USING GEODETIC AND LASER SCANNER MEASUREMENTS FOR MEASURING AND MONITORING THE STRUCTURAL DAMAGE OF A POST BYZANTINE CHURCH

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ABSTRACT

This paper presents a detailed description of the methodologies and the actions taken for determining and monitoring the deformations and micro-movements of the post-byzantine Church of Megali Panayia in Samarina. An assessment is also provided of the first results obtained from the repeated observations and their combined adjustment, followed by a discussion on their reliability. The reported analysis focuses on the following crucial aspects: the accuracy and the stability of georeferencing, which is fundamental to make comparisons between different multi-temporal measurements; and the computation of deformation based on the survey observations between the established control points. Emphasis is given to the presentation of the results using a detailed 3D model of the church, created from terrestrial laser scanner point clouds. The methodology applied involves the analytical combination of multi-source measurements, i.e. GPS observations and classical surveying observations using appropriate high-end theodolites and total stations, in order to achieve the highest possible accuracy. The use of a time-of-flight laser scanner provided data for the 3D model creation. From the re-measuring of this network four times during a five month period and by comparing and analyzing the results of all measurements, it was possible to detect and obtain crucial information about the building's structural behaviour and the surrounding terrain's displacements and deformations. The results show a tendency of the surrounding ground to slide towards the northeast direction in a very slow tempo. However the observed displacements between characteristic control points on the building itself tend to demonstrate a slow varying periodic effect. The internal accuracy achieved in absolute positioning of the network points is of the order of ±2-3 millimeters.

INTRODUCTION

Samarina, reportedly the highest village in Greece, at an elevation of some 1450 m, is situated on the wooded slopes of Mount Smolikas in the Pindos Mountains, approximately 70 km west of Grevena in northwestern Greece. This small town with its Vlach population enjoyed (up to the early 19th c.) three successive centuries of exceptional economic growth and cultural development. The level of culture reached by this town is still evident in the excellence of the religious paintings found in the many churches in the area. Samarina's post Byzantine church of Megali Panayia (Great St. Mary's church) has been built in 1816 and is the area's main religious landmark, as well as a wonder of nature. The church is famous for its painted ceilings, frescoes, and a finely carved iconostasis (templon), but also that the roof of the altar, covering the apse, in the east side of the 40 m long building, "accommodates" a big pine tree with no sight of the tree's roots to be found within the church or outside the wall (Figure 1). The church is constructed of local stone and has very shallow foundations sitting on unfavorable ground, composed mainly of clay, silt and peat, with the solid rock found in depths of more than 15 m from the surface.

Figure 1: Samarina's Megali Panayia (South façade)

Figure 2: Typical examples of damage caused by structural deformation in the interior (left) and the exterior (right) of the church of Megali Panayia in Samarina

Wintertime is very harsh in Samarina. The geological and geomorphological characteristics of the greater area around the village are considered to play a vital role in the various geotechnical problems, i.e. severe cracks, which have made their appearance on a number of houses and other constructions and stone works, and inevitably contribute to similar structural problems on the Church of Megali Panayia. Continuous rainfalls and snow cover, causing soil erosion and ground loading effects, are considered to be additional causes of structural problems, which have occurred to the church over the past few years. Large visible cracks inside and outside of the building, as the examples shown in Figure 2, have alarmed the curators of the monument and have caused severe worries concerning the security of the building and that of the pilgrims. Hence some immediate action is considered imperative. However, before any maintenance intervention, a quantitative analysis of the evident and likely critical deformations of the church building was deemed to be of utmost importance. Therefore, before deciding upon the best possible solution for the support of the damaged monument and the engineering measures to be taken for preventing the risk of possible sliding of the building due to the soil erosion, a highly accurate geodetic network for monitoring over a period of time the deformations of the church building and the surrounding ground was considered necessary. The main objecting was, through the synthesis of various types of repeated measurements and the variations of the coordinates of observed points, to detect and monitor the building's deformations through time and relate any changes in its geometry to likely terrain displacements and the rebounding of the terrain caused by the winter induced variations of the underground water levels in the surrounding area of the monument.

METHODOLOGY

In order to design the necessary monitoring network inside and outside of the church, there were many different considerations, which had to be taken into account. These were mainly posed by the high accuracy requirements for the determination of the deformations, the architectural and structural features of the church, the morphology of the surrounding area, the immediate environment around the church building, as well as by several restrictions dictated by the Archeological Service, such as for instance not to use any destructive techniques which could inflict the slightest additional physical damage on the church.

