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COMPUTER MODELLING AND SEISMIC PERFORMANCE ASSESSMENT OF A BYZANTINE BASILICA

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Abstract. *Within the scope of this study, seismic performance of the St. Mamas Church (Morphu, Cyprus) that dates back to 16th Century was assessed. This structure is one of the finest examples of a three aisled basilica with a central dome that was constructed based on Franco-Byzantine type architectural style. The structure is single-story with a plan area of 19m x 8m and a height of 7.5m that utilizes thick perimeter walls together with a system of cylindrical vaults and a spherical dome. The main construction material used at this structure is local highly sandy calcarenite. With the non-destructive tests carried out, mechanical properties of the material used were identified. Among the non-destructive tests utilized; Rock Hammer, Ultrasonic Pulse Velocity, Infrared Thermography as well as Ground Penetrating Radar Tests can be listed. Based on the detailed information gathered on the structure, its 3D Finite Element model was built. With this model, seismic performance of the church was assessed with the aim of identifying critical parts of this structure. In this paper, our findings are compared with results obtained by other researchers for structures of similar geometry in Europe.*

1 INTRODUCTION

St. Mamas Church of Morphou, Cyprus is a structure that dates back to 16^{th} Century [1]. The era in which St. Mamas Church was constructed corresponds to the latest phase of Lusignan architecture on the island (AC. 1360 to 1581). This phase is characterized by poor building quality, a reflection of impoverished conditions caused by Genoese aggression. Any stimulating contacts with Western Europe appear to have ended and the repertory of construction became almost exclusively insular in this phase [2]. However still, St. Mamas Church is considered to be one of the finest examples of the Franco-Byzantine type architecture on the island [2], which combines features of Byzantine (the dome) and Gothic (the pointed arch) characteristics in it (Figure 1b).

Structurally, St. Mamas Church belongs to the type of three aisled basilica with a dome [2]. It is a single-story structure with a plan area of 19m x 8m and a height of 7.5m that utilizes 1m thick perimeter walls together with a system of cylindrical vaults and a spherical dome (Figure 1). The aisles are separated by two colonnades of five columns. The central aisle is wider than the side aisles and bears a dome on its eastern part. The sanctuary apse is semi-circular and is of the same width as the central aisle. This structure including bearing walls, internal columns as well as the roof system is made up of the local highly sandy, calcarenite stone. In the late 19^{th} Century, a bell tower was added to the structure at the Northeastern corner as well as the porticoes on the north and west sides of the structure [1].

Historical records indicate that St. Mamas Church sustained severe damage from a fire during the Ottoman rule [2]. Some authors suggest that the dome did not exist in the initial construction but was added later by the Byzantine patrons during reconstruction of the collapsed roof after the fire. However the ornamentation of the column capitals at the bays below the dome is different, which is an indication that the position of the dome was decided beforehand during the initial construction of the structure. Today, the roof structure is in a rather deprived state due to high moisture level caused by malfunctioning drainage system. From the search of historical records, no mention of previously sustained earthquake damage could have been found, although [3] indicates that the Morphou region was shook severely by the two rather recent earthquakes of 13 June 1933 and 6 November 1968.

The present St. Mamas Church stands on the ruins of two early Christian basilicas and of a Byzantine church [1]. The non-destructive tests carried out on the structure that will be discussed in detail in the following section as well as the geotechnical and seismic tests carried out at the Church site are supportive of this statement. The geotechnical and seismic tests also indicated that soil condition at the location of interest is of NEHRP D [4] type. Due to the presence of ruins as well as the rather poor soil conditions, cracks are not absent on some of the inside columns supporting the dome structure above.

Within the scope of this study, the material and structural properties of the St. Mamas Church was investigated with the aim of assessing its seismic vulnerability as well as documenting the deterioration of this unique structure. My findings obtained based on non-destructive tests, as well as static and dynamic analyses carried out are discussed in the following sections of this paper.

(a)

(b)

