SECOND EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING AND SEISMOLOGY, ISTANBUL AUG. 25-29, 2014

SEISMIC SAFETY AND STABILITY OF EXISTING STRUCTURE OF ST. MARY PERIBLEPTOS CHURCH IN OHRID

Veronika SHENDOVA[1], Zivko BOZINOVSKI[2], Roberta APOSTOLSKA[3], Aleksandar ZLATESKI[4], Goran JEKIKJ[5]

ABSTRACT

Within the frames of the project on *Restoration and Revitalization of St. Mary Peribleptos Monastic Complex in Ohrid*, financed by the Fund for Protection of Cultural Heritage at the USA Embassy in Skopje and coordinated by the Cultural Heritage Protection Office of the Republic of Macedonia, the Institute of Earthquake Engineering and Engineering Seismology (UKIM-IZIIS) has carried out an analysis of the stability of the existing structures of the individual units of the complex for the effect of gravity and seismic loads. A multi-disciplinary approach has been applied that involves definition of the seismic potential of the location, detailed experimental non-destructive investigations of the existing state of the structures and the built-in materials as well as further analysis of the structures for gravity and seismic effects. The paper presents the results from the performed analyses of the church of St. Mary Peribleptos in its existing and retrofitted state.

INTRODUCTION

The town of Ohrid with its surrounding represents a mosaic of Orthodox religious structures which, according to their function represent not only spiritual but also cultural and social medium of modern living. It is therefore that further existence and protection of these structures in the past and in the present has been and is of particular interest for society and science.

The monastic complex of St. Mary Peribleptos in Ohrid dates back to the beginning of the XIII century, (Fig.1). Dominant and central place in the monastic complex has the church of St. Mary Peribleptos (A). In addition to the church, the complex also includes a belfry dating back to 1923, (C), a gallery of icons in the renovated structure of the formerly existing horse stable, (B), the north lodgings (D), the building of the former Museum of Slavic Literacy, the grave of the well-known national revival leader Grigor Prlichev (1830-1893), three cypresses and a fountain built in 1935.

The church of St. Mary Peribleptos is situated on the plateau south from Upper Gate, in the old part of Ohrid (Fig. 2). Due to its extraordinary architecture, the church is considered to be among the most important medieval monuments, while the fresco paintings in its interior representing the first authentic work of fresco painters Mihailo and Evtihij, are among the most important achievements of the fine arts of the XIII century in Macedonia. According to the inscription above the west entrance leading to the narthex, the church was erected in 1295.

[1] Professor, PhD, Institute of Earthquake Engineering and Engineering Seismology, UKIM-IZIIS, SS Cyril and Methodius Unvesrsity, Skopje, Republic of Macedonia, veronika@pluto.iziis.ukim.edu.mk
[2] Professor, , PhD, UKIM-IZIIS, Skopje, zivko@pluto.iziis.ukim.edu.mk
[3] Professor, , PhD, UKIM-IZIIS, Skopje, beti@pluto.iziis.ukim.edu.mk
[4] Ass., Researcher M.Sc., UKIM-IZIIS, Skopje, azlate@pluto.iziis.ukim.edu.mk
[5] Ass., M.Sc., UKIM-IZIIS, Skopje, jekic@pluto.iziis.ukim.edu.mk

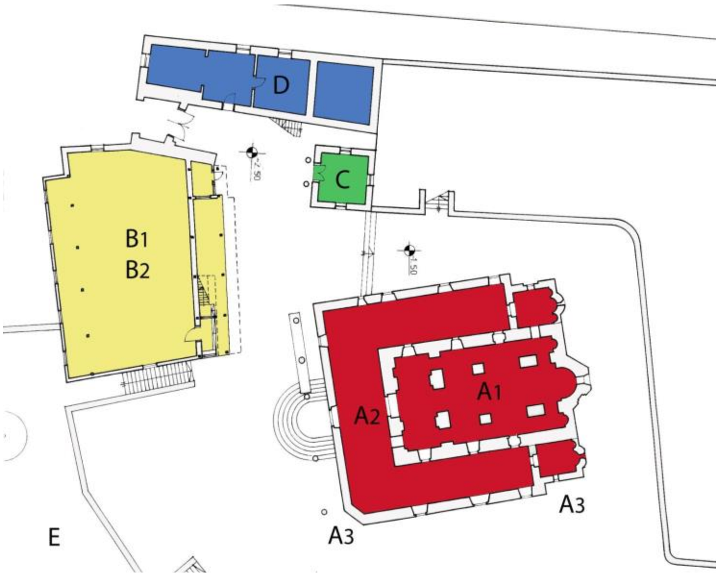


Figure 1. Monastic complex St. Mary Peribleptos in Ohrid

Figure 2. Southeast facade of the church of St. Mary Peribleptos in Ohrid

As part of the project on restoration and revitalization of the monastic complex financed by the Ambassador's Fund for Cultural Preservation at the USA Embassy in Skopje and coordinated by the Cultural Heritage Protection Office of the Republic of Macedonia, detailed analysis of the stability of the existing structural systems of the three individual structural units of the monastic complex "St. Mary Preibleptos" (the church of St. Mary Peribleptos, the Belfry and the Monastic lodgings facilities) for gravity and seismic effects has been provided. A multidisciplinary integrated approach, which has been developed at UKIM-IZIIS based on decades of experience in the field of seismic protection of cultural historic monuments, has been applied. It involves the following:

- Detailed inspection of the entire existing technical documentation;
- In situ insight and inspection of the structure;
- Identification and analysis of existing and variable gravity loads on the bearing structure;
- Definition of seismic potential of the site;
- Definition of design criteria for the seismic safety and stability of the structure;
- Investigation of dynamic characteristics by Ambient Vibration Method;
- Testing of physical-mechanical characteristics of built-in materials

The knowledge gained through these investigations has been used in the individual phases of further analyses for the purpose of obtaining more accurate input parameters and carrying out a reliable structural analysis and further evaluation of its seismic safety and stability, which is directly connected with the measures to be taken for their consolidation and possible strengthening in future.

