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RECONSTRUCTION, SEISMIC STRENGTHENING AND REPAIR OF THE ST. ATHANASIUS CHURCH I LESHOK – CASE STUDY

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SUMMARY

On August 21, 2001, during the armed conflicts in R. Macedonia, the monastic church of St. Athanasius in Leshok experienced strong detonation, which resulted in its almost complete demolition. A Main Project on the Reconstruction, Seismic Strengthening and Repair of the church has been elaborated by the Institute of Earthquake Engineering and Engineering Seismology in collaboration with the Republic Institute for the Protection of Cultural Monuments, Skopje. From structural aspects, there have been two approaches taken in the attempt to renovate and reconstruct the structure. In this paper, based on the performed detail analysis of the structure under gravity and seismic load, (i) solutions for repair and strengthening of the existing damaged part of the monastic church and (ii) solution for seismic strengthening of the ruined part of the church to be reconstructed are presented.

INTRODUCTION

The church of St. Athanasius in the village of Leshok is situated within the monastic compound St. Bogoroditsa in the village of Leshok - Tetovo area and as such has been put under the protection of the Law on Protection of Cultural Monuments of R. Macedonia. As a result of the armed conflicts that took place on August 21, 2001, a greater part of this church was torn down, while the still existing part is characterized by severe damage.

The church of St. Athanasius in the village of Leshok was constructed in the thirties of the twentieth century (Fig. 1). From structural aspect, it represented a three conched structure with an elongated narthex on the west side and belfries. The outline proportions of the plan were 14.00 / 22.60 metres, while the total height of the part with the church belfries and the main

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dome was 13.40 m and 18.40 m, respectively. The walls were constructed of stone masonry in lime mortar with a total thickness of 80 cm. All the vaults, the tambours and the domes were constructed of brick masonry.

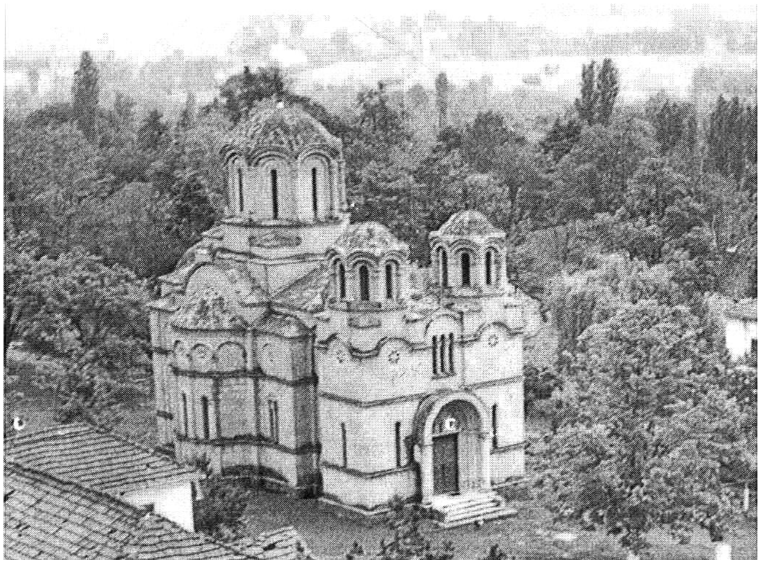


Figure 1. Northwest view of the church of St. Athanasius

DESCRIPTION OF THE EXISTING STATE OF THE STRUCTURE

The armed conflicts that took place on August 21, 2001 led to almost complete destruction of the part of the naos with the two side conches (main central part), the altar part and the part of the narthex over the terrain level, with some remains of the facade walls, whereas the part with the belfry still exists but is characterized by severe structural and nonstructural damage, (Fig.2).

Figure 2. Damage to the belfries, northwest view

The vaulted structure over the gallery area is completely torn down, while the only remains of the floor structure are the timber floor beams with visible deformation of wood. In the part of the two preserved belfries, there are visible large cracks along the height of the bearing walls, in the staircase area, the walls of the tambours and the domes with a width of over 2 cm (Fig. 2 - 4).

