Reconstruction and Seismic Strengthening of the Blown Up Cathedral Church of the Holy Trinity in Mostar

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SUMMARY:
The cathedral church of the Holy Trinity in Mostar was exposed to shellfire on 7th June 1992, while, on 15th June it was put on fire and finally blown up. Presented in the paper is the analysis of the seismic stability of the designed structure of the church, then definition of the concept of necessary strengthening of the bearing structure and analysis of the stability of the strengthened structural system under gravity and seismic loads. Three general states have been treated: designed structure of plane stone masonry; strengthened structure by horizontal steel element and strengthened structure by horizontal and vertical steel strengthening elements, (confined masonry). The selected structural strengthening with confined masonry enables increase of the strength, stiffness and deformability capacity of the church as well as ability for dissipation of seismic energy. The construction of such designed structure started in March 2011.

Keywords: historic monument, confined masonry, capacity analysis approach, reconstruction, seismic strengthening

1. INTRODUCTION

The cathedral church of the Holy Trinity in Mostar built in the period 1863 to 1873 was shelled on June 7, 1992, while on the 15th June, the belfry was torn down and the church was put on fire and finally blown up. The remains of the church were cleared in 2005 (Fig.1.1). Later, a decision on renovation of the church involving full reconstruction and maximum possible use of the existing preserved material has been made. Based on this decision, the Main Project on Renovation of the Cathedral Church in Mostar (architecture and structure) has been elaborated. In this project, the bearing structure is designed to be constructed of massive stone masonry in cement lime mortar. The renovation of the church started with the construction of the newly designed, reinforced concrete foundation over which reconstruction of the structure is planned (Fig.1.2).

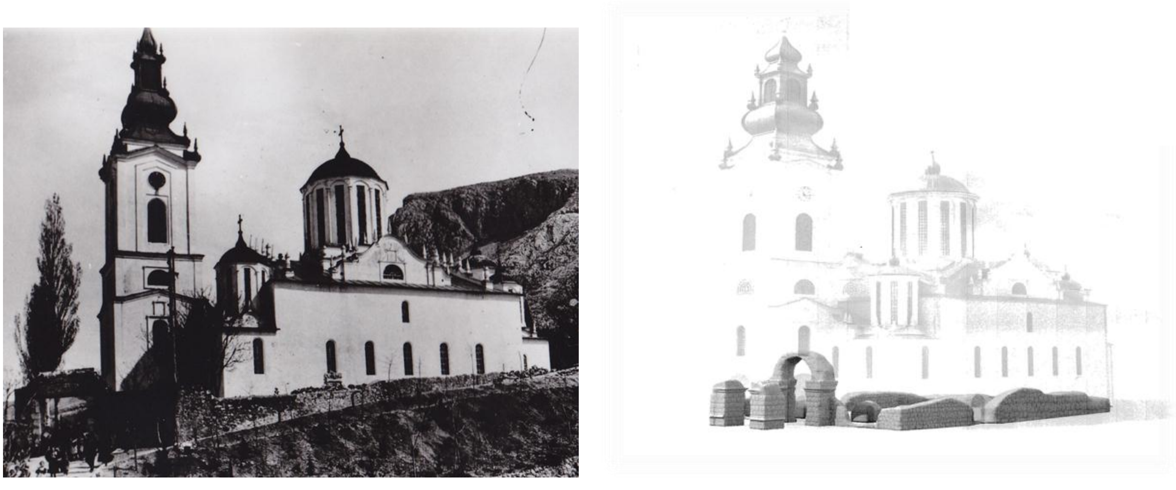


Figure 1.1. The original church and view of the torn down church

After the preparation of the Main Project, in accordance with the categorization of the church as a structure of the first category, the Republic Hydro-meteorological Institute of the Serb Republic performed seismic microzonation and defined the seismic parameters of the considered location. For such defined seismic parameters, prior to the construction works, it was necessary to carry out analysis of the seismic stability of the designed bearing structure of the church.

Figure 1.2. Newly designed RC foundation in the course of construction

Upon getting evidence on the necessity of structural strengthening, variant solutions of strengthening have been proposed and analysed. Following the selection of the most adequate (from the aspect of stability and economy) possible solution, the stability of the strengthened structure under gravity and seismic effects has been analysed. Three general states of the bearing structure have been treated: (PS) – designed structure constructed of plain stone masonry; (HE) – strengthened structure by horizontal steel elements and (OS) – strengthened structure by horizontal and vertical steel strengthening elements, (confined masonry). The applied methodology of analysis has been developed by IZIIS based on the most recent knowledge on behaviour of masonry structures enriched with analytical and experimental own and world experience and implementation of this knowledge in reconstruction of more important cultural-historic monuments. The results from the performed analyses have shown that the selected concept of strengthening of the structure enables optimization of the design structural system by adequate selection of strengthening elements and provides the necessary integrity and stability of the structure for the designed level of seismic protection.