Existing state of St. Mary Peribleptos Church: From structural aspect, the church represents a single dome structure with a plan in the form of an inscribed developed cross (Fig. 1, A1). The bearing walls constructed of massive masonry consisting of limestone and brick in lime mortar represent a typical Byzantine style with two faces. The central dome rests, through an octagonal tambour, on four rectangular columns in the central part of the naos. On the east side, there is an apse, which is semicircle inside and three sided on the outside. In the west part of the church, there is a narthex with a blind dome in the central part. There are several entrances: two on the west and the north side of the narthex, two on the north and the south side of the naos and one on the west wall.

In the XIV century, the church was enlarged by small chapels added on the north and the south side of the apse, as single-nave premises with an apse and openings – niches – in the diaconicon and the prosthesis. Around the church, there was established a monastic complex, while in the vicinity of the church, there was erected an archiepiscopal palace, which disappeared in a fire in the midst of the XIX century. The porch on the southwest and the northern side was constructed in the course of the last phase of construction of the church (Fig. 1, A2) when openings were made in the existing bearing walls in the narthex and the naos. The walling of the inner windows of the original walls of the church was not carried out in a way that the wall can be considered as continuous without opening. In the analyses, these are considered as openings in the walls.

Conservation works on preservation of the architecture and the structure of the monument were carried out in the course of the 1950-ties and in the midst of the 1980-ties of the last century. During the last interventions, the cracks at characteristic places in the vaulted parts due to loss of function of the timber ties, were repaired (Fig. 3).

Figure 3. Existing cracks in the church repaired in 1986

Then, steel ties were inserted in the central area below the dome and anchored in the wall mass of the bearing walls. The reinforced concrete slabs placed over the vaults, the diaconicon, the prothesis, the reinforced concete rings in the base of the tambour (Fig. 4, 5) as well as the cement layer and tar paper over the dome were constructed in 1986 in compliance with the then existing level of technical knowledge and represent massive structural interventions.

This type of work is certainly “incorrect” both in terms of “historical respect” for the building and its original construction techniques and in terms of the actual capacity to prevent leaks and protect the decorations inside the church. This was confirmed by the massive leaks which affect both the vault of the north chapel and the far eastern end of the north wall of the naos, which is due to original construction errors but also to the use of unsuitable materials and the lack of adequate maintenance. Consequently, one of the main task within the project was to consider the possibilty of removing of all the reinforced concrete and cement layers.

From structural aspect, it can be concluded that the existing bearing structure is in stable conditions with minimal structural damage to the roof vaulted parts dating back prior to the restoration in 1986. These have not been enlarged since then. The steel ties of the main vaults were appropriately incorporated in 1986 and are still functioning. In accordance with the present knowledge, reinforced concrete should not be applied in strengthening and protecting monuments of this type. However, although the reinforced concrete elements are constructed as free standing and separated from the existing walls by an air space, (Fig. 5), due to presence of reinforcement, these cannot be removed without the use of equipment that will cause vibrations and will most probably damage the existing masonry.

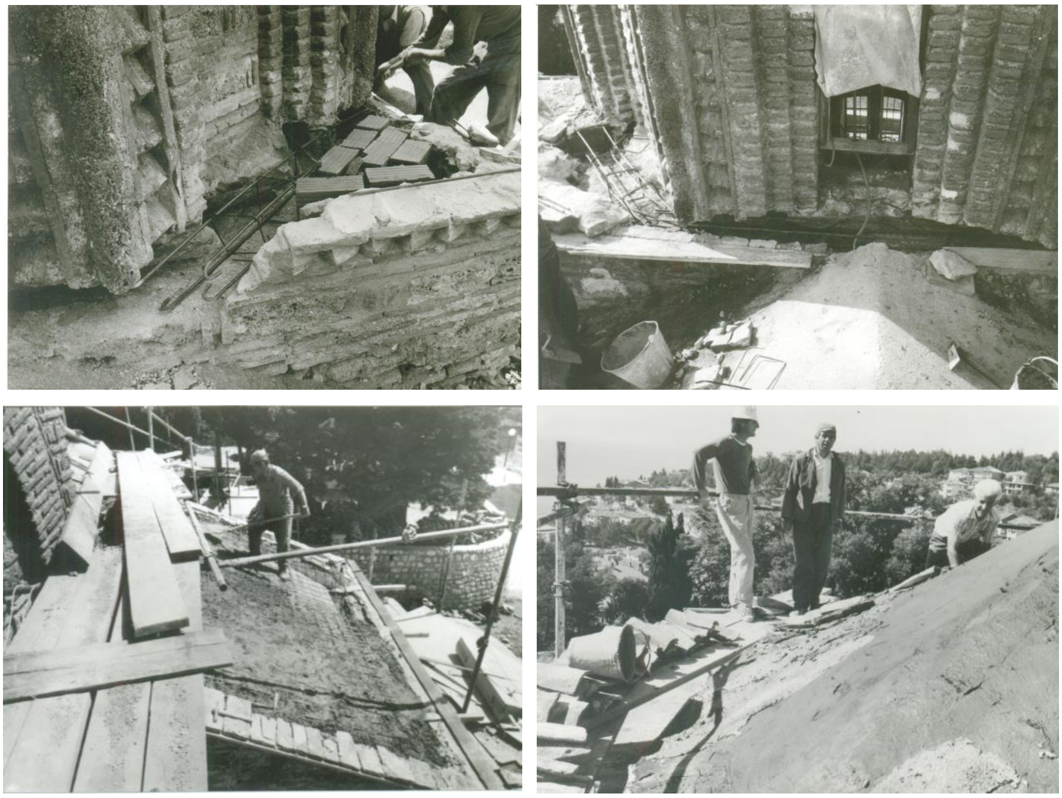


Figure 4. Incorporation of reinforced concrete elements (tambour, vault, dome) (1986)

