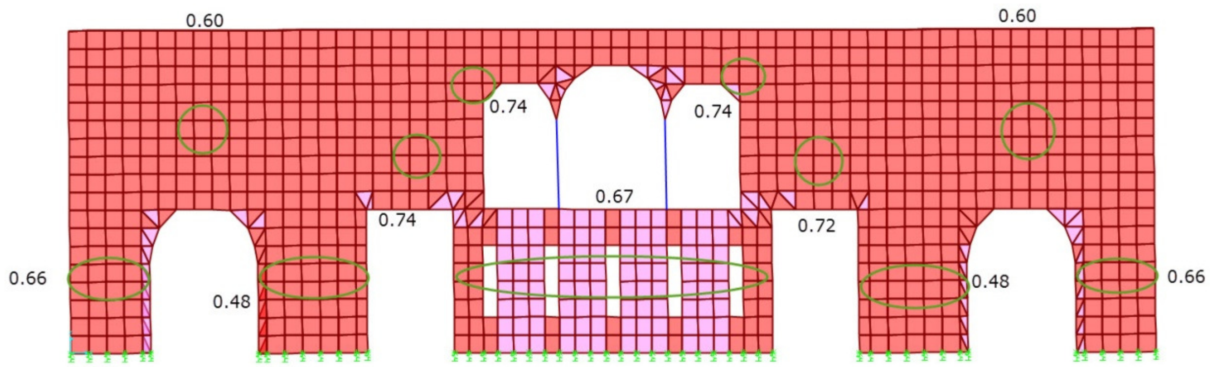


Figure 9c. Values of the ratio **Rτ** towards assessing the performance of the West peripheral wall against in-plane shear for load condition G+Ex . Case of rigid foundation

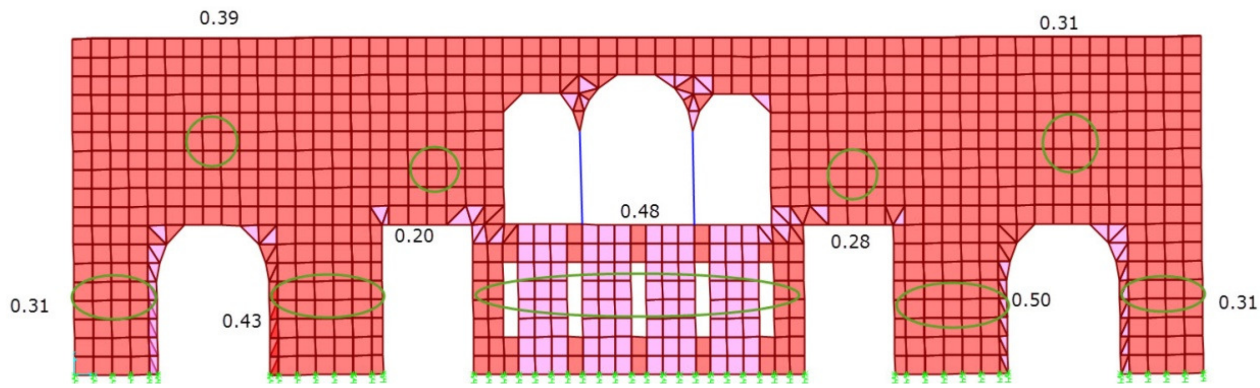


Figure 9d. Values of the ratio **Rτ** towards assessing the performance of the West peripheral wall against in-plane shear for load condition G+Ex . Case of flexible foundation

As can be seen in these figures 9a to 9d there are many locations in both the North and the West peripheral walls where these shear strength over shear demand ratio values are smaller than 1, which signifies the appearance of shear mode of failure. This observation is in line with the damage that the masonry walls of this church sustained during the 1978 Thessaloniki earthquake sequence. Moreover, as can be seen by comparing the shear strength over shear demand $\boldsymbol{R\tau}$ ratio values for the same location of these two peripheral walls (North wall, figures 9a and 9b or West wall, figures 9c and 9d) between the two cases of rigid and flexible foundation it can be seen that the flexibility of the foundation results in $\boldsymbol{R\tau}$ values noticeably smaller than the case of flexible foundation. This observation supports the partial conclusion that the soil-foundation flexibility was an additional factor contributing to the observed earthquake damage.