2. METHODOLOGY OF ANALYSIS OF THE STRUCTURE

Generally, three types of analyses have been carried out: (1) linear static and seismic analysis of the stress-strain state by use of the finite element method, (2) analysis of the bearing elements up to ultimate states of strength, stiffness, deformability and ability of the elements and the system as a whole to dissipate seismic energy by linear and nonlinear behaviour, and (3) analysis of the dynamic response of the bearing system for actual earthquakes with intensity and frequency content expected at the location of the church.

2.1. Analysis of Stress-Strain State by the Finite Element Method

Detailed definition of stresses and strains in the structure under different loads is possible only by analysis of a 3D finite element model of a structure. In the concrete case, static, equivalent seismic (according to JUS and EC8 standards) as well as spectral analysis of the church structure (ERS with a set of synthetic earthquakes obtained by seismic microzonation of the location) has been performed by using the SAP 2000. The massive bearing walls, the walls of the belfry and the tambour have been modelled by a three dimensional finite element (SOLID) with eight nodes. All the vaults and the domes have been modelled by surface SHELL elements considering the considerably smaller thickness, while the visible ties and steel elements used for strengthening have been modelled by a linear 3D-FRAME, i.e., 3D-TRUSS elements. Although elastic, this analysis of stress-strain state enables identification of potential local instabilities where exhaustion of compressive or tensile strength or excessive deformations take place, which is very useful in the further process of decision making regarding improvement of the seismic stability.

2.2. Analysis of Bearing and Deformability Capacity

The bearing and deformability capacities are the main initial parameters by which is defined the behaviour of a structure as a whole and the individual structural elements. Definition of bearing capacity of a structure is actually determination of the ultimate shear force at individual characteristic levels. This force, compared with the associated equivalent seismic force, provides the safety factor against failure. The mathematical model which was applied in further analyses of the Cathedral Church was based on modelling of all the individual wall elements in the analysed direction. The wall elements were modelled as fixed at the base for the designed state of the structure, while for the strengthened structure, by grouping of individual wall elements fixed at both ends. The bending and shear bearing capacity expressed as maximum ultimate horizontal force in the wall in the case of shear or bending failure is presented in a tabular form by the following expressions, Table 1.1:

Table 1.1. Bending and shear bearing capacity

	Bending capacity	Shear capacity
Plain masonry	$Q_u^{sov} = M_u/(\alpha h)$ $M_u = 0.5\ \sigma_o t l^2\ (1-\sigma_o/f_c)$ Cantilever - α=1 fixed - α=1/2	$Q_u^{sm} = A\tau_u$ $\tau_u = f_t/b\ (\sigma_o/f_t + 1)^{1/2}$
Confined masonry	$Q_u^{sov} = M_u/(\alpha h)$ $M_u = 0.5\ \sigma_o t l^2\ (1-\sigma_o/f_c) + 2A_s F_{sy}\ (l/2 - l')$ Cantilever – α=1 fixed - α=1/2	$Q_u^{sm} = A\ \tau_\upsilon$ $\tau_u = f_t\ \{h/2l + [(h/2l)^2 + N/A/f_t + 1]^{½}\}$

The interpretation of the results obtained in this way enables observation of the behaviour of each individual wall and the structure as a whole. For the described procedure, the SDUAMB (Static and Dynamic Ultimate Analysis of Masonry Buildings) computer programme has been developed at IZIIS. In this programme, the input data are the geometry of the walls, the characteristics of the materials and the seismic coefficient at the base, while the output data are the bearing and deformability capacity as well as the safety factor against occurrence of the first cracks and the safety factor against failure for each individual wall and the structure as a whole. With this programme, the bearing and deformability capacity of the church structure has been analysed for all three conditions, taken separately, along with corresponding modelling of the walls.

2.3. Dynamic Analysis of the Structure

The main dynamic model of the structure represents a schematized system of concentrated masses, assuming concentration of distributed structural characteristics at individual levels and connected such that enable displacement in horizontal direction only. The masses, the stiffness, the displacements at the elasticity limits, the plasticity line as well as the time history of ground acceleration represent input data in the dynamic analysis. In the concrete case, the input hysteretic diagram is obtained by summing up the elasto-plastic characteristics of the individual walls, whereat the bearing capacity of each wall has been limited to the smaller value of shear and bending capacity, (Table 1.1). The dynamic response of the structure is obtained for two orthogonal directions separately in the following form:

- Time histories of relative displacements, relative velocity and absolute accelerations at each level during an earthquake;
- Maximum values of displacement, velocity, accelerations and time of their occurrence.

The results from the dynamic analysis define the behaviour of the structure through the demanded displacement, δ_{RQ} and the demanded ductility, μ_{RQ}, obtained for a certain earthquake and different maximum input accelerations. At this point, it is necessary that the relative displacements and ductility be within the limits defined by the regulations or based on the defined design criteria.