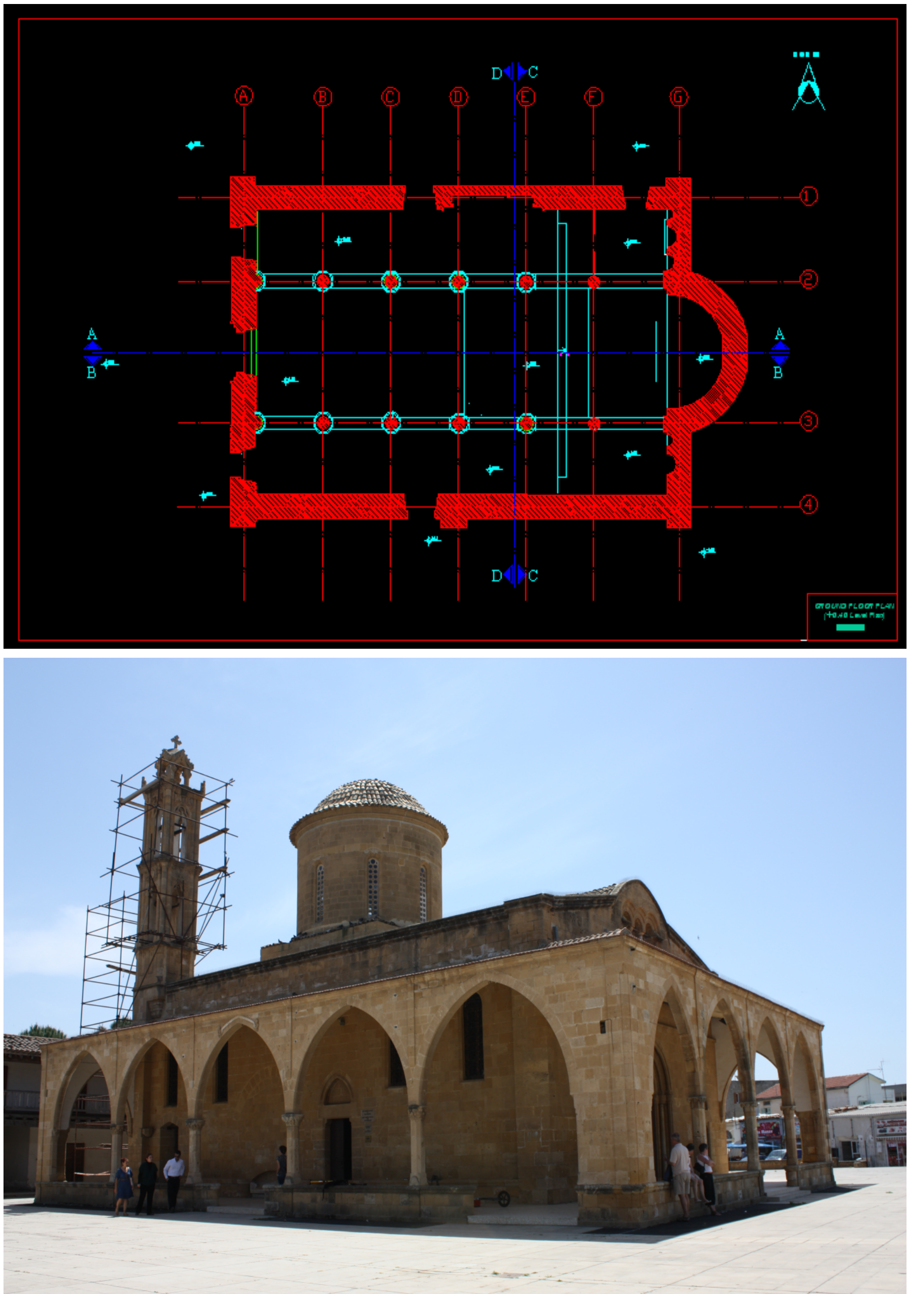


Figure 1: St. Mamas Church (a) plan view (Northern and Western porticoes are not included to the drawing), (b) picture of Northern and Western facades of the structure.

2 NON-DESTRUCTIVE TESTING

In this study, a number of non-destructive tests were employed; including rock hammer, ultrasonic pulse velocity, infrared thermography, ground penetrating radar tests with the aim of assessing the mechanical properties of the calcarenite building material of the structure. Variability of these properties throughout the structure, in addition to the degree of deterioration of the building material in different parts of the structure was studied as well.

Rock hammer test was applied to internal columns and peripheral walls of the structure. The instrument utilized for this purpose was an L-type rebound hammer with an impact energy value of 0.74nm. From each structural element 20 readings were taken, mean and standard deviation values of these readings were later computed to reach conclusions regarding relative strength values of the studied structural elements. Obtained results are summarized in Table 1 for internal columns.

	F2	F3	E2	E3	D2	D3	C2	C3	B2	B3	A2	A3
μ	9.2	13.0	11.2	12.3	10.7	17.6	11.6	14.8	9.2	14.8	8.8	6.8
σ	5.54	7.13	5.74	6.87	5.41	7.68	6.18	6.45	4.23	8.76	4.65	2.57

Table 1: Rock hammer test results (μ: mean, σ: standard deviation) for the interior columns of St. Mamas Church (The reader should refer to Figure 1a for exact locations of these columns).

Results given in Table 1 indicate clearly the considerable variability observed. This was expected as the tested material is natural hence non-uniform. Also due to structure's age, time added additional heterogeneities caused by regional decay of the material under question. The middle columns of axis 3 (F3, E3, D3, C3) have relatively higher mean strength values in comparison to the rest. This can be one reason for observed cracking on columns D2, E2, F2 (supporting the dome above) whereas their mirror image counterparts on axis 3 being crack free. The rock hammer test results obtained for peripheral walls (both interior and exterior readings) have an average value of 11 with a coefficient of variance of 20%, in this respect similar to the results obtained from the interior columns. Regarding peripheral walls, local weaknesses were observed at the sanctuary apse, where axes G and 2 meet as well as G and 3.