5 CONCLUSIONS

- 1. The dynamic behaviour and earthquake performance of churches is a very wide subject. This paper attempts to address the earthquake performance of Basilica churches in parts of Greece during the last 30 years. Towards this objective use is made of in-situ observations together with relatively simple numerical tools. The numerical study includes soil-foundation deformability. The earthquake performance of three specific stone masonry Basilica churches is investigated. The second and third of these churches are simply covered by a wooden roof whereas the first has also a stone masonry vaulted superstructure under its wooden roof. All three churches developed significant structural damage typical of churches of this Basilica typology as was observed in Greece during the last 30 years.
- 2. The eigen-periods, eigen-modes and the deformation patterns to horizontal earthquake actions of the examined churches were predicted numerically. These numerical simulations resulted in large out-of-plane displacements at the top of the longitudinal peripheral walls for all examined churches. The presence of stone masonry vaulting systems increases the stiffening effect at the top of the peripheral wall level; however, such vaulting systems also add considerable masses at a high level that generate large seismic forces.
- 3. The predicted regions most vulnerable to damage are near the door and window openings for the in-plane behaviour; this agrees reasonably well qualitatively with observed damage, although the numerical simulation is based on elastic behaviour. For a superstructure consisting of vaults and domes, large seismic forces are generated leading to stress concentration at the bases of the domes and vaults and at the keys of the arches. Further investigation is needed to establish appropriate limit-state criteria for these stone masonry elements.
- 4. The foundation deformability was investigated introducing linear deformable springs at the foundation level. The results of such a numerical approximation combined with the followed methodology of limit state criteria of strength / demands ratio values ($\boldsymbol{R\tau}$, $\boldsymbol{R\sigma}$, $\boldsymbol{R_M}$) demonstrate in all the examined cases that the foundation deformability, when combined with the gravitational forces and seismic actions, leads to $\boldsymbol{R\tau}$, $\boldsymbol{R\sigma}$, ***and*** $\boldsymbol{R_M}$ ratio values that are considerably smaller than for the case of non-deformable foundation thus verifying the detrimental effect of the foundation deformability in all the examined cases.

- 5. Retrofitting counter measures that aim at improving the earthquake performance of such structures must employ comprehensive means for improving the soil-foundation deformability and the interaction with the masonry walls resting on such foundation. This problem is also amplified by the presence of underground water pressures during certain time of the year with substantial rain fall or snow melting. Unfortunately, in many cases these counter measures of improving the soil-foundation-structure interaction are rather superficial.

REFERENCES

[1] Manos G.C., 2011. "Consequences on the urban environment in Greece related to the recent intense earthquake activity", Int. Journal of Civil Engineering and Architecture, Dec. 2011, Volume 5, No. 12 (Serial No. 49), pp. 1065–1090.

[2] Manos G.C., Soulis V.J, Karamitsios N., Felekidou O., 2009. "Numerical Simulation of the Dynamic and Earthquake Behaviour of Greek Post-Byzantine Churches with and without Base Isolation", PROHITECH 2009, 21 to 24 June 2009, Rome, Italy.

[3] Manos G.C., Soulis V., Felekidou O., 2010a. "Numerical Study of the Dynamic and Earthquake Behaviour of Byzantine and Post-Byzantine Basilicas", 8th International Masonry Conference, 2010, Dresden, Germany.

[4] Manos G.C., Soulis V., Felekidou O., Matsou V. 2010b. "A Numerical Investigation of the Dynamic and Earthquake Behaviour of Byzantine and Post-Byzantine Basilicas", 9th U.S. National and 10th Canadian Earthquake Engineering Conference, 2010, Toronto, Canada.