3. SEISMIC PARAMETERS OF THE SITE AND SEISMIC SAFETY CRITERIA

3.1. Definition of Seismic Parameters

The elaboration of the Study for Seismic Micro-zoning of the Location of the Cathedral Church in Mostar according to the JUS and EC8 standards by the Republic Hydro-meteorological Institute of Banja Luka, enabled definition of the total seismic hazard at the church location and analysis of the response of the local soil to seismic effects for the purpose of definition of the parameters of the horizontal elastic spectrum of local soil response (Fig. 3.1), the corresponding time histories of acceleration and deformability properties of the soil.

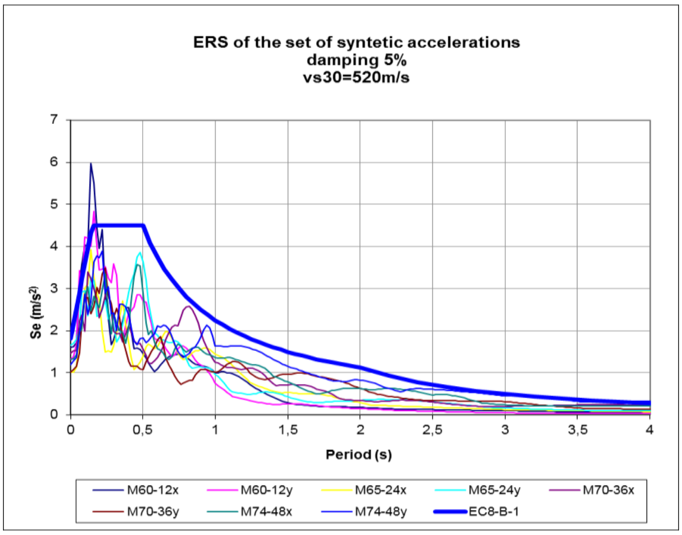


Figure 3.1. Elastic response spectrum of the set of synthetic accelerations

The seismic hazard at the location has been defined for a return period of 100, 475 and 1000 years with values of 0.07g, 0.16g and 0.22g, respectively. Categorization of the local soil for local soil type B has been carried out based on the data available from geological and geotechnical investigations and the shape of local time history of acceleration.

A set of 6 horizontal histories of acceleration scaled to satisfy the EC-8-1 criteria has been generated for the location. In accordance with the JUS standards, the numerical values of K_c have been computed for three return periods, namely, 100, 475 and 1000 years, at the free surface of the terrain with 0.028, 0.062 and 0.088, respectively. With this, there have been defined the input data for further computation of the stability of the structure according to the JUS and EC8 standards for analysis of bearing and deformability capacity and dynamic analysis with consideration of actual earthquakes:

- JUS standards, (PIOVS): $K=K_0\,K_c\,K_d\,K_p$

K_0 – category of the structure - I category- $K_0=1.50$
K_c – seismicity of the terrain - $K_c^{100}=0.028$, $K_c^{475}=0.062$, $K_c^{1000}=0.088$
K_d – soil dynamics - $K_d=1.00$
K_p – ductility - $K_p^{\text{plain masonry}}=2.0$, $K_p^{\text{confined masonry}}=1.60$

- Plain masonry: $K^{100}=0.084$; $K^{475}=0.186$; $K^{1000}=0.264$
- Confined masonry: $K^{100}=0.067$, $K^{475}=0.130$; $K^{1000}=0.211$

- Eurocode 8, (EC8): $K = \alpha\, S\, \beta_0 / q = 0.30$

α - maximum ground acceleration - $\alpha^{100}=0.07$, $\alpha^{475}=0.16$, $\alpha^{1000}=0.22$,
S – soil parameter - $S^{B}=1.20$
β_0 – amplification factor - $\beta_0^{\mu=5\%}=2.5$
q – behaviour factor - $q^{plain\ masonry}=1.5$, $q^{confined\ masonry}=2.0$

  - Plain masonry: $K^{100}=0.14$; $K^{475}=0.32$; $K^{1000}=0.46$
  - Confined masonry: $K^{100}=0.10$; $K^{475}=0.24$; $K^{1000}=0.34$

3.2. Design Seismic Safety Criteria

For analysis of the principal structural system of the cathedral church, three levels of seismic intensity have been defined for different return periods for which the seismic stability criteria have also been defined. The development of nonlinear mechanism in the structural system leads to a big increase of deformations wherefore the seismic activity level is defined as seismic risk related to ultimate strains in the system:

- *Level I:* For earthquakes of lower intensity and more frequent return period, the dynamic behaviour of the structure should not cause vibrations leading to damage to both structural and secondary, non-structural elements (the structural response is in the elastic range, the ductility demand is $\mu<1$);
- *Level II*: For the design earthquake, the structure should generally remain in the linear range with possible limited nonlinear deformations of individual elements of the system, meaning limited stiffness deterioration and energy dissipation (initial nonlinear behaviour, ductility demand of $\mu<1.5$);
- *Level III*: For the maximum expected earthquake, the structural and nonstructural elements are in the nonlinear range, while the stiffness and the resistance of the structure are considerably reduced. However, such earthquakes must also not disturb the stability of the bearing structure as a whole, i.e., the inflicted damage must be repairable (nonlinear behaviour, maximum ductility demand is $\mu<2.0$).