Ultrasonic pulse velocity test was applied to internal columns of St. Mamas Church only, at 6 different levels along each element height with an approximate spacing of half a meter. This test was especially applied around observed cracks with the aim of determining whether these cracks are running deep into the element or are just surface cracks. Obtained results are summarized in Table 2. Distribution of mean velocity values (in m/s) are in agreement with above discussed rock hammer test results. With this test, it was also observed that except from cracks of column D2, other reported cracks are probably not running very deep into the column elements. It should be noted that average results obtained for the calcarenite of St. Mamas Church in this study are almost twice as high as the velocity values obtained by [5] for the calcarenite of Saint George of Latins Church at Famagusta, Cyprus.

	F2	F3	E2	E3	D2	D3	C2	C3	B2	B3	A2	A3
μ	4956	5148	3310	4318	3735	4077	5974	6334	4659	5469	2585	3374
σ	841	1526	1524	1290	2961	1665	2631	403	1855	1730	355	1321

Table 2: Ultrasonic pulse velocity test results (μ: mean, σ: standard deviation) for the interior columns of St. Mamas Church (The reader should refer to Figure 1a for exact locations of these columns).

The malfunctioning drainage system of St. Mamas Church was already mentioned above. As presence of water accelerates deterioration of masonry and hence weakens it, variations in moisture content were documented by employing the infrared thermography technique. Results obtained indicate positive temperature gradients along the height of internal column elements suggesting moisture intake from the foundation of the structure through capillary action. However the lowest of all temperature measurements taken corresponded to the two side aisle vaults (Figure 2). On these vaults, severe discoloration was observed; the measurements obtained are the proof that these discolorations are not just stains but due to high moisture content of the masonry at these locations.

Figure 2: Pictures depicting two aisle vaults along which the lowest temperatures within the structure were detected.

The main objective of carrying out the ground penetrating radar surveys on the structure floor was to investigate the foundation type of the structure as well as existence of any possible underground structures. For this purpose a ground penetrating radar unit with a 400 MHz antenna was used that is capable of providing moderate resolution up to 5m depth. The structure floor was surveyed regularly in both North-South and West-East directions. Results indicate that the foundation system is pad foundation type with no interconnecting ground beams. Also numerous archeological remains were detected throughout the plan area of the structure. These were found to exist at two main depth levels: (i) 0.75-1.25m and (ii) below 1.5m, probably corresponding to the early Christian and Byzantine church remains mentioned above (Figure 3). Existence of such remains/cavities may give rise to settlement problems; hence the observed difference in behavior of axes 2 and 3 columns can also be attributed to differential settlement caused by these detected remains/cavities. However it should be underlined that remains detected are not localized along axis 2 only but rather distributed throughout the whole floor area.

Above, the variability of properties of structural material utilized at St. Mamas Church was discussed. In order to determine compressive strength and modulus of elasticity parameters corresponding to the structural material under investigation, rock hammer and ultrasonic pulse wave test results had to be calibrated. For this purpose three different sources of information was used: (i) official calibration data of rock hammer employed in this study [6], (ii) mechanical properties measured by [7] for local (not aged though) calcarenite, (iii) mechanical properties measured by [8] for local calcarenite (samples taken from St. Nicholas of Famagusta, Cyprus) with lower sand content. Based on this information, a compressive strength value of 10MPa, tensile strength value of 1MPa (assumed to be 1/10 of the compressive strength) and modulus of elasticity value of 2500 MPa was used in the analyses phase of this study. For columns D2, E2, F2, these values were reduced by 20% as a reflection of observed surface hardness and porosity variations. A density value of 1.7 gr/cm^3 was used throughout the whole structure.