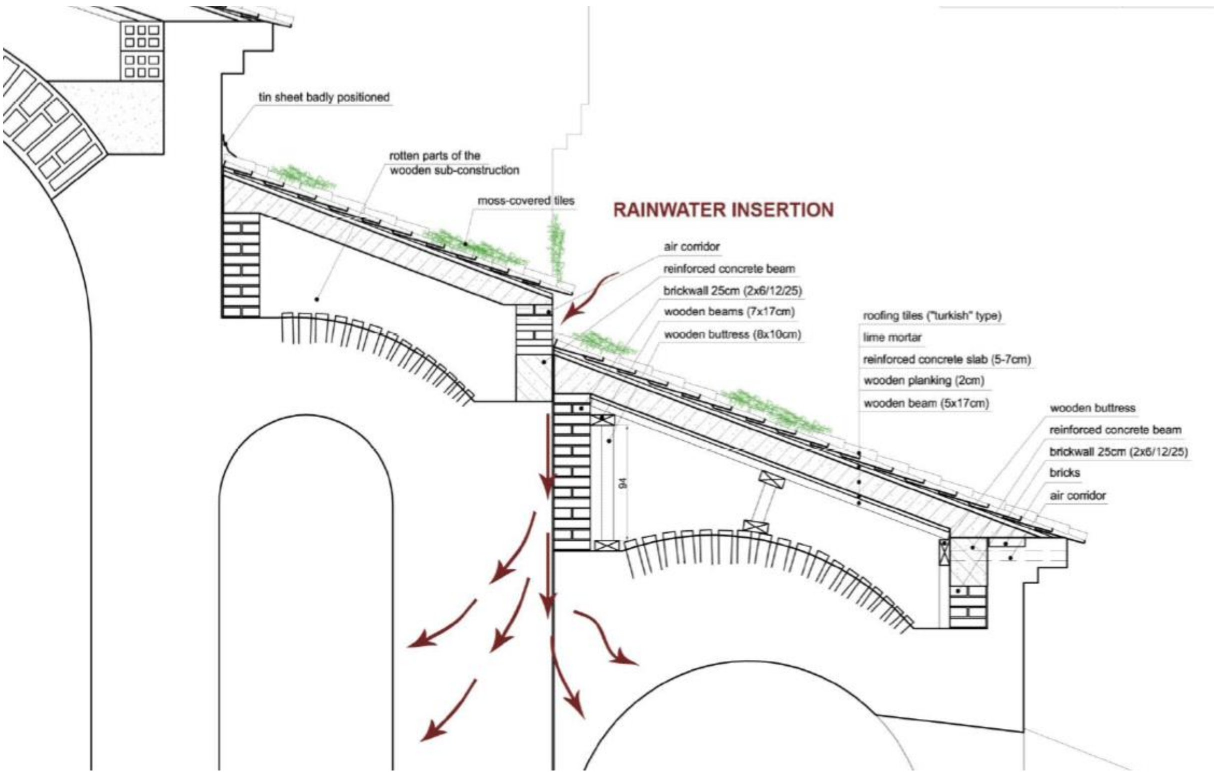


Figure 5. Construction details of the roofing over prosthesis and northern chapel

Still, most of the visible damages to the fresco paintings have not occurred as a direct consequence of placement of the reinforced concrete slabs, but as a result of the inappropriate technical solution and incorporation of architraves, damages to facade expansion joints and non-existence of a drainage system, (Fig. 5). Based on the above, it was concluded that the present conditions of the church structure show that it is stable and safe under gravity serviceability loads.

PREVIOUS INVESTIGATIONS OF THE SITE AND THE STRUCTURE

Seismic Potential of the Site: The investigations carried out for the purpose of defining the seismic potential of the site for the needs of analyses of stability of St. Mary Peribleptos church in Ohrid under seismic effects involved field, laboratory and office studies performed in compliance with the latest achievements in the field of earthquake engineering, (Sheshov et al., 2011, Geotehnika, 2011). The main concept of the applied procedure has been modeling of the local soil effects by field geophysical studies and performing the so called site response analysis of representative geodynamic models by using the input data on the seismic hazard, the results of which enable correct definition of the input seismic parameters for dynamic analysis of the structure. Based on the analyses of the seismic risk, the seismic parameters for seismic analysis has been defined corresponding to the serviceability period of the structure and the seismic risk level, (Table 1). Taking into account the amplitude-frequency effects of the local soil conditions as well as the frequency properties of the structure, recommended to be used in the dynamic analysis where the following earthquake records:

- *Robic N-S*, recorded during the Friuli earthquake (Italy) of 15.09.1976 with M=6.1.
- *El Centro N-S*, selected as a representative excitation for an earthquake with a wide range of frequencies (USA, 1940)
- *Ulcinj N-S*, recorded during the Montenegro Earthquake of 15.04.1979 with M=7.0.