For the integrated detection of the deformations and the micro-movements of the church, it was decided that periodical measurements were necessary for a total period of one year, so that the impact of the various seasons of the year on the movement of the ground would be investigated. So far four measurement epochs have been completed, in intervals of about 45 days, from June to October 2009.

Church Features leading to Special Considerations

The church of Megali Panayia is a typical example of post-Byzantine building in Greece and other Eastern European countries with the most common type of roof known as a "spatial post and beam" system (Tsakanika-Theohari, 2007), which functions in a completely different way from the other well know types of king post trusses that are more common at Italy and other European countries. The most important structural features that are common in most of these roofing systems are: the spatial main load bearing system, the closely set rafters and the need of internal load bearing walls for large spans.

The outside dimensions of the church are approx. 40 x 17.50 m, with the two sided roof reaching a height of 10 metres. The main building which has a width of 13.5 m consists of the *narthex* (vestibule), the *nave* and the sanctuary (or the *altar*) and a roofed porch as a fore-narthex. Along the South side, there are the chapel of St. Peter and St. Paul (Figure 3), a long roofed porch with an arched masonry wall and a store room (Figure 3). Inside, the nave is separated from the sanctuary by an impressive wall of icons, curved panels and religious paintings (the *iconostasis* or *templon*) which are in full harmony with the decorated interior of the church which is full of frescoes portraying the whole body of the orthodox church, from Christ downwards, and have the dual purpose to give inspiration to the worshipper and to act as windows to the spiritual world. The side walls are decorated with stand life-size portrayals of the saints on the lowest level, or more complex scenes portraying incidents from the Gospels or the Day of Judgment on the upper walls and vaults.

Establishing the Monitoring Network

The measurements to be carried out were to answer two vital concerns: (a) whether the building is tilting or moving on a suspected slide-prone slope, at what direction, what is the magnitude of that movement, as well as what is the time evolution of such movements, and (b) whether any deformations of risk to the building do occur and how big they are. Therefore, the team of scientists and engineers who would eventually be called upon to make the decision on what engineering measures must be taken had specified that the accuracies for the necessary geodetic measurements were to be at the level of a few millimeters. In order to meet these requirements, it was decided to establish a network of fixed points around the monument (referred in the sequel as the "local GPS control network"), whose position would be monitored using multi-day/multi-period GPS observations and state-of-the-art analysis techniques similar to those used for geodynamic applications. Then, additional network points (the "terrestrial control network") in the interior of the church would be determined with highly accurate conventional (terrestrial) geodetic measurements of angles, lengths and height variations between these points. Such measurements were repeated so far four times in intervals of about 45 days, from June to October 2009.