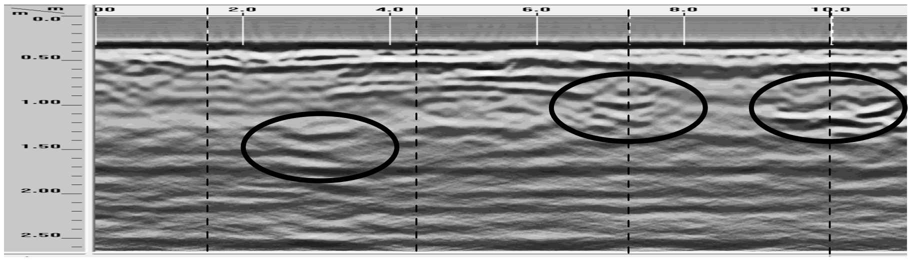


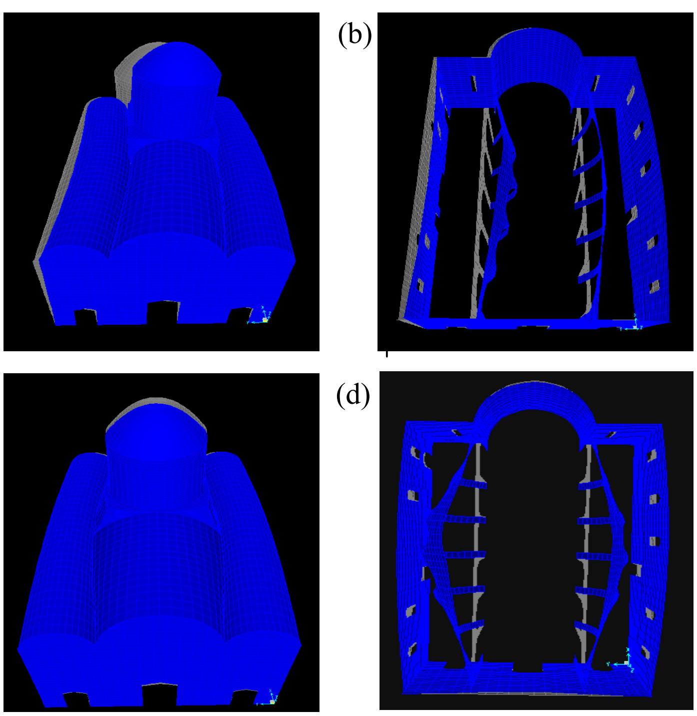
Figure 3: Ground penetrating radar survey result along axis 2. Horizontal axis represents distance along axis 2, vertical axis represents depth. Dotted black lines represent locations of columns B2, C2, D2, E2 from left to right respectively. Black ellipses close identified remains along axis 2.

3 DYNAMIC PROPERTIES

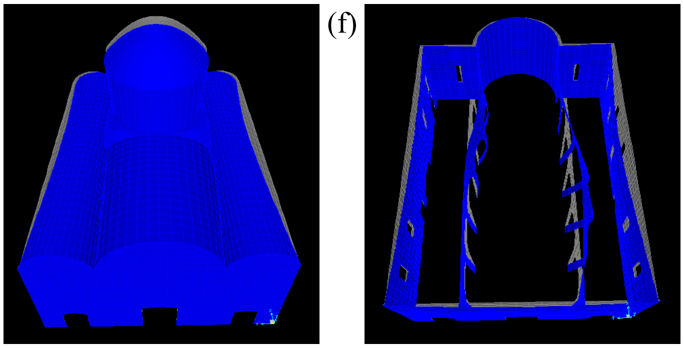
A finite element model of the structure was developed using SAP2000 software. In this model of plan area 19m x 8m, general height 7.5m that reaches 17m at the dome; all structural elements were represented by shell elements. Thicknesses of these elements vary considerably throughout the structure, i.e. 1m for peripheral walls, 0.55m for internal columns, 0.35m for the vaults and the spherical dome. The dimensions of various structural elements were quite accurately obtained with the help of a laser scanning instrument; except for the vault and dome thicknesses. As accurate dimensioning would require causing damage to the structure, these thicknesses were assumed to be 0.35m in this study. The bell tower and the porticoes were not included into the model of the church explicitly as these parts were added 300 years later to the original church building and hence are structurally independent. The effect of bell tower was taken into account indirectly as additional mass at the Northeastern corner of the structure. All peripheral and internal walls were set fixed at the foundation level in the model.

(a) (b) ***T=0.21s, U_y=59%***

(c) (d) ***T=0.20s, U_y=0.05%***



(e) (f)



T=0.17s, U_x=13%

Figure 4: First three eigen-modes of the structure. First column of figures illustrate the structure as a whole, second column of figures do not include the roof system (hidden from view to illustrate directly the interior elements)

Figures 4a-4f depict the first three eigen-modes of the structure obtained by the linear elastic modal analysis carried out. The translational eigen-mode in the transverse North-South direction (y direction) is the one with the longest eigen-period (Figures 4a-4b, T=0.21s). The structural response in this mode displaces the longitudinal walls mainly in an out-of-plane sense, which is achieved by the in-plane deformation of transverse peripheral walls. This first mode mobilizes approximately 59% of the total mass of the structure. This is later followed by a higher translational eigen-mode in the transverse direction that mobilizes only very small portion of the total mass and mainly causes out-of-plane deformation at internal longitudinal walls (Figures 4c-4d, T=0.20s). The translational eigen-mode in the longitudinal East-West direction (x direction) is finally reached at the third longest eigen-period (Figures 4e-4f, T=0.17s). The structural response in this mode displaces the longitudinal peripheral walls mainly in in-plane manner, with the cylindrical drum at the roof displacing in the East-West direction and the transverse peripheral walls mainly resisting with out-of-plane action. This third mode mobilizes approximately 13% of the total mass of the structure.