Table 1. Seismic parameters for analysis of the structure

Serviceability period of the structure	Seismic risk level (%)	Maximum acceleration a_{max} (g)
100	30	0.28
	10	0.31

The performed site analyses point to several important issues. The predominant periods of the site are in the range of T=0.13-0.15s, meaning that there is no danger as to occurrence of resonance effects. The structure is founded in a terrain of considerable bearing capacity and compactness which is considered favourable during future earthquakes wherefore considerable soil displacements are not expected.

Seismic Safety Criteria: In practice, if the seismic parameters of a site are available, the safety of the structure is defined for different levels of seismic effects through linear and nonlinear deformations. For the St. Mary Peribleptos church as a structure of the first category, the following design seismic safety criteria have been defined:

- *Level I:* Under more frequent earthquakes of a lower intensity, the dynamic behavior of the structure during an earthquake must not allow vibrations that cause damage to both structural and secondary, nonstructural elements (elastic behavior, ductility demand $\mu<1$);
- *Level II:* Under earthquakes of a greater intensity, i.e., under the design earthquakes, the structure should generally remain in the linear range of behavior, with possible limited nonlinear deformations in individual elements of the system, meaning limited stiffness deterioration and energy dissipation (initial nonlinear behavior, ductility demand μ=1.5-2.0);
- *Level III:* Under the maximum expected earthquake effects, the structural and nonstructural elements of the structure are deep in the nonlinear range of behavior, while the stiffness and the resistance of the structure are considerably reduced. However, such earthquakes must not disturb completely the stability of the bearing structure, i.e., the damage should be repairable (nonlinear behavior, ductility demand μ=2.0-2.5).

Defined by these parameters is the required behavior of the bearing structure under dynamic effects caused by an earthquake, i.e., the level of the designed seismic protection of the structure.

Investigation of Dynamic Characteristics of the Structure by the Ambient Vibration Method: This experimental in-situ method is widely applied for the needs of definition of dynamic characteristics of existing structures. The method is based on measurement of structural vibrations caused by ambient forces. This is a very fast and relatively simple procedure that does not disturb the normal functioning of the structure.

The church was measured in 18 points located on the roof, tambour, windows, beams and ground floor in both orthogonal directions. Additional measuring points were selected on the walls of the chapels as well as on the wooden beams connecting the walls of the portico and the walls of the church, in order to see if they have the same dominating frequencies, (Fig. 6, Krstevska et al., 2011). The obtaiined dominant frequencies are well expressed and they are presented in Table 2 together with the corresponding damping coefficients.

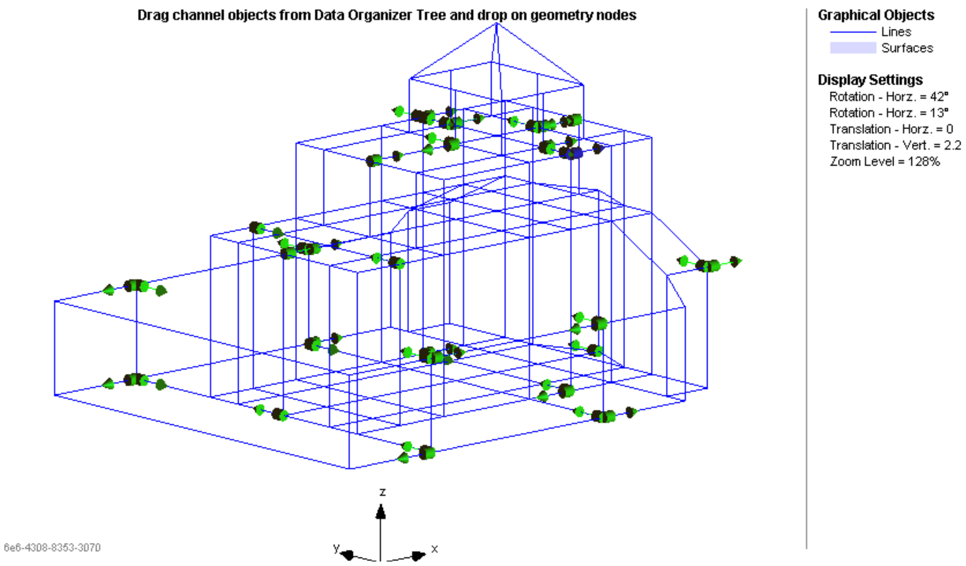


Figure 6. Spatial presentation of the test set-up for the church, the chapels and the portico

Table 2. Dominant frequencies and damping coefficients

Mode	Frequency (Hz)	Damping ration (%)
FDD Mode 1	5.08	3.80
FDD Mode 2	6.45	2.20
FDD Mode 3	7.60	1.80
FDD Mode 4	8.80	2.30
FDD Mode 5	10.9	1.30

The resonant frequency in lateral (transversal) direction for the church is 5.08Hz, in longitudinal direction is 6.45Hz and for torsion it is f=7.6Hz as well f=10.9Hz, while the frequency f=8.8Hz belongs to lateral vibation of the longitudinal walls of the portico. The church itself has clear and well expressed natural frequencies and corresponding mode shapes of vibration. The church complex (the church, the paraklisses and the portico) works mostly together, (Fig. 7). as a result of the existing wooden beams connecting the walls of the church and of the portico, but also independently at the higher frequencies because of existing separation joints. The values obtained for damping ranging within 1.3-3.8% are relatively lower than the usual ones for this type of structures which speaks for itself about the compactness of the original masonry.

The parameters obtained with these investigations have been used in the subsequent analysis for verification of the strength-deformability characteristics of the structures and calibration of the computational mathematical models.