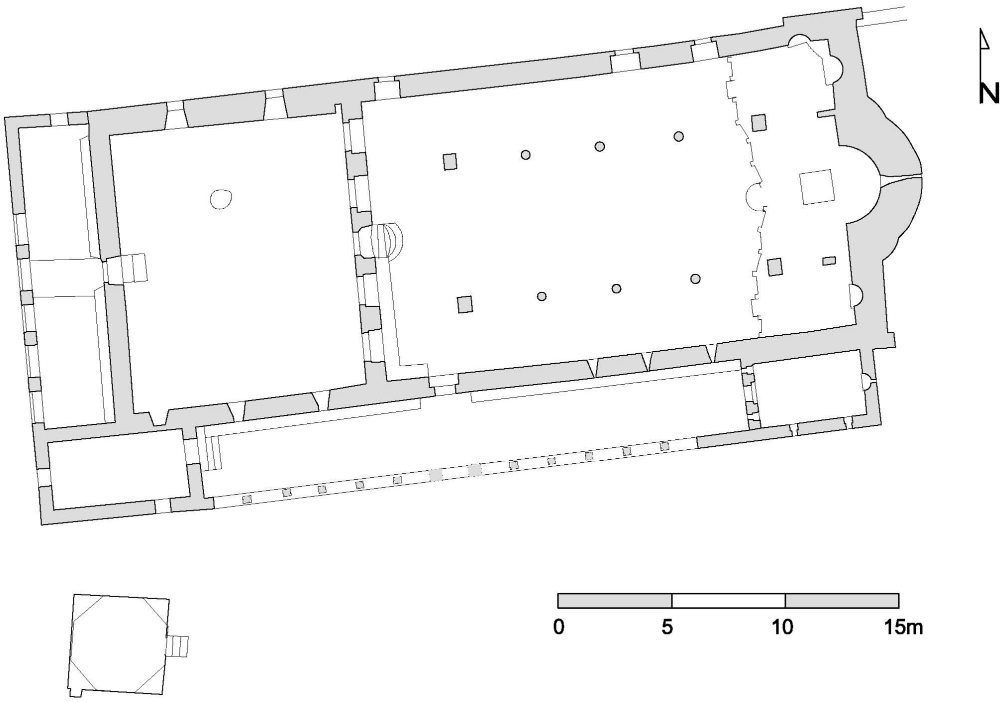


Figure 3: Floor plan of the Church of Megali Panayia

The monitoring geodetic network which was established around the church is composed of special permanent markers, namely semi-portable pillars (Figure 4), for setting for positioning the various instruments used (e.g. GPS receivers, electronic theodolites and total stations), nine more station points in the church's interior, marked using small brass benchmarks floor (Figure 5) and of permanent survey targets on the walls of the church, both inside and outside, that were set up in different places where identity, pointing accuracy and permanence could be ensured. The former type of markers was selected so that to ensure high repeatability of the measuring process, in terms of minimising any centering errors during the instrument setting over the markers each measuring period. A special procedure was used in order to measure precisely (±0.3mm) the instrument height over each pillar (Lambrou et al, 2007). The targets for the points on the walls were specialized retroreflective tape targets (Figure 6), which would ensure accuracy of ±1mm in distance measurement. Their exact positions were decided with the co-operation of specialized architects and civil engineers, who would eventually use and exploit the results of the measurements of the repeated surveys (Figure 5). They were durable, nondestructive, stable and insensitive to external influences (temperature variations, corrosion and moisture) and they were suitably chosen so that not to cause any damage to the church walls and be barely visible to the uninformed observer.

The main criteria for the design of the survey network were twofold: (α) to ensure that the repeated measurements would be performed under the same controlled conditions and they would be referenced to a common reference system in every measurement period, and (b) to have the capability to measure suitable points on the building walls in such a way that all suspected deformations could be detected and monitored with an accuracy better than 2-3 mm.

Figure 4: Special markers for the network points

Figure 5: Station point inside the church

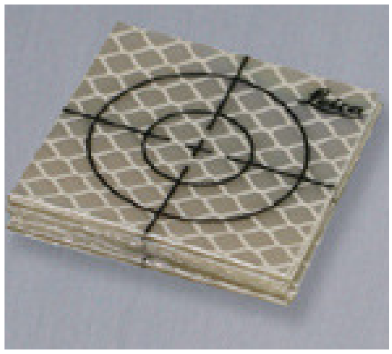


Figure 6: Special retroreflective tape targets

The Local GPS Control Network – Design Characteristics and Measurement Requirements

The local GPS control network consisted of six stations surrounding the church building with inter-station distances ranging from about 19 to 69 m. These stations could obviously experience, if any, the likely deformation and strain effects in the surrounding terrain due to any movements resulting, for example, from geological phenomena such as the ground's rebound response to snow loading (Architectural Institute of Japan, 2004; Hansen and Brown, 1986) and the pockets of high strain rates that could occur in localized volumes in the area surrounding the church ("inner zone"). Therefore, the small local network was simultaneously observed from five additional fixed stations (fiducial reference points) which were situated outside of the influential area of the factors that can affect the church's building and the foundation ground that it is situated on.

As fiducial reference points, we chose the following five nearby EUREF stations: AUT1, ORID, DUBR, MATE, TUC2. These stations are established and monitored continuously by international agencies such as IGS and EUREF and are classified as class-A stations, i.e. sites with well-known coordinates and coordinate velocities given in the ITRF and ETRF reference systems, and are suitable for determining the coordinates of other unknown points (Bruyninx et al, 2009). The collected GPS observations in these stations are readily available from the web site of the EUREF Permanent Network (EUREF, 2009) within 24 hours of their acquisition time. Generally, the six stations of the local GPS network were observing for 7 to 8 hour sessions each for three days, during each of the four measurement campaigns that were carried out. All local GPS sessions were conducted using dual frequency GPS (Javad and Spectra Precision) receivers with L_1/L_2 geodetic type antennas, which allow the precision levels of GPS measurements to be at the sub-centimeter level horizontally, and ±1 cm vertically, provided that the collected data is analyzed carefully using sophisticated GPS processing techniques and software (Barnes and Cross, 1998).

The Deformation Monitoring Network

GPS has been used in many similar situations of structural monitoring of large structures with considerable range of displacements, as well as combined with other sensors (Aguilera et al., 2007; Knecht and Manetti, 2001). In spite of this, however, it has a significant limitation, as well. Since signals are received from satellites, coordinates cannot be measured indoors or through obstacles. In the present project, this inability has led us to use classical geodetic methods based on angles, distances and height variation measurements between the points of an extensive 3-D terrestrial network (denoted in the sequel as the "deformation network") established using 15 station points (the six points of the GPS network and the nine more in the interior of the church) and an additional 76 target points situated in the outside vertical walls and the interior of the church. These points were selected so that to be able to extract from the measurements the maximum amount of information which could reveal any likely variations (e.g., detail relative and absolute movements and structural irregularities in the church building) between the 45-day observing periods. All of these points were actually network points and were measured with the high accuracy total station, using specialized forced centering devices and specially developed instrument height measuring procedure, in order to keep all error sources to a minimum.