4 STATIC ANALYSIS RESULTS

In this part of the study, static behavior of the structure was assessed under three different loading cases. The first of these loading cases is dead load (G) that includes weight of the tile coverings on the roof as well as the weight of the bell tower. From this loading case, live loads were explicitly left out as in the absence of snow loading, which is the case for the structure of interest, the magnitude of live loads were negligible in comparison to dead loads. The second and third loading cases are the earthquake forces E_x and E_y applied in the x and y directions respectively. These earthquake forces include application of an acceleration value of 1g (g=9.81m/s^2) in the x and y directions. Main objective here was to determine limiting deformations in the two horizontal and vertical directions.

Results are shown in Figures 5a-5f below. Under the effect of dead load only, maximum vertical deformation of 3mm was observed at the spherical dome above the drum; whereas level of vertical deformation at the columns below supporting the drum was 2mm (Figure 5a-5b). The structure is considerably more flexible in the transverse direction than the longitudinal direction. Results also indicate that the internal colonnades are so flexible in both longitudinal and transverse directions that they contribute very little to the resistance in both of these directions. Under E_x, maximum deformation along the longitudinal axis of the structure at the dome level was determined to be 12mm whereas under E_y maximum deformation along the transverse axis of the structure at the dome level was almost twice as much approximately

22mm. Under both E_x and E_y, the deformations at the roof level below the drum were considerably lower than the aforementioned maximums, in the range of 3-4mm for E_x and 8-10mm for E_y. Under E_x, it is mainly the longitudinal peripheral walls resisting with in-plane action. Whereas under E_y, transverse peripheral walls resist with in-plane action and the longitudinal peripheral walls resist with out-of-plane action.

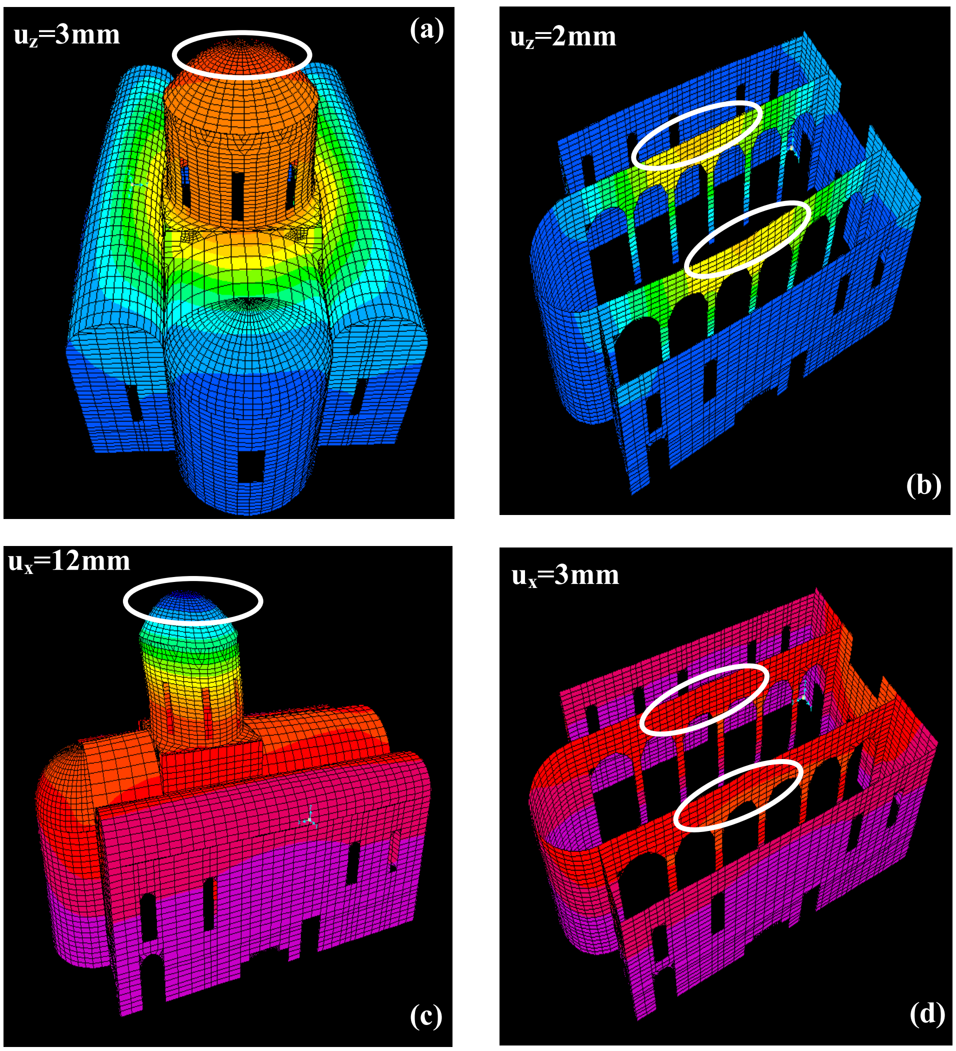
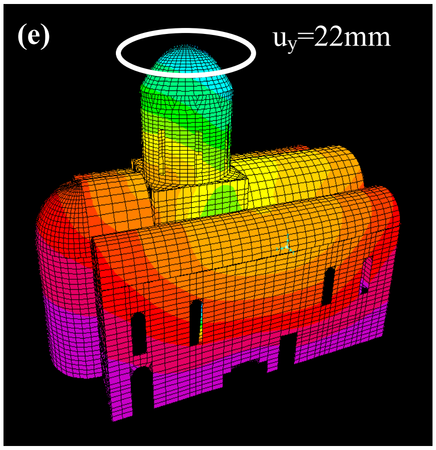