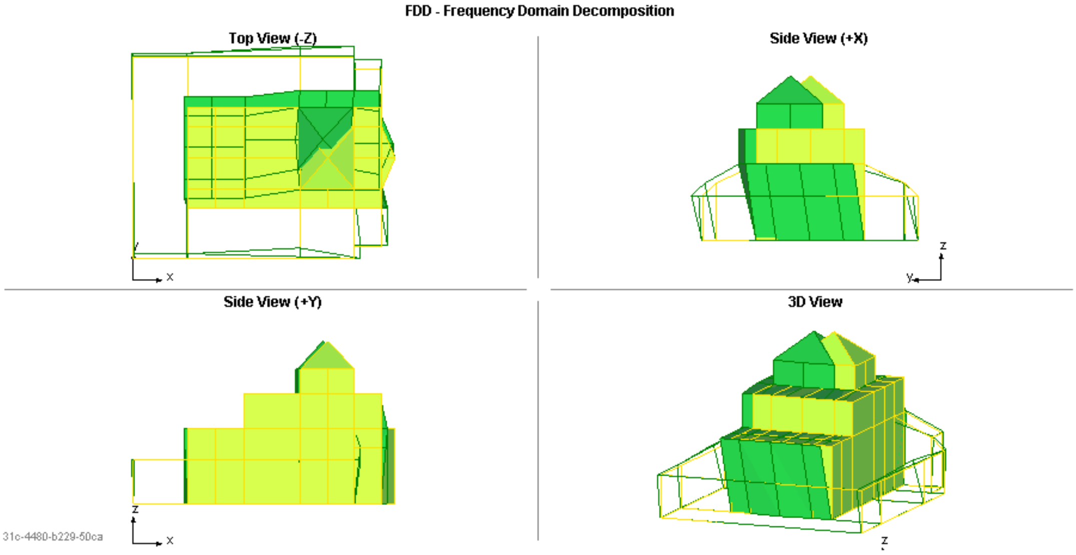


Figure 7. Mode shape of vibration at a frequency of 5.08Hz

Testing of Physical-Mechanical Characteristics of Built-in Material by Specimens taken from the Foundation Walls of the Church: Considering the fact that one of the most important parameters of behavior of masonry structures are the physical-mechanical characteristics of the mortar and for the purpose of, as precise as possible, definition of the static and seismic stability of structures, there have been performed laboratory tests on three specimens of built-in mortar taken from the foundation walls of the church dating back to the XIII century. The tests involving the grain size distribution, the bulk mass, the water saturation and the compressive strength have been performed at the laboratory of the Institute for Testing Materials and Development of New Technologies, (UKIM-ZIMAD 2012). Based on the statistic processing of the results obtained for the three series of tested mortar, the following mean values have been obtained:

- Bulk mass: 1.78-1.87 g/cm^3
- Water saturation: 10.4-20.6 %
- Compressive strength: β^{min}=2.99 MPa; β^{sr}=4.33 MPa; β^{max}=5.67 MPa;
- Most probable compressive strength: β=4 MPa

The obtained values for the compressive strength of minimum 2.99 MPa speak for themselves about the extraordinarily high strength characteristics of the lime morter which contributes to greater stability of the structure.

ANALYSIS OF ST. MARY PERIBLEPTOS CHURCH STRUCTURE

The knowledge gained through the above mentioned investigations has been used in the individual phases of the analyses for the purpose of obtaining more accurate input parameters and carrying out a reliable analysis of the structures, (Shendova et al., 2013). Generally, the procedure consisted from: (i) elastic – static analysis carried out on a 3D FE mathematical model, (ii) analysis of elements up to ultimate state of strength, deformability and ability of the bearing elements and the system as a whole to dissipate seismic energy and (iii) analysis of the dynamic response of the system for actual seismic effects with intensity and frequency content that are expected on the considered location.

Based on the knowledge obtained from available documentation, visual inspections and particularly testing of the church structure by use of the ambient vibration technique, it has been concluded that the church in its original form dating back to the XIII century represents a structural entity by itself, with separate behavior of the later enlargements. It is therefore that the subject of further analyses has been the original church from the XIII century.

3D Finite Element Method Analysis: For the existing church structure, a three-dimensional static and equivalent seismic analysis has been performed by use of the SAP 2000 (Structural Analysis Program, UC Berkeley, California) computer programme. For the needs of this analysis, a mathematical model (Fig. 8) has been formulated whereat SOLID elements have been used for modelling of the walls, while SHELL elements have been used for modelling of the tambour, the pendentives, the vaults and the dome. The mathematical model refers only to the masonry part of the church considering that the subjects of analysis are the original bearing elements. Consequently, the effect of the reinforced concrete elements added during the conservation works in 1986 has been considered through forces concentrated at the nodes of the roof surfaces that, in accordance with the possibilities offered by the programme, are included in the computation of the total mass of the mathematical model.