In addition, in each intervention to be done on important historic monuments, one should observe certain principles and rules among which the main principle is to provide maximum protection of the structure by minimal interventions. In the concrete case, the objective has been to satisfy the defined safety criteria by minimal modification of the designed church structure, including the necessary additional strengthening elements.

4. COMPARISON OF RESULTS ON THREE GENERAL CONDITIONS OF THE STRUCTURE

With the elaboration of the Main Design on Renovation of the Cathedral Church by the Republic Institute for Protection of Cultural-Historic and Natural Heritage, Banja Luka, the entire architecture and the principal structural system of the church have been defined (Fig. 4.1). The Cathedral Church has a cross-like plan with a high belfry in the west part and a triple apse in the east part. The total length of the church along with the belfry and the altar is 45.32м, while its width is 25.66м. The cross-section of the arms of the inscribed cross is surmounted by the central dome with a dodecagonal plan with a span of ~9м, on a tambour with a square plan. Over the altar, there rise three domes with octagonal plans that are lower in respect to the central dome. Other two domes with octagonal plans are situated above the narthex. The width of the bearing walls varies from 130 to 206 cm, while the width of the tambour walls ranges from 40 to 60 cm. The inner width of the church naos is 21.53m, while the height of the central vault is 17.96 m.

With the main project on the structural phase, the church is designed as a massive masonry structure constructed of different types of stone in cement lime mortar. The main structural elements are the domes, the arches, the vaults, the wooden floor structures, the columns, the walls and the foundation.

For the purpose of making decision as to the recipe for the cement lime mortar based on analysis of the bearing structure, three variant characteristics of mortar have been adopted for analysis of the designed structure as follows:

- *Variant V1:* fc=2 MPa; ft=0.2 MPa, E=2000 MPa, G=0.25E = 500 MPa;
- *Variant V2:* fc=4 MPa; ft=0.4 MPa, E=3500 MPa, G=0.25E = 875 MPa;
- *Variant V3:* fc=8 MPa; ft=0.8 MPa, E=6000 MPa, G=0.25E = 1500 MPa.

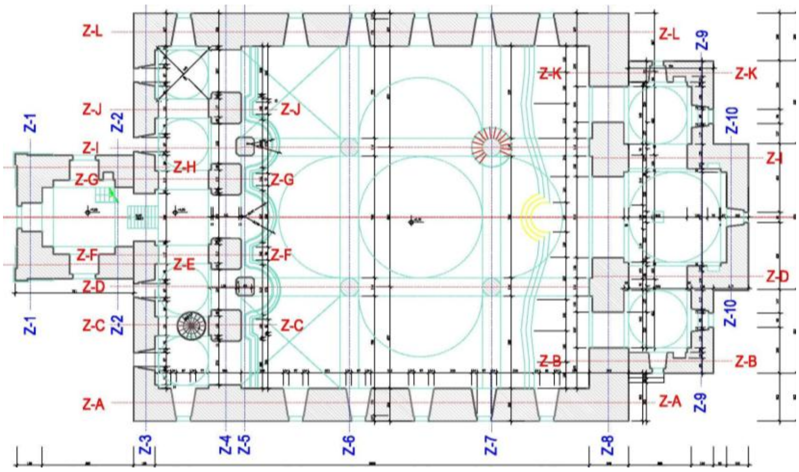


Figure 4.1. Ground floor plan of the designed church structure

After proving the necessity of improvement of the designed structure, the proposed first variant (V4) is strengthening by implementation of only horizontal steel elements – ties made of rigid “L” and “I” profiles at five characteristic levels of the principal structure (at levels: +4.44м, +9.60м, +14.15м, +20.50, +29.50) and at six levels of the belfry (at levels: +7.50, +13.00, +18.50, +26.30, +29.70 и +32.30). These elements are placed along the length of the bearing walls in transverse and longitudinal direction and are connected by welding where they border on each other. In this way, the integrity of the principal structure is considerably improved at characteristic levels and contributes to synchronized behaviour of the individual walls. The objective of placement of these ties is to sustain the tensile stresses after exhaustion of the bearing capacity of the masonry and occurrence of the first cracks and thus preventing further damage to the walls.

However, the results from the performed analyses have shown that it is necessary to include additional vertical strengthening elements to provide the designed level of seismic protection. With this, the bearing structure of the church constructed of plain masonry is turned into a confined masonry which has been proved to behave better during earthquake effects.