Figure 5: Deformations under (a)-(b) dead load G and (c)-(d) Earthquake load E_x. First column of figures illustrate the structure as a whole, second column of figures do not include the roof system (hidden from view to illustrate directly the deformation of interior elements)



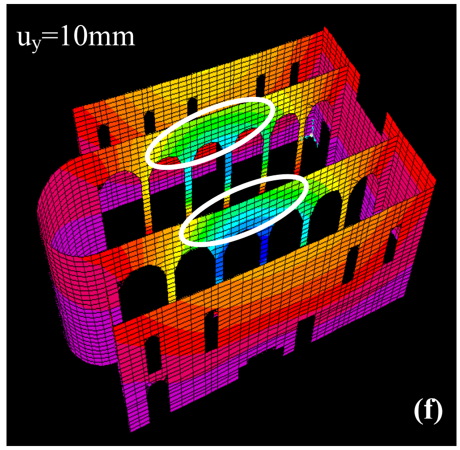


Figure 5 cont.: Deformations under (e)-(f) Earthquake load E_y. Figure on the left illustrate the structure as a whole, figure on the right do not include the roof system (hidden from view to illustrate directly the deformation of interior elements)

5 DYNAMIC ANALYSIS RESULTS

In this final phase of the study, dynamic behavior of the structure was assessed by carrying out a response spectrum analysis. According to the findings of [9], the peak ground acceleration value at the site of interest is in the range of 0.2-0.3g corresponding to a return period of 475 years. As mentioned in the introduction part of this manuscript, detailed seismological and geotechnical tests were carried out at the site of St. Mamas Church within the scope of this study. The geotechnical study included, carrying out a standard penetration test up to a depth of 30m and classification of formations based on samples taken during the test. With the seismological tests, a 2D S- and P-wave velocity profile along an axis of 50 m parallel to the structure was developed. Based on the results of both tests, soil condition at the site of interest was classified as NEHRP D, or class C of Eurocode 8. By utilizing the base response spectrum function of Eurocode 8 together with the peak ground acceleration value and the aforementioned soil type, together with the building importance factor of 1.3, the site specific response spectrum for the St. Mamas Church was developed. In addition to the horizontal loading (E_x when applied in the longitudinal direction, E_y when applied in the transverse direction) described by the response spectrum, dead load was applied to the structure with the following combinations: (1) $G+1.4E_x$, (2) $G+1.4E_y$, (3) $G+0.9E_x+0.3E_y$, (4) $G+0.9E_y+0.3E_x$. Obtained normal and shear stress distributions were investigated in detail and critical structural elements were identified.