One of the main challenges of the project was the design of the complete 3D local network, and achieving redundant inter-visibility between targets considering the restrictions imposed by the short object distances which ranged between ten and thirty meters. Most of these target points were observed from more than one station points, enabling in this way their determination through a combined network adjustment using suitable software that allowed the adjustment of the collected GPS and terrestrial surveying observations separately and/or in combination.

Linked to and adjusted using the local GPS network stations to impose suitable constraints, the points in this deformation network were, almost all of them, used as control points allowing, though repeated measurements, to monitor changes and possible displacements in the structurally sensitive areas of the building. All survey measurements were carried out using recently calibrated electronic instruments, such as the Leica TDA 5005 and Leica TCRM 1201 Total Stations, which provide angular measurements with a precision of $\pm 1.5^{cc}$ and $\pm 3^{cc}$ respectively and distance measurements with a precision of ±1 mm when either using reflectors or retroreflective tape targets. Such precision levels ensure that the inter-station distances between any points of the deformation network could be determined with precision of the order of ±0.5 mm.

In this modeling strategy, the control and accurate determination of any type of deformation can be ensured, by analyzing carefully the 2-D and 3-D results for the absolute and relative coordinates of these points and determining reliably any likely variations between the components of the inter-station vectors between them. Hence, it is possible to demonstrate their viability for the sought structural monitoring of the church throughout the periods of the repeated observations conducted so far, i.e. between June and October 2009.

Laser Scanning Measurements - 3D textured model

For visually presenting the results in a more understandable and descriptive way, it was decided to construct a virtual photorealistic 3D model of the church and depict on that the displacements on various measuring periods for all measured points. For that purpose a Leica ScanStation 2 terrestrial time-of-flight laser scanner was used. Both the inside and outside of the church were fully scanned and in addition a few hundreds of high resolution digital images were captured, in order to render the final 3D model.

In total 8 scans were performed for the outside, 4 for the porches and 14 for the inside (Figure 7), collecting approximately 11 million points for the exterior and the porches and 18 million points for the interior scans points covering practically all details of the monument.

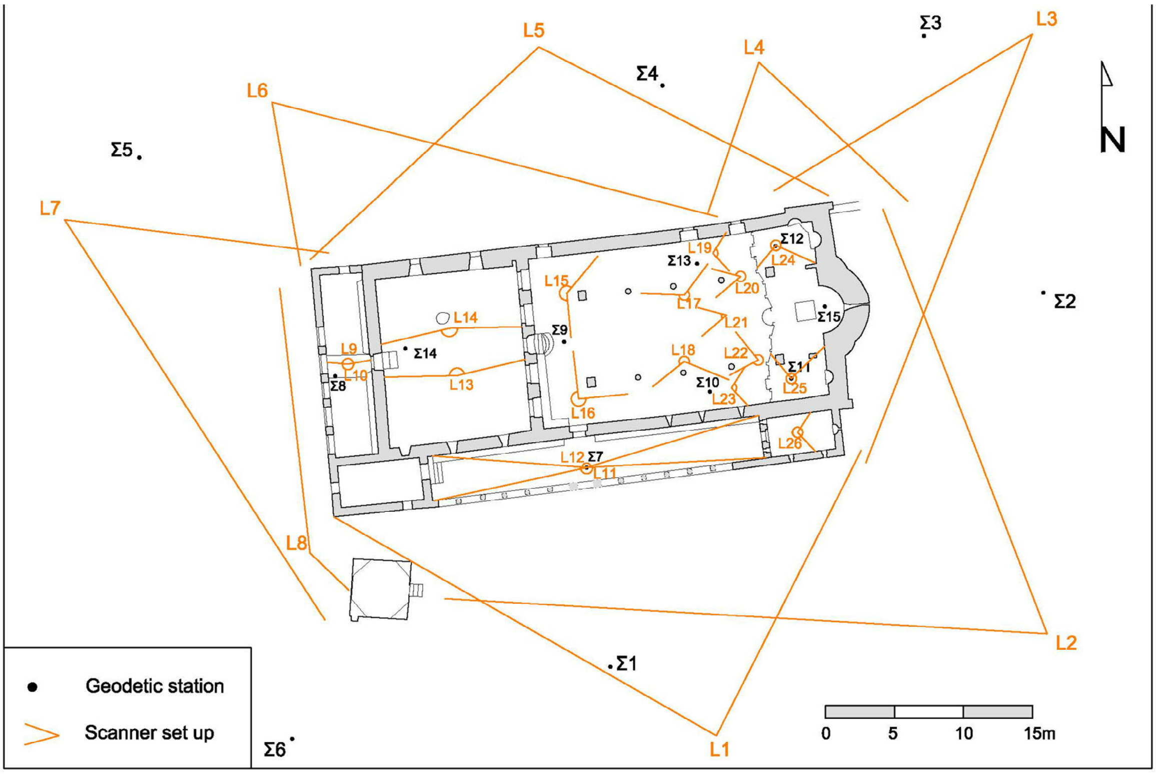
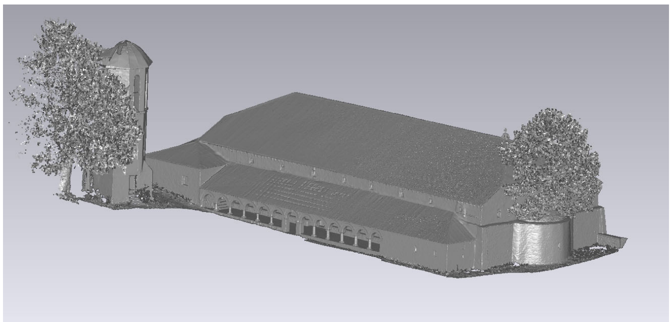