Figure 3. Damage to the vault in the narthex

Fig. 4. Damage to the floor structure

REPAIR, STRENGTHENING AND RECONSTRUCTION OF THE STRUCTURE

Definition of Concept for Repair and Strengthening of Existing Structure

The seismic safety criteria and the method of strengthening of a particular historic structure are defined based on investigation of the structural response to expected earthquakes and its seismic stability, taking into account its main characteristics and artistic historic value

Design Seismic Safety Criteria

As in the case of designing new structures, during repair and strengthening of existing structures of damaged historic monuments or preventive seismic strengthening of structures of vital importance, it is necessary to define the design seismic criteria as follows:

- Level I: Under earthquakes of a lower intensity and shorter return period, the dynamic behaviour of the structure during an earthquake must not lead to vibrations and induce damage to both structural and secondary, nonstructural elements (the behaviour should completely be in the elastic range with required ductility of $\mu < 1$);

Level II: Under earthquakes of a higher intensity, so called design earthquakes, the structure should generally remain in the linear range of behaviour, with possible limited nonlinear deformations of individual elements of the system, which means limited stiffness deterioration and energy dissipation (initial nonlinear behaviour with required ductility of $\mu < 1.5$)

Level III: Under maximum expected earthquake effects, the structural and nonstructural elements of the structure are deeply in the nonlinear range of behaviour, while the stiffness and the resistance of the structure are considerably reduced. However, such earthquakes must also not disturb completely the stability of the bearing structure, i.e., the damages induced should be repairable (nonlinear behaviour with required ductility of $\mu < 2$).

In addition, in each intervention to be carried out for such type of structures, certain principles and rules must be respected. Among these, the main principle is to provide a maximum safe structure by minimal interventions. For each historic monument, the design criteria are defined on the basis of special conditions that depend on the historic, architectural and artistic value of the structure, the seismic hazard and the possibility of application of a corresponding measure for repair and strengthening. In the concrete case, for the considered location of the structure with defined seismic zone and good local soil conditions, the expected maximum earthquake intensity for a return period of 500 to 1000 years ranges between 0.20 g - 0.24 g.

Concept for Repair, Strengthening and Reconstruction of the Structure

Based on the knowledge gained from all the previous investigations and the defined design criteria, having the consent of the principal architect-conservator, the following has been adopted:

1. For the damaged existing part of the structure, a *concept for repair and structural strengthening* up to the necessary level of seismic safety has been adopted considering the purpose of the structure and the justification (architectural-conservatory) of preservation of its integrity. The solution for repair and structural strengthening anticipates (i) injection of all the cracks and (ii) incorporation of strengthening elements (vertical RC jackets along the inner side of the walls of the staircase core and the columns of the tambours, horizontal RC belt courses at the level of the floor structure and at the base of the domes, RC slab below the floor level, as well as steel ties besides the timber beams in the floor structure of the gallery, (Figs. 5 and 6);

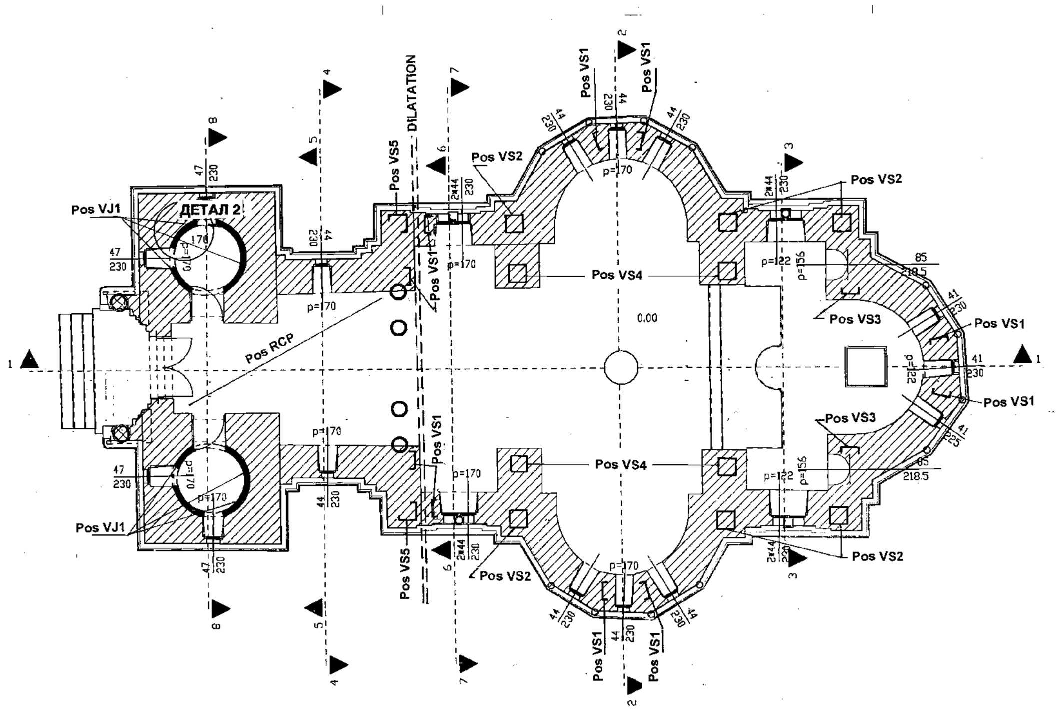


Figure 5. Reconstruction, Repair and Seismic Strengthening of the Church Plane at level +1.90m