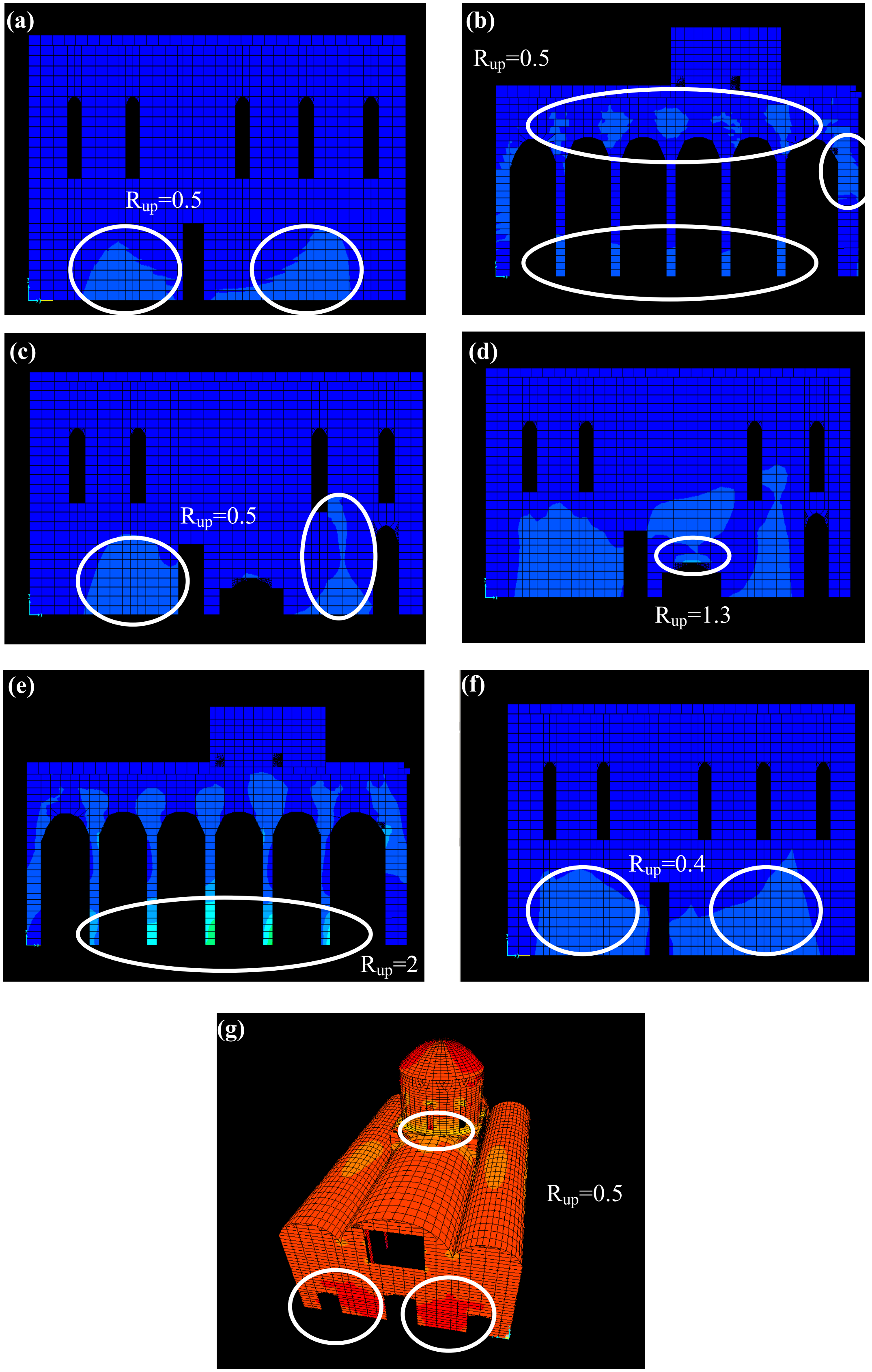
In Figure 6, distribution of stresses is shown for peripheral walls, internal colonnades, and the roof structure. Under the average strength values given in section 2 of this manuscript, the structure is not expected to sustain any damage due to seismic loading. However as the non-destructive testing results illustrate, mechanical properties of the masonry vary considerably throughout the structure. If the lower limit strength values are considered rather than the average values obtained, several structural elements appear to be critical throughout the structure. The lower limit strength values that were employed in this study were 3MPa for compressive strength and 0.3 MPa for tensile strength. For columns D2, E2 and F2, 20% lower strength values were used to reflect non-destructive test results. The critical elements identified are the internal columns carrying the drum and the dome of the structure (D2, E2, F2, F3, E3, and F3 of Figure 1a), vicinity of openings (doors, windows, sarcophagus of St. Mamas) on the peripheral walls, base of peripheral walls and the drum-lower roof intersections of the roof structure. Based on the aforementioned lower limit strength values, masonry over limit factors (ratio of observed stress level to assumed lower bound strength value) were also computed for these critical regions. These masonry over limit factors (R_{up}) are all indicated on Figure 6 as well.

Three aisled basilicas are abundant on the island of Cyprus, as well as neighboring regions such as Balkans. One of the main aims of this study was to compare results of this study with other previously carried out studies on dynamic behavior of basilicas of similar architecture to identify commonalities as a forward attempt towards developing a vulnerability relation for this type of structures that can be utilized in aiding separation of high seismic risk structures from the rest.

In [10], behavior of three different basilicas was studied. Two of these structures had similar plan areas to that of St. Mamas Church, whereas the third structure was considerably larger. The main difference between the structures of [10] and the St. Mamas Church is that St. Mamas Church is considerably taller with a roof structure that is entirely made up of masonry which reaches a height of 17m at the semi-hemispherical dome above the drum. Whereas two basilicas of [10] (with the similar plan area as St. Mamas Church), have wooden roofs at a height of only approximately 7m. This difference between the roof structures appears to be the main reason behind different dynamic behaviors. As a result, St. Mamas Church has longer dynamic periods, as well as eigen-modes dominated by out-of-plane deformation of internal colonnades carrying the drum and the dome of the roof structure. Similarly, internal columns reach critical stress values under seismic loading whereas such behavior was not observed in the study of [10].

In [11-13], the plan area of the structure studied is very similar to that of the St. Mamas Church as well; however it only has a height of 5m with a wooden roof. Expectedly, the eigen-modes are not dominated by the out-of-plane deformation of internal colonnades supporting the roof structure. However results indicate damage incurred by internal columns due to excessive tensile stress once the peak ground acceleration level of 0.25g was surpassed (with tensile strength level corresponding to the lower bound strength level employed in this study).