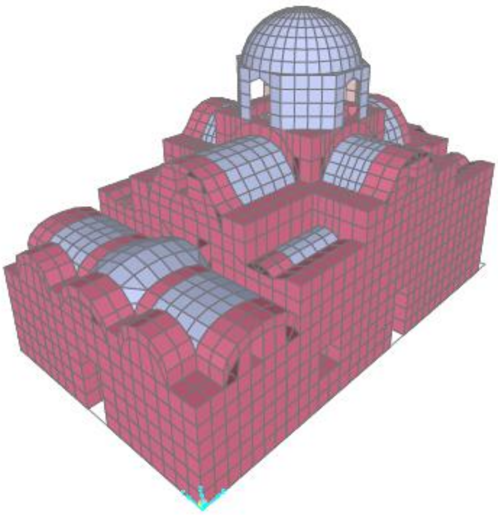


Figure 8. 3D mathematical model of the existing church structure

For such modelled structure, analysis under dead weight effect and equivalent seismic forces computed according to the regulations in Republic of Macedonia has been made, whereat the obtained seismic coefficient at the base has been K = 0.15, i.e.:

$$K=K_0\, K_s\, K_d\, K_p = 0.15 \qquad (1)$$

whereat:

K_0 – category of the structure, (I category) - $K_0 = 1.50$
K_s – seismicity of the terrain, (MCS VIII) - $K_s = 0.05$
K_d – dynamics, (soil category I) - $K_d = 1.00$
K_p – ductility, (masonry structures) - $K_p = 2.00$

By varying the elastic characteristics of masonry for the purpose of obtaining the values of the natural periods similar to the experimentally obtained ones, calibration of the mathematical model of the church structure has been made. For such modelled structure (with elasticity modulus E=1000 MPa), natural periods of vibration of T^{lat} =0.189s, T^{long}=0.181s, T^{tor}=0.149s have been obtained. The total displacements due to seismic force in longitudinal and transverse direction amount to δx=0.114cm and δy=0.156cm, respectively and these are lower than the maximum ones allowed by the regulations δ max= H/600 = 1.73cm. The results from such analysis provide a clear insight into the static quantities of the individual structural elements and their comparison with the strength characteristics of the built-in materials. The similarity between the analytically (T^{lat}=0.189s) and experimentally (T^{lat}=0.196s) obtained fundamental dynamic characteristics speaks for itself about the realistic modelling of the church structure and the reliability of the performed analysis.

Analysis of Bearing and Deformation Capacity: To define the real strength and deformability characteristics depending on the quantity and quality of the built-in material, the methodology has been developed by which, for each individual element of a story, are obtained the displacement and the transversal force at yielding, δy and Qy, the maximum displacement δu and corresponding ultimate transversal force Qu at maximum displacement, i.e., the initial stiffness and the stiffness after the yielding point. In that way, the Q-δ relationships are obtained for each element and for each storey taken separately, whereat the deformation capacity of the storey is also defined through the displacement capacity δu and the capacity of displacement ductility defined as μ=δu/δy.

Applying this methodology, the structure of the St. Mary Peribleptos church has been analysed. The input parameters for the ultimate strength characteristics fc and ft, have been adopted based on previous laboratory tests of physical mechanical characteristics of the mortar. For the elasticity and shear moduli, there has been used the knowledge obtained from the 3D FE analysis after performed identification with the experimentally obtained dynamic characteristics.

The individual bearing walls in both orthogonal directions have been modelled as fixed at the base with cumulative axial forces due to calculated dead weight and associated vertical load. The structure has been considered with a total of four levels, a total weight of 16193 kN and a total seismic force at the base amounting to 2423 kN. In the analysis of the bearing and deformability capacity, the following input parameters have been adopted:

- Elasticity modulus E= 1 000 000 kPa
- Shear modulus G= 250 000 kPa
- Ultimate compressive strength fc= 2000 kPa
- Ultimate tensile strength ft = 200 kPa

Table 3 shows the cumulative results from the analysis of the bearing capacity (Qu) and deformability, (δu, μu).

Table 3. Bearing and Deformation Capacity, existing state of the church

Level	Weight G [kN]	Capacity Qy [kN]	Stiffness K [kN/m]	Displacement at yielding δy [cm]	Ultimate displacement δu [cm]	Ductility capacity μ [δy/δu]
x-direction						
4	555.	240.	4031.	0.06	0.08	1.33
3	1532.	684.	6073.	0.11	0.28	2.54
2	3126.	1178.	8632.	0.21	0.24	1.14
1	10980.	3379.	3471.	0.71	1.76	2.48
$\sum_{1-4}$	16193.	20.8%				
y-direction						
4	555.	254.	2403.	0.11	0.12	1.10
3	1532.	583.	5097.	0.11	0.27	2.45
2	3126.	1485.	8742.	0.17	0.30	1.88
1	10980.	2973.	3494.	0.81	1.65	1.94
$\sum_{1-4}$	16193.	18.3%				

Dynamic Analysis of the Structure: For the structure of the St. Mary Peribleptos church, a nonlinear dynamic analysis has been performed applying modelling by concentrated masses that assumes concentration of distributed structural characteristics at characteristic levels. The bilinear hysteretic model obtained from the Q-δ storey diagrams has been applied as input parameter. To obtain the dynamic response, there have been used seismic parameters defined for the site on the basis of performed geotechnical surveys, i.e, three different types of earthquake (El Centro N-S, Ulcinj N-S, Robic N-S) have been used with design and maximum expected acceleration of 0.28g and 0.31g, respectively. As a result of the dynamic analysis, storey displacements, i.e., ductilities required by the earthquake are obtained and these should comply with the defined design criteria.

Table 4 shows the results from the analysis of the dynamic response of the structure for the defined seismic parameters, whereat the required relative storey displacements (δmax) and the required ductility (μ=δmax /δy) for the individual levels, have been obtained.