[5] Manos G.C., Soulis V. J & Karamitsios N., 2012. "The Performance of Post-Byzantine churches during the Kozani-1995 Earthquake – Numerical Investigation of their Dynamic and Earthquake Behaviour", 15WCEE, (2012) Portugal.

[6] Manos G.C., Soulis V., Diagouma A. (2008) "Numerical Investigation of the behaviour of the church of Agia Triada, Drakotrypa, Greece", Journal in Advances in Engineering Software 39, 284-300.

[7] G.C. Manos and L. Kotoulas "Dynamic and Earthquake Performance of Greek Post-Byzantine Churches made of stone masonry accounting for Foundation Deformability", Proc. 9th Int. Masonry Conference, July, 2014.

[8] George C. Manos and Lampros Kotoulas (2014) "Earthquake Performance of Greek Post-Byzantine Churches with Foundation Deformability " Proc. 2nd Int. Conf. on Protection of Historical Constructions, pp. 297-303, 2014, ISBN 978-975-518-361-9.

[9] G. C. Manos, L. Kotoulas, V. Matsou, (2013a) "Numerical Investigation of the influence of Earthquake Forces and Foundation Settlement on the Performance of Post-Byzantine Churches", STREMAH-2013, Southampton, England, June 2013.

[10] Manos G.C., Kotoulas L., Matsou V., Felekidou O., (2013b). "Dynamic behaviour of Greek Post-Byzantine churches with foundation deformability and evaluation of their earthquake performance", 4th ECOMASS Thematic Conference on Computational Methods in Structural Dynamics and Earthquake Engineering – CompDyn 2013, 12-14 June 2013, Kos Island, Greece.

[11] Manos G.C., Kotoulas L., Matsou V., Felekidou O., (2015) "Dynamic and Earthquake Behaviour of Greek Post-Byzantine Churches with Foundation Defornability – Experimental Investigation of Stone Masonry Materials Properties", Springer Publishing, I. N. Psycharis (seds.), I.N. Psycharis et al. (eds.), Seismic Assessment, Behavior and Retrofit of Heritage Buildings and Monuments, Computational Methods in Applied Sciences 37, DOI 10.1007/978-3-319-16130-3_9.

[12] Manos G.C. & Kozikopoulos E., 2015b. "The Dynamic and Earthquake Response of Basilica Churches in Kefalonia-Greece including Soil-Foundation Deformability and Wall Detachment", CompDyn 2015, Crete, Greece.

[13] Dimitriadis V. "An ordinance for the erection of the first church in Gianitsa", Mekedonika, Vol. 9, Society for Mecedonian Studies, Thessaloniki 1969.

[14] Manos G.C. & Kozikopoulos E., 2015a. "In-situ measured dynamic response of the bell tower of Agios Gerasimos in Lixouri-Kefalonia, Greece and its utilization in the numerical predictions of its earthquake response", CompDyn 2015, Crete, Greece.

[15] Manos G.C. & Karamitsios N., 2013a. "Numerical simulation of the dynamic and earthquake beheaviour of Greek post-Byzantine churches with and without base isolation", Earthquake Engineering Retrofitting of Heritage Structures, Design and evaluation of strengthening techniques, pp. 171-186, Edited By: S. Syngellakis, Wessex Institute of Technology, UK, ISBN: 978-1-84564-754-4, eISBN: 978-1-84564-755-1, 2013.

[16] Manos G.C., 1994. "Seismic Code of Greece, Chapter 17, International Handbook of Earthquake Engineering: "Codes, Programs and Examples", edited by Mario Paz, by Chap-man and Hall, ISBN 0-412-98211-0.

[17] Euro-Code 6 part 1 2003: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, FINAL DRAFT prEN 1998-1, Dec. 2003.

[18] Euro-Code 8 2003: Design of structures for earthquake resistance - Part 1: General rules, seismic actions and rules for buildings, FINAL DRAFT prEN 1998-1, Dec. 2003.

[19] Greek Seismic Code Provisions 2000, EPPO, Earthquake Planning and Protection Organization Athens, Greece, December 1999.