Figure 7: The scanner stations diagram

Respectively, for the analysis of the collected laser scanning observations, the *Cyclone* and *Geomagic* software suite were used for the registration of point clouds, based on special reflective targets whose coordinates had been measured by field surveying, and for the production of a surface 3D model of the exterior and interior of the church. The number of necessary triangles to describe the outside surface was approximately 3,000,000 while for the interior the number was larger, i.e. more than 13,000,000 triangles. Samples of the constructed models are presented in Figure 8.



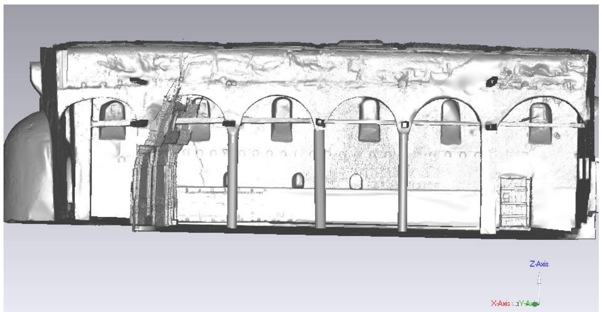


Figure 8: The 3D models constructed from the laser scans; Left: exterior and Right: interior

The creation of a textured model followed, using TOPCON Image Master software, with input (a) the 3D model, (b) the high resolution digital images, that cover all its surfaces, and (c) control points for the determination of the exterior orientation (registration) of each image, which are either the reflector targets or characteristic points of the 3D model. Each registered image was draped in part of the model, a photo-mosaic was created, by cutting the common parts between the images, and finally the textured model was produced in VRML format. For the interior of the church the creation of a textured model was not possible, due to the number of objects (religious operational objects, seats, etc) that are shown in the images and hide the walls.

RESULTS

Adjustments of the GPS Measurements

All GPS data used in this study were processed using Trimble's Total Station Control software. The GPS data collected from the local GPS control network were analyzed in combination with the GPS data downloaded from the five EUREF stations, together with precise orbital ephemerides and satellite clocks information obtained through the web from any of the data dissemination centers of the International GNSS Service (IGS), e.g. http://igscb.jpl.nasa.gov/components/prods_cb.html. This analysis gave us the ability to obtain precise geodetic measurements with dense spatial sampling and to determine the sought absolute and relative coordinates of the six local GPS stations with high levels of accuracy and repeatability. Some indicative results are the achieved accuracies of the six stations of the local GPS control network, as computed UTM coordinates using a fiducial network adjustment with all available data from both the local stations and the data available from the 5 EUREF stations, in the second period of observations (29, 30 and 31 July 2009): σ_N = 3.7 – 4.1 mm, σ_E = 3.3 -5.1 mm, σ_h = 5.3 – 8.6 mm. Results of the same level of accuracy, also, achieved for the other observation periods.