Table 4. Required maximum displacement and ductility, existing state of the church

Storey	Required δ_{max} (cm)					
	Ulcinj N-S		Robic		El Centro	
	0.28g	0.31g	0.28g	0.31g	0.28g	0.31g
x-direction						
4	0.051	0.052	0.029	0.031	0.044	0.047
3	0.120	0.125	0.070	0.074	0.106	0.114
2	0.186	0.191	0.115	0121	0.176	0.188
1	1.619	2.027	0.880	0.972	2.132	2.533
y-direction						
4	0.091	0.095	0.051	0.398	0.085	0.090
3	0.155	0.169	0.087	0.643	0.138	0.145
2	0.190	0.197	0.120	0.920	0.170	0.176
1	2.915	3.123	0.861	0.337	2.754	3.109

Storey	Required ductility $\mu=\delta_{max}/\delta_y$					
	Ulcinj N-S		Robic		El Centro	
	0.28g	0.31g	0.28g	0.31g	0.28g	0.31g
x-direction						
4	0.842	0.872	0.480	0.513	0.735	0.782
3	1.031	1.078	0.603	0.642	0.911	0.979
2	0.901	0.927	0.559	0.589	0.852	0.911
1	2.704	3.282	1.245	1.374	3.430	3.998
y-direction						
4	0.802	0.843	0.449	1.094	0.755	0.795
3	1.164	1.272	0.653	1.426	1.036	1.088
2	0.839	0.870	0.527	1.897	0.748	0.775
1	3.639	3.899	1.075	1.000	3.438	3.881

Conclusions from the Performed Analysis: The elastic and seismic analysis performed by a static equivalent method on a 3D FE model has shown that the total displacements at the dome level due to seismic force amounting to 15% of the total weight of the structure are lower than those allowed by the regulations in RM. By calibration of the mathematical model (harmonization of the analytically and the experimentally obtained fundamental periods), an elastic modulus amounting to E=1000 MPa has been obtained. This knowledge has been applied in adopting the input parameters for the characteristics of the material in the subsequent analyses.

The bearing capacity of the structure amounts to 20.81% and 18.3% of the total weight of the structure in longitudinal and transverse direction, respectively (Table 3). Accordingly, the bearing capacity of the structure in both directions is greater than that required according to the regulations for earthquakes with a return period of 500 years amounting to 15% of the total weight of the structure, but is lower than that required for the expected maximum earthquake with a return period of 1000 years amounting to 30% of the total weight of the structure.

The ductility capacity ranges 1.14–2.54 for longitudinal and 1.10–2.45 for transverse direction, which is greater than the required ductility ($\mu=\delta max /\delta y$, Table 4), except for the first level. The required ductilities for the first level for the design and maximum earthquake are greater than 1.5 and 2.5, respectively, i.e., the design criteria for seismic safety are not satisfied.

The relative storey displacements for the first level do not satisfy the requirements regarding earthquakes that are expected at this location considering the fact that the required relative displacements (δmax, Table 4) during design and maximum expected earthquake reach values of up to 2.132cm and 2.533cm in longitudinal direction, i.e., 2.915cm and 3.123cm in transverse direction.

From the above stated, it can be concluded that the existing structure possesses sufficient bearing capacity, but does not possess sufficient deformation capacity which is characteristic for such massive and non-ductile structures as are the traditional masonry ones. Accordingly, the existing church structure does not satisfy completely the established safety design criteria for the expected maximum earthquake.

PROPOSED SEISMIC UPGRADING OF THE CHURCH STRUCTURE

For improvement of the behaviour and resistance to dynamic effects and to provide synchronous behaviour of the structure that would activate all the walls equally during an earthquake, it is necessary to provide sufficient integrity at the top level of the bearing walls. In the concrete case, it is proposed not to remove the reinforced concrete plates but to use them to provide the necessary integrity by way of hinged connections of the plates with the bearing walls. Such connection will play two important roles during future dynamic effects:

1. it will prevent uncontrolled displacement of the reinforced concrete plates considering that they are not anchored into the existing walls but are freely supported, and;
2. it will enable activation of all the bearing walls and behaviour of the structure as a whole.

The proposed technical solution for this structural consolidation consists of implementation of vertical steel bars along the edge of the reinforced concrete plate over the prothesis, the diaconicon and over the church corners, diagonal steel anchors along the edge of plate over the corners as well as steel tie placed in horizontal joint (previously cleaned from facade mortar) along the perimeter of the vaults, at depth of 5cm to provide continuity of connection at the corresponding level, (Fig. 9). The hinged connections should be made of stainless steel bars that are inserted in previously prepared openings through the plate and the bearing wall, filled with epoxy mortar or alternatively, with lime mortar with additives for accelerated achievement of strength (crushed brick, pozzolana). After opening of the holes and prior to the placement of the bars, injection of the upper parts of the bearing walls with mixtures based on lime mortar is proposed.

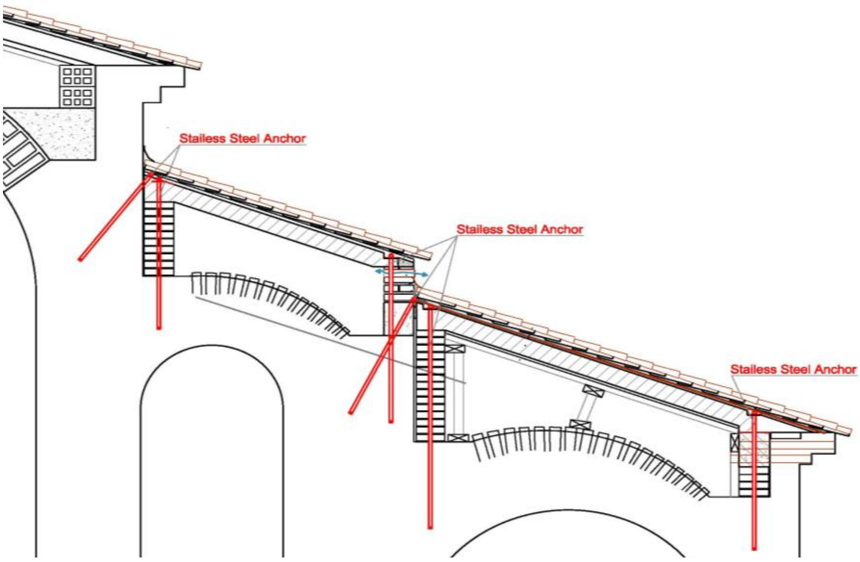
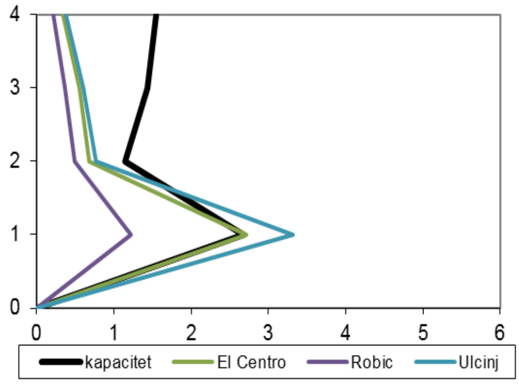