Based on the required strength and deformability characteristics of the elements and the system as a whole, a number of variant solutions have been considered and the most adequate (from the aspect of stability and economy) has been selected, (V5). The intensity and the location of the vertical elements have been defined on the basis of a detailed analysis of the bearing system and the possibility for their placement in a way not to disturb the architecture of the structure. Part of the vertical elements has been placed structurally to provide system lines in both orthogonal directions.

4.1. Comparison of Bearing and Deformability Capacity of the Structure

Figures 4.2 through 4.4 show the results on the characteristic levels obtained by the analysis of bearing and deformability capacity described in chapter 2.2 simultaneously for the three variant solutions of the bearing structural system (V2, V4 andV5).

The comparative presentation of the results obtained from this analysis clearly shows that the insertion of horizontal ties (HE-V4) along the length of the walls enables increase of the bearing capacity and stiffness but reduces the deformability of the characteristic levels. However, including also vertical ties (OS-V5) at the ends of the walls and around the openings enables considerable increase of strength and deformability of the structure at all levels in both orthogonal directions as the result of improvement of integrity and bending resistance.

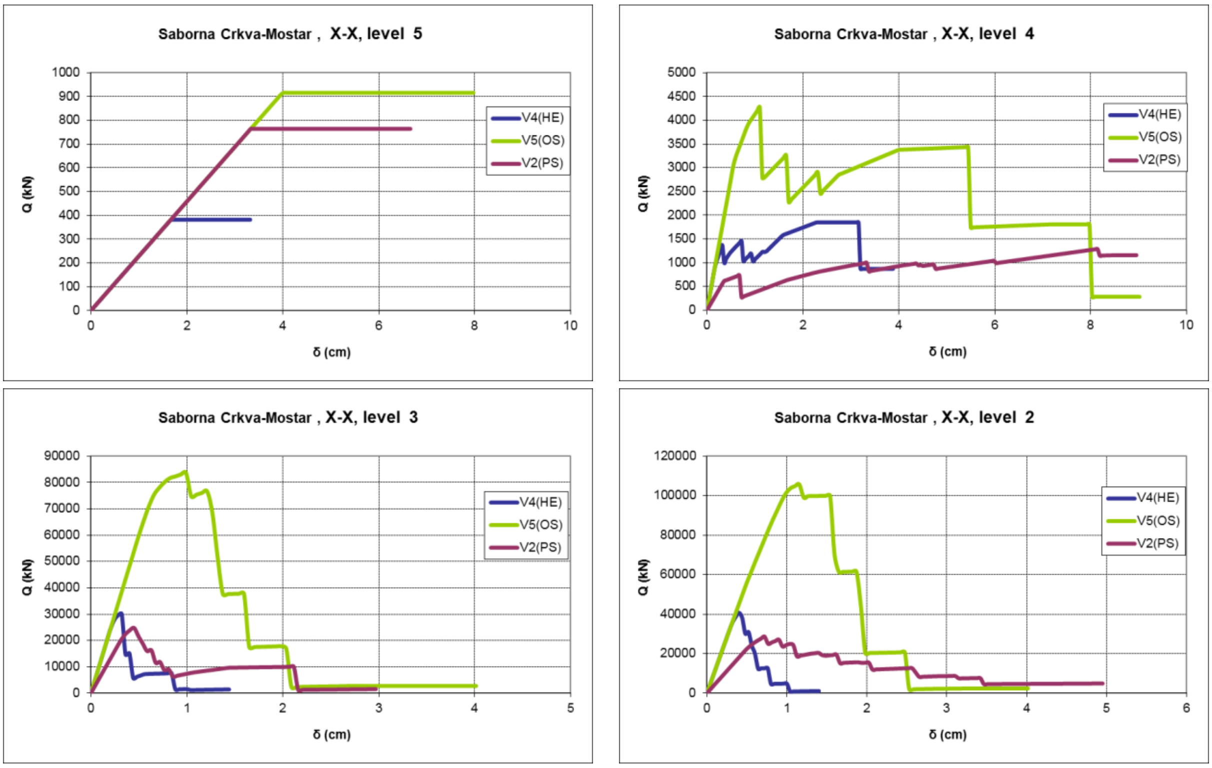


Figure 4.2. Comparative presentation of bearing capacity, longitudinal direction (level 2-5)