Table 1: Between observation periods differences in the local coordinates (UTM Northing and Easting) and the geometric heights of the stations of the local GPS control network

	Coordinate differences June - July			Coordinate differences June - September			Coordinate differences June - October		
Point	δN (m)	δE (m)	δh (m)	δN (m)	δE (m)	δh (m)	δN (m)	δE (m)	δh (m)
S1	-0.012	-0.004	-0.009	0.004	0.005	-0.003	-0.004	-0.003	-0.037
S2	-0.015	-0.009	-0.004	-0.003	-0.007	0.007	-0.009	-0.013	-0.016
S3	-0.012	-0.006	0.000	0.001	-0.001	0.009	-0.008	-0.005	-0.016
S4	-0.020	-0.012	-0.005	0.001	-0.008	0.005	-0.007	-0.010	-0.013
S5	-0.014	-0.015	-0.007	0.000	-0.010	0.004	-0.012	-0.012	-0.012
S6	-0.018	-0.009	0.002	-0.005	-0.009	0.014	-0.015	-0.005	-0.001
Mean	**-0.015**	**-0.009**	**-0.004**	**0.000**	**-0.005**	**0.006**	**-0.009**	**-0.008**	**-0.016**
StDev	**0.003**	**0.004**	**0.004**	**0.003**	**0.006**	**0.006**	**0.004**	**0.004**	**0.012**
Min	**-0.020**	**-0.015**	**-0.009**	**-0.005**	**-0.010**	**-0.003**	**-0.015**	**-0.013**	**-0.037**
Max	**-0.012**	**-0.004**	**0.002**	**0.004**	**0.005**	**0.014**	**-0.004**	**-0.003**	**-0.001**

	Coordinate differences July - September			Coordinate differences July - October			Coordinate differences September - October		
Point	δN (m)	δE (m)	δh (m)	δN (m)	δE (m)	δh (m)	δN (m)	δE (m)	δh (m)
S1	0.015	0.010	0.006	0.008	0.002	-0.028	-0.007	-0.008	-0.034
S2	0.012	0.002	0.011	0.005	-0.004	-0.012	-0.006	-0.007	-0.022
S3	0.013	0.004	0.010	0.004	0.001	-0.016	-0.009	-0.004	-0.026
S4	0.021	0.005	0.011	0.013	0.002	-0.008	-0.008	-0.002	-0.019
S5	0.014	0.005	0.011	0.003	0.002	-0.005	-0.012	-0.002	-0.017
S6	0.013	0.000	0.012	0.003	0.004	-0.003	-0.009	0.004	-0.016
Mean	**0.015**	**0.004**	**0.010**	**0.006**	**0.001**	**-0.012**	**-0.009**	**-0.003**	**-0.022**
StDev	**0.003**	**0.003**	**0.002**	**0.004**	**0.003**	**0.009**	**0.002**	**0.004**	**0.007**
Min	**0.012**	**0.000**	**0.006**	**0.003**	**-0.004**	**-0.028**	**-0.012**	**-0.008**	**-0.034**
Max	**0.021**	**0.010**	**0.012**	**0.013**	**0.004**	**-0.003**	**-0.006**	**0.004**	**-0.016**

Table 1 shows the respective differences observed between the four observational periods, in the UTM plane coordinates (Northing and Easting) and the geometric heights of the local GPS control network stations. The mean differences noted in Table 1 are mostly due to the slight inconsistencies (biases) in realizing the 'exact same' reference system which is inherent in the modeling of the precise GPS orbits used for the data adjustment. When these biases are removed, the observed minimum differences range between -3 mm and -21 mm, and respectively the maximum differences range between 3 mm and 21 mm.

Also from these adjustments, we were able to also compute the differences between the four observing periods in the components of the inter-station vectors formed between the pairs of the six points of the local GPS, both in terms of differences in their Cartesian components (δΔX, δΔY, δΔZ) and in their Northern and Eastern plane (UTM) coordinates (δΔN, δΔE) and geometric heights (δΔh). These are not given in this paper due to space limitations. However, it suffices to mention that the noted variations in the components of the inter-station vectors, between two successive observational periods, were at the sub-cm level.

Determination of deformations

Observations and measurements for all four periods were performed in an identical way, in order to ensure maximum accuracy and reliability on one hand and comparability of the results on the other. The adjustments were also performed in identical ways, obviously for the same reasons. Station 2 was selected to be considered fixed as was the direction of S2-S3 sight. The estimated uncertainties for the horizontal and vertical angles were $\pm 8^{cc}$ and for distances ±2mm; the resulting uncertainties for the coordinates determination were σ_X=±0.2 - ±2mm, σ_Y=±0.3 - ±2mm and σ_Z=±0.1 - ±1mm. The conclusions presented here are not the final ones, as additional adjustments are currently performed and new observation periods will be accomplished for the completion of the scheduled measurements within the year.