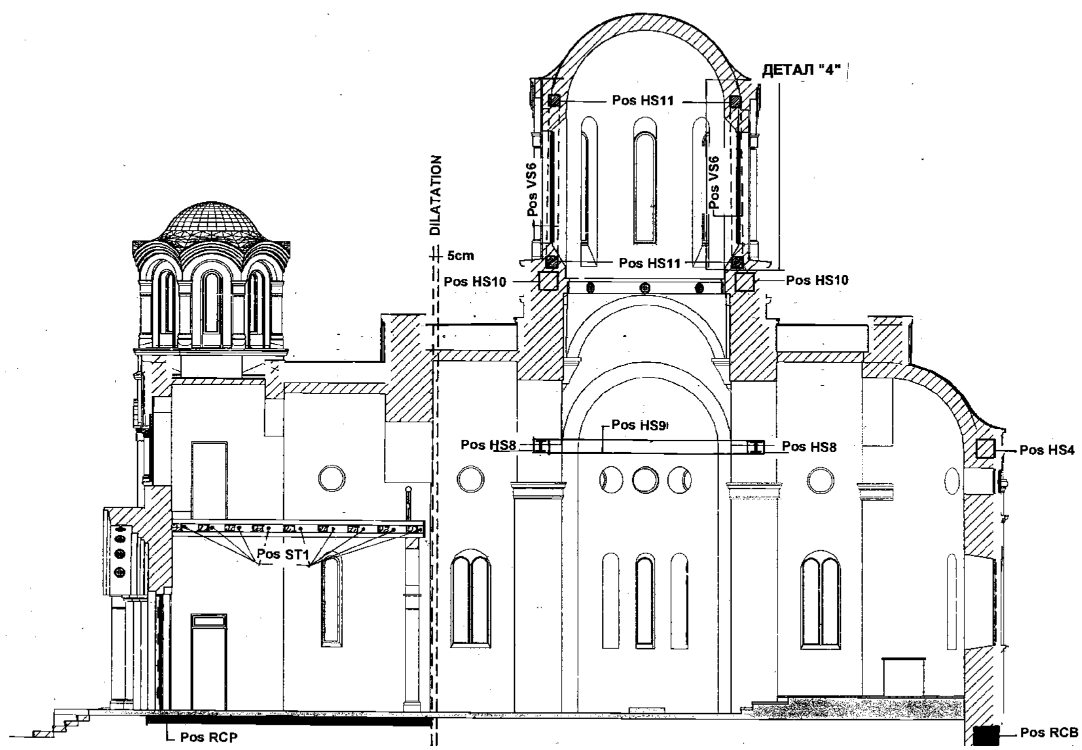


Figure 6. Reconstruction, Repair and Seismic Strengthening of the Church Longitudinal Cross Section

2. For the demolished part of the structure, a *concept of complete reconstruction* by maximum possible use of selected material has been adopted, whereat elements for structural strengthening for providing the necessary level of seismic safety have also been anticipated: (i) incorporation of RC belt course below the floor level, in the existing foundation walls, below the massive walls for the purpose of connection with the vertical strengthening elements, (ii) incorporation of vertical strengthening steel elements at the necessary height, at the ends of the massive walls and around the openings, (iii) incorporation of vertical strengthening steel elements into the tambour columns composed of deformed reinforcement, (iv) incorporation of horizontal steel elements along the massive walls, in the base of the tambour , as well as in the base of the dome, (Figs. 5 and 6);
3. Due to the different treatment of the structural units constituting the integral structure, an *expansion joint* between them (that will be made invisible from the inside of the structure) is anticipated to be constructed and dictate the concentration of damages in the structure during future earthquakes, (Figs.5 and 6).

ANALYSIS OF THE STRUCTURE

The methods for analysis of both structural units under gravity and seismic load include:

1. Analysis of the bearing and deformability capacity and dynamic analysis for maximum expected intensities of earthquake effects of Amax=0.24g for the return period of tp=1000 years;.
2. Static and Equivalent Seismic Analysis by the finite element method.