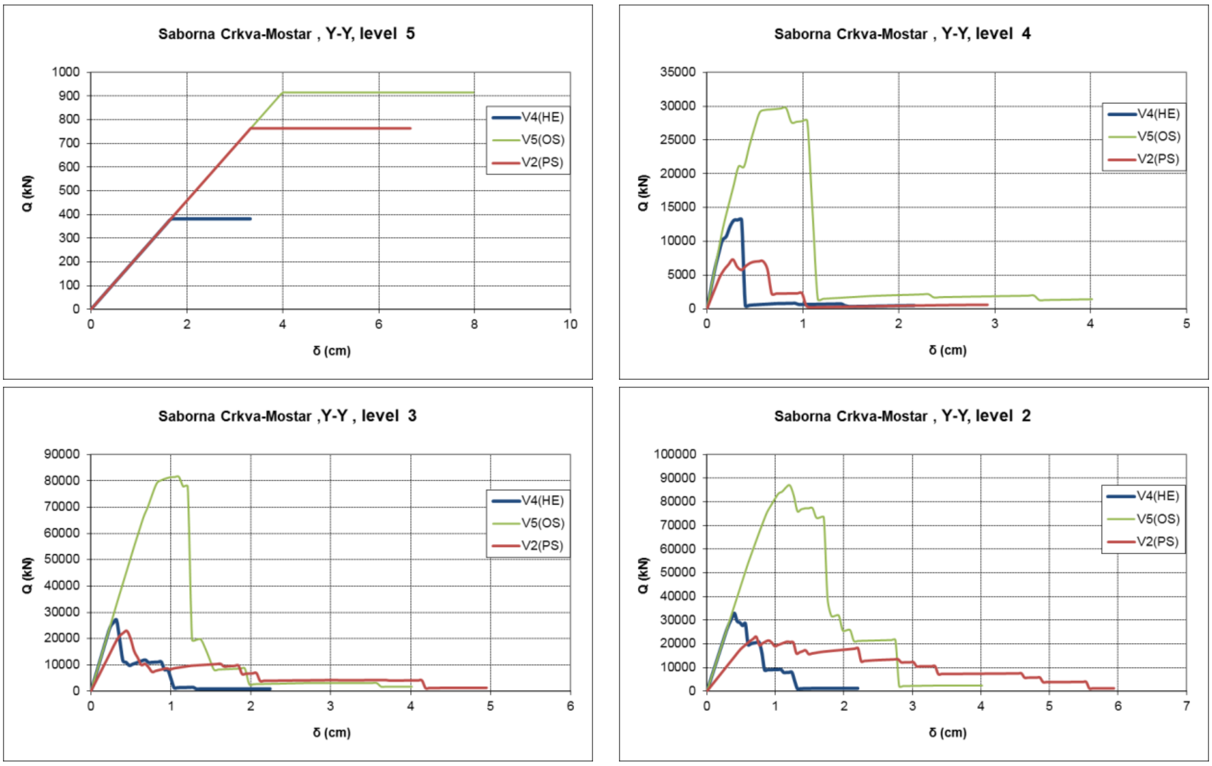


Figure 4.3. Comparative presentation of bearing capacity, transverse direction, (level 2-5)

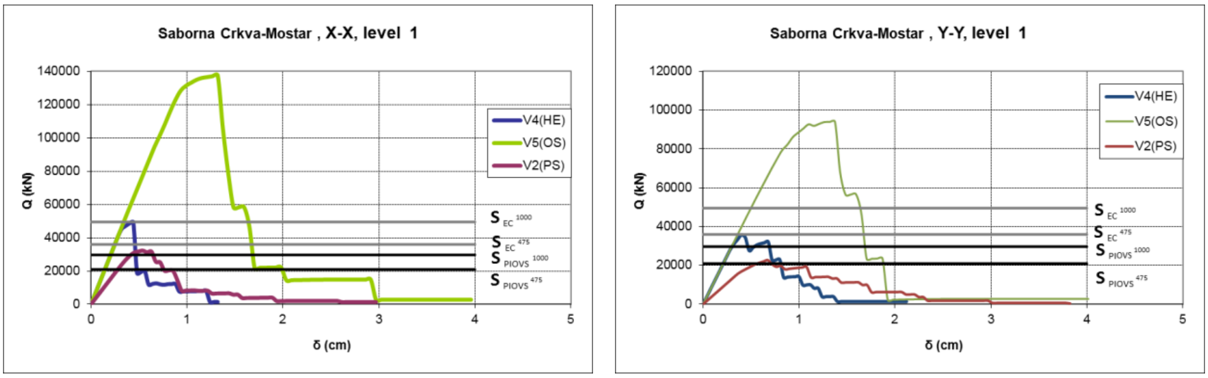


Figure 4.4. Comparative presentation of bearing capacity, level 1

With this, it has been proved that the church structure constructed of confined masonry in both directions, possesses sufficient bearing capacity in accordance with the defined criteria since the strength of the most critical first level has been higher than the total seismic force according to both JUS and EC8 standards (Fig. 4.4).