Table 2: Planar (Δr) and vertical (ΔH) displacements of the network points

	Sep - Oct 2009		**July - Sep 2009**		**June - July 2009**	
Point	**Δr (mm)**	**ΔH (mm)**	**Δr (mm)**	**ΔH (mm)**	**Δr (mm)**	**ΔH (mm)**
Points outside the Church						
S1	4.8	+10.4	16.1	-2.9	18.3	+1.8
S2		0	0	0	0	0
S3	3.1	6.7	1.0	-7.0	1.2	-1.6
S4	2.7	-4.9	4.6	-1.5	14.3	1.5
S5	5.9	-3.7	8.6	0.1	18.3	-1.8
S6	8.5	-4.9	12.2	-3.2	15.6	-2.2
Points inside the Church						
S7	7.1	-9.3	9.4	5.0	14.6	-0.3
S8	7.2	-2.6	14.1	1.6	6.3	-1.9
S9	13.5	-2.7	17.9	2.2	23.9	-3.1
S10	3.8	-3.2	6.7	2.4	18.2	-3.4
S11	10.0	-4.0	23.1	1.0	25.4	-2.3
S12	4.3	-1.6	12.6	0.1	18.4	-5.2
S13	6.2	-3.4	9.2	1.6	19.7	-2.5
S14	7.9	-4.5	15.7	-0.1	12.2	0.9
S15	8.6	-4.0	10.4	0.9	11.5	-1.9

In Table 2 the planar (Δr) and vertical (ΔH) displacements of the 15 network points are presented, as differences of their positions between observation periods. Relevant results have been derived for all control points in the interior and the exterior of the church. Their analysis shows that

- In the first epoch interval (June - July 2009) the planar displacements are of the order of 10mm with a direction mostly to the northeast for the northern and the eastern façade of the church as the southern façade appears to move to the south and the western one to a northwestern direction. Vertically, the tendency is a lowering of the points on the walls by a considerably smaller amount, at the order of 5mm.
- During the second epoch interval (July – September 2009) the planar displacements are again more or less of the order of 10mm. Maximum displacement in elevation difference observed is 5mm, but in an upward direction this time. The direction of the horizontal displacements for all

external façades is to the northwest, while all the internal points show movement to a northeastern direction.

- During the third epoch interval (September – October 2009) the planar displacements are confined to 5mm while the points retreat vertically up to 6mm. The direction of all point movements shows a west-northwest tendency.

The above described displacements show the evolution of a periodic phenomenon, which calls for more observation periods in order to draw safer conclusions.

Presentation of the results

The presentation of the deformations in such a complicated object, with so many observation periods, is critical for comprehending the results and drawing of conclusions useful to the specialized architects and civil engineers, who will use them to select the most appropriate method for their interventions in order to preserve the monument. Obviously, the spatial visualization of the results-deformations in each network or control point is necessary (Georgopoulos et al., 2007). Firstly, two dimensional vector and raster drawings of horizontal and vertical sections, giving special emphasis on the way of presenting the deformations between the observation periods were made. In Figure 9 a horizontal section close to the ground floor of the church is shown, using an appropriate presentation of the deformations of the 15 network points.