Figure 9. Proposed seismic upgrading of the church structure

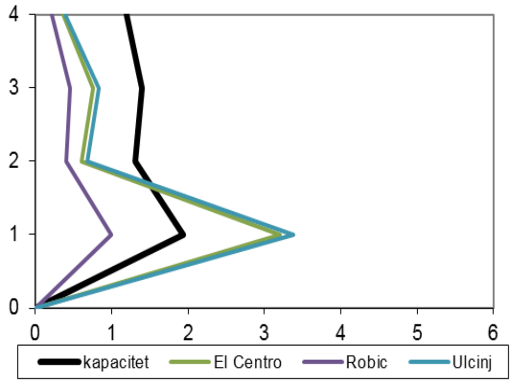
For the improved church structure, control analyses have been performed. The modification of the existing input data refers only to the ultimate conditions of support at the tops of the walls due to the presence of the anticipated hinged connections with the existing reinforced concrete roof plates.

From the presented results, it is evident that the anticipated minimal interventions enable improvement of the behaviour of the church structure, particularly the upper levels for which the bearing capacity and deformability is greater than the required (Fig. 10). As to the first level, it possesses sufficient bearing capacity but due to the high stiffness (presence of all the walls with the greatest thickness), the maximum earthquake requires greater ductility than those possessed by the structure, particularly in the longitudinal direction.

The insufficient deformability of the first level of the church structure can be overcome only by large interventions as is addition of new ductile structural elements for strengthening. However, considering that the authenticity of the church is a priority and an imperative, it has been assessed that the intervention in this sense is not allowable, particularly that, after detailed investigations and analyses, it has been shown that the church structure is stable and safe under gravity serviceability loads for the design level of seismic effects.



longitudinal direction



transversal direction

Figure 10. Required ductility for input a_{max}=0,31g in respect to ductility capacity of the church

CONCLUSIONS

For the existing structural systems of the church of St. Mary Peribleptos in Ohrid complex experimental, laboratory and analytical investigations have been performed for the purpose of defining its stability and safety under gravity and seismic effects. It was concluded that the church is presently stable and safe against gravity serviceability loads.

From structural aspect, the interventions of 1986, particularly the steel ties have played an important role for such behavior of the church. The reinforced concrete belt courses and plates have also had an important role in the behavior of the structure during the past period contributing, although not directly (they are freely supported on the existing walls), to better integrity of the structure through lateral support of the bearing walls at several levels. The additional weight of the reinforced elements increases the mass of the structure and therefore induces greater seismic forces, but on the other hand, it increases the stress due to the vertical load in the walls as well their bearing capacity. Considering that the structure is founded in rock of high bearing capacity, these additional weights negligibly increase the stress in the soil.

As to the stability and safety of the structure under seismic loads, the performed analyses have shown that the structure possesses a certain bearing capacity and deformability but does not satisfy completely the established design criteria for safety of a structure of the first category, or more precisely, there is insufficient deformation capacity for the expected maximum earthquake. Anticipated therefore are minimal interventions to improve the seismic safety and stability that will prevent uncontrolled displacement of the reinforced concrete plates and will enable activation of all the bearing walls and synchronous behavior of the structure as a whole. The control analysis of the improved structure shows that the proposed minimal interventions improve the behaviour of the church structure, particularly in the upper levels.

REFERENCES

Sheshov V., Dojcinovski D., Cvetanovska J., Edip K., Gjorgjeska I., Olumceva T., (2011), "Assessment of Seismic Potential of the St. Mary Peribleptos Church Site in Ohrid", *Report IZIIS 2011-36,* Skopje. August 2011

Krstevska L., Tashkov Lj., Shendova V., (2011), "Experimental In-situ Testing of Monastery Complex St. Marry Peribleptos in Ohrid, Macedonia, by Ambient Vibration Method" *Report IZIIS 2011- 38*, Skopje, August 2011

Geotehnika DOO, Skopje, (2011) "Geotechnical Investigation of the St. Mary Peribleptos Church Site in Ohrid", *Report IZIIS 2011-39*, Skopje, August 2011

UKIM-ZIMAD Institute for material testing, (2012) "Investigation of physical mechanical characteristics of mortar samples taken from the St. Marry Peribleptos church structure", *Report IZIIS 2012-58*, Skopje, November 2012

Shendova V., Bozhinovski Z., Apostolska R., Stojanoski B., Zlateski A., Gjorgjievska E., Jekikj G., (2013) "Analysis of Stability of Existing St. Mary Peribleptos Church Structure Under Gravity and Seismic Effects", *IZIIS Report 2013-3*, Skopje, January 2013