4.2. Comparison of Dynamic Response of the Structure for the Defined Seismic Parameters

To obtain the dynamic response of the structure, 9 different types of earthquakes have been used: 6 synthetic earthquake records defined by seismic microzonation as well as records of three other earthquakes, namely, Petrovats 1979, Ulcinj, 1979 and El Centro, 1940. The response has been investigated for the maximum input ground acceleration of a_{max}=0.16g and a_{max}=0.22g in accordance with the defined seismic hazard for return periods of 475 and 1000 years. As a result of the dynamic analysis, displacements and ductility demanded by the earthquake ($\mu^{earthquake} = \delta^{max} / \delta^{y}$) are obtained and these should comply with the design safety criteria, i.e., they should be less than μ^{100}<1.0, μ^{475}< 1.5 and μ^{1000}< 2.0.

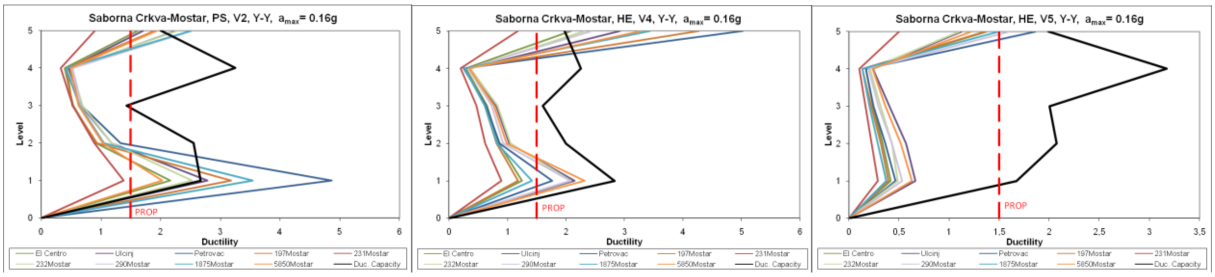


Figure 4.5. Dynamic response to the design earthquake, (V2, V4 and V5, transverse direction)

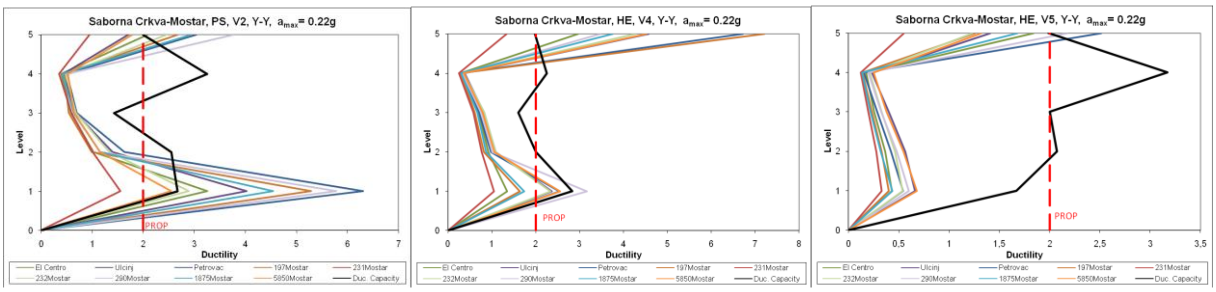
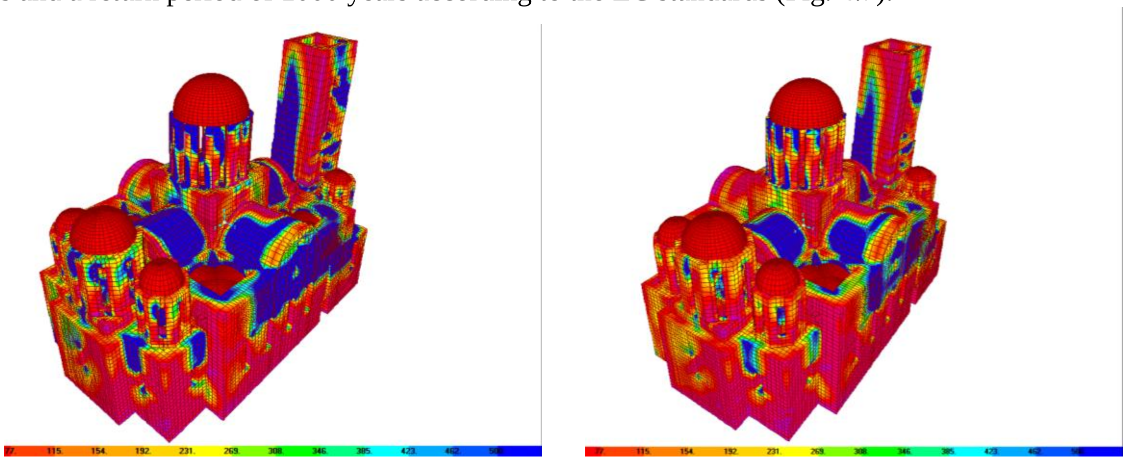


Figure 4.6. Dynamic response to the maximum earthquake, (V2, V4 and V5, transverse direction)