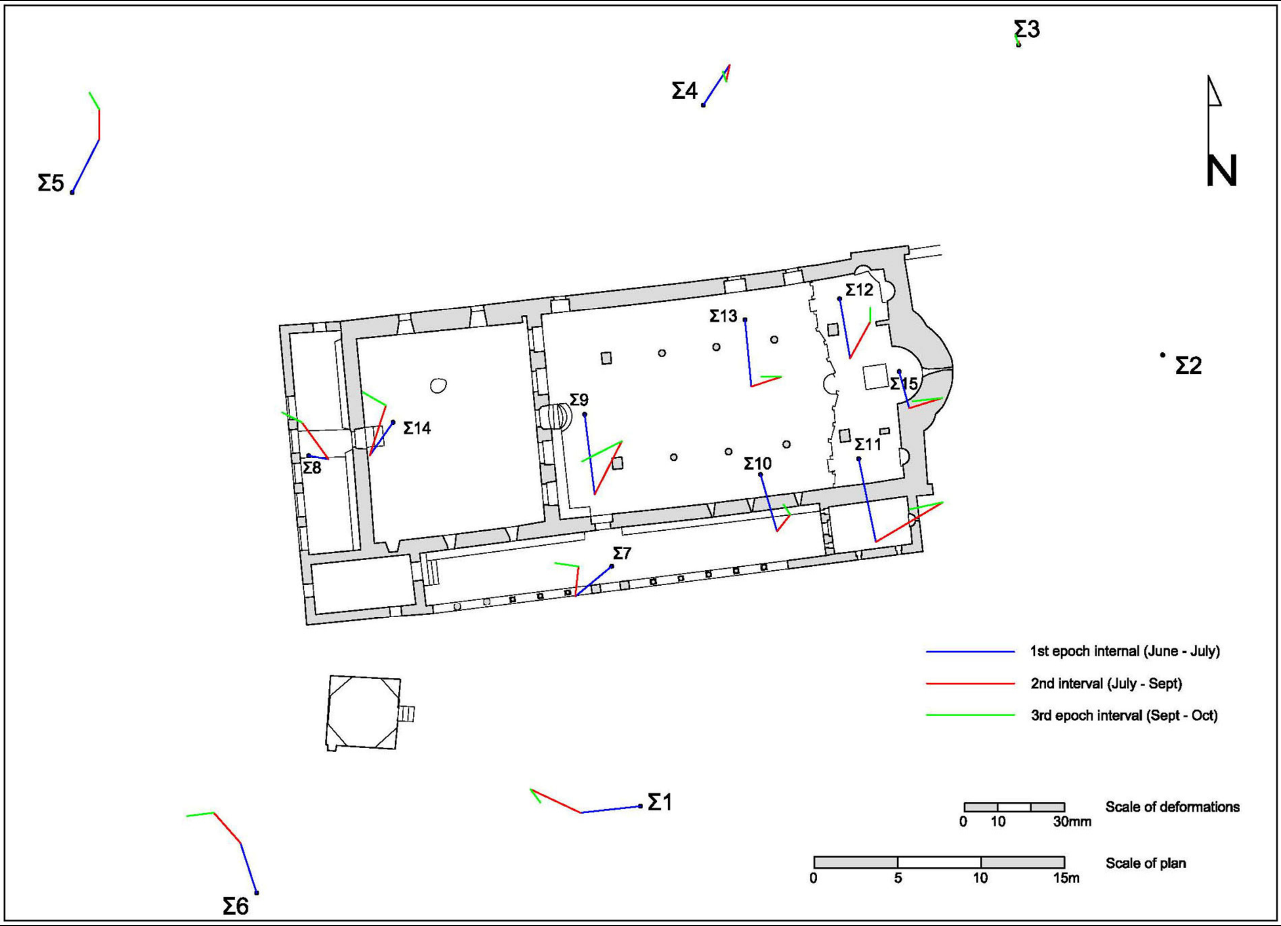


Figure 9. Displacements of the 15 network points between the successive observation periods

Further, the use of the 3D textured model of the church for a better and more representative visualization of the deformations was decided. As already mentioned, a 3D laser scanning survey of both the exterior and the interior of the building was conducted in order to create a 3D model of the church on which the differences in the coordinates of the control points from the repeated geodetic measurements would be plotted and the dimensional variations between the building's former and previous states would be further studied.

The exterior surfaces of the 3D model were rendered using the high resolution registered digital images and the points on the walls were positioned on the VRML version of the 3D model, using little spheres, in the Geomagic software. For the representation of the displacements, linear vectors were used with different colour for each period interval and with an exaggerated scale, e.g., 50 times. The result is shown in Figures 10 and 11. In Figure 10 part of the rendered 3D model of the external northern façade is shown, with several point positions and the displacements between the observation periods. In Figure 11 an enlargement of an area of the same façade is presented and the displacements between observation periods are also shown. It is remarkable how similar these displacements are.

Figure 10: Part of the external northern façade

Figure 11: Detail of the northern façade

EVALUATION OF RESULTS

In this paper the results of combined geodetic and laser scanning measurements for monitoring of the deformations of more than 90 control points located on the exterior and interior walls of a post Byzantine church, that has suffered serious structural damage, are presented. The aimed accuracies for the determination of deformations were of the size of a few millimeters, while it is known that the surrounding area of the church, to a great extent, is not stable. The appropriate methodology was applied using GPS and classical geodetic measurements.

It has been shown so far that the accurate measurements and their adjustment prove that there are definite deformations in the surrounding area but also on the church building itself. Sometimes these displacements are at the order of 1-2 centimeters, but the general tendency is that these deformations tend to follow a periodic cycle. The detailed analysis of these data is still in progress and a final analysis and interpretation of the results is still awaiting additional series of GPS and surveying measurements planned to take place in the spring of 2010, following the winter period when it remains to be seen whether one would observe the adverse effects of heavy snowfalls in the area and the impact of the snow and soil water loads are likely to cause any seasonal stresses, deformation and displacements on the building of the church and the surrounding terrain.

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