Analysis of the bearing and deformability capacity and nonlinear dynamic analysis

The methodology for defining the bearing capacity of the structure in the form of ultimate storey shear force that compared to the equivalent seismic force yields the factor of safety against failure is a procedure which is widely applied in equivalent static analyses of masonry structures at IZIIS. The following quantities have been adopted as input parameters: (i) Elasticity modulus E = 1200000 kPa; (ii) Shear modulus G = 460000 kPa; (iii) Ultimate compression strength fc = 1000 kPa and (iv) Ultimate tensile strength ft = 100 kPA.

Due to the importance of the structure and the fact that it represents a historic heritage, the value of K = 0.30 has been adopted as the total seismic coefficient of the base. This value is higher than the values computed in compliance with our and the European codes:

Our regulations: $K = K_o\ K_s\ K_d\ K_p = 1.5 \times 0.05 \times 1.00 \times 1.60 = 0.12$
Eurocode 8: $K = \alpha\ S\ \beta_o / q = 0.24 \times 1.00 \times 2.5 \times 2.0 = 0.30$

From the obtained results, it is concluded that there is sufficient bearing and deformability capacity of each individual wall whereat the safety factor at occurrence of the first cracks and particularly occurrence of failure is greater than 1 (Fj >>1, Fu > 1) for all the storeys. Table 1 shows the summarized results for both structural units for both directions per storeys (storey stiffness Ki, ultimate bearing capacity Qu, factor of safety against failure Fu) that represent, at the same time, the input parameters for dynamic analysis.

Table 1. Bearing and Deformability Capacity

	Results for Repaired and Strengthened Existing Part					
	Ki (kN/cm)		Qu (kN)		F=Qu/S	
	x-x	y-y	x-x	y-y	x-x	y-y
storey 3	1283	1283	496	496	2.21	2.21
storey 2	14386	14072	2567	2502	2.57	2.51
storey 1	10148	10640	3249	3407	1.56	1.63

	Results for Reconstructed Part					
	Ki (kN/cm)		Qu (kN)		F=Qu/S	
	x-x	y-y	x-x	y-y	x-x	y-y
storey 2	1158	1158	1323	1058	2.86	2.29
storey 1	16498	5448	4439	4283	1.36	1.31

Applying modeling by concentrated masses that assumes concentration of distributed structural characteristics at characteristic levels, a nonlinear dynamic analysis has been performed by application of a corresponding storey hysteretic model obtained by summing up the elastoplastic characteristics of each of the bearing walls, whereat the bearing capacity of each of them has been limited to the lower value of bending and shear bearing capacity.

To obtain the dynamic response, three different types of earthquake (Petrovets 1979, Ulcinj 1979 and El Centro, 1940) with maximum input acceleration of 0.24 g and return period of 1000 years have been applied. Obtained as results from the dynamic analysis are the storey displacements, i.e., the ductilities required by the earthquake that have to comply to the design criteria.

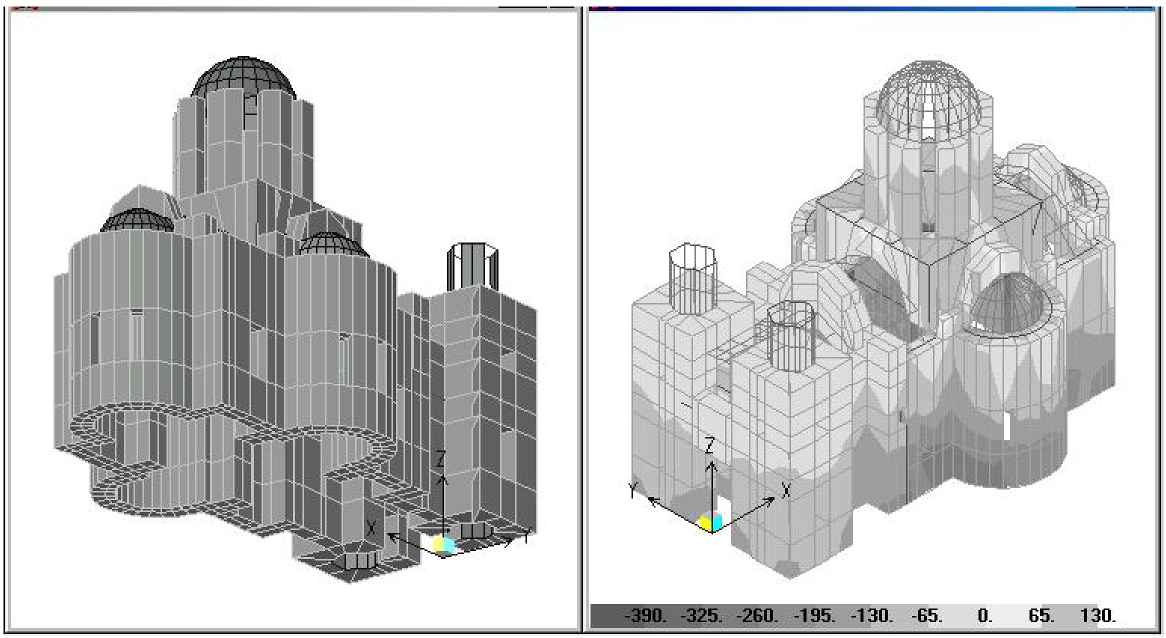
It has been concluded that, for both the structural units, the absolute storey displacements are less than 1 cm. Hence, it can be asserted that such a designed repair and structural strengthening of the existing part of the structure, as well as design part of the structure to be reconstructed, provides sufficient bearing and deformability capacity, i.e., that the dynamic behaviour complies to the set out design criteria because a ductility of $\mu < 2$ has been obtained even for the maximum expected earthquake with a return period of 1000 years.