By comparative analysis of the dynamic responses, it can clearly be concluded that the behaviour of the structure constructed of confined masonry is considerably more favourable in respect to the other two variants (figures 4.5, 4.6). Despite the strict design criteria, the demanded ductility of the structure strengthened by horizontal and vertical elements for all the analysed earthquakes is within the limits of the allowed ductility. It is only that the response of the fourth level in longitudinal direction is more intensive than the allowed (μ>1.5 for a_{max}=0.16g, μ >2 for a_{max}=0.22g), but despite this, the structure possesses the demanded ductility capacity. A drastic improvement of response is characteristic for the transverse direction, particularly for the first, the most critical level. While the first level in the case of the designed structure is deep in the nonlinear range under the maximum expected earthquakes (μ=3-6 for different earthquakes), in the conditions of a strengthened structure, it is in the elastic range of behaviour.

4.3. Comparison of Stress-Strain State of the Structure

In this chapter, the results from the analysis of the structure by use of the finite element method are presented. Selected as the most important one in showing the efficiency of implementation of horizontal and vertical elements are the presentations of the main tensile stress for the applied seismic forces and a return period of 1000 years according to the EC standards (Fig. 4.7).



Designed-V2 strengthened-V5

Figure 4.7. Main tensile stresses (dead weight + $Sy^{EuroCode}$)

With this analysis of the stress-strain state, one can identify the potential local instabilities where exhaustion of compressive or tensile strength or excessive deformations takes place. The dark blue zones shown in Figure 4.7 indicate zones in which the tensile stress is higher than the tensile strength of the designed mortar, i.e., zones where occurrence of cracks could be expected under the maximum expected earthquake. The comparative presentation shows that these zones are reduced in the case of the strengthened structure in respect to the design structure considering both standards. This doesn't mean that these will occur in reality, because it will be prevented by the presence of ductile elements (horizontal and vertical ties), but it cannot be modelled by the programme that does not enable modelling of nonlinearity of material for SOLID elements, i.e., excluding masonry with stresses greater than the ultimate ones. In reality, upon occurrence of the first crack in masonry, the placed ties will be activated whereat extension of the cracks will be prevented, while the failure mechanism will be transferred in the lower zones instead taking place by separation of individual walls

Figure 4.8. Beginning of the construction of the church in May 2011

The reconstruction of the cathedral church with the bearing system constructed of confined masonry started in May 2011 (Fig. 4.8, 4.9).

Figure 4.9. Construction of the church up to the level of first horizontal ties

5. CONCLUSIONS

- Within the frames of renovation of the Cathedral Church in Mostar, detailed analysis of the stability of the designed structural system has been performed and the necessity for structural strengthening has been proved. Variant solutions have been proposed and analysed. The most adequate solution has been selected and analysis of the stability of the strengthened structure has been performed. Three general structural conditions have been treated: the designed structure constructed of plain stone masonry in cement lime mortar, the designed structure strengthened by horizontal steel elements, and the structure strengthened by horizontal and vertical steel elements.
- In each of the three individual cases, three types of analyses have been carried out: linear static and seismic analysis of the stress-strain state by use of the finite element method; analysis of the bearing elements up to ultimate states of strength, stiffness, deformability and ability of the elements and the system as a whole to dissipate seismic energy by linear and nonlinear behaviour; and analysis of the dynamic response of the bearing system to actual earthquakes, with intensity and frequency content expected at the location of the church.
- The analysis has shown that, with its strength, stiffness and deformability characteristics, the designed structure as well as the structure strengthened by horizontal elements only do not satisfy the seismic safety criteria and are not in compliance with the most recent knowledge on behaviour of masonry structures exposed to gravity and seismic effects.
- Strengthening by use of horizontal and vertical steel elements turns the structural system constructed of plain masonry into a confined masonry system that exhibits a considerably more favourable behaviour under dynamic effects. All the performed analyses have shown that the designed strengthening system which complies with the conservation demands enables the necessary integrity of the structure at the characteristic levels and increases its strength, bearing and deformability capacity to the designed level of seismic protection.

REFERENCES

Shendova V., Bozinovski Z., Stojanoski B., Gjorgievska E.(2011). Analysis of Seismic Stability with Technical Solution for Repair and Seismic Strengthening of the Orthodox Church in Mostar. *Report IZIIS 2011-22*

Shendova V., Rakicevic Z., Garevski M., Apostolska R., Bozinovski Z. (2012). Implementation of experimental developed methodology for seismic strengthening and repair in historic monuments. *FP7 SERIES Workshop on the Role of research infrastructures in seismic rehabilitation,* Istanbul, 2012

Shendova V., Gavrilovic P. (2010) New Materials and Techniques in Retrofitting Historical Buildings and Monuments. *14 European Conference on Earthquake Engineering*, 14ECEE, Ohrid, 2010