In [14], structures with similar architecture were investigated both analytically and experimentally. As the plan area of the structure under consideration of [14] is much smaller than that of St. Mamas Church, direct one to one comparisons will not be made here with the findings of [14]. However in [14], it is clearly indicated that structures having a masonry roof system including a central drum and dome will sustain damage first at this critical region, in line with the observations made regarding St. Mamas Church.


(a)
R_{up}=0.5
(b)
R_{up}=0.5
(c)
R_{up}=0.5
(d)
R_{up}=1.3
(e)
R_{up}=2
(f)
R_{up}=0.4
(g)
R_{up}=0.5

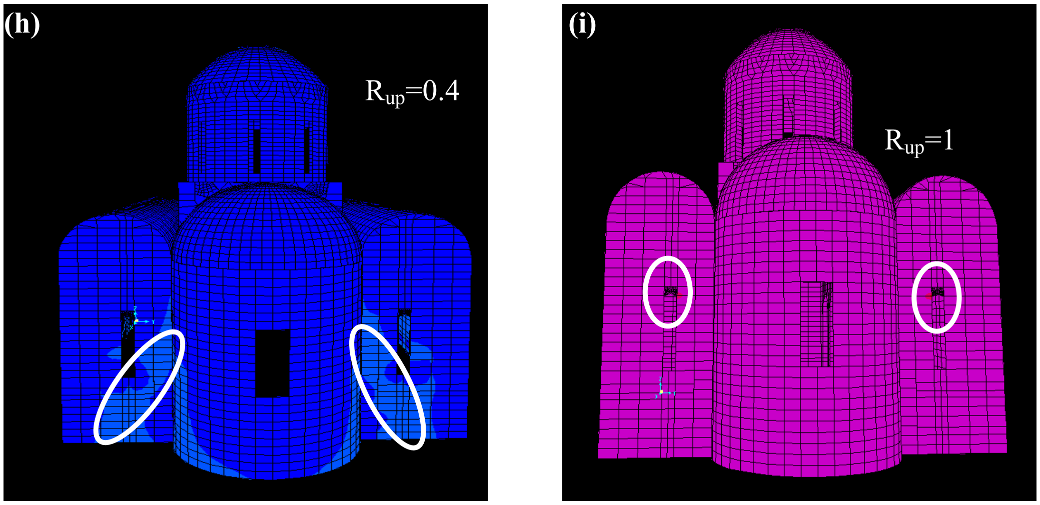


Figure 6: Stress distributions under (a)-(c) combination 3 G+0.9E_x+0.3E_y and (d)-(i) combination 4 G+0.9E_y+0.3E_x. As the structure is weaker in the transverse direction, the combination 4 yielded more critical results. Most critical regions are marked with the white ellipses, for these regions masonry over limit factors (R_{up}, ratio of observed stress to assumed lower bound strength level) are given as well.

6 CONCLUSIONS

Within the scope of this study, both experimental and analytical work had been carried out on St. Mamas Church of Morphou, Cyprus with the main objective of assessing its dynamic behavior. The following conclusions were reached:

- The non-destructive tests carried out at the structure indicated that mechanical properties of the masonry used vary considerably with coefficient of variation values computed reaching up to 60%. Based on the test results, the average compressive strength value of 10 MPa, modulus of elasticity value of 2500 MPa and tensile strength of 1 MPa were assigned to the structure. However in order to take into account the considerable variability of mechanical properties of masonry, lower bound strength values of 3 MPa for compressive strength and 0.3 MPa for tensile strength were also used.

- Dynamic analysis results indicate that as the internal colonnades of the structure are quite flexible, the structure is weaker in the transverse direction than longitudinal direction. Under the seismic loading with a return period of 475 years, the most critical elements of the structure were identified as the internal columns supporting the drum and dome of the roof system. The stress level at these columns is expected to reach twice the lower limit strength values determined. The second level critical localities are surroundings of openings such as windows, doors, sarcophagus of St. Mamas on the North façade of the church and the drum-lower roof connections of the roof system. Results indicate clearly that presence of the drum and the dome has governing effect on the dynamic behavior of the overall structure.

- Dynamic analysis results indicate the vulnerability of internal columns, hence observed cracking on these columns can not be associated with a differential settlement problem alone.

- Structures with the same architectural form as that of St. Mamas Church are abundant in Europe. Hence several studies assessing dynamic behavior of similar structures can be found in the literature. Comparison of results indicate that grouping these structures according to floor area (geometry does not vary considerably), roofing structure, material properties and developing vulnerability relations for each group would be a valuable contribution towards preservation of such monuments; as such relations would enable the preliminary categorization of structures according to their seismic risk level. With this statement of course, the intention is not to undervalue the effect of uniqueness of each structure and the effect of considerable variability of building material properties on their dynamic behaviors. However with the aim of approaching the problem of preservation from a more macro perspective, existence of such vulnerability relations would prove to be useful in preliminary categorization of these structures regarding their seismic riskiness.

7 ACKNOWLEDGEMENTS

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