Static and Equivalent Seismic Analysis by the finite element method

Based on the defined structural systems of the two individual structural units and the defined strengthening structural elements of the structure, a static and equivalent seismic analysis has been performed by using the finite element method and the computer software package SAP 2000 (Structural Analysis Programme, UC Berkeley, California). Taking into account the complexity and the specific nature of the structural system and the materials built-in the model of the structure on one hand and the possibilities offered by the programme package on the other, an attempt has been made to define a model by finite elements that shall most appropriately represent the structure. A moderately dense mesh of a total of 5191 nodes and 1901 elements has been adopted, involving the global geometrical characteristics of the model without paying attention to the inhomogeneity of the material, (Fig. 7a).

The bearing massive walls and the tambour walls have been modeled by the SOLID three-dimensional finite element with eight nodes, i.e., by a total of 938 elements. The steel vertical and horizontal strengthening elements were modeled by a 3D-FRAME, i.e , 3D-TRUSS elements , i.e., by a total of 362 elements. All the vaults and the domes have been modeled by a total of 601 SHELL elements.

For such modeled structure, a static analysis has been performed for the effect of dead weight and equivalent seismic forces computed according to the regulations, whereat the following values have been adopted as specific weight of masonry; (i) stone masonry $\gamma = 22.5$ kN/m3 and (ii) brick masonry $\gamma = 18.5$ kN/m3.



a) 3-D View b) Stress State due to Dead Load

Figure 7. Results form the Finite Element Analysis

The results from this analysis given in the form of digital and graphic presentations of stress and strain states for different loading cases, give an insight into the designed values of all the static quantities referring to the individual types of elements, which justifies the selected solution for repair and structural strengthening, (Fig. 7b). The power of the programme as to the definition and detailed presentation of stress-strain state under different load combinations is also evident.

CONCLUSION

On 21.08.2001, during the armed conflict in R. Macedonia, the monastic church of St. Athanasius in Leshok was almost completely demolished by a strong detonation. From structural aspects, there have been two approaches taken in the attempt to renovate and reconstruct the structure again. For the existing, damaged part, repair and structural strengthening is anticipated whereas the demolished part is anticipated to be thoroughly reconstructed with elements of structural strengthening.

The structural system of the monastic church of St. Athanasius in Leshok is composed of two structural units separated by an expansion joint of 5 cm because of the different approach in the solutions for repair and structural strengthening as well as dictating of the concentration of damage during future possible earthquakes. For both individual structural units, a methodology for repair and strengthening has been proposed with due respect to the main principle holding for this kind of historic structures, which is "minimal interventions - maximum protection".

An analysis has been carried out for both structural units in accordance with the valid existing regulations and the European prestandards. Two methods were used:

1. Analysis of the bearing and deformability capacity of the structure and performance of a nonlinear dynamic analysis for maximum expected actual earthquake effects with intensity of amax = 0.24 g with a return period of 1000 years.
2. Static and equivalent seismic three-dimensional analysis of the structure by means of the computer package SAP 2000.

The results from the analysis presented in the report show that:

- Both structural units constituting the integral structure possess a sufficient bearing and deformability capacity up to the designed level of seismic protection;
- Both structural units satisfy the design safety criteria. Under the expected design earthquake (level II, amax = 0.20 g with return period of 500 years), it is expected that the structure will behave completely in the elastic rage, while under the maximum expected earthquake (level III, amax = 0.24 g with return period of 1000 years), it is possible that the structure suffers concentration of damage in the expansion joint as well as other nonstructural damage.
- It is generally concluded that such designed structure satisfies the prescribed requirements and criteria for such type of historic